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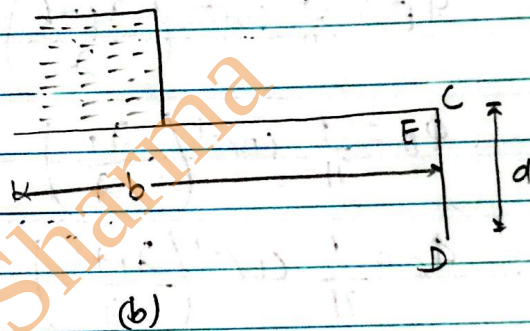
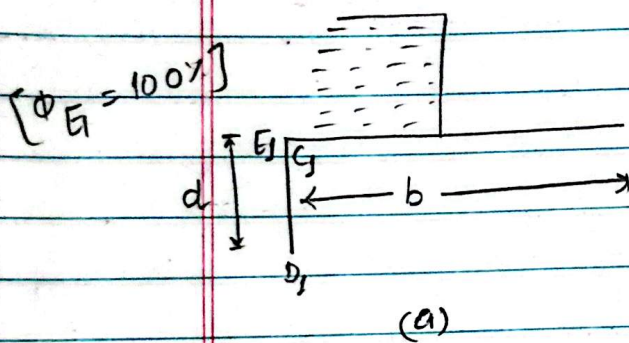
Surendra Sharma

Khosla's method of independent variables

Khosla developed a simple and quick approach for designing hydraulic structure such as weir/barrage called method of independent variables

3 conditions:-

- (1) St. hz. floor with negligible thickness provided with end piles.



$$\begin{cases} \Phi_{C1} = 100 - \Phi_E \\ \Phi_{D1} = 100 - \Phi_D \end{cases}$$

$$\Phi_E = \frac{1}{\pi} \cos^{-1} \left(\frac{\lambda - 2}{\lambda} \right)$$

$$\Phi_D = \frac{1}{\pi} \cos^{-1} \left(\frac{\lambda - 1}{\lambda} \right)$$

where

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2}$$

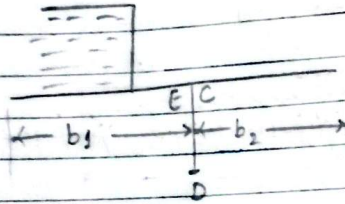
$$\alpha = \frac{b}{d} \text{ (respective)}$$

Φ_E, Φ_D & Φ_C are % pressure head at E, D and C of pile line respectively

(a) Up end pile

(b) Down end pile

2. St. hz. floor with negligible thickness provided with intermediate sheet piles



$$\phi_E = \frac{1}{\pi} \cos^{-1} \left(\frac{\lambda_1 - 1}{\lambda} \right)$$

$$\phi_D = \frac{1}{\pi} \cos^{-1} \left(\frac{\lambda_1}{\lambda} \right)$$

$$\phi_C = \frac{1}{\pi} \cos^{-1} \left(\frac{\lambda_1 + 1}{\lambda} \right)$$

where,

$$\lambda = \frac{\sqrt{1 + \alpha_1^2} + \sqrt{1 + \alpha_2^2}}{2}$$

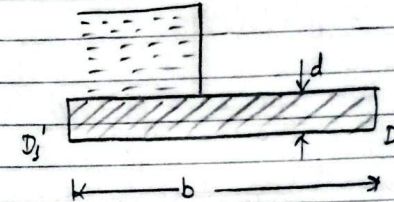
$$\lambda_1 = \frac{\sqrt{1 + \alpha_1^2} - \sqrt{1 + \alpha_2^2}}{2}$$

$$\alpha_1 = b_1/d$$

$$\alpha_2 = b_2/d$$

Fig: Intermediate sheet pile.

3. St. hz. floor depressed below bed w/o any vertical cutoff sheet pile.



Here, $\alpha = b/d$ b = floor length
 d = thickness of depressed floor.

$$\phi_E = \frac{1}{\pi} \cos^{-1} \left(\frac{\lambda - 2}{\lambda} \right)$$

$$\phi_D = \frac{1}{\pi} \cos^{-1} \left(\frac{\lambda - 1}{\lambda} \right)$$

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2}$$

$$\left\{ \begin{aligned} \phi_{D'} &= \frac{2}{3} (\phi_E - \phi_D) + \frac{3}{\alpha^2} \\ \phi_{D_1} &= 100 - \phi_{D'} \end{aligned} \right\}$$

Safe Gradient: G_E

$$G_E = \frac{H}{d} \times \frac{1}{\pi \sqrt{\lambda}}$$

H = head loss / seepage head at d / r

d = depth of dls cutoff / sheet pile.

$C_1 \rightarrow 1$ no pile
 $C_3 \rightarrow 3$ no pile
 $\frac{dL}{L} \rightarrow \dots$ numbering

Correction for % pressures

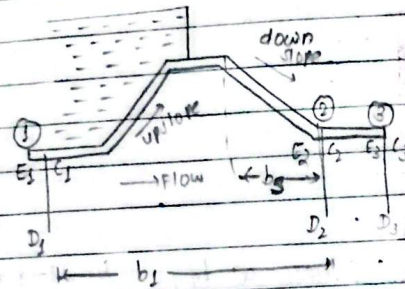


Fig 1

1. Correction for mutual interference of piles (C-int)

eg: ① pile को बिना गारा C_1 मा ② pile को E_2 गारे
 H-को असर आउँछ।

① र ③ को interference हुन्छ।
 यो consecutive piles को मात्र interference हुन्छ।

$$C_{int} = \pm 19 \sqrt{\frac{D}{b'}} \times \left(\frac{d+D}{b} \right)$$

b' = distance between two pile lines (interfering)

b = total floor length

d = depth of pile on which effect is considered

D = depth of pile, the effect of influence of which on neighbouring piles.

+ve $\rightarrow$ for influence in opposite direction of flow

(eg. पानीको बहाव: ① to ② तिर but interfere)

② ले उहाँ ① लाई -ve गर्ने +ve गर्ने)

-ve $\rightarrow$ for influence in the direction of flow.

(flow direction as interference dir)

Flow ① $\rightarrow$ ②

Interference ① ले उहाँ ② मा.

2. Correction for thickness of floor: C-thickness

considerably negligible thickness

$$C_{thickness} \text{ for } \phi C_1 = \frac{[\phi D_1 - \phi C_1]}{\text{depth of pile}} \times \text{floor thickness}$$

Calculated pressure at E_2' is more than $E_2 \rightarrow$ correction for E_2 is -ve

Calculated pressure at C_1 is more than C_1' , so correction is +ve

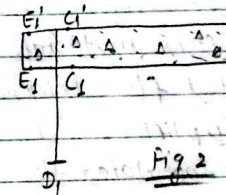


Fig 2

3. Correction for slope: C-slope

It is applied only for sloping floor for the pile line points if they are just at the starting or ending point of the slope.

$\Rightarrow$ +ve for down slope as that of flow direction

$\Rightarrow$ -ve for up slope as that of flow direction.

Since, pile no ② is at ending pt. of slope so correction for slope is applied for E_2 only (Fig 1)

$$C_{slope} \text{ for } \phi E_2 = \text{Correction factor} \times \frac{b_2}{b_1}$$

$[b_1 = 1/2 \text{ length of slope}]$

Slope	1:1	2:1	3:1	4:1	5:1	6:1	7:1	8:1
Correction factor	11.2	6.5	4.5	3.3	2.8	2.5	2.3	2.0

Safe against uplift

For safety against uplift, after finding percentage pressure heads at various bay points, the residual head 'h' in terms of depth of water is calculated. The thickness of floor 't' required to counter balance the uplift head is determined by using formula

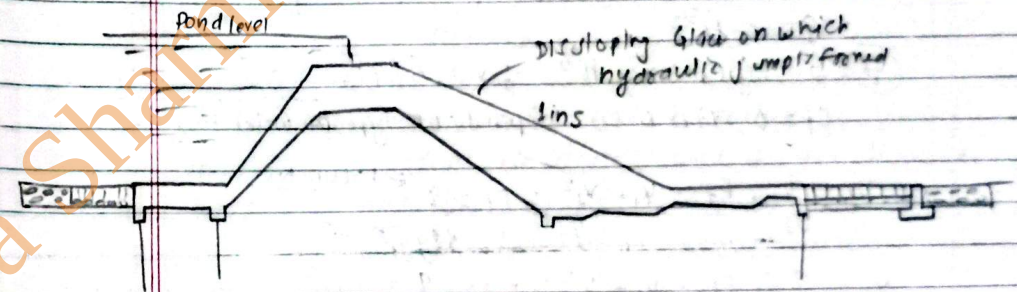
$$t = \frac{h}{(G-1)}$$

G = sp. gravity of floor material.

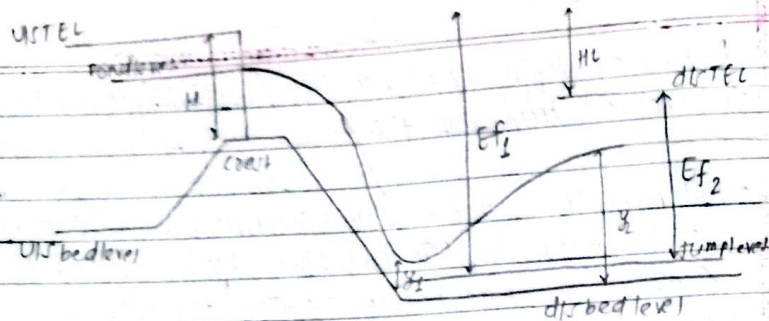
- If calculated $t \leq$ provided floor thickness of structure, then safe against uplift
- Otherwise, unsafe. If calculated floor thickness ' t ' > provided floor thickness of structure.

Design of sloping glacis weir bay (crest, length and thickness of impervious floor).

- Sloping glacis weir has downward slope provided from crest level towards d/s bed level where hydraulic jump is formed to dissipate energy of flowing water.
- In modern type of concrete weir provided with vertical cutoffs / sheet piles at u/s or d/s or well as at intermediate points as the requirement.



Hydraulic jump.



→ Here, $E_{f1} = E_{f2} + HL$.

→ $q = \text{discharge intensity} \quad [q = \frac{Q}{L}]$

$$q = C \cdot H^{3/2}$$

$$Q = \frac{2}{3} C_d L \sqrt{2g} H^{3/2} ; V = \sqrt{2gHL}$$

$$C_d = 0.577 \text{ to } 0.80 \text{ (depends on type of weir)}$$

→ $V_1 = \frac{q}{y_1} ; E_{f1} = y_1 + \frac{V_1^2}{2g} ; F_1 = \frac{V_1}{\sqrt{gy_1}} \leftarrow \text{Froude No.}$

$$V_2 = \frac{q}{y_2} ; E_{f2} = y_2 + \frac{V_2^2}{2g} ; F_2 = \frac{V_2}{\sqrt{gy_2}}$$

→ $HL = \frac{(y_2 - y_1)^3}{4y_1 y_2}$

$$y_2 = \frac{y_1}{2} [-1 + \sqrt{1 + 8F_1^2}] ; F_1 = \frac{V_1}{\sqrt{gy_1}}$$

$$y_1 = \frac{y_2}{2} [-1 + \sqrt{1 + 8F_2^2}] ; F_2 = \frac{V_2}{\sqrt{gy_2}}$$

↑
check

Design of weir/Barrage:-

1. Crest level of weir/barrage bays = 1 to 1.5m above crest level of undersluice.

[Crest level of undersluice = bed level of the river]

2. Afflux is generally taken as 1m. which helps to reduce length of weir but increases river training works.

3. Water way and Discharge per meter wetted perimeter (Q)

• Water way is calculated by using Lacey's wetted-perimeter,
 $P = 4.75 \sqrt{Q}$

• Generally, total length of water way L is taken as 1.2P to 1.4P i.e. $L = 1.2P$ to $1.4P$ including pier and bays of undersluice, divide wall & other barrage bays, or is multiplied by looseness factor given.

$$L = L_f \times P$$

• discharge intensity $q = \frac{Q}{L}$

4. Generally,

$$Q_{us} = \text{Max} \left[\begin{array}{l} 20\% \text{ of designed flood} \\ 2 \times \text{canal discharge} \\ \text{Max} \approx \text{winter flood discharge} \end{array} \right]$$

Q should be parted from under sluice and rest is from barrage bays/weir.

5. $R = 1.35 \left[\frac{q^2}{f} \right]^{1/3}$, $R = \text{Lacey's hydraulic mean radius}$,

$$f = \text{silt factor} = 1.76 \sqrt{d_{mm}}$$

6. Approach velocity, $V = \frac{q}{R}$; velocity head = $\frac{V^2}{2g}$;

$$U/S TEL = \text{U/S HFL} + \text{vel. head}; V = \left(\frac{q f^2}{140} \right)^{1/6}$$

head over under sluice $\phi_{\text{crest}} = \text{ULS TEL} - \text{undersluice crest level}$

Head over barrage bays = $\text{ULS TEL} - \text{crest level of weir/barrage}$

7. Discharge passing through weir/barrage

$\Rightarrow Q = 1.71 (L - k \cdot n \cdot H) H^{3/2}$ for broad crested weir.
If head over the crest $H > 1.5 \times \text{crest width}$,
it behaves as sharp crested.

where,

$H = \text{head over crest including velocity head}$

$\Rightarrow Q = 1.84 (L - k \cdot n \cdot H) H^{3/2}$ for sharp crested weir,
 $k = \text{coeff. of end contraction} = 0.10$
 $n = \text{no. of end contraction} = 2 \times \text{no. of bays}$

8. Pond level is taken as 1 to 2m above canal FSL
which is minimum water level reqd. in under
sluice pocket at ULS of canal head regulator.

9. Thickness and length of impervious floor is calculated by using Khosla's Theory.

10. Pond level's and HFL level's conditions should be analyzed to determine the min. level of formation of hydraulic jump with initial and final depth of the jump.

The critical condition is considered i.e. min^m level at which jump will form.

11. Minimum Length of d/s concrete floor $L_{\min} = 5(y_2 - y_1)$
where y_1 and y_2 are depth of before and after jump.

The depth ϕ of cutoff/sheet piles are calculated by considering depth of scour R .

$\rightarrow$ Depth of bottom of scour hole from ULS flow level = XR , where $X = 1.25$ to 1.5

$\rightarrow$ Depth of bottom of scour hole from d/s flow level = XR , $X = 1.5$ to 2 .

$\rightarrow$ Depth of ULS pile = ULS bed level - bottom level of ULS pile

$\rightarrow$ Depth of d/s pile = d/s bed level - bottom level of d/s pile

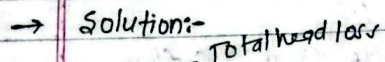
Protection works:

(i) Inverted filter for both ULS and d/s = $1.5D$ to $2D$ is provided just after end of concrete bat. $D = XR - y$.

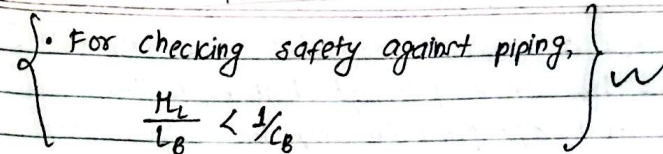
(ii) Launching apron: length of d/s launching apron = $1.5D$, where $D = (2R - y)$, thickness $(t) = 2.25D / \text{length of apron}$, where, $y = \text{d/s water depth}$.

(iii) length of ULS launching apron = $2.25D/t$, where, $D = (1.5R - y)$, $y = \text{ULS water depth}$.

(Q) From the following figure draw the hydraulic gradient line using Bligh's creep theory. Design the floor of the given weir and also check whether it is safe against piping or not. Take $C_u = 5$, $G = 2.25$ for concrete floor.



$$\text{At pile line 3 (after pile 3)} = H_4 = \frac{29}{29} \times (2 \times 8) = 1.103 \text{ m}$$



- To find thickness of floor:
 Ordinate of MGL above floor at 6m/d/s under weir, h_w ,
 find out

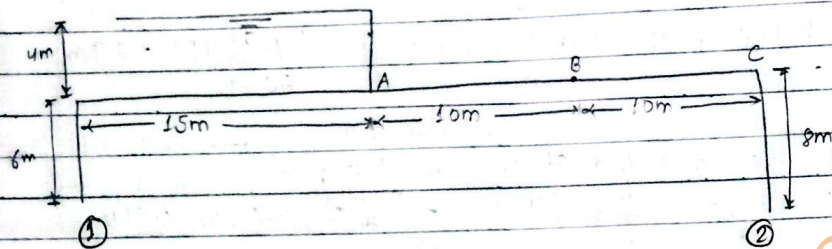
$$t_w = \frac{4}{3} \frac{(h_w)}{(4-1)}$$
 Bligh's method

which is very high but it can be reduced to a thickness of 1.5m because of the weight of water there is extra weight of water to balance uplift

Kharla

- (9) An impervious floor of a weir on permeable foundation soil is of 35m long and has sheet piles at both ends. The u/s pile is 6m deep and d/s pile is 8m deep. The weir creates a net head of 4m, neglecting the thickness of the weir floor, calculate the uplift pressure at the junctions of the inner faces of the piles with the weir floor using Kharla theory. Find thickness of floor required at B. take $g = 2.24$ for concrete. If permissible G.E. is $\frac{1}{7}$ check whether the

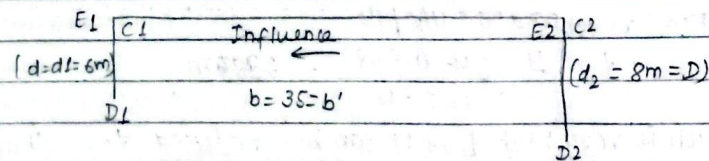
structure is safe against piping or not?



→ Solⁿ:-

- ↳ Only End pile
- ↳ no thickness given

total head = 4m = H



Calculations of percentage pressure head at inner faces of piles.

For pile no. 1 u/s pile, end pile condition

$d = 6m$ (depth of u/s pile)

$b = 35m$ (total floor length)

$\alpha = b/d = 35/6 = 5.83$

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} = \frac{1 + \sqrt{1 + 5.83^2}}{2} = 3.457$$

$$\phi_E = \frac{1}{\pi} \cos^{-1} \left(\frac{\lambda - 2}{\lambda} \right) = \frac{1}{\pi} \cos^{-1} \left(\frac{3.457 - 2}{3.457} \right) = 0.36 = 36\%$$

$$\phi_{C1} = 100 - \phi_E = 100 - 36 = 64\%$$

- (i) correction for ϕ_{C1} due to mutual interference of d/s pile no. 2 on pile no. 1;

$$C_{int} = 19 \sqrt{\frac{D}{b'}} \left(\frac{d+D}{b} \right)$$

$D = 8m, b' = b = 35m, d = 6m$

$$= 19 \sqrt{\frac{8}{35}} \left(\frac{8+6}{35} \right)$$

$$= 3.63\% \quad (+ve)$$

① pile पर पानी गहरा ② pile पर
ठोस पानी & back to ① में गे

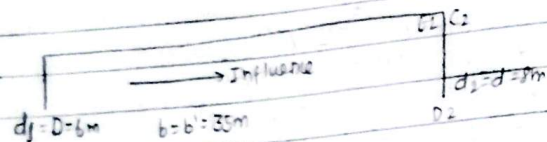
[∴ interference is opposite of flow direction]

[② के influence ① में]

↳ thickness and slope correction are nil. [∴ No floor thickness & slope provided]

$$\therefore \text{corrected } \phi_{C1} = 64\% + 3.63\% = 67.63\%$$

For pile no. 2, dls pile, end pile condition:



Depth of dls pile = $d = 8\text{m}$

total floor length, $b = 35\text{m}$

$$\alpha = \frac{b}{d} = \frac{35}{8} = 4.375$$

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} = \frac{1 + \sqrt{1 + 4.375^2}}{2} = 2.74$$

$$\phi_E = \phi_{E2} = \frac{1}{\pi} \cos^{-1} \left(\frac{\lambda - 2}{\lambda} \right) = 41.30^\circ$$

correction of ϕ_{E2} due to mutual interference of
uls pile no 1 on pile no. 2;

Then, $d = 8\text{m}$, $b = 6\text{m}$, $b = b' = 35\text{m}$

$$C_{int} = 19 \sqrt{\frac{D}{b'}} \left(\frac{d+D}{b} \right)$$

$$= 19 \sqrt{\frac{6}{35}} \left(\frac{8+6}{35} \right)$$

$$= 3.15\%$$

(-ve)

(flow & interference same direction)

(No thickness & slope)

$$\phi_{E2} = 41.30 - 3.15 = 38.15\%$$

Now, pressure head at $C_1 = h_{C_1} = H \times 67.63\%$

$$= 4 \times 0.6763 = 2.71\text{m of water}$$

Pressure head at $E_2 = h_{E_2} = H \times 38.15\%$

$$= 4 \times 0.3815 = 1.53\text{m of water}$$

$$\% \text{ pressure at pt. B } (\phi_B) = \phi_{E2} + \left(\frac{\phi_{C_1} - \phi_{E2}}{35} \right) \times 10$$

$$= 38.15 + \left(\frac{67.63 - 38.15}{35} \times 10 \right)$$

$$= 46.57\%$$

$$\text{'OR'} \quad \phi_{C_1} - \left(\frac{\phi_{C_1} - \phi_{E2}}{35} \right) \times 25 = 46.57\%$$

Residual head at B = $h_B = 0.4657 \times 4 = 1.86\text{m of water}$

$$\text{Thickness of floor required at B} = t_B = \frac{h_B}{\frac{5-1}{2.24-1}} = \frac{1.86}{2.24-1} = 1.50\text{m}$$

(33% not taken by khoria)

⇒ TO check safety against piping:

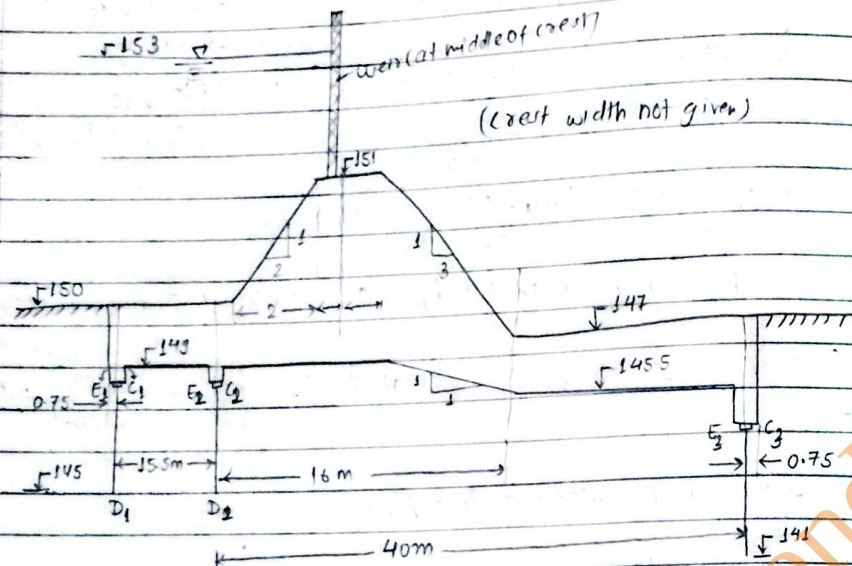
$$G_E = \frac{H}{d} \times \frac{1}{\pi \sqrt{\lambda}}$$

dls pile condition: $d = 8\text{m}$, $H = 4\text{m}$, $b = 35\text{m}$, $\alpha = 4.375$

$$G_E = \frac{4}{8} \times \frac{1}{\pi \sqrt{2.74}} = \frac{1}{10.41} < \frac{1}{7} \text{ safe } G_E$$

Hence, structure is safe against piping.

- (9) Check whether the structure is safe against uplift and piping or not using Khosla's method from the attached figure of a sloping glacis weir. Take permissible velocity hydraulic gradient of $1/7$ and specific gravity of concrete as 2.25.



$$\begin{aligned} \text{Head loss} &= \text{RL of u/s} - \text{RL of d/s bed (water level)} \\ &= \text{water level} \\ &= 153 - 147 \\ &= 6\text{m.} \end{aligned}$$

(Initially floor thickness $\frac{5}{8}$ m at start of pile)

(I) Calculation of percentage E_1

pressures (corrected)

$b = 57\text{m}$
($0.75 + 15.5 + 40 + 0.75$)

$d = 5\text{m}$

(Total floor length extreme left to extreme right)

(a) For u/s pile, end pile condition.

$$\text{Total floor length (b)} = 40 + 15.5 + 2 \times 0.75 = 57\text{m}$$

$$\text{Depth of pile, } d = 150 - 145 = 5\text{m}$$

$$\alpha = \frac{b}{d} = \frac{57}{5} = 11.40$$

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} = \frac{1 + \sqrt{1 + 11.40^2}}{2} = 6.22$$

$$\begin{aligned} \phi_E &= \frac{1}{\pi} \cos^{-1} \left(\frac{\lambda - 2}{\lambda} \right) = \frac{1}{\pi} \cos^{-1} \left(\frac{6.22 - 2}{6.22} \right) = 0.263 = 26.3\% \\ \phi_D &= \frac{1}{\pi} \cos^{-1} \left(\frac{\lambda + 1}{\lambda} \right) = \frac{1}{\pi} \cos^{-1} \left(\frac{6.22 + 1}{6.22} \right) = 18.3\% \end{aligned}$$

Now,

$$\phi_{E_1} = 100\%$$

$$\phi_{D_1} = 100 - \phi_D = 100 - 18.3 = 81.7\%$$

$$\phi_{C_1} = 100 - \phi_E = 100 - 26.3 = 73.7\%$$

- (1) → starting pt of slope. ⇒ slope corr.
 (2) → thickness correction reqd.

⇒ Correction for ϕC_1

⇒ (a) Correction due to Mutual interference of intermediate pile no. 2 on pile no. 1.

(interference at depth thickness consider जहाँ गलती level at pile end में)

$d = 149 - 145 = 4m$ [वस्तुतः effect पसिदाह → pile (1)]

$D = 4m$ [जसले जहाँ interfere जहाँ]

$b = 15.50m$ (distance b/w two interfering piles)

$b = 57m$ (total floor bed distance)

$$C_{int} = 19 \sqrt{\frac{D}{b}} \left(\frac{d+D}{b} \right)$$

$$= 19 \sqrt{\frac{4}{15.50}} \left(\frac{4+4}{57} \right)$$

$$= 1.36\% \text{ (+ve)}$$

(interference & flow opposite dir)

(b) Correction due to floor thickness

$$C_{thickness} = \frac{(\phi D_1 - \phi C_1)}{\text{depth below top of floor}} \times \text{thickness of floor}$$

$$= \left(\frac{81.7 - 73.7}{150 - 145} \right) \times (150 - 149)$$

$$= 1.60\%$$

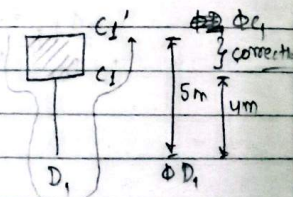
(+ve)

(पसिदाह C_1 pt. जहाँ गलती 150 में)

level में है तो calculate

जहाँ जहाँ है तो calculate

जहाँ जहाँ है तो calculate



(pressure at C_1 more than C_1')

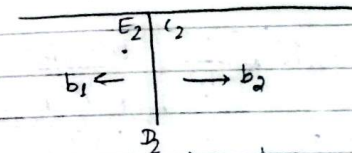
Use correction ⇒ only for inner junction point

(c) Correction for slope is nil because C_1 is not the starting & ending point of slope.

$$\therefore \text{corrected } \phi C_1 = 73.7\% + 1.36\% + 1.6\% = 76.66\%$$

(other two i.e. ϕE_1 & ϕD_1 no correction required)

⇒ For pile no. 2, intermediate pile condition



(Initially, floor bed level consider जहाँ जहाँ है. thickness consideration)

$d = 5m$, $b = 57m$, $b_1 = 15.5 + 0.75 = 16.25m$, (2nd pile to left extremely)

$b_2 = 40 + 0.75 = 40.75m$ (2nd pile to right extremely)

$$\alpha_1 = \frac{b_1}{d} = \frac{16.25}{5} = 3.25$$

$$\alpha_2 = \frac{b_2}{d} = \frac{40.75}{5} = 8.15$$

$$\lambda = \frac{\sqrt{1+\alpha_1^2} + \sqrt{1+\alpha_2^2}}{2} = \frac{\sqrt{1+3.25^2} + \sqrt{1+8.15^2}}{2} = 5.81$$

$$\lambda_1 = \frac{\sqrt{1+\alpha_1^2} - \sqrt{1+\alpha_2^2}}{2} = -2.41$$

$$\text{corr} \left\{ \phi E_2 = \phi E = \frac{1}{\pi} \cos^{-1} \left(\frac{\lambda_1 - 1}{\lambda} \right) = \frac{1}{\pi} \cos^{-1} \left(\frac{-2.41 - 1}{5.81} \right) = 70\% \right.$$

$$\left\{ \phi D_2 = \phi D = \frac{1}{\pi} \cos^{-1} \left(\frac{\lambda_1}{\lambda} \right) = \frac{1}{\pi} \cos^{-1} \left(\frac{-2.41}{5.81} \right) = 63.6\% \right.$$

$$\text{corr} \left\{ \phi C_2 = \phi C = \frac{1}{\pi} \cos^{-1} \left(\frac{\lambda_1 + 1}{\lambda} \right) = \frac{1}{\pi} \cos^{-1} \left(\frac{-2.41 + 1}{5.81} \right) = 57.80\% \right.$$

$D = (\text{आफनो मायिल्लो level}) - (\text{Interfere गर्नेको तल्लो level})$

Correction for ϕE_2
(a) Mutual interference due to pile 1 on pile 2.

$d = 4\text{m}$, $D = 4\text{m}$] interfere गर्ने pile length

$$b' = 15.50\text{m}$$

$$b = 57\text{m}$$

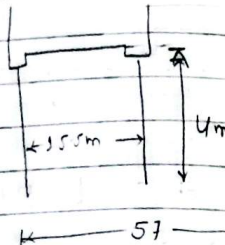
$$C_{\text{int}} = 19 \sqrt{\frac{D}{b'}} \left(\frac{d+D}{b} \right)$$

$$= 19 \sqrt{\frac{4}{15.50}} \left(\frac{4+4}{57} \right)$$

$$= 1.36\%$$

(-ve)

flow & interference same direction



(b) Correction due to floor thickness,

$C_{\text{thickness}}$

$$\left(\frac{\phi E_2 - \phi D_2}{\text{depth of pile from floor top}} \right) \times \text{Thickness}$$

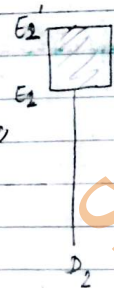
$$= \left(\frac{70 - 63.6}{5} \right) \times 1$$

$$= 1.28\%$$

(-ve)

(E_2 less than E_2') $\rightarrow$ -ve

Pressure at E_2 is less than E_2'



(c) As E_2 is not starting pt of slope (as it is left of pile 2) nor ending (pile ले छेको छ)

$$\therefore \text{Correction } \phi E_2 = 70\% - 1.36\% - 1.28\% = 67.36\%$$

Correction for ϕC_2

(a) Mutual interference due to pile 3 on pile 2.

(pile 1 ले interfere गर्ने सवर्द्धन ar C_2 pt बाई pile 2 ले छेको छ)

$d = 4\text{m}$, $D = 149 - 141 = 8\text{m}$ (2 pile को upper level देखि 3 pile को lower level)

(interfering pile को interfering length)

$$b' = 40\text{m} \text{ (blw two piles)}$$

$$b = 57\text{m} \text{ (total flow)}$$

$$C_{\text{int}} = 19 \sqrt{\frac{D}{b'}} \left(\frac{d+D}{b} \right) = 19 \sqrt{\frac{8}{40}} \left(\frac{4+8}{57} \right) = 1.79\% \text{ (+ve)}$$

(b) Corrections due to floor thickness

$$C_{\text{thickness}} = \left(\frac{63.6 - 57.8}{150 - 145} \right) \times (150 - 149) = 1.16\% \text{ (+ve)}$$

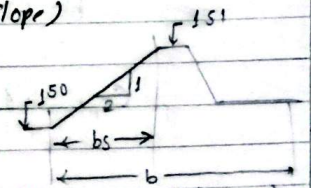
(c) Correction for slope (C_2 starting pt of slope)

Slope = 2:1 ; upslope along flow

$$b_s = (151 - 150) \times 2 = 2\text{m}$$

$$b' = 40\text{m}$$

$$C.F. = 6.5 \text{ for } 2:1$$



(b) $\rightarrow$ slope को direction बाई pile 2 को upper level

$$C_{\text{slope}} = C.F. \times \frac{b_s}{b'} = 6.5 \times \frac{2}{40} = 0.33\% \text{ (-ve)}$$

Upslope

$$\therefore \text{corrected } \phi C_2 = 57.80\% + 1.79\% + 1.16\% - 0.33\% = 60.42\%$$

⇒ For pile no 3, end condition:-

$$d = 147 - 141 = 6\text{m}$$

$$D = 145.5 - 145 = 0.5\text{m}$$

$$\alpha = \frac{b}{d} = \frac{57}{6} = 9.50$$

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} = 5.28$$

$$\phi_E = \phi_{E3} = \frac{1}{\pi} \cos^{-1} \left(\frac{\lambda - 2}{\lambda} \right) = 28.70\%$$

$$\phi_D = \phi_{D3} = \frac{1}{\pi} \cos^{-1} \left(\frac{\lambda - 1}{\lambda} \right) = 19.90\%$$

$$\phi_{C3} = 0\%$$

Correction for ϕ_{E3} ,

(a) Mutual interference due to pile 2 on pile 3

$$d = 145.5 - 141 = 4.50\text{m}$$

$$D = 145.5 - 145 = 0.50\text{m}$$

$$b' = 40\text{m}$$

$$b = 57\text{m}$$

$$C_{int} = 19 \sqrt{\frac{D}{b'}} \left(\frac{d+D}{b} \right) = 19 \sqrt{\frac{0.50}{40}} \times \left(\frac{4.5+0.5}{57} \right)$$

$$= 0.18\% \text{ (-ve)}$$

(b) Correction due to thickness of floor.

$$C_{thickness} = \frac{28.70 - 19.90}{147 - 141} \times (147 - 145.50) = 2.20\%$$

$$(-ve)$$

© Correction for slope is nil because $E3$ is not the starting & ending pt. of slope

$$\therefore \text{corrected } \phi_{E3} = 28.70\% - 0.18\% - 2.20\%$$

$$= 26.32\%$$

⇒ To check safe against piping:

d/s end pile condition

Total seepage head, $H = 153 - 147 = 6\text{m}$ (no water on d/s flow)

$$\left\{ \begin{array}{l} d = \text{end pile depth} = 147 - 141 = 6\text{m} \\ \lambda = 5.28 \end{array} \right.$$

$$\text{Exit gradient } (G_E) = \frac{H}{d} \times \frac{1}{\pi \lambda}$$

$$= \frac{6}{6} \times \frac{1}{\pi \times 5.28}$$

$$= \frac{1}{7.217} < \frac{1}{7}$$

Okay

(safe)

safe against piping

⇒ To check safe against uplift.

Pile line	US End pile			Inter pile			Dis end pile		
Points	E1	D1	C1	E2	D2	C2	E3	D3	C3
% Pressure corrected ϕ	100	88.7	76.66	67.36	63.60	60.42	26.32	19.90	0
Total Head, H	6m								
Residual head, $h = H - h_p$	6m	4.90	4.6	4.04	3.82	3.62	3.58	3.19	0
Head loss ($H_c = H - h$)	0	1.10	1.4	1.96	2.18	2.38	4.42	4.81	6.0

⇒ Let crest width be 2m.

$$\% \text{ pressure at crest of wall, } \phi = 60.42 - \left\{ \frac{60.42 - 26.32}{40} \right\} \times (2+1)$$

$$= 57.84\%$$

$$\text{Residual head at crest, } h = 6 \times 57.84\% = 3.47\text{m}$$

$$\text{Thickness required, } t = \frac{h}{(G-1)} = \frac{3.47}{(2.25-1)} = 2.78\text{m} > 2\text{m}$$

(151-149)
uplift.

(Khosla's curve can also be referred for finding ϕ)
from $(\frac{1}{2} = \frac{d}{b}) \Rightarrow \phi = \phi_D \& \phi_E$ } But correction required
 $\frac{\text{up pile}}{100} \quad \frac{\text{dis pile}}{100}$

$$\text{At 16m dis from } C_2$$

$$[(151-150) \times 2 + 2 + (151-147) \times 3] = 16\text{m}$$

$$\phi \text{ at 16m dis of } C_2 = 60.41 - \left\{ \frac{60.41 - 26.32}{40} \right\} \times 16$$

$$= 46.78\%$$

$$\text{Residual/uplift head 'h' at 16m dis from } C_2$$

$$= 6 \times 46.78\%$$

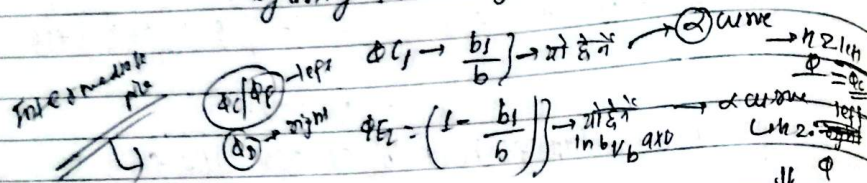
$$= 2.81\text{m}$$

$$\text{Thickness required} = t = \frac{h}{(G-1)} = \frac{2.81}{(2.25-1)} = 2.25\text{m} > 1.5\text{m}$$

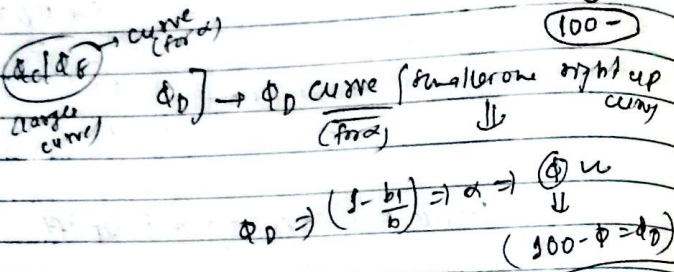
(147-145g)

Hence, not safe against uplift.

(Q.3) Calculate the χ pressure head at the key points of the given fig of sloping glacis
 → By using Charlas graph



End plate
 otherwise



$$\phi_D = \begin{cases} \text{i) } b_1/b < 0.5 \Rightarrow \text{right in } \phi_D \text{ curve at } \frac{b_1/b}{100} \text{ to } 100\% \text{ for } (1 - \frac{b_1}{b}) \Rightarrow \text{on } \phi_D \text{ curve \& find } \phi_D \text{ by } (100 - \phi) \\ \text{ii) } b_1/b > 0.5 \Rightarrow \phi_{\text{taken}} = \phi_D \text{ okay} \end{cases}$$

$$\phi_E \left\{ (1 - \frac{b_1}{b}) \Rightarrow \phi = 1 (\phi_E = 100 - \phi) \Rightarrow \text{Always} \right.$$

$$\phi_C \left\{ \frac{b_1}{b} \Rightarrow \phi = \phi_C \right\}$$

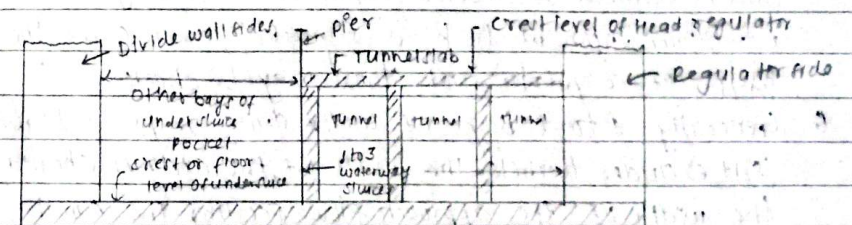
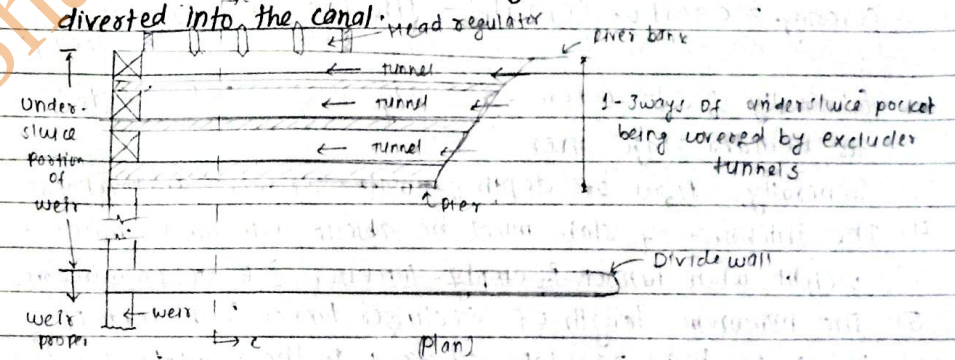
✓ correction required after finding ϕ at different points

(Q.3)

End pile
structure

Design of silt excluder

- Silt excluder is constructed in the river bed in front of head regulator to prevent coarse silt from entering the canal. The basic idea behind the design is that the lower levels layer of the flowing water carry a higher concentration of silt than in the upper layers and therefore if the upper layers of the water only can be passed into the canal, all the rolling bed silt and the silt in the lower layers is excluded from the excluder tunnel through the gate of undersluice.
- Excluder is a diaphragm slab supported on a number of rectangular tunnels. Tunnels are placed parallel to the head regulator and discharge dis through the undersluice gate. The water above the silt excluder slab containing less and fine suspended is then diverted into the canal.



section at C-C (Scale different in plan & section)

Fig. Silt excluder

Design:
 (Q.) → Discharge of silt excluded = 15 to 20% of canal discharge.
 The discharge through the tunnels will depend upon the head measured above the center line of the tunnel.
 Straight tunnels ~~are~~ can be treated as box culverts.

1. Minimum flow velocity, $v = 2.0$ to 4.5 m/s to keep free from sediment deposit. value of 3 m/s generally adopted.

2. Sectional area, $A = Q/v$, $A = B \times D$
 B = width of tunnel & D = ht. of tunnel

3 Ht. of opening is determined by the clearance available from the floor of pockets of u/s of under sluice to the crest of head regulator minus the thickness of roof covering.

$$= \text{crest of regulator} - \left(\begin{array}{l} \text{thickness of slab} \\ + \text{undersluice crest level} \end{array} \right)$$

Generally 0.5 to 0.6 m for sandy river and 0.80 to 1.8 m for boulder stage river.

Generally, $1/3^{\text{rd}}$ of depth of water minus slab thickness.

4. The thickness of slab must be able to withstand water weight when tunnel is empty. Thickness ≥ 0.3 m (minimum).

5. The minimum length of excludes tunnel is as that of length of head regulator adjacent to the regulator crest and is reduced to shorter length subsequently for other tunnels i.e. tunnel adjacent to head regulator must cover the full length of regulator and others may be shorter.

6. Generally 1 to 3 bays of under sluice is provided with silt excludes tunnels. The width of the excludes should cover the width of the approach flow feeding the canal.

7. Tunnels are separated by walls of thickness 0.30 to 0.80 m sufficient strength to withstand the impact of coarse particles of sand, gravels, pebbles and even boulders.

Design of Silt ejector

→ Silt ejector or silt extractor is the silt controlling device which is provided on the bed of the off-taking ~~to~~ main canal d/s side some distance far away from the head regulator to extract silt entered into the canal and ~~disposed~~ ^{disposed} back to the river.

→ The silt ejector should not be sited too near the head regulator as the residual turbulence may cause the sediment load to remain in suspension and prevent its ejection to the desired extent.

→ At the same time, it should not be far away from the head reach otherwise the sediment may settle down in earlier reach and reduce the channel capacity ~~downstream~~ upstream.

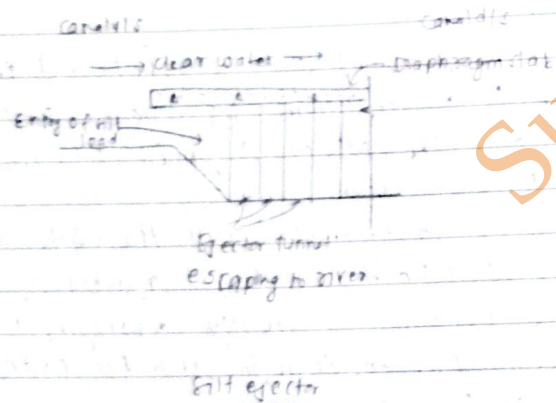
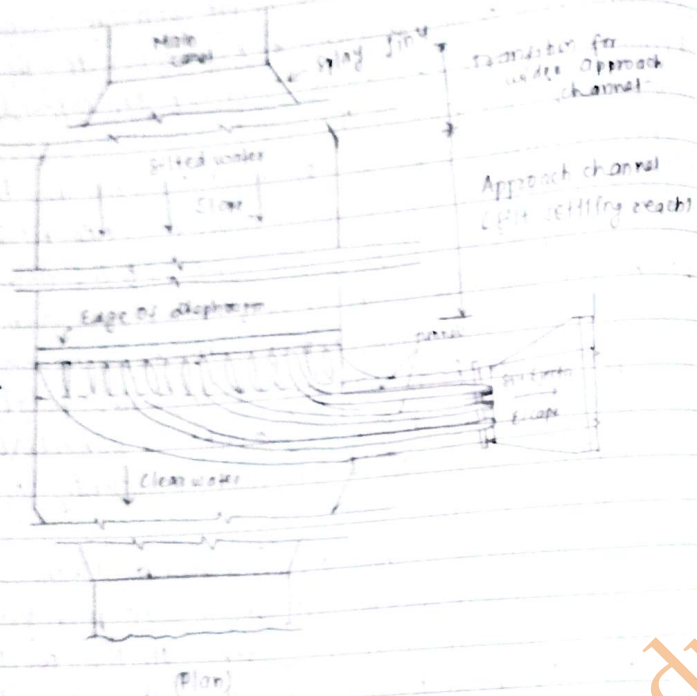
→ It consists of horizontal diaphragm slab above the canal bed which separates the bottom layer of flowing water where silt occurs.

→ The canal bed is slightly depressed ~~down~~ under the diaphragm.

→ Under the diaphragm slab tunnels are provided which receive the silt particles and the water that is washed or removed from the bed to the river from the regulator ~~to~~ gate of escape channel.

→ These main compartments shall be subdivided into smaller compartments or sub-tunnels by vanes of radii varying from 3 to 4 times the width of sub-tunnels to avoid cross flow in the transition section.

→ The ejector should normally span the entire width of the canal and shall be divided into a number of compartments of tunnels by vanes gradually converging to or to accelerate the escaping flow for delivering it to the outfall channel on one side of the canal.



• Design of silt ejector consists:

→ (1) $Q_{\text{ejector}} = 20\%$ of canal discharge

(2) Concentration of coarse sediment usually persists in $1/3^{\text{rd}}$ to $1/4^{\text{th}}$ of the depth of flow. As such, the height of the diaphragm in most of the ejectors has been kept in this range.

(3) The bed of canal depressed by 0.3 to 0.5 m (approximately 1 in 100 slope of bed) gradually at the mouth of the ejector so that bed load may be trapped.

(4) Height of opening of diaphragm slab = 20 to 25% of canal water depth.

(5) A working head of about 1 m is generally sufficient b/w uls canal water level of ejector and the outlet channel at the exit of ejector tunnel to extract the desired sediments.

(6) The depth of main tunnels should be kept about 1.8 to 2.2 m to facilitate inspection and repairs work.

(7) The section of the sub-tunnel at the entry should be so chosen that the velocity of flow at the intake is slightly higher than the velocity of bottom filaments of water upstream of the ejector.

(8) The section at the exit of the main tunnels shall be so designed as to attain a velocity of 2.5 to 6 m/s depending on the grade of sediment to be ejected. Generally, 3 m/s.

Tutorials

1. Design a sloping glass weir for the following site condition.
Maximum discharge intensity on weir crest, $q = 10 \text{ m}^3/\text{s}/\text{m}$
HFL before construction of weir = 250m

Pond level = 249.00m

RL of river bed = 244.50m

Ht. of crest shutter = 1m

Dis Bed deterioration = 0.50m [HFL of Bed level at weir crest is 0.50m at bed afflux]

Lacey's silt factor (f) = 0.90

Anticipated water level in dis of river when weir is discharging with pond level U/S = 246.50m

Permissible exit gradient = $G_E = 1/7$

Afflux = 1m (permissible)

(Water U/S to dis U/S ~~is~~ back to U/S with

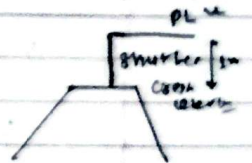
so, water level increased afflux)

Before HFL = 250m

Due to afflux, HFL will be 251m

Solution:-

(a) Crest level of weir = pond - ht. of shutter
 $= 249.00 - 1.0\text{m}$
 $= 248.00\text{m}$



(b) Calculation of velocity head

$Q = \frac{2}{3} C_d \sqrt{2g} L H^{3/2}$ for rectangular crest weir

$Q/L = \frac{2}{3} C_d \sqrt{2g} H^{3/2}$

$q = 1.71 H^{3/2}$; $C_d = 0.577$

But discharge intensity, $q = Q/L$

$10 = 1.71 H^{3/2}$

$\Rightarrow [H = 3.25\text{m}]$

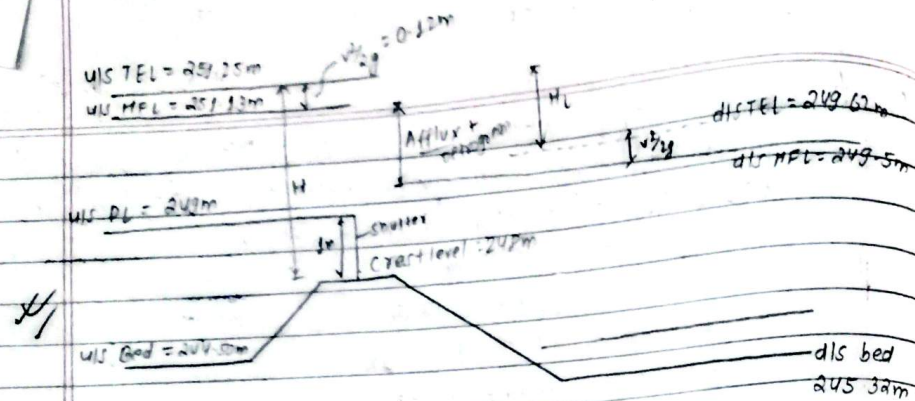


Fig: HFL condition with bed retrogression of 0.5m

$H \rightarrow$ Head over the crest

$$\therefore \text{U/S TEL} = \text{crest level} + H = 248.00 + 3.25 = 251.25\text{m}$$

$$\begin{aligned} \text{From Lacey's formula, scour depth } R &= 1.35 \left(\frac{q^2}{f} \right)^{1/3} \\ &= 1.35 \left(\frac{10^2}{0.90} \right)^{1/3} \\ &= 6.49\text{m} \end{aligned}$$

$$\text{velocity of flow} = V = \frac{q}{R} = \frac{10}{6.49} = 1.54\text{m/s}$$

$$\therefore \text{velocity head} = \frac{V^2}{2g} = \frac{1.54^2}{2 \times 9.81} = 0.12\text{m}$$

(c) Loss of head at TEL HFL condition with 0.50m retrogression of d/s bed level.

$$\begin{aligned} \text{U/S HFL} &= \text{U/S TEL} - \text{vel. head} = 251.25 - 0.12 \\ &= 251.13\text{m} \end{aligned}$$

$$\begin{aligned} \text{d/s TEL} &= \text{HFL before construction of weir} + \text{velocity head} \\ &= 250 + 0.12 \\ &= 250.12\text{m} \end{aligned}$$

$$[d/s \text{ HFL} = \text{U/S HFL} - \text{Afflux} - \text{bed retrogression}]$$

$$\therefore \text{Afflux} = \text{U/S HFL} - d/s \text{ HFL} = 251.13 - 250 = 1.13\text{m} \approx 1\text{m} \text{ okay}$$

$$\begin{aligned} \text{d/s HFL after retrogression of } 0.50\text{m of d/s bed} \\ &= 250 - 0.5 \\ &= 249.50\text{m} \end{aligned}$$

$$\begin{aligned} \text{d/s TEL after retrogression} &= d/s \text{ TEL} - 0.50 \\ &= 250.12 - 0.50 \\ &= 249.62\text{m} \end{aligned}$$

(bed 0.5m ले तब कच्ची पानी सही 0.5m ले तब कच्ची)

$$\begin{aligned} \therefore \text{Loss of head at HFL condition with retrogression} \\ &= \text{U/S TEL} - \text{d/s TEL after retrogression} \\ &= 251.25 - 249.62 \\ &= 1.63\text{m} \end{aligned}$$

(d) Loss of head at Pond level condition with 0.50m retrogression.

$$\begin{aligned} \text{When weir is discharging with pond level U/S} \\ \text{Head over the crest of weir} &= H = \text{Pond level} - \text{crest level} \\ &= 249.0 - 248.0 \\ &= 1.0\text{m} \end{aligned}$$

$$\begin{aligned} \text{Discharge intensity, } q &= 1.71 H^{3/2} = 1.71 \times (1)^{3/2} \\ &= 1.71\text{m}^3/\text{s}/\text{m} \end{aligned}$$

Due to d/s bed retrogression of 0.50m the water level will be depressed to a value of $246.50 - 0.50 = 246.0\text{m}$ at d/s (d/s water level when Q at pond level U/S)

$$\begin{aligned} \text{Loss of head, } H_L &= \text{Pond level} - \text{d/s water level at that time} \\ H_L &= 249.00 - 246.0 = 3.00\text{m} \end{aligned}$$

HFL & Ponding → 3 conditions: (1) $E_1 = H_1$ → and E_2
 [Bench curve] → (2) $E_1 = H_1$ → and E_2

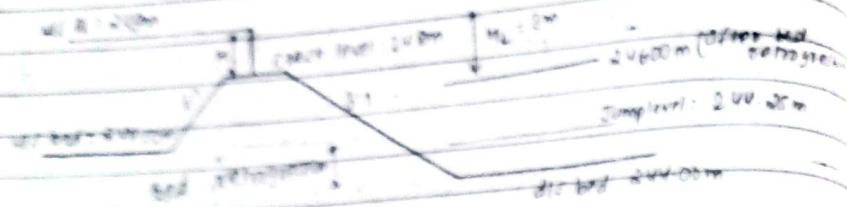


Fig - A condition with bed retrogression of 0.5m
 [the crest is on the slope]

© Hydraulic jump Calculation

Parameters	HFL condition with retrogression	Pond level condition with retrogression
Discharge intensity, q	10	1.71
Head loss	1.63	3.0
dis specific energy E_{f2} from Bench curve	4.3	1.75
Level at which jump will form	$d/s \text{ TEL} - E_{f2}$ $249.62 - 4.3 = 245.32\text{m}$	$246 - 1.75$ $= 244.25\text{m}$
up specific energy E_{f1}	$E_{f2} + H_L$ $= 4.3 + 1.63 = 5.93\text{m}$	$1.75 + 3 = 4.75\text{m}$
Pre jump ht. y_1 $E_{f1} = y_1 + \frac{V_1^2}{2g}$ $[V_1 = 2/y_1]$	1.02m	0.19m
Post jump ht. y_2 $E_{f2} = y_2 + \frac{V_2^2}{2g}$ $[V_2 = 9/y_2]$	3.97 $\approx$ 4m	1.697 $\approx$ 1.70m
Length of concrete floor beyond the jump $= 5(y_2 - y_1)$	$5(4 - 1.02)$ $= 15\text{m (say)}$ [average of two]	$5(1.7 - 0.19)$ $= 7.55\text{m}$

$$\begin{aligned} \text{HFL} \quad E_{f1} &= y_1 + \frac{V_1^2}{2g} \\ E_{f1} &= y_1 + \frac{q^2}{(y_1^3 \times 2g)} \\ 5.93 &= y_1 + \frac{100}{(y_1^3 \times 19.62)} \\ \Rightarrow y_1 &= 3.97\text{m} \end{aligned} \quad \left\{ \begin{aligned} \text{PL} \quad E_{f1} &= y_1 + \frac{V_1^2}{2g} \\ E_{f1} &= y_1 + \frac{q^2}{(y_1^3 \times 2g)} \\ 4.75 &= y_1 + \frac{1.71^2}{(y_1^3 \times 19.62)} \\ y_1 &= 0.19\text{m} \end{aligned} \right.$$

(f) Design of glacis

Let width of crest of weir be 2m, providing the u/s slope as 1:1 and d/s slope of 3:1.
 Provide bed level of d/s side of the level 0.95m below of the jump of $244.25\text{m} = 244.25 - 0.95 = 244.00\text{m}$.
 crest level of weir = 246.00m
 $\therefore$ d/s h2 length of glacis = $3 \times (\text{crest level} - \text{d/s bed level})$
 $= 3 \times (246 - 244)$
 $= 12.00\text{m}$
 $\therefore$ u/s h2 length of slope = $1 \times (\text{crest level} - \text{u/s bed level})$
 $= 1 \times (248 - 244.5)$
 $= 3.5\text{m}$
 d/s h2 floor level beyond the glacis = $\underline{3.5\text{m}}$

(g) Calculation of depth of piles
(based on scour depth) (d/s pile: 1.5~2)

(a) For u/s pile: ($x = 1 \sim 1.5$)

Lacey's formula, $R = 1.35 \left(\frac{q^2}{f} \right)^{1/3} = 1.35 \left(\frac{10^2}{0.90} \right)^{1/3} = 6.49 \text{ m}$

u/s scour below u/s HFL = $xR = 1.5 \times 6.49 = 9.735 \text{ m}$
($\because x = 1 \sim 1.5$ for u/s)

EL of bottom of u/s scour hole/pile = $251.13 - 9.735$
= 241.755 m

depth of u/s pile = u/s bed level - u/s scour level
= $244.50 - 241.755$
= 2.745 m

Hence, provide u/s pile of depth 2.75 m

(b) For d/s pile

$R = 6.49 \text{ m}$, d/s HFL = 249.50 m

Depth of scour below d/s HFL = $xR = 2 \times 6.49$
= 12.98 m

($\because x = 1.5 \sim 2$ for d/s)

Bottom level of d/s pile = $249.50 - 12.98$
= 236.52 m

Depth of d/s pile = d/s bed level - pile bottom level
= $244 - 236.52$
= 7.48 m

Provide d/s pile depth of 8 m

(for ^{design} exit gradient)

Provide the intermediate pile of 8 m at bottom of d/s glacier toe as second line defence with its bottom level to scour depth hole as that of d/s end pile 236.52 m .

(h) calculation of total length of impervious floor

Exit gradient = $1/7$

d/s end pile depth, $d = 8 \text{ m}$

Pond level with no flow at d/s condition is used to calculate seepage head.

Seepage head, $H = \text{u/s pond level} - \text{d/s bed level}$
= $249.0 - 244.0 = 5 \text{ m}$

Now,

$$q_e = \frac{H}{d} \left(\frac{1}{\pi \sqrt{\lambda}} \right)$$

$$\frac{1}{7} = \frac{5}{8} \left(\frac{1}{\pi \sqrt{\lambda}} \right)$$

$$\Rightarrow [\lambda = 1.939]$$

$$\text{But, } \lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} = 1.94$$

$$\Rightarrow \alpha = 2.7$$

$$\alpha = \frac{b}{d} = 2.7$$

$$\Rightarrow [b = 2.7 \times 8 = 21.6 \text{ m}] \leftarrow \text{Total floor length}$$

Floor length already provided from other consideration as = H/L. length of u/s glacier + crest width + H/L. length of d/s glacier + floor length

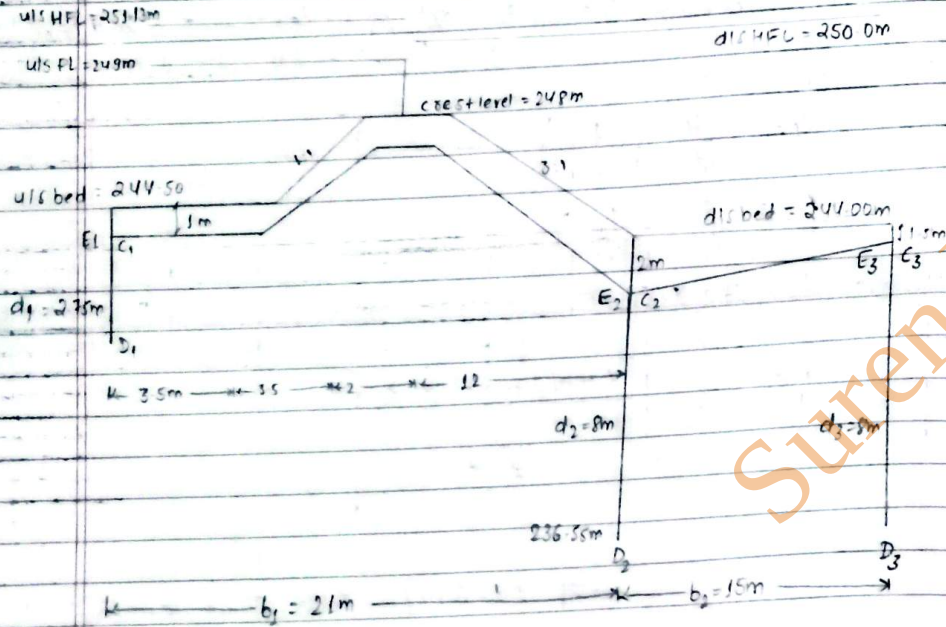
beyond toe of glacier

$$= 3.5 + 2 + 12 + 15 = 32.50 \text{ m which is greater than } 21.60 \text{ m}$$

(If floor length calculated then we would put ult floor length
 $= \alpha d - \text{floor length}$
 $= \alpha d - \text{calculated floor length}$

→ So, no need of providing ult hz floor length because floor length of 32.50m is very very large than that calculated considering Khosla's theory however 3.5m is provided.

∴ Provide, hz impervious floor length
 $= 32.5 + 3.5$
 $= \underline{36m}$



(assumed thickness).

(1) Calculations of floor thickness

This is calculated by using Khosla's Seepage theory. The Percentage pressures at various key points of cutoff is calculated for HFL, PL and no flow on d/s conditions

→ Assuming floor thickness at ult, intermediate and d/s pile line be 1m, 2m and 1.5m respectively.

(a) Calculation of percentage pressures (total floor length = b = 36m)

Pile line & points	Depth of pile (d)	$\alpha = b/d$	b_1	b_1/b	$1/\alpha$	Percentage pressures				
						Before correction	Int. corr ⁿ	Th. corr ⁿ	Slope corr ⁿ	Corrected
ULF pile	2.75	13.09	21		0.08	$E_1 = 100\%$	-			$\phi E_1 = 100\%$
D_1						83%	-			$\phi D_1 = 83\%$
C_1						74%	2.64	3.27		$\phi C_1 = 79.91\%$
Int. Pile	8.0	4.5	21	0.583						
E_2				1-0.583		59%	-0.70	-3.25	2.22	$\phi E_2 = 57.27\%$
D_2				1-0.583		46%	-	-	-	$\phi D_2 = 46\%$
C_2				0.583		31.5%	4.0	3.625	-	$\phi C_2 = 39.12\%$
d/s pile	8.0	4.5	15	-	0.22					
E_3						42%	-4.52	-3.5	-	$\phi E_3 = 33.98\%$
D_3						28%	-	-	-	$\phi D_3 = 28\%$
C_3						0%	-	-	-	$\phi C_3 = 0\%$

↑
Khosla graph

(b) Elevation of HG line for different flow condition

Condition	Hs Static head (m)	Ht/Elevation of HG line above datum									
		Pile 1 U/s end		Pile 2 Intermediate				Pile 3 d/s end			
		$\phi E_1 =$ 100%	$\phi D_1 =$ 83%	$\phi C_1 =$ 79.91%	$\phi E_2 =$ 57.27%	$\phi D_2 =$ 46%	$\phi C_2 =$ 39.125%	$\phi E_3 =$ 33.98%	$\phi D_3 =$ 28%	$\phi C_3 =$ 0%	
Pond level	249-246 " 3m	Ht.	3	249	240	1.72	1.38	1.17	1.02	0.84	0.00
flow		Elev.	249	248.49	248.40	247.72	247.38	247.17	247.02	246.84	246.00
HFL	251.13- 249.50 = 1.63m	Ht.	1.63	1.35	1.30	0.93	0.75	0.64	0.55	0.46	0.00
flow		Elev.	251.13	250.85	250.80	250.43	250.25	250.14	250.05	249.96	249.50
Pond level with no flow d/s	249-244 " 5m	Ht.	5	4.15	4	2.86	2.30	1.96	1.7	1.40	0.00
		Elev.	249	248.15	248	246.86	246.30	245.96	245.70	245.40	244.00

(c) Prejump profile: RL of crest = 248.00m, d/w slope 3:1 (H:V)

Distance from d/w end of crest	RL of glacis	HFL condition $q = 10 \text{ cumec}$ U/s pond level = 251.25m $E_{f1} = 251.25 - \text{glacis level}$	Pond level flow condition $q = 3.71 \text{ cumecs}$ U/s pond level = 249.00m $E_{f1} = 249.00 - \text{glacis level}$		
3.0	247.00	4.25	1.32	2.0	0.30
6.0	246.00	5.25	1.11	3.0	0.23
8.04 (Jump location for high flood)	245.32	5.93	1.02	-	-
9.0	245.00	6.25	-	4.0	0.20
11.25 (Jump location for pond level)	244.25	-	-	4.75	0.19

The hydraulic jump is plotted for HFL condition and PL condition from where static head H_s is calculated.

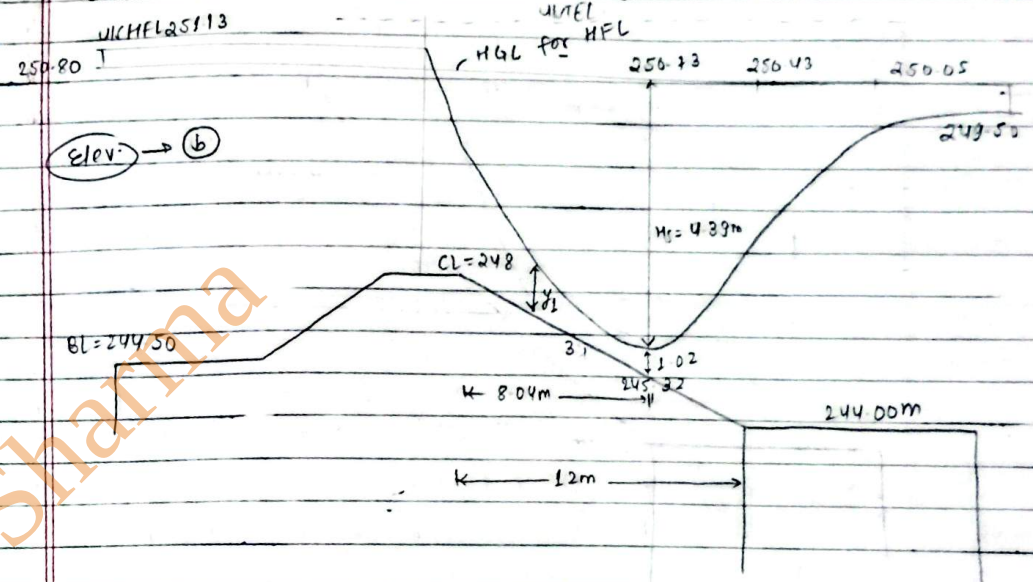


Fig. Plotting of Hydraulic jump for HFL flow.

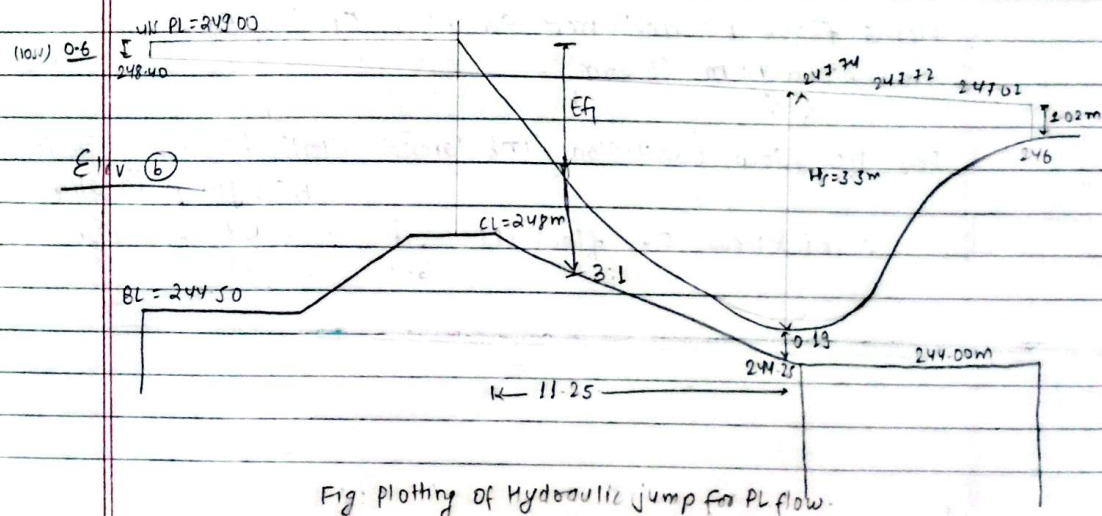


Fig. Plotting of Hydraulic jump for PL flow.

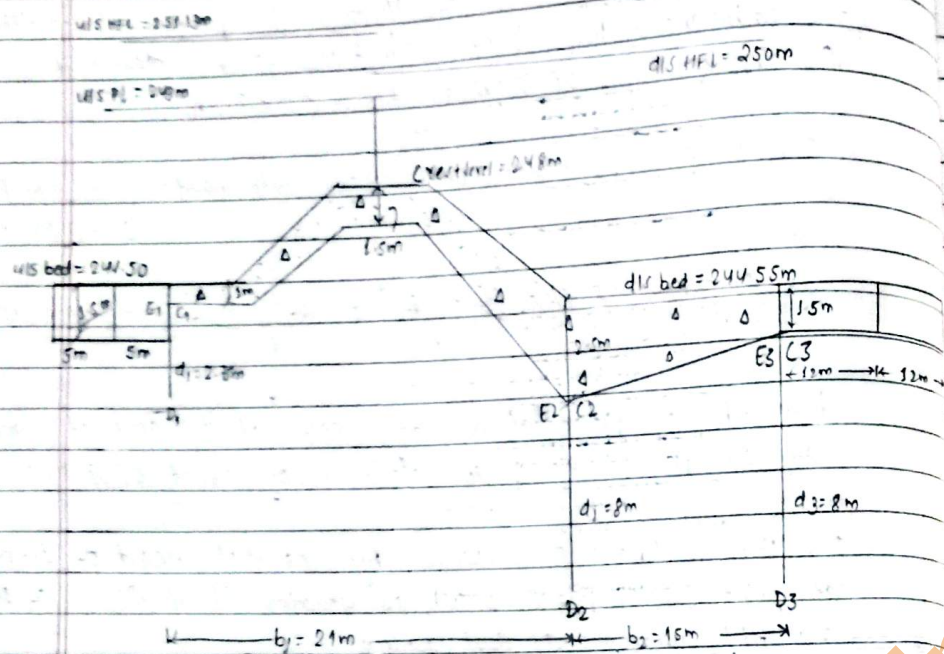


Fig. Showing thickness of concrete bed of sloping glacis weir.

(g) Protection works

(1) Block protection with inverted filter

(i) u/s scour hole below u/s floor $= d_1 = D = \text{scour depth} = 2.75m$
 $(xR-y)$

hence, provide 1m thick stone blocks laid over 0.50m thick boulder soling of $1.5 \times D = 1.5 \times 2.75 = 4.125 \approx 5m$ length

(ii) For d/s scour hole $= d_2 = 8m = D$

length of block protection $= 1.5D = 1.5 \times 8 = 12m$

hence, provide 1m thick stone blocks of 12m laid over 0.50m thick inverted filter of graded aggregated boulder soling.

(2) Launching apron:

(i) length of u/s launching apron $(L) = 1.5d_1^{(xR-y)}$ as it is $\approx D$
 $= 1.5 \times 2.75$
 $= 4.125m$

hence, provide launching apron of length, $l = 5m$ and thickness of $(2.25 \times 2.75)/L = \frac{2.25 \times 2.75}{5} = 1.23m \approx 1.5m$ say

(ii) length of d/s launching apron $= 1.5 \times d_2^{(xR-y)}$
 $= 1.5 \times 8 = 12m$
 Hence, provide launching apron of length 12m and thickness of 1.5m

(Q2) From the following data provided fix the suitable value of the waterways and crest level of weir, under-sluice and canal head regulator. Assume suitable any other data if necessary.

River bed level = 200.00m

Lacey's silt factor = $f = 1$

Discharge of river $Q_{\text{river}} = 16,000 \text{ m}^3/\text{s}$

Discharge of canal, $Q_{\text{canal}} = 200 \text{ m}^3/\text{s}$

FSL of canal at its head = 203.00m

HFL before construction = 205.70m

→ Soln: (Refer steps of design of weir/barrage)

(1) Calculation of crest level

(a) Crest level of the under sluice is provided at the bed level of the river = 200.00m.

~ (b) Crest level of weir is provided 1.0 to 1.5m higher than the crest level of under sluice. So, let us provide crest level of weir at 1.20m higher than under sluice.
crest = $200.00 + 1.20 = 201.20 \text{ m}$.

~ (c) Crest level of head regulator is provided at 1.2 to 1.5m higher than that of under sluice if silt excluder is not provided. So, providing crest level of head regulator at = $200.00 + 1.50 = 201.50 \text{ m}$.

(2) calculations of water ways for weir and under sluice.

$$\begin{aligned} \text{Darcy's wetted perimeter (P)} &= 4.75 \sqrt{Q_{\text{river}}} \\ &= 4.75 \sqrt{16000} \\ &= 600.83 \\ &= 601\text{m.} \end{aligned}$$

Let's provide a water way approx equal to $1.2P$ by total in such a way that approximately 15 to 20% of discharge is passed through under sluice and total provided waterway should pass the entire discharge. ($1.2 \times 601 \approx 721.2\text{m}$)

Assuming water way as below,

(a) Under sluice portion:

$$9 \text{ bays of } 15\text{m each} = 9 \times 15 = 135\text{m}$$

$$8 \text{ piers of } 2.0\text{m each} = 2 \times P = 16\text{m}$$

$$\text{Overall waterway} = 151\text{m}$$

(b) Other barrage bays portion (weir proper)

$$45 \text{ bays of } 12\text{m each} = 540\text{m}$$

$$43 \text{ piers of } 2\text{m each} = 43 \times 2 = 86\text{m}$$

$$\text{Overall waterway} = 540 + 86 = 626\text{m}$$

Assume divide wall of 3m width; hence total waterway provided b/w the abutments (L) = $151 + 626 + 3$
 $= 780\text{m.}$ (should be within $1.2P \sim 1.4P$)

⇒ Let us assume the afflux of 1m.

So, checking the flood to be passed through the waterway with 1m afflux.

$$\text{HFL before construction} = 205.70\text{m}$$

$$\text{U/S HFL} = 205.70 + 1.0 = 206.70\text{m}$$

$$\text{d/s HFL} = \text{HFL before construction} = 205.70\text{m}$$

$$\begin{aligned} \text{Avg. discharge intensity, } q &= \frac{Q_{\text{river}}}{L} = \frac{16000}{780} \\ &= 20.51 \text{ m}^3/\text{s/m} \end{aligned}$$

$$\begin{aligned} \text{Scour depth, } R &= 1.35 \left(\frac{q^2}{f} \right)^{1/3} \\ &= 1.35 \times \left[\frac{20.51^2}{1} \right]^{1/3} \\ &= 10.12\text{m} \end{aligned}$$

$$\text{Velocity of approach (V)} = \frac{q}{R} = \frac{20.51}{10.12} = 2.03 \text{ m/s}$$

$$\text{Velocity head} = \frac{V^2}{2g} = \frac{2.03^2}{2 \times 9.81} = 0.216\text{m} \approx 0.21\text{m}$$

$$\text{U/S TEL} = \text{U/S HFL} + \text{vel. head} = 206.7 + 0.21 = 206.91\text{m}$$

(15-10)
 (A) discharge through under sluice (Gen. 20% of full discharge should be passed)

Head over under-sluice crest (H) = U/S TEL - crest level of under-sluice

$$= 206.91 - 200.00 \\ = 6.91 \text{ m}$$

Discharge passing through under sluice (Q_1)
 $= 1.71 (L - K n H) H^{3/2}$

K = Coeff of end contraction = 0.10

n = 2 * no. of under-sluice bays = 2 * 9 = 18 m

L = Clear waterway length = 135 m

$$Q_1 = 1.71 (135 - 0.1 \times 18 \times 6.91) (6.91)^{3/2} \\ Q_1 = 3786.62 \text{ m}^3/\text{s}$$

(B) Discharge through weir portion

Provide width of weir be 2 m. The head over weir portion = 206.91 - 201.20
 $[H = 5.71 \text{ m}]$

$H \gg 1.5 \times \text{crest width} = 1.5 \times 2 = 3 \text{ m}$

Thus, weir behave as sharp crested

Discharge through barrage/weir bays; (Q_2)

$$Q_2 = 1.84 [L - K n H] H^{3/2}$$

Here, $L = 540 \text{ m}$ (weir)

$K = 0.10$

$n = 2 \times 44 = 88$ (45 del 2405 8821 0311 2405 m divide wall 576 24051 44m ladda 576 50, 2405 44) excluding 44

$H = 5.71 \text{ m}$

$$Q_2 = 1.84 (540 - 0.10 \times 88 \times 5.71) \times 5.71^{3/2} \\ = 12295.56 \text{ m}^3/\text{s}$$

$$\therefore Q_1 + Q_2 = 3786.62 + 12295.56 \\ = 16082.17 \text{ m}^3/\text{s} > 16000 \text{ m}^3/\text{s}$$

okay.

The provided water way for under sluice and weir portion is okay.

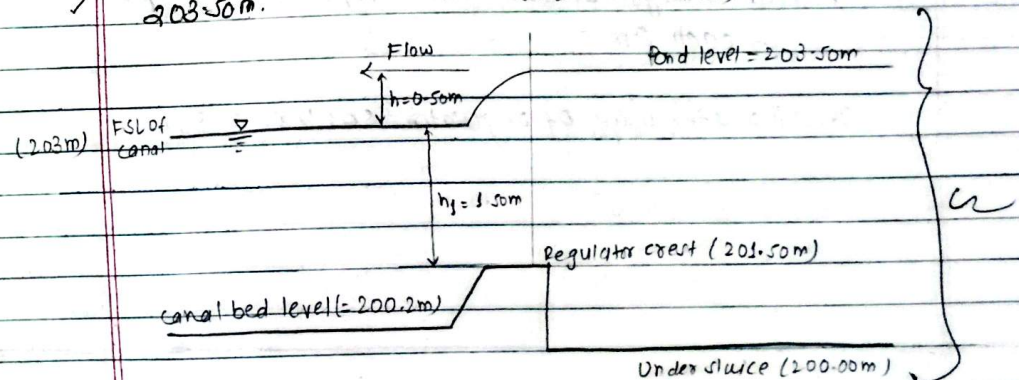
(If $Q_1 + Q_2 < 16000 \text{ m}^3/\text{s}$ then waterway must be redesigned)

Anyhow, $Q_1 + Q_2$ must be $> 16000 \text{ m}^3/\text{s}$

(3) Calculations of waterways for head regulator

(Gen. canal bed level > river bed level)

Let us provide bed level of canal at 200.20 m slightly higher than under-sluice crest level and pond level be at 203.50 m. (PL above FSL)



Discharge through regulator is given by:

$$Q_{\text{canal}} = \frac{2}{3} C_{d1} \sqrt{2g} B (h+h_v)^{3/2} + C_{d2} B h_1 \sqrt{2g} (h+h_v)$$

$$Q_{\text{canal}} = 1.71 B h^{3/2} + 3.54 B h_1 h^{1/2}$$

[B → length of head regulator]

Neglecting velocity of approach, $h_v = 0$,

$$C_{d1} = 0.577, C_{d2} = 0.80$$

Then,

$$200 = \frac{2}{3} \times 0.577 \times \sqrt{2g} B \times (0.50)^{3/2}$$

$$+ 0.80 \times 1.50 \times B \times \sqrt{2g} \times 0.50$$

$$\text{or, } 200 = 0.608 + 3.76B$$

$$\Rightarrow [B = 45.87 \text{ m} \approx 50 \text{ m (say)}]$$

where, B = length of clear water way of regulator

Provide 5 bays of 10m each separated by 4 piers of each 2m.

$$\text{Overall water way of regulator} = 50 + 4 \times 2 = 58 \text{ m}$$

(93) Design a suitable head regulator at off taking point of canal with the following data provided.

FSL discharge of canal = 180 m³/s;

Silt factor, $f = 1$

Anticipated max^m FSL of canal = 260.20m

Safe exit gradient for canal bed material = 1/5

The pond level of flow = 260.60m

Approach velocity of river = 1.9 m/s

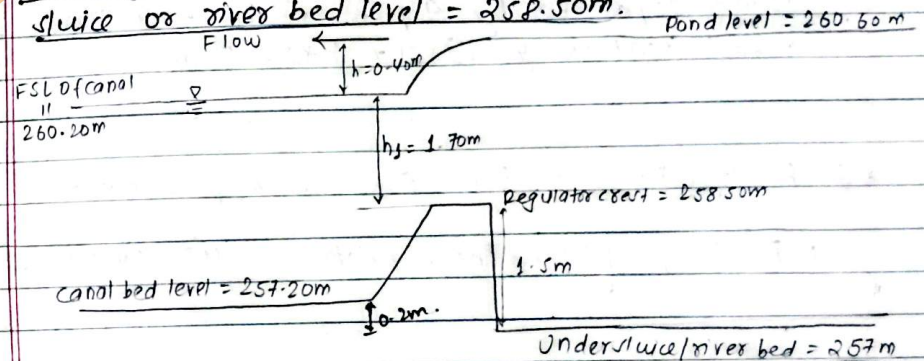
The HFL before construction of barrage = 262.20m;

Afflux = 1m

Bed level of the river = 257.00m

→ Solution:-

Providing crest level of head regulator 1.5m above under sluice or river bed level = 258.50m.



(a) Fixing waterway for the regulator to pass FSD of the canal.

Discharge through regulator is given by,

$$Q_{\text{canal}} = \frac{2}{3} C_d \sqrt{2g} B (h+h_r)^{3/2} + C_d B h_r \sqrt{2g(h+h_r)}$$

Neglecting velocity of approach, then $h_r = 0$

$$Q_{\text{canal}} = \frac{2}{3} C_d \sqrt{2g} B (h)^{3/2} + C_d B h_r \sqrt{2gh}$$

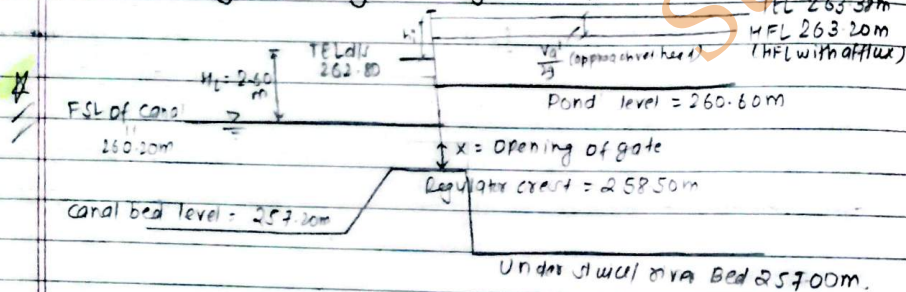
$$180 = \frac{2}{3} \times 0.577 \sqrt{2g} B (0.40)^{3/2} + 0.80 \times B \times 1.70 \sqrt{2g \times 0.40}$$

$$[\therefore B = 42.50\text{m}]$$

Provide 5 bays of 9m each giving clear waterway of 45m separated by 4 piers of 2m each. Then, overall waterway = $(9 \times 5) + 4 \times 2 = 53\text{m}$

(b) Hydraulic calculations for various flow conditions

(i) Discharge passing the regulator for HFL condition.



Diff from well

Discharge passing the gate of regulator of opening x meter is calculated using submerged orifice formula

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

(HFL - diff)

(clear water way)

$$h = \frac{263.20 - 260.20}{1} = 3.0\text{m}$$

$$180 = 0.62 \times (45 \times x) \sqrt{2 \times 9.81 \times 3}$$

$$\Rightarrow [x = 0.84\text{m}]$$

$$\text{velocity of flow through gate, } V = \frac{Q}{A} = \frac{180}{(45 \times 0.84)} = 4.76\text{m/s}$$

different from well

$$\text{Loss of head at entry} = \frac{0.50 V^2}{2g} = \frac{0.50 \times 4.76^2}{2 \times 9.81} = 0.58\text{m}$$

$$\text{TEL at just d/s of gate} = 263.38 - 0.58 = 262.80\text{m}$$

$$\text{Head loss} = 262.80 - 260.20 = 2.60\text{m}$$

(gate at entry side entry loss ke jaha TEL bade)

$$\text{Discharge intensity, } q = \frac{Q}{B} = \frac{180}{45} = 4\text{m}^3/\text{s/m}$$

(discharge per length of gate) $\hookrightarrow$ पानी कितने खण्डको मात्र length

(ii) Discharge passing the regulator for PL condition.

$$\text{head loss } H_L = PL - FSL = 260.60\text{m} - 260.20\text{m} = 0.40\text{m}$$

$$\text{discharge intensity, } q = \frac{Q}{B} = \frac{180}{45} = 4\text{m}^3/\text{s/m}$$

(c) Hydraulic jump calculations for two flow conditions

Particulars	HFL condition	PL condition
Discharge intensity, q	4.0	4.0
Head loss, H_L	2.60m	0.00m
d/s sp. energy E_{f2} from Mach curve	2.7	2.05
U/s sp. energy $E_{f1} = E_{f2} + H_L$	5.31	2.45
Level at which jump will form = d/s FSL - E_{f2}	$260.20 - 2.7 = 257.50m$ (d/s FSL - E_{f2} = lower level)	$260.20 - 2.05 = 258.15m$
y_1 corresponding to E_{f1}	0.4	0.68
y_2 corresponding to E_{f2}	2.57	1.80
length of concrete floor required: $5(y_2 - y_1)$	<u>10.85m</u>	5.60m

Provide d/s floor at 257.20m with a horizontal length of 12m

(d) Calculations of depth of sheet piles from scour hole consideration

$$q = 4 \text{ m}^3/\text{s/m}$$

$$\text{scour depth, } R = 1.35 \left(\frac{q^2}{f} \right)^{1/3} = 1.35 \left(\frac{4^2}{1} \right)^{1/3} = 3.40m$$

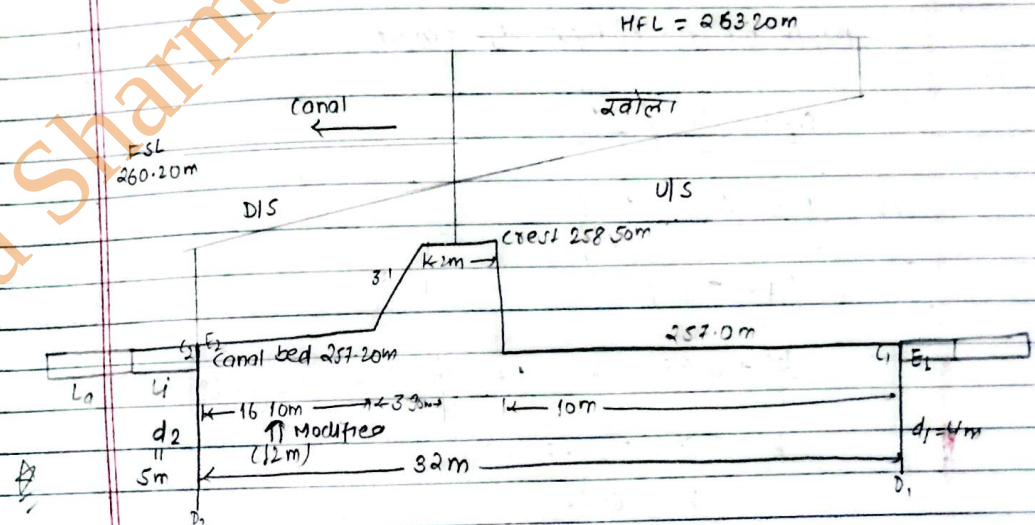
Let's provide d/s cutoff up to $2R$ from d/s FSL of canal of 260.20m

$$RL \text{ of bottom of d/s cutoff} = 260.20 - 2 \times 3.40 = 253.40m$$

Depth of scour d/s below floor (D) =

$$\begin{aligned} RL \text{ of canal bed} - RL \text{ of bottom of cutoff} \\ = 257.20 - 253.40 \\ = 3.80m \end{aligned}$$

Hence, provide d/s pile of depth, $d_s = 5.0m$



For d/s pile taking the level of pile bottom from HFL be 1.5R.

$$RL \text{ of bottom of d/s scour} = 263.20 - 1.5 \times 3.40 = 258.10m > 257.00m$$

(Hence pile is safe from scour)

So, provide d/s pile of 80% of d/s pile; $\therefore d_1 = 0.8 \times 5 = 4m$ (say)

(e) calculation of total floor length.

The maximum static head will occur on HFL condition when there is no water in the canal during the closure of regulate gates.

$$\begin{aligned} \text{So, } H_s &= \text{HFL} - \text{BL of canal} \\ &= 263.20 - 257.20 \\ &= 6\text{m} \\ &= H \end{aligned}$$

Depth of dir cutoff $d_2 = 5\text{m} = d$

$$G_E = \frac{H}{d} \left(\frac{1}{\pi \sqrt{\lambda}} \right)$$

$$\Rightarrow \frac{1}{5} = \frac{6}{5} \left(\frac{1}{\pi \sqrt{\lambda}} \right)$$

$$\Rightarrow [\lambda = 3.65]$$

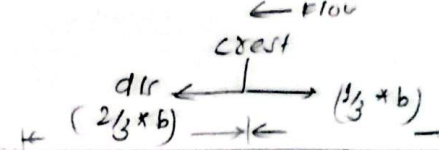
$$\frac{b}{d} = 3.65 \quad \lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} = 3.65 \Rightarrow [\alpha = 6.22]$$

$$\Rightarrow b = 3.65 \times 5 = 18.25\text{m}$$

$$\frac{b}{d} = 6.22$$

$$\Rightarrow [b = 6.22 \times 5 = 31.1\text{m}]$$

Hence, provide a total floor length of 32m



Floor is adjusted as below,

dir horizontal length = 12m

dir glacis length with 3:1 slope = $3 \times (258.50 - 257.20)$
= 3.90m

crest width = 2m

total length of dir floor is 17.90m < $2/3$ of total floor
= 21.33m
 $\approx 22\text{m}$

So, provide dir floor of 22m increasing hz. floor beyond glacis to 16.10m from 12m

Now, ukr hz. floor or floor balance ukr = $(32 - 22) = 10\text{m}$

(f) Calculation of thickness of floor

The floor thickness is calculated by using Khosla theory by calculating % prestress at key points of ukr and dir pile lines with their correction and HG line is plotted accordingly for HFL, thickness is determined by using formula; $t = \frac{h}{(G-1)}$

(Do yourself as in weir for HFL condition ukr and no flow in canal dir only)

(g) Protection works

(a) Length of launching apron $dls (L_a) = 1.5 \times D$
 $= 1.5 \times 3.8$
 $= 5.7m$

Provide 7m length of apron
thickness of apron $= \frac{2.25D}{L_a} = \frac{2.25 \times 3.8}{7} = 1.22m$

Hence, provide thickness of 1.5m.

(b) D/s Inverted filter of gravel/aggregate of thickness of 0.50m over which block protection of thickness 1m is provided for a length of 7m

(a) Length of launching apron $uls (L_a) = 1.5 \times d_s = 1.5 \times 4$
 $= 6m$

(b) u/s inverted filter of gravel/aggregate of thickness of 0.50m over which block protection of thickness 1m is provided for a length of 6m.

(Q3) From the following data design the profile of undersluice section

Discharge of river = $3000 \text{ m}^3/\text{s}$

Max^m winter flood discharge = $300 \text{ m}^3/\text{s}$

RL of river bed = 249.50 m

HFL before construction of weir = 255.50 m (d/s HFL)

Pond level = 254.50 m

$f = 0.90$; Canal discharge = $200 \text{ m}^3/\text{s}$

Afflux = 1 m ; Looseness factor = 1.10

d/s bed retrogression = 0.50 m

Concentration factor = 1.20 i.e. 20%

Permissible exit gradient = $1/7$

Flood stage discharge curve is given.

PL condition on
ULS side of flow
d/s side of flow
discharge ULS

→ Solution:-

$$\Rightarrow \text{Lacey's approach velocity, } V = \left(\frac{Qf^2}{140} \right)^{1/6}$$

$$= \left(\frac{3000 \times 0.90^2}{140} \right)^{1/6}$$

$$= 1.61 \text{ m/s}$$

$$\text{Wetted perimeter} = P = 4.75\sqrt{Q}$$

$$= 4.75\sqrt{3000}$$

$$= 260.17 \text{ m}$$

(L) Looseness factor = 1.10

$$\text{Hence, actual width of water way} = 1.10 \times 260.17$$

$$= 286.19 \text{ m}$$

(L) 20% discharge - undersluice
80% discharge - weir

(a) Calculation of water way for undersluice & weir.

Let us provide 2 bays of 10 m each separated by 1 piece of 2.5 m for the undersluice portion.

$$L_{\text{sluice}} = 2 \times 10 = 20 \text{ m clear water way.}$$

And 18 bays of 15 m each for weir proper separated by 17 pieces of 2 m each.

$$L_{\text{weir}} = 18 \times 15 = 270 \text{ m clear water way.}$$

$$\text{Providing Total waterway} = 20 + 270 = 290 \text{ m} > 286.19 \text{ m}$$

(b) Calculation of discharge passing through the weir and undersluice & section.

→ Discharge passing from undersluice is maximum of three below:

$$(i) \text{ 2 times canal discharge} = 2 \times 200 = 400 \text{ m}^3/\text{s}$$

$$(ii) 20\% \text{ of flood discharge} = 0.20 \times 3000 = 600 \text{ m}^3/\text{s}$$

$$(iii) \text{ Max}^m \text{ winter flood discharge} = 300 \text{ m}^3/\text{s}$$

Hence, provide discharge for undersluice = $Q_{\text{sluice}} = 600 \text{ m}^3/\text{s}$

d/s HFL = 255.50 m

$$\text{velocity head} = \frac{V^2}{2g} = \frac{(1.61)^2}{2 \times 9.81} = 0.13 \text{ m}$$

$$\text{d/s TEL} = 255.50 + 0.13 = 255.63 \text{ m}$$

afflux = 1 m

$$\text{ULS HFL} = \text{d/s HFL} + \text{afflux} = 255.50 + 1.0 = 256.50 \text{ m}$$

[ULS d/s
TEL d/s HFL]

(Borrow of $\frac{1}{2}$ m $\rightarrow$ U/S head afflux)

$$U/S TEL = 256.50 + 0.13 = 256.63m$$

- $\rightarrow$ Provide crest level of undersluice at bed level of river = 249.50m
- $\rightarrow$ Head over undersluice crest, $h_{sluice} = U/S TEL - \text{crest level}$
 $= 256.63 - 249.50$
 $= 7.13m$

(i) Check for discharge passing undersluice bays

$$Q_{sluice} = 1.71 (L - knH) H^{3/2}$$

$$= 1.71 (20 - 0.1 \times (2 \times 1) \times 7.13) \times 7.13^{3/2}$$

$$= 604.00 m^3/s > 600 m^3/s$$

(If $Q_{sluice} < 600$,
 No of bays $\frac{600}{Q_{sluice}}$ ok)

(ii) Check for discharge passing weir bays

$$Q_{weir} = 3000 - 604 = 2396 m^3/s$$

$$\text{Discharge intensity of weir} = \frac{Q}{L} = \frac{2396}{270} = 8.87 m^3/s/m$$

$$q = 1.84 H^{3/2}$$

Thus,

$$\text{Head over weir crest, } H = \left[\frac{8.87}{1.84} \right]^{2/3} = 2.85m$$

$$\text{Crest level of weir} = U/S TEL - H$$

$$= 256.63 - 2.85$$

$$= 253.77m$$

Provide crest of weir at 253.70m

Now,

$$\text{Head over crest, } H = 256.63 - 253.70 = 2.93m$$

$$\rightarrow Q_{weir} = 1.84 [270 - 0.1 \times (2 \times 17) \times 2.93] \times 2.93^{3/2}$$

$$= 2399.69 \approx 2400 m^3/s \text{ say}$$

$\therefore$ total discharge passing sluice and weir proper

$$= Q_{sluice} + Q_{weir}$$

$$= 604 + 2400$$

$$= 3004 m^3/s > 3000 m^3/s \text{ Okay}$$

Hence, provided water way for weir and undersluice is Okay.

- (c) ~~Calculate~~ Design of undersluice - conditions
- HFL condition without discharge concentration and retrogression of d/s bed of river.

$$\text{Approach velocity, } V = 1.61 m/s$$

$$\text{Velocity head, } = 0.13m$$

$$D/s HFL = 255.50m$$

$$d/s TEL = 255.50 + 0.13 = 255.63m$$

$$U/S TEL = d/s TEL + \text{afflux} = 256.63m$$

$$\text{Head above crest, } H = 256.63 - 249.50 = 7.13m$$

$$U/S \text{ discharge intensity, } q = \frac{1.71 \times H^{3/2}}{7.13} = 32.56 m^3/s/m$$

(blw pier)

$$\text{Head loss, } H_L = 256.63 - 255.53 = 1m$$

(ii) HFL condition with discharge concentration of 20% and retrogression of d/s bed of river of 0.50m

$$Q_{\text{new}} = 1.2Q = 1.2 \times 32.56 = 39.07 \text{ m}^3/\text{s/m}$$

$$\text{d/s HFL due to } 0.50 \text{ m retrogression of bed} = 255.50 - 0.50 = 255.0 \text{ m}$$

$$\text{d/s TEL} = 255 + 0.13 = 255.13 \text{ m}$$

$$\text{head over crest of weir} = H = \left(\frac{39.07}{1.71} \right)^{2/3} = 8.05 \text{ m}$$

$$\text{u/s TEL} = 249.50 + 8.05 = 257.55 \text{ m}$$

$$\text{Head loss } H_L = \underset{\text{u/s TEL}}{257.55} - \underset{\text{After bed retrogression}}{255.13} = 2.42 \text{ m}$$

(iii) Pond level condition w/o discharge concentration of 20% and retrogression of d/s bed of river of 0.50m

$$\text{u/s pond level} = 254.50 \text{ m}$$

$$\text{head over crest of undersluice} = 254.50 - 249.50 = 5 \text{ m}$$

$$\begin{aligned} \text{head over barrage/weir during pond level} \\ = 254.50 - 253.70 \\ = 0.80 \text{ m} \end{aligned}$$

⇒ Total discharge passing through sluice and weir.

$$Q = 1.71 (20 - 0.10 \times (2 \times 1) \times 5)^{3/2} + 1.84 (270 - 0.10 \times (2 \times 17) \times 0.80)^{3/2} = 715.15 \text{ m}^3/\text{s}$$

↓
Find d/s wl from stage-discharge curve

⇒ For discharge of 715.15 m³/s the level of water at d/s side = 251.75 m from stage discharge curve.

$$\text{Approach velocity, } V = \left(\frac{Qf^2}{140} \right)^{1/5} = \left(\frac{715.15 \times 0.90^2}{140} \right)^{1/5} = 1.27 \text{ m/s}$$

$$\text{velocity head} = \frac{1.27^2}{2 \times 9.81} = 0.08 \text{ m}$$

$$\text{d/s TEL} = 251.75 + 0.08 = 251.83 \text{ m}$$

$$\text{u/s TEL} = \text{PL} + \text{vel. head} = 254.50 + 0.08 = 254.58 \text{ m}$$

$$\text{head over crest} = H = \underset{\text{u/s TEL}}{254.58} - 249.50 = 5.08 \text{ m}$$

$$Q = 1.71 \times (5.08)^{3/2} = 19.58 \text{ m}^3/\text{s/m}$$

$$\therefore \text{Head loss } H_L = 254.58 - 251.83 = 2.75 \text{ m}$$

(iv) Pond level condition with discharge concentration and retrogression of d/s bed of river

$$Q_{\text{new}} = 1.20 \times 19.58 = 23.50 \text{ m}^3/\text{s/m}$$

$$\text{head over crest of sluice, } H = \left[\frac{23.50}{1.71} \right]^{2/3} = 5.74 \text{ m}$$

$$\text{u/s TEL} = 249.50 + 5.74 = 255.24 \text{ m}$$

d/s water level is depressed by 0.50m

$$\text{d/s water level} = 251.75 - 0.50 = 251.25 \text{ m}$$

$$\text{d/s TEL} = 251.25 + 0.08 = 251.33 \text{ m}$$

$$\text{head loss } H_L = 255.24 - 251.33 = 3.91 \text{ m}$$

(c) Calculation of Hydraulic jump parameters

Items	HFL condition		Pond Level condition	
	without retrogression and concentration	with retrogression and concentration	without retrogression and concentration	with retrogression and concentration
Discharge intensity, q	32.56	39.07	19.58	23.50
Head loss, H_L	1.0	2.42	2.75	3.91
dis TEL	255.63	255.13	251.83	251.33
Sp. Energy E_f (from Blench curve)	8.25	9.70	6.75	7.80
Sp. Energy $E_{f1} = E_{f2} + H_{L1}$	9.25	12.12	9.50	11.71
Level at which jump will form	247.38	245.43	245.08	243.53 (min)
Pre-jump ht, y_1 $E_{f1} = y_1 + \frac{q^2}{2gy_1^3}$	2.8	3.0	1.60	1.70
Post-jump ht, y_2 $E_{f2} = y_2 + \frac{q^2}{2gy_2^3}$	7.6	8.6	6.25	7.27
Length of jump ($y_2 - y_1$)	4.8	5.6	4.65	5.57
dis concrete floor beyond the toe of glacis $5(y_2 - y_1)$	24.0	28.0	23.25	28.50 (longest)

Hence, provide d/r bed level (floor) at 243.40m and d/r hz. concrete floor beyond the glacis portion of 30m.

(d) Calculation of depth of sheet pile/cutoff

discharge intensity of under sluice, $q = \frac{604}{20} = 30.20 \text{ m}^3/\text{s/m}$

$$\text{Lacey's hydraulic radius, } R = 1.35 \left(\frac{q^2}{f} \right)^{1/3}$$

$$= 1.35 \times \left(\frac{30.2^2}{0.90} \right)^{1/3}$$

$$= 13.56 \text{ m}$$

Level of dis scour hole below d/r HFL $1.5R$

$$= \text{d/r HFL} - 1.5R$$

$$= \text{d/r HFL} - 1.5R$$

$$= 255 - 1.5 \times 13.56$$

$$= 234.66 \text{ m}$$

$$\approx 234.60 \text{ m}$$

Depth of d/r pile, $d_2 = \text{d/r floor level} - \text{bottom level of pile}$

$$= 243.40 - 234.60$$

$$= 8.80 \text{ m}$$

Hence, provide d/r sheet pile of depth 9m.

Level of uls scour hole below uls HFL
 $= 256.18 - 1.25 \times 13.56$
 $= 239.23m$ $\rightarrow$ uls = 1.25
 $\approx 239.00m$ (say)

Now, depth of uls pile = $249.50 - 239 = 10.50m$

(e) Calculation of total floor length. [uls PL & d/s no flow condition]

permissible exit gradient = $\frac{1}{7}$

depth of d/s pile = 9m

Static head for no flow condition in d/s side,

$H_s = PL - \text{d/s bed level}$
 $= 254.50 - 243.40$
 $= 11.10m$

$G_F = \frac{H_s}{d} \left(\frac{1}{\pi \sqrt{\lambda}} \right) = \frac{11.1}{9} \left(\frac{1}{\pi \sqrt{\lambda}} \right) = \frac{1}{7}$

$\Rightarrow \lambda = 7.55$

$\lambda = \frac{1 + \sqrt{1 + q^2}}{2} = 7.55$

$\Rightarrow \alpha = 14.06$

$\alpha = \frac{b}{d} = \frac{b}{9} = 14.06$

$\Rightarrow [b = 126.48m]$

Provide bed length of total 127m.

Crest level of sluice = 249.50m

Provide slope of discharge 8:1.

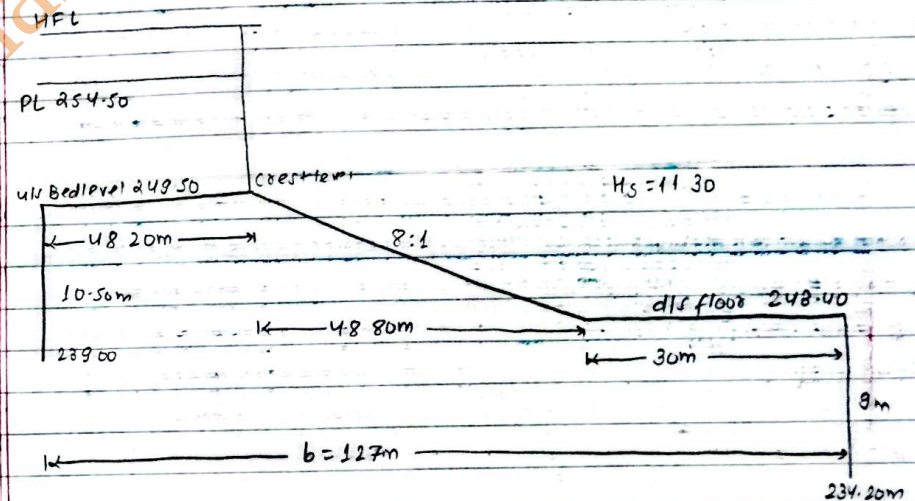
Then, horizontal length of glacis floor =

$8 \times (249.50 - 243.40)$

$= 8 \times 6.3$

$= 48.80m$

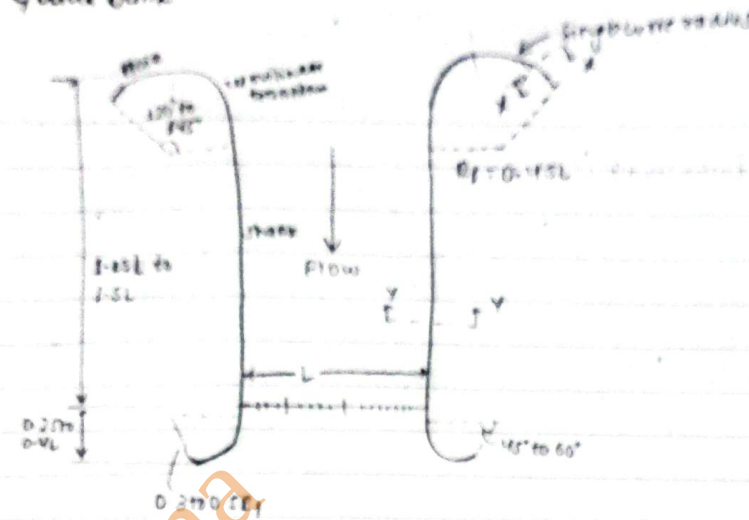
Hence a balance of $(127 - 30 - 48.80) = 48.20m$ is provided for uls floor length as shown in figure.



[Thickener - kharla] } as per spec.
 [Bed protection...]

#Chapter 6: River Training WORKS (RTW)

Guide bank



(a) Straight guide bank (Plan)

• Design of guide bank

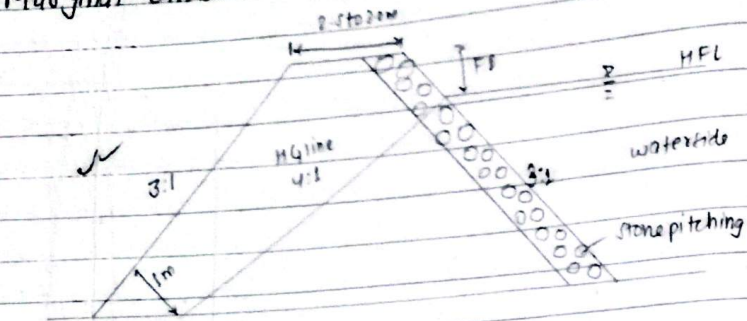
Largest wetted perimeter (P) = $4.75\sqrt{Q}$

Ht. distance b/w two guide banks (L) = $1.2P$

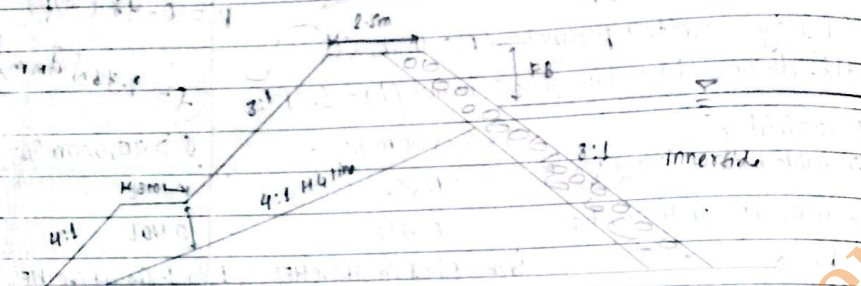
Hydraulic mean radius $R = 0.48 (Q/f)^{1/3}$
 $(f = 1.76\sqrt{d_{mm}})$

Particulars	$Q \leq 20,000 \text{ m}^3/\text{s}$	$Q > 20,000 \text{ m}^3/\text{s}$
1. Len. of guide bank uls side of hydrau structure	$1.25L$	$1.5L$
2. Len. of bank dlr of H-structure	$0.25L$	$0.40L$
3. Free board	Gen: 1 to 1.5m above HFL	1 to 1.5m above HFL
4. top width of Guide bank	$\geq 3\text{m}$	$\geq 3\text{m}$
5. Side slope	$2:1 (H:V)$	$2:1 (H:V)$
6. Radius of uls curve head (R)	$0.45L$	$0.45L$
7. Radius of dlr curve head (r)	$0.225L = R/2$	$0.225L = R/2$
8. Angle of sweep uls curve head	$120^\circ \text{ to } 145^\circ$	$120^\circ \text{ to } 145^\circ$
9. Angle of sweep dlr curve head	$45^\circ \text{ to } 60^\circ$	$45^\circ \text{ to } 60^\circ$
10. Stone pitching thickness (t)	$0.06 Q^{1/3}$	$0.06 Q^{1/3}$
11. Width of launching (WLa) apron bank	$= 1.5D, D = XR - Y$ $(X = 2.25 (\text{nose}); 1.5 (\text{curvilinear trans}); 1.25 (\text{stone}))$	
12. Thickness of apron	$T = 1.9t, t = \text{thickness of stone pitching}$	

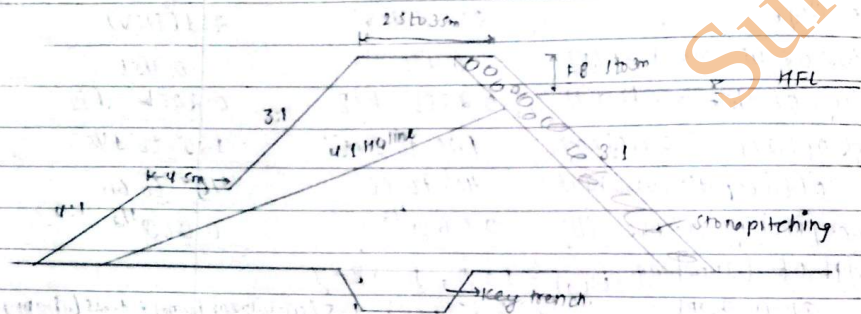
Marginal Embankment/Levees



For ht. below 2.5m



For ht. 2.5 to 3.25m



For ht. above 3.25m

Design of Spurs:- (similar to guide banks)

1. Length of spur, $L = 1.5$ to 2 times depth of flow
& usually restricted to 20% of width of river.

2. The embedded length of spur into the bank (l_e) = $\frac{1}{4}$ th to $\frac{1}{3}$ rd of its total length.

3. Inclination of spur i.e. angle of inclination (θ) = 10 to 30° with a normal line from the bank.

4. Spacing b/w two adjacent spurs = $S = 2$ to 2.5 times L
'or' $S = 7$ to 7.5 times h

where, h = vt. projection of spur exposed in the river,

$$h = L \times \cos \theta, \text{ where, } L = L - l_e$$

6. Nos. of spur, $N = \frac{L_b}{S} + 1$; one or two additional spurs should be provided ulr of eroded bank for its protection.

L_b = length of bank to be protected.

7. Top width of spur, $T_w = 3$ to 6 m

8. Free board, $FB = 1$ to 1.5 m above HFL for non-submerged spur.

9. Slope pitching is provided over 20 to 30 cm of graded filter. Stone pitching slope for shank $2:1$ and $3:1$ to $5:1$ for head/nose.

10. Thickness of pitching, $t = 0.069^{1/3}$

11. Width of launching apron

(i) For shank/straight portion,

$$W_s = 1.5D;$$

$D = XR - y$, provided for both dir and dir along its full length, for shank, $x = 1.25$

(ii) For nose/head), $W_n = 2.5D;$

$$D = XR - y, \text{ where } x = 2.25$$

12. Thickness of apron, $T = 1.25t$ for shank and $1.5t$ for head/nose

(Q1) The following hydraulic data refers to a bridge site of a river. Design the Bell's bund/ Guide bank including launching apron to train the river.

High flood discharge of the river = $30,000 \text{ m}^3/\text{s}$

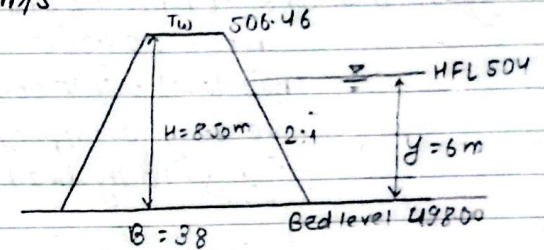
HFL of the river = 504.00 m

River bed level = 498.00 m

Avg. diameter of river bed materials = 1 cm

Afflux = 1 m

velocity of flow = 3 m/s



→ Solution:-

(a) Calculation of distance b/w two guide banks

discharge, $Q = 30,000 \text{ m}^3/\text{s}$

dia. of bed material, $d = 1 \text{ cm} = 10 \text{ mm}$

$$\text{Stt factor, } f = 1.76 \sqrt{d_{\text{mm}}} = 1.76 \sqrt{10} = 5.57$$

$$\text{Lacey's wetted perimeter} = P = 4.75 \sqrt{Q}$$

$$= 4.75 \sqrt{30000}$$

$$= 822.72 \text{ m}$$

Waterway of the bridge including piers taking 20% extra for piers; $L = 1.20 \times P$

$$= 1.20 \times 822.72$$

$$= 987.26 \approx 987 \text{ m}$$

$\therefore$ Distance b/w guide banks = 987 m

(b) Calculation of length of guide bank $(9720,000 \text{ m}^3/\text{s})$
 u/s length of guide bank $= 1.5L = 1.5 \times 987$
 $= 1480.50 \text{ m}$
 d/s length of guide bank $= 0.40L = 0.40 \times 987$
 $= 394.80 \text{ m}$
 $\therefore$ Total length of guide bank $= 1480.50 + 394.80$
 $= 1875.30 \text{ m}$

(c) Calculation of ht. of guide bank

$$\text{HFL} = 504.00 \text{ m}$$

$$\text{Bed level} = 498.00 \text{ m}$$

$$\left. \begin{array}{l} \text{velocity of approach, } V = 3 \text{ m/s} \\ \therefore \text{velocity head} = \frac{V^2}{2g} = \frac{3^2}{2 \times 9.81} = 0.46 \text{ m} \end{array} \right\}$$

$$\text{Afflux} = 1 \text{ m}$$

$$\begin{aligned} \text{Top level of bank} &= \text{HFL} + \text{vel. head} + \text{Afflux} + \text{F.B.} \\ &= 504.00 + 0.46 + 1 + 1 \\ &= 506.46 \text{ m} \end{aligned}$$

$$\text{Ht. of guide bank} = H = 506.46 - \text{Bed level}$$

$$= 506.46 - 498$$

$$= 8.46 \text{ m}$$

$$\text{Provide ht. of bank, } H \approx 8.50 \text{ m (say)}$$

(d) Top and bottom width of guide bank
 Let us provide top width of guide bank $(T_w) = 4 \text{ m} (> 3 \text{ m})$
 side slope of guide bank $= 2:1$
 then,
 Bottom width $(B) = 2 \times (2 \times H) + T_w$
 $= 2 \times (2 \times 8.5) + 4$
 $= 38 \text{ m}$

(e) Radius of Impregnable head

$$\begin{aligned} \text{Radius of u/s Impregnable / curve head (R)} &= 0.45L \\ &= 0.45 \times 987 \\ &= 444.15 \text{ m} \end{aligned}$$

$$\begin{aligned} \text{Radius of d/s impregnable / curve head (r)} &= R/2 \\ &= \frac{444.15}{2} \\ &= 222.075 \text{ m} \end{aligned}$$

The angle of sweep for u/s curve may be provided as 120° $[120 \text{ to } 145^\circ]$

The angle of sweep for d/s curve may be provided as 60° $[45^\circ \sim 60^\circ]$

(f) Calculation of thickness and inclined length of slope pitching.

$$\begin{aligned} \text{thickness of stone pitching on side slope, } t &= 0.06 Q^{1/6} \\ &= 0.06 (30000)^{1/6} \\ &= 1.86 \text{ m} \end{aligned}$$

Hence, provide thickness, $t = 1.90 \text{ m}$

$$\begin{aligned} \text{Inclined ht. of stone pitching} &= \sqrt{2^2 + 1^2} \times t \\ &= 1.9 \text{ m} \end{aligned}$$

(g) Calculation of volume of stone per m length of pitching.

$$V = \text{inclined ht.} \times t = 1.9 \times 1.9 = 36.1 \text{ m}^3/\text{m}$$

(h) Calculation of scour depth and width and thickness of launching apron.

$$\begin{aligned} \text{Hydraulic mean radius, } R &= 0.48 (Q/f)^{1/3} \\ &= 0.48 (30,000 / 5.57)^{1/3} \\ &= 8.41 \text{ m} \end{aligned}$$

Scour depth below bed level, $D = XR - y$

For Nose portion, $L_{\text{depth of water}}$

$$D = 2.25 \times 8.41 - (504 - 498) = 13.597 \approx 13.60 \text{ m}$$

For width of apron (W_a) = $1.5D = 1.5 \times 13.66 = 20.40 \text{ m}$

For curvilinear transition,

$$D = 1.5 \times 8.41 - (6) = 6.62 \text{ m}$$

$$W_a = 1.5 \times 6.62 = 9.93 \approx 10 \text{ m}$$

For Shank

$$D = 1.25 \times 8.41 - 6 = 4.51 \text{ m}$$

$$W_a = 1.5 \times 4.51 = 6.77 \approx 7 \text{ m}$$

$$\begin{aligned} \text{Now, Thickness of apron, } T &= 1.9t = 1.9 \times 1.9 \\ &= 3.61 \text{ m} \\ &\approx 3.65 \text{ m} \end{aligned}$$

Launching apron and slope pitching is provided on both sides i.e. outer and inner side for Nose and curved portion and on water side (inner side) only for shank portion.

(i) Calculation of volume of stone required for per m length of launching apron.

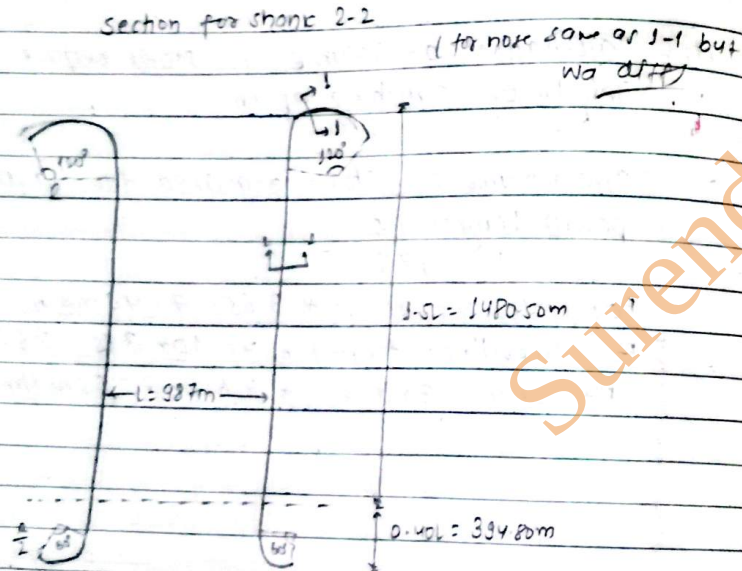
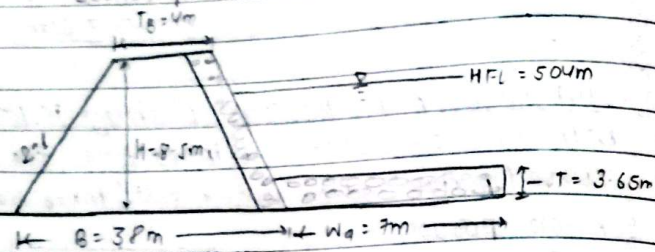
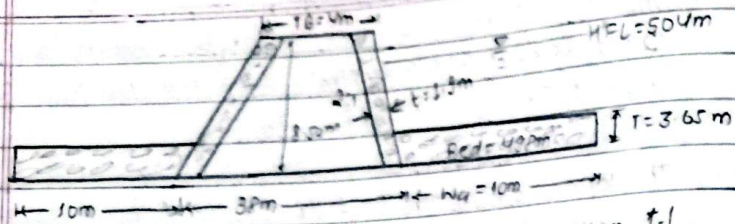
The volume of stone required for launching apron per m length is

$$V = W_a \times T$$

$$\text{For Nose, } V = 20.40 \times 3.65 = 74.46 \text{ m}^3/\text{m}$$

$$\text{For curvilinear transition, } V = 10 \times 3.65 = 36.50 \text{ m}^3/\text{m}$$

$$\text{For Shank, } V = 7 \times 3.65 = 25.55 \text{ m}^3/\text{m}$$



Plan

Numerical

(Q3) Find out the number of spurs to control one side of the stream bank of 150m length. The average flood flow is $5000 \text{ m}^3/\text{sec}$. It is given that the length of spur is 10m and angle of projection is 30° from the vertical.

→ Solution:-

Total length of eroded bank, $L_b = 150 \text{ m}$
 Projection angle from vertical, $\theta = 30^\circ$
 Total length of spur; $L = 10 \text{ m}$

Assuming that one third of total length is embedded into the bank,

$$L_e = \frac{L}{3} = \frac{10}{3} = 3.33 \text{ m}$$

$$\begin{aligned} \text{Then, length of spur exposed into river (l)} &= L - L_e \\ &= 10 - 3.33 \\ &= 6.67 \text{ m} \end{aligned}$$

Vertical projection of spur exposed in the river stream
 $h = l \cos \theta = 6.67 \cos 30^\circ = 5.78 \text{ m}$

Let us assume the spacing of spur (S) = $7h$
 $= 7 \times 5.78 = 40.4 \text{ m}$

Thus, number of spurs required to protect eroded downstream = $N = \frac{L_b}{S} + 1 = \left(\frac{150}{40.4} \right) + 1 = 4.5125$ No. of spurs.

Assuming that 2 spurs are provided towards the upstream face additionally,

Then, total number of spurs required = $5 + 2$
= 7 spurs

Surendra Sharma

Numerical on Guide bank / Bell's bund

9. A bridge is to be constructed across the river, having the following hydraulic data.

$$\text{Max}^{\text{m}} \text{ discharge} = 17000 \text{ m}^3/\text{sec}$$

$$\text{Highest flood level} = 288 \text{ m}$$

$$\text{River bed level} = 280 \text{ m}$$

$$\text{Average dia. of river bed material} = 0.1 \text{ mm}$$

Find out desired waterway for the bridge. Design bell's guide bund including launching apron.

Soln.

→ Step 1: Finding length of waterway

$$P = 4.75 \sqrt{Q}$$

$$= 4.75 \sqrt{17000}$$

$$= 619.32 \text{ m}$$

$$\text{or, } L = 619.32 \text{ m}$$

Now,

providing 20% extra length for pier.

$$L = 1.2 \times 619.32 = \underline{744 \text{ m}}$$

Step 2: Length of guide bank

$$(a) \text{ U/s length} = 1.25 \times L = 1.25 \times 744 = 930 \text{ m}$$

$$(b) \text{ D/s length} = 0.25 \times L = 0.25 \times 744 = 186 \text{ m}$$

Step 3: Finding radius of curve head

From gales recommendation,

(i) Radius of U/s curved head of guide bank

$$= 0.45 \times L = 0.45 \times 744 = 334.8 \text{ m} \approx 335 \text{ m}$$

(ii) Radius of D/c curved head = $\frac{D_{ur}}{2}$
 $= \frac{335}{2}$
 $= 167.5m$

→ Thus, providing sweep angle for
 U/c curved head 125°
 [Range: 120° to 140°]

→ Providing sweep angle of 60° for D/c curved head.

Step 4: cross-section of Guide bank.

→ Assuming top width of guide bank = 4m

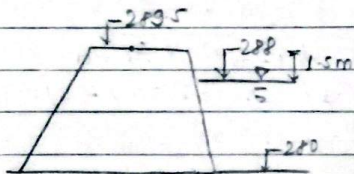
→ Assuming side slope as 2:1

→ HFL = 288m

→ Assuming Free board = 1.5m

Neglecting velocity head & afflux.

Top level of guide bund = $288 + 1.5 = 289.5m$



Height of guide bund above bed of river
 $= 289.5 - 280$
 $= 9.5m$

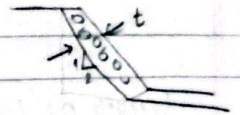
Slope protection

thickness of stone pitching on side slope of guide bank

$$t = 0.06 Q^{1/3}$$

$$= 0.06 \times 17000^{1/3}$$

$$[t \approx 1.55m]$$



Volume of stone required for pitching per m length

$$= \sqrt{S} \times D \times t$$

$$= \sqrt{S} \times \text{depth of guide bank}$$

$$= \sqrt{S} \times 9.5 \times 1.55$$

$$= 33m^3/m$$

Step 6: launching apron:-

Lacey Regime scour depth

is given by,

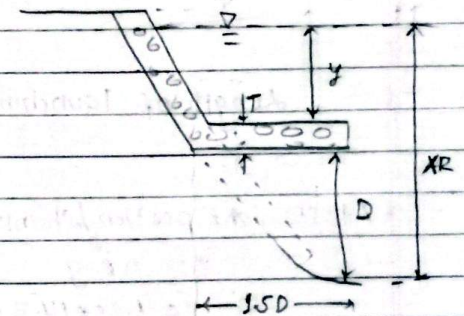
$$R = 0.47 \left(\frac{Q}{f} \right)^{1/3}$$

$$f = 1.76 \sqrt{d_{mm}}$$

$$f = 1.76 \times \sqrt{0.1}$$

$$f = 0.55$$

$$R = 0.47 \left(\frac{17000}{0.55} \right)^{1/3} = 14.696 \approx 14.7m$$



depth of water above bed (y) = HFL - Bed level
 $= 288 - 280$
 $= 8m$

At nose.

$$\begin{aligned}\text{Depth of scour (D)} &= XR - y \\ &= 2.25 \times 14.7 - 8 \\ &= 25.075\text{m}\end{aligned}$$

$$\begin{aligned}\text{Length of launching apron} &= 1.5 \times D \\ &= 1.5 \times 25.075 \\ &= 37.6\text{m}\end{aligned}$$

$$T = 1.9 \times t = 1.9 \times 1.55 = 2.945\text{m}$$

At transition:

$$\alpha = 1.5$$

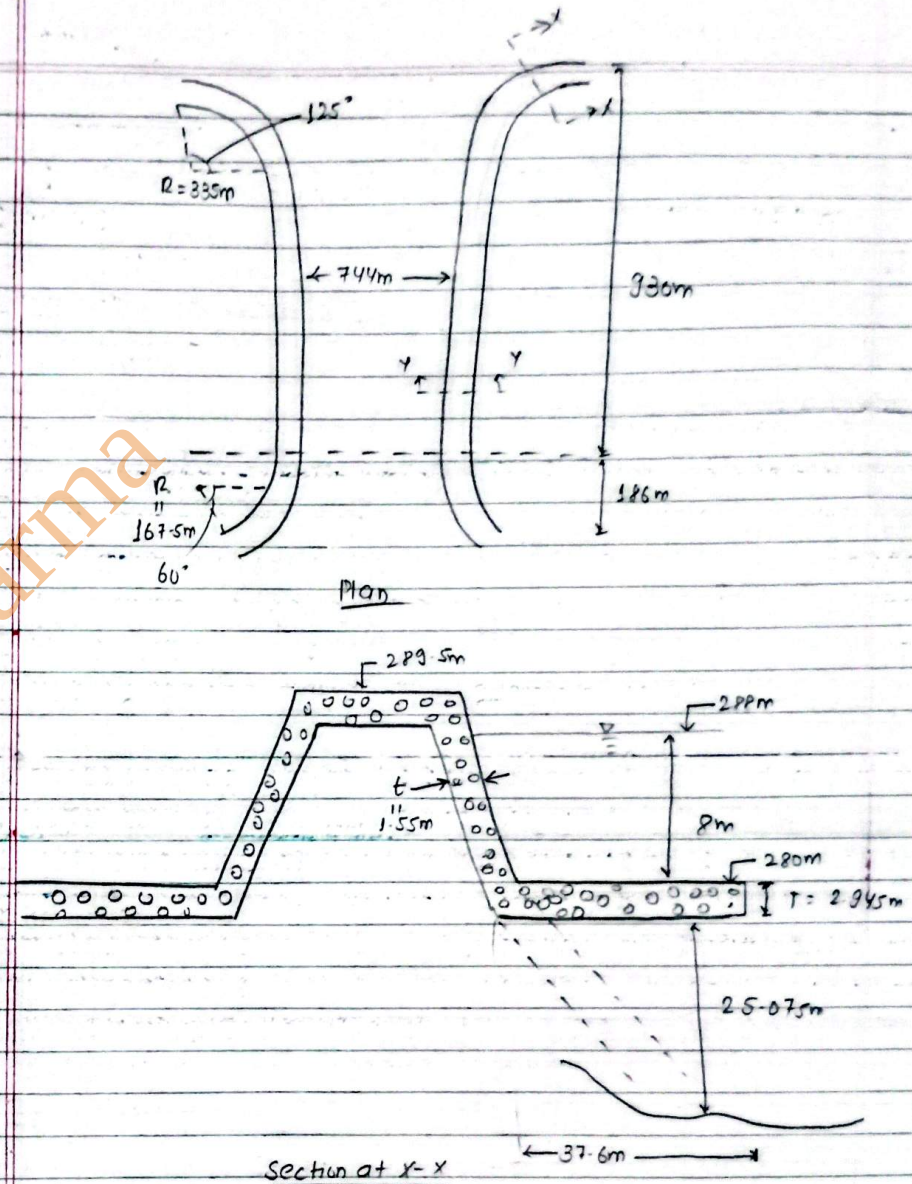
$$\begin{aligned}\therefore \text{depth of scour (D)} &= \alpha R - y \\ &= 1.5 \times 14.7 - 8 \\ &= 14.05\text{m}\end{aligned}$$

$$\begin{aligned}\text{Length of launching apron} &= 1.5 \times 14.05 \\ &= 21.075\text{m}\end{aligned}$$

At straight portion / abutment:

$$\begin{aligned}D &= \alpha R - y \\ &= 1.25 \times 14.7 - 8 \\ &= 10.375\text{m}\end{aligned}$$

$$\begin{aligned}\text{Length of launching apron} &= 1.5 \times D \\ &= 15.6\text{m}\end{aligned}$$



(9) Design a siphon aqueduct if the following data at the crossing of canal and drainage are given.

Discharge of canal = $50 \text{ m}^3/\text{sec}$

Bed width of canal = 30 m

Full supply depth of canal = 1.5 m

Bed level = 106 m

Side slope of canal = $1.5H:1V$

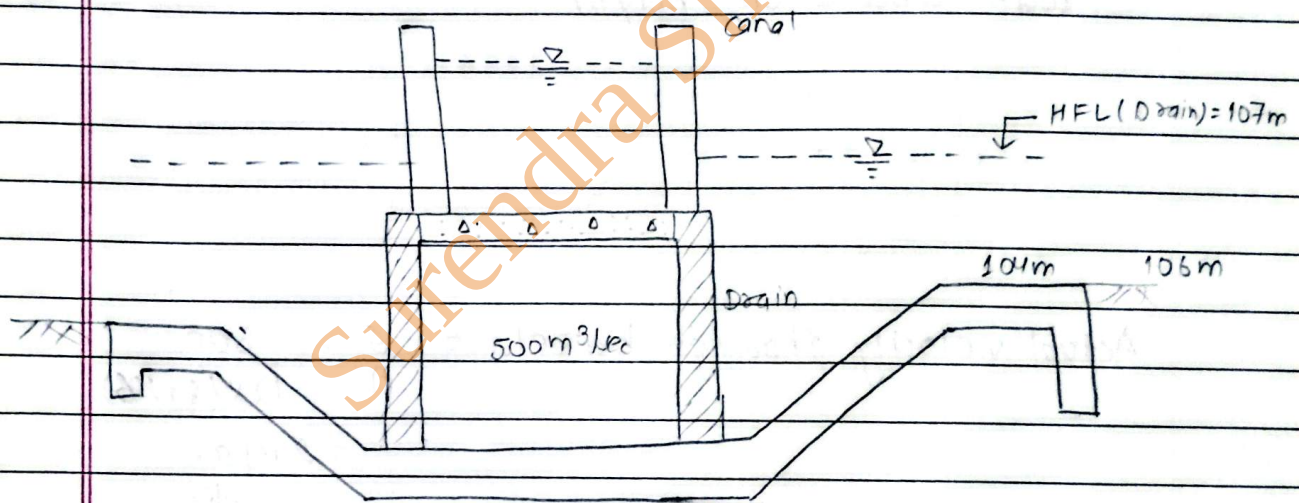
High flood discharge of drainage = $500 \text{ m}^3/\text{sec}$

HFL of drainage = 107 m

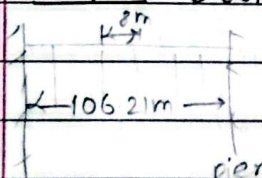
Bed level of drainage = 104 m

General Ground level = 106 m

→ Solⁿ:-



Step (a) Design of drainage waterway



$$P = 4.75 \sqrt{Q} = 4.75 \sqrt{500} = 106.21 \text{ m}$$

Say, providing 12 bays of 8 m each.

& provide width of each pier = 1.5 m

$$\therefore \text{Length of waterway} = 12 \times 8 + 11 \times 1.5$$

$$= 112.5 \text{ m}$$

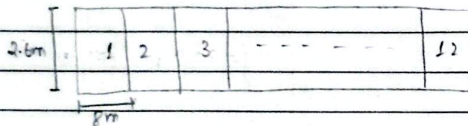
⇒ Limiting velocity of siphon barrel is 2 m/s

$$A = \frac{Q}{V} = \frac{500}{2} = 250 \text{ m}^2$$

$$\begin{aligned} \therefore \text{Area of each barrel} &= \frac{\text{Total Area}}{\text{No. of barrel}} \\ &= \frac{250}{12} \\ &= 20.833 \text{ m}^2 \end{aligned}$$

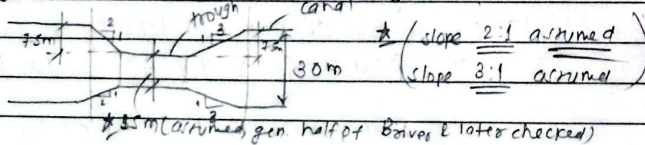
$$\text{Height of barrels} = \frac{\text{Area}}{\text{width}} = \frac{20.833}{8} = 2.6 \text{ m}$$

Thus, providing, 12 rectangular barrels of 8m width and 2.6m height



$$\begin{aligned} \text{Actual velocity through barrel} &= \frac{Q}{A} = \frac{500}{12 \times (8 \times 2.6)} \\ &= 2 \text{ m/sec} \quad (\text{ok}) \end{aligned}$$

Step 3(b) Design of canal water waterway.

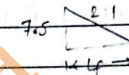


Normal bed width of canal = 30m

Let for flumed section,
width required = 15m

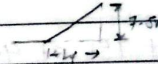
$$\begin{aligned} \text{Velocity through trough} &= \frac{Q_{\text{canal}}}{d_{\text{canal}} \times B_{\text{canal}}} \\ &= \frac{50}{1.5 \times 15} \\ &= 2.2 \text{ m/s} \end{aligned}$$

length of transition in contraction,

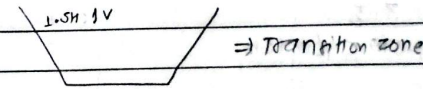


$$L_f = 2 \times 7.5 = 15 \text{ m}$$

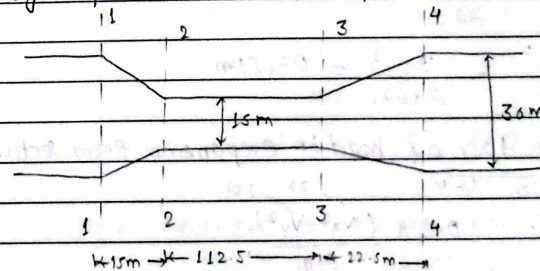
length of transition in expansion,



$$L_f = 7.5 \times 3 = 22.5 \text{ m}$$



Step 2 Design of bed level of different sections:



At section 4-4,

Area of trapezoidal section = $(B + 1.5y)y$

$$A = (30 + 1.5 \times 1.5) \times 1.5 = 48.375 \text{ m}^2$$

$$V_4 = \frac{Q_4}{A_4} = \frac{50}{48.375} = 1.03 \text{ m/s}$$

$$\text{vel. head} = \frac{V_4^2}{2g} = \frac{1.03^2}{2 \times 9.81} = 0.0544 \text{ m}$$

Bed level of canal = 106m
water depth = 1.5m

$$\therefore \text{RL of water surface at 4-4} = 106 + 1.5 = 107.5 \text{ m}$$

$$\text{RL of TEL at 4-4} = 107.5 + 0.0544 = 107.544 \text{ m}$$

At section 3-3,

$$y = 1.5 \text{ m}, L = 15 \text{ m}$$

$$A = 1.5 \times 15 = 22.5 \text{ m}^2$$

$$V_3 = \frac{50}{22.5} = 2.22 \text{ m/s}$$

$$\text{vel. head} = \frac{V_3^2}{2g} = \frac{2.22^2}{2 \times 9.81} = 0.251 \text{ m}$$

Assuming loss of head in expansion from section 3-3 to section 4-4

$$= 0.3 \times \frac{(V_3^2 - V_4^2)}{2g}$$

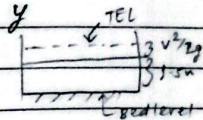
$$= 0.059 \text{ m}$$

RL of TEL at 3-3 = RL of TEL at 4-4 + Loss in expansion

$$= 107.544 + 0.059 = 107.613 \text{ m}$$

$$\text{RL of water level at 3-3} = 107.613 - 0.251 = 107.362 \text{ m}$$

$$\text{RL of Bed level} = \text{RL of water level} - y = 107.362 - 1.5 = 105.862 \text{ m}$$



At section 2-2

((3-3) + particulars)

From Manning's formula,

$$H_L = \frac{n^2 V^2 L}{R^{4/3}} \quad (n = 0.016, \text{ concrete})$$

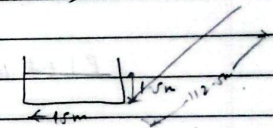
$$L = 112.5 \text{ m}$$

$$R = \frac{A}{P} = \frac{15 \times 1.5}{15 + 2 \times 1.5}$$

$$= 1.25$$

$$\text{Velocity} = \frac{Q}{P} = \frac{50}{15 \times 1.5} = 2.22 \text{ m/s}$$

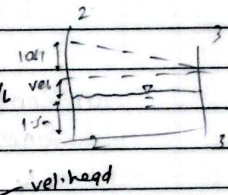
$$H_L = \frac{0.016^2 \times 2.22^2 \times 112.5}{1.25^{4/3}} = 0.105 \text{ m}$$



$$\text{R.L. of TEL at 2-2} = \text{R.L. of TEL at 3-3} + H_L = 107.613 + 0.105 = 107.718 \text{ m}$$

$$\text{RL of water level at 2-2} = 107.718 - 0.251 = 107.467 \text{ m}$$

$$\text{RL of Bed level} = 107.467 - 1.5 = 105.967 \text{ m}$$



At section 1-1

Loss of head due to contraction

$$= 0.2 \left(\frac{V_2^2 - V_1^2}{2g} \right)$$

$$= 0.2 \left(\frac{2.22^2 - 1.03^2}{2 \times 9.81} \right)$$

$$= 0.0385 \text{ m}$$

$$EL \text{ of TEL at 1-1} = EL \text{ of TEL at 2-2} + h_c$$

$$= 107.718 + 0.0385$$

$$= 107.756 \text{ m}$$

$$EL \text{ of water level} = 107.756 - 0.0544$$

$$= 107.7016 \text{ m}$$

$$EL \text{ of bed} = 107.7016 - 1.5$$

$$= 106.2016 \text{ m}$$

Step 3: Design of Transition

(width of zone b/w 1-1 & 2-2)

& b/w 3-3 & 4-4

(a) Contraction transition:

Since, the depth is constant, so the transition shall be designed on the basis of Mitrani hyperbolic transition, eqn.

$$B_a = \frac{B_n \cdot B_f \cdot L_f}{L_f - \alpha(B_n - B_f)}$$

$$B_n \cdot L_f - \alpha(B_n - B_f)$$

$$B_n = \frac{30 \times 15 \times 15}{30 \times 15 - \alpha(30 - 15)}$$

$$B_n = \frac{6750}{450 - 15\alpha} = \frac{450}{30 - \alpha}$$

$$B_n \rightarrow 30 \text{ m}$$

$$B_f \rightarrow 15 \text{ m}$$

$$L_f = 15 \text{ m} \& 22.5 \text{ m}$$

$$\alpha \rightarrow \text{distance from 2-2}$$

$$\text{or } 3-3$$

For various values of α lying b/w 0 to 15m, various values of B_n are found from eqn d)

α	0	2	4	6	8	10	12	14	15
$B_n = \frac{450}{30 - \alpha}$	15	16.07	17.37	18.75	20.45	22.5	25	28.125	30

Expansion transition,

$$B_n = \frac{B_n \cdot B_f \cdot L_f}{L_f - \alpha(B_n - B_f)}$$

$$B_n \cdot L_f - \alpha(B_n - B_f)$$

$$= \frac{30 \times 15 \times 22.5}{30 \times 22.5 - \alpha(30 - 15)}$$

$$30 \times 22.5 - \alpha(30 - 15)$$

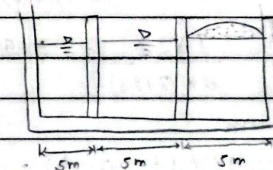
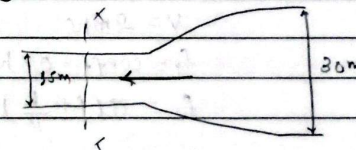
$$B_n = \frac{675}{45 - \alpha}$$

$$45 - \alpha$$

α	0	4	8	12	16	20	22.5
$B_n = \frac{675}{45 - \alpha}$	15	16.46	18.24	20.45	23.24	27	30

Step 4: Design of Trough

Let's divide the trough into 3 equal compartments each 5m wide



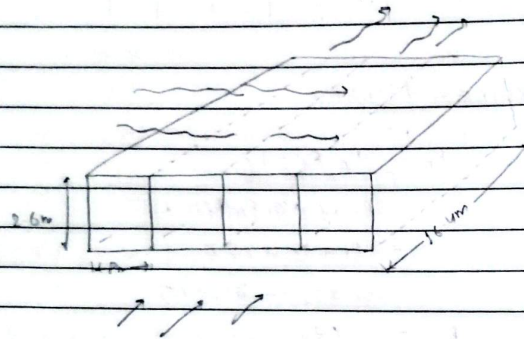
Providing the separation wall of 0.3m width

$$\text{Total height of trough} = 0.6 + 1.5 = 2.1 \text{ m}$$

Providing outer wall of width 0.4m.

$$\therefore \text{Overall outer width of trough} = 15 + 0.3 \times 2 + 0.4 \times 2 = 16.4 \text{ m}$$

$$\text{Length of siphon barrel} = 16.4 \text{ m}$$



Steps: Head losses through siphon barrel.

From unwin's formula,

$$\text{head loss, } h = \left[1 + f_1 + f_2 \times \frac{L}{R} \right] \times \frac{V^2}{2g} - \frac{V_2^2}{2g}$$

[V_2 , approach vel. negligible]

$$V = 2 \text{ m/s}$$

$$f_1 = \text{coeff. of head loss at entry} = 0.505$$

$$f_2 = a \left(1 + \frac{b}{R} \right)$$

from table, $a = 0.00316$, $b = 0.030$ for plaited material.

$$R = \frac{A}{P} = \frac{8 \times 2.6}{2 \times (8 + 2.6)} = 0.9811 \text{ m}$$

$$L = 16.4 \text{ m}$$

$$f_2 = 0.00316 \left(1 + \frac{0.03}{0.9811} \right) = 0.003256$$

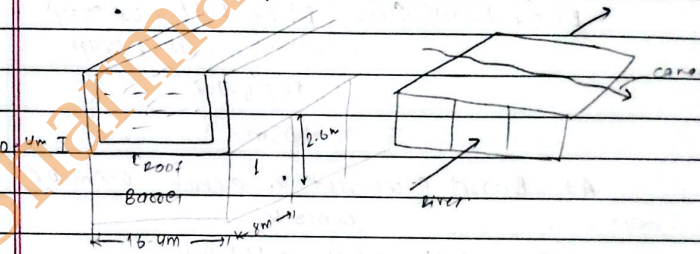
$$h = \left[1 + 0.505 + 0.003256 \times \frac{16.4}{0.9811} \right] \times \frac{2^2}{2 \times 9.81}$$

$$[h = 0.318 \text{ m}]$$

$$\text{HFL of drainage} = 107 \text{ m}$$

$$\therefore \text{HFL of drainage at } 1 = 107 + h_1 = 107 + 0.318 = 107.318 \text{ m}$$

Step 6: Uplift pressure on roof of barrel



$$\text{RL of bottom of trough} = 106 - 0.4 = 105.6 \text{ m}$$

$$\therefore \text{loss of Head at entry of barrel} = 0.505 \times \frac{V^2}{2g} = 0.102 \text{ m}$$

$$\begin{aligned} \text{Uplift on roof} &= \text{HFL} - \text{loss of entry} - \text{level of underside of roof slab} \\ &= 107.318 - 0.102 - 105.6 \\ &= 1.616 \text{ m of water} \end{aligned}$$

$$\text{Assume, } \gamma_w = 10 \text{ kN/m}^3$$

$$\therefore \text{Uplift on roof} = 1.616 \times 10 = 16.16 \text{ kN/m}^2$$

$$\text{Assume } \gamma_{\text{concrete}} = 24 \text{ kN/m}^3$$

$$\begin{aligned} \text{DLW load on concrete} &= 0.4 \times 24 \text{ kN/m}^3 \\ &= 9.6 \text{ kN/m}^2 \end{aligned}$$

Thus,

$(16.16 - 9.8) = 6.36 \text{ kN/m}^2$ of uplift has to be resisted by reinforcement to be provided at the top in the roof slab of basins.

Step 7: Uplift in bottom of basin.



(a) Static head

$$\begin{aligned} \text{LL of basin floor} &= \text{LL of bottom of rough} \\ &\quad - \text{Ht of rough} \\ &= 105.6 - 2.6 \\ &= 103 \text{ m} \end{aligned}$$

At Basin floor, let us assume 0.8m of concrete.

$$\begin{aligned} \text{LL of bottom of floor} &= 103 - 0.8 \\ &= 102.2 \text{ m} \end{aligned}$$

$$\text{Bed level of drain} = 104 \text{ m}$$

$$\begin{aligned} \text{Static uplift on floor} &= \text{Bed level of drain} \\ &\quad - \text{LL of bottom of floor} \\ &= 104 - 102.2 \\ &= 1.8 \text{ m of water} \end{aligned}$$

(b) Seepage head

$$\begin{aligned} \text{Total seepage head} &= \text{FSL of canal} - \text{Bed level of drain} \\ &= (106 + 1.5) - 104 \\ &= 3.5 \text{ m} \end{aligned}$$

The residual head at point a, at centre of first basin has been calculated by Bligh's theory.

Consider total length of drainage floor = 30 m

$$ab = 15 + 4.$$

$$bc = \frac{30}{2} = 15$$

$$\therefore \text{Total creep length} = 19 + 15 = 34 \text{ m.}$$

$$\text{Residual head at } h_r = \frac{H}{ab + bc} \times bc$$

$$h_b = \frac{3.5}{34} \times 15 = 1.544 \text{ m}$$

$$\begin{aligned} \therefore \text{Total uplift} &= \text{Static head} + \text{seepage head} \\ &= 1.8 + 1.544 \end{aligned}$$

$$= 3.34 \text{ m of water}$$

$$\text{Uplift pressure} = 33.44 \text{ kN/m}^2$$

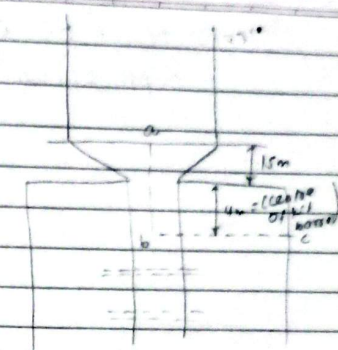
$\therefore$ Uplift resisted by floor wt.

itself is

$$0.8 \times 24 = 19.2 \text{ kN/m}^2$$

(c)

$$\begin{aligned} \text{Uplift to be resisted by basin} &= 33.44 - 19.2 \\ &= 14.24 \text{ kN/m}^2 \end{aligned}$$



Step 8: Design of cut-off and protection work.

$$\text{Depth of scour (P)} = 0.47 \times \left(\frac{Q}{f} \right)^{1/3}$$

(River at scour = 500 m³/s)

$$= 0.47 \times \left(\frac{500}{f} \right)^{1/3}$$

(1) assume

$$= 3.73 \text{ m}$$

Provide depth of cut-off for scour hole of 1.5P, on both sides i.e. •

$$\text{Depth of cut-off at U/S} = 1.5 \times 3.73 = 5.6 \text{ m}$$

$$\text{do at D/S} = 1.5 \times 3.73 = 5.6 \text{ m}$$

$$\text{RL of bottom of U/S cut-off} = \text{U/S HFL} - 5.6 = 101.718 \text{ m}$$

$$\text{do at D/S} = 107 - 5.6 = 101.4$$

$$\text{Length of U/S protection} = 2 (\text{RL of U/S bed} - \text{RL of bottom of U/S cut-off})$$

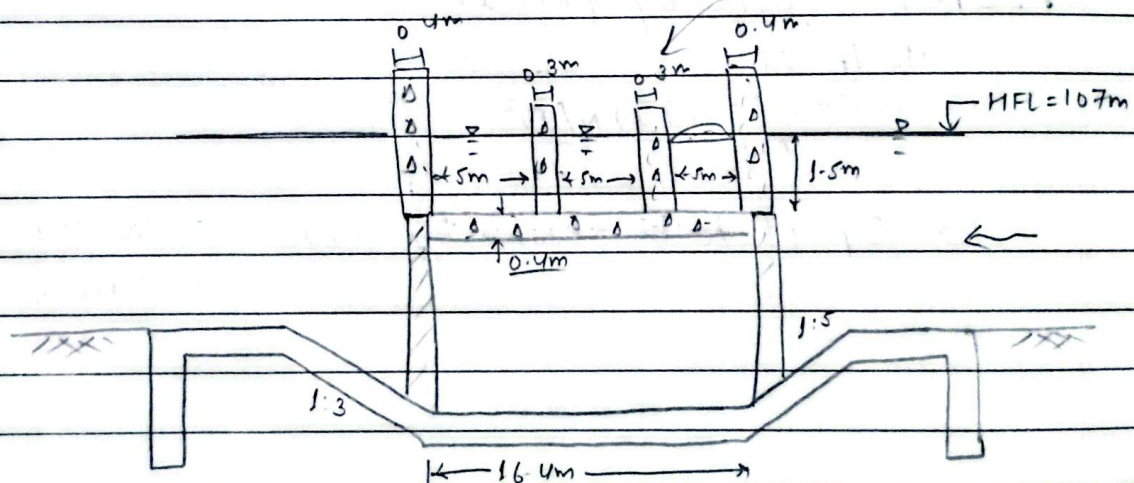
$$= 2 (103 - 101.718)$$

$$= 2.56 \text{ m}$$

$$\text{Length of D/S protection} = 2 (\text{RL of D/S bed} - \text{RL of bottom of D/S cut-off})$$

$$= 2 (104 - 101.4)$$

$$= 5.2 \text{ m}$$



Tutorial 3

(Q) Design an irrigation canal to carry a discharge of 10 cumecs for longitudinal slope 1 in 5000 adopting a side slope of 1:1, critical velocity ratio of 1 and rugosity coefficient of 0.0225.

↓
Kennedy's theory

→ Solution,

Canal is designed on basis of Kennedy's theory.

Here, We have discharge $Q = 10 \text{ m}^3/\text{s}$.

bed slope, $S = 1 \text{ in } 5000$

$CVR(m) = 1$

Rugosity coefficient (N) = 0.0225

We know,

critical velocity, $V_c = 0.55 m D^{0.64}$

→ 1st trial value of depth of flow, $D = 1.5 \text{ m}$.

Now, $V_c = 0.55 \times 1 \times 1.5^{0.64} = 0.71 \text{ m/s}$ ←

Sectional area, $A = \frac{Q}{V_c} = \frac{10}{0.71} = 14.08 \text{ m}^2$

$$A = BD + nD^2$$

$$\text{or, } 14.08 = B \times 1.5 + 1 \times 1.5^2$$

$$\Rightarrow [B = 7.89 \text{ m}]$$

$$\begin{aligned} \text{Wetted perimeter, } P &= B + 2D\sqrt{1+n^2} \\ &= 7.89 + 2 \times 1.5 \sqrt{1+1^2} \\ &= 12.13 \text{ m} \end{aligned}$$

$$\text{Hydraulic mean radius, } R = \frac{A}{P} = \frac{14.08}{12.13} = 1.16 \text{ m}$$

Assuming value	
Q	D
< 10	1
10-20	1.5
20-30	2.0
30-40	2.8
40-50	3
> 50	3.5

Actual velocity of flow, $V_a = C\sqrt{RS}$

$$C = \frac{\left(\frac{1}{N} + 23 + \frac{0.00155}{S}\right)}{1 + \left(23 + \frac{0.00155}{S}\right) \frac{N}{\sqrt{R}}}$$

$$= \frac{\left(\frac{1}{0.0225} + 23 + \frac{0.00155}{1/5000}\right)}{1 + \left(23 + \frac{0.00155}{1/5000}\right) \frac{0.0225}{\sqrt{1.16}}}$$

$$[C = 45.78]$$

Actual $\rightarrow V_a = 45.78 \times \sqrt{1.16 \times \frac{1}{5000}} = 0.697 \approx 0.70 \text{ m/s}$

$$\frac{V_a}{V_c} = \frac{0.70}{0.71} = 0.986 \neq 1 \quad [\because V_c > V_a \text{ so } V_c \text{ should be reduced by taking decreasing value of } D]$$

(2 digit)

So, Adopting another trial value of $D = 1.40 \text{ m}$.

$$V_c = 0.55 \times 1 \times 1.4^{0.64} = 0.68 \text{ m/s}$$

$$A = \frac{10}{0.68} = 14.70 \text{ m}^2$$

$$B = (14.70 - 1 \times 1.4^2) / 1.40 = 9.10 \text{ m}$$

$$P = 9.10 + 2 \times 1.40 \sqrt{1+1^2} = 13.06 \text{ m}$$

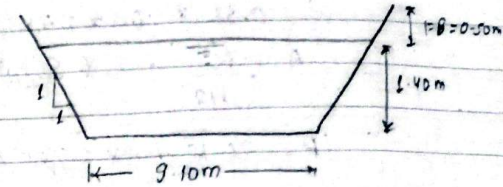
$$R = \frac{14.70}{13.06} = 1.125 \text{ m}$$

$$C = \frac{\left(\frac{1}{0.0225} + 23 + \frac{0.00155 \times 5000}{S}\right)}{1 + \left(23 + \frac{0.00155 \times 5000}{S}\right) \frac{0.0225}{\sqrt{1.125}}} = 45.508$$

$$V_a = 45.508 \times \sqrt{1.125 \times \frac{1}{5000}} = 0.68 \text{ m/s}$$

$$\frac{V_a}{V_c} = 1 = m \text{ (okay)}$$

Therefore, depth of canal $D = 1.40 \text{ m}$, $B = 9.10 \text{ m}$
(canal bed depth).
(w/o F.B.).



(2.) Using Kennedy's approach determine the dimension of section of an irrigation canal for the following data.
B/D ratio = 11.50, $m = 1.03$, $Q = 30 \text{ m}^3/\text{s}$, side slope (H:V) = 0.50:1 (n:1)

$\rightarrow$ Solution:-

$$\text{Given, } B/D = 11.50$$

$$B = 11.50D$$

$$V_c = 0.55 \text{ m } D^{0.64}$$

$$Q = A \times V_c$$

$$\text{or } 30 = (BD + mD^2) \times 0.55 \text{ m } D^{0.64}$$

$$\text{or } 30 = (11.50D + 0.50D^2) \times 0.55 \times 1.03 \times D^{0.64}$$

$$\therefore D = 1.755 \text{ m}$$

$$\& B = 11.50 \times 1.755 = 20.18 \text{ m}$$

Now,

$$V_c = 0.55 \times 1.03 \times 1.755^{0.84} = 0.812 \text{ m/s}$$

$$A = \frac{Q}{V_c} = \frac{50}{0.812} = 61.58 \text{ m}^2$$

$$P = 20.12 + 2 \times 1.755 \sqrt{1 + 0.50^2} = 24.104 \text{ m}$$

Hydraulic Mean Radius $R = \frac{A}{P} = \frac{61.58}{24.104} = 2.55 \text{ m}$

$$\left\{ V_c = C \sqrt{RS} \right\} \text{ or Manning's}$$

Let us take longitudinal slope, $S = 1/5400$

$$\left\{ \text{For } \frac{V_c}{V} = 1.03 \right\}$$

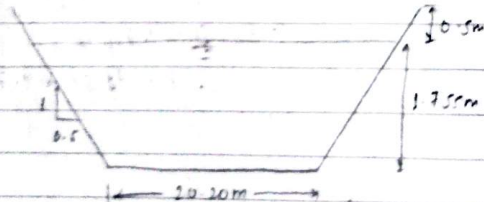
$$\text{or } \left[\left(\frac{1}{0.025} \right) + 23 + \frac{0.00155}{S} \right] \approx \sqrt{2.55 \times S}$$

$$1 \cdot \left(23 + \frac{0.00155}{S} \right) \frac{0.0225}{\sqrt{2.55}}$$

$$= 1.03 \times 0.812$$

$$\Rightarrow \left[S = 1.96 \times 10^{-4} \approx \frac{1}{5102} \right]$$

Therefore adopt, $D = 1.755 \text{ m}$, $B = 20.12 \approx 20.20 \text{ m}$ and longitudinal slope of 1 in 5100.



③ Design a regime channel for a discharge of $50 \text{ m}^3/\text{s}$ with silt factor of 1.25 and side slope of 1:1 by Lacey's method.

→ Solution,

$$\text{Given } Q = 50 \text{ m}^3/\text{s}$$

$$\text{silt factor } f = 1.25$$

$$\text{side slope } (n:1) = 1:1$$

From Lacey's formula,

Relation b/w V , Q and F is given by,

$$V = \left(\frac{Q f^2}{140} \right)^{1/6}$$

$$\therefore \text{velocity of flow, } V = \left(\frac{50 \times 1.25^2}{140} \right)^{1/6} = 0.907 \text{ m/s}$$

$$\text{Cross-sectional area, } A = \frac{Q}{V} = \frac{50}{0.907} = 55.126 \text{ m}^2$$

$$\text{For trapezoidal section, } A = BD + nD^2 = BD + D^2$$

$$55.126 = BD + D^2 \quad \text{--- (I)}$$

$$P = 4.75 \sqrt{Q} = 4.75 \sqrt{50} = 33.587 \text{ m}$$

Again,

$$P = B + 2D \sqrt{1+n^2} = B + 2D \sqrt{2}$$

$$33.587 = B + 2.828D$$

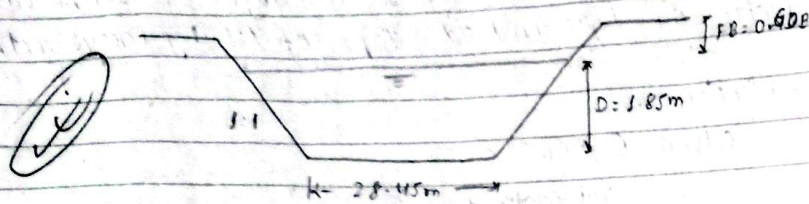
$$B = 33.587 - 2.828D \quad \text{--- (II)}$$

From (I) & (II),

$$D = 1.82 \text{ m} \approx 1.85 \text{ m}$$

$$B = 33.587 - 2.828 \times 1.85 = 28.55 \text{ m}$$

$$\text{Again, } S = \frac{1}{3340} \left(\frac{f^{5/3}}{Q^{1/6}} \right) = \frac{1}{3340} \left(\frac{1.25^{5/3}}{50^{1/6}} \right) \approx \frac{1}{9420}$$



⑤ Design an irrigation canal in alluvial soil to carry a discharge of $60 \text{ m}^3/\text{s}$, the avg. size of particle in channel is 0.5 mm .

→ Solution.

$$Q = 60 \text{ m}^3/\text{s}$$

$$\text{dia of particle, } d_{\text{mm}} = 0.5 \text{ mm}$$

$$\text{Find silt factor } f = 5.76 \sqrt{d_{\text{mm}}}$$

with known Q and f , do in similar way as above.

⑥ A channel is to be designed to carry a discharge of 600 cumecs . The bed slope is kept as $1 \text{ in } 600$. The soil is coarse alluvium of grain size 5 cm . assuming the channel to be unlined with unprotected banks and trapezoidal section. Determine the section of the channel ϕ for soil may be taken as 37° .

→ Solution:-

Since, grain size $> 6 \text{ mm}$, tractive force approach used.

Assume, side slope θ of the channel $< \phi$, Let's take $\theta = 30^\circ$.

Longitudinal slope $S = 1 \text{ in } 600$

$$Q = 600 \text{ cumecs}$$

$$\text{dia. of particle, } d = 5 \text{ cm} = 0.05 \text{ m}$$

The critical shear stress on slope, be τ_c' and bed shear stress be τ_c .

Again, we know,

$$k = \frac{\tau_c'}{\tau_c} = \sqrt{\frac{1 - \sin^2 \theta}{\sin^2 \phi}}$$

$$= \sqrt{\frac{1 - \sin^2 30^\circ}{\sin^2 37^\circ}}$$

$$= 0.556 < 1$$

okay.

$$\text{But (on bed), } \left\{ \begin{array}{l} \tau_c = \gamma_w R S \\ \tau_c = \gamma_w \left(\frac{d}{11} \right) \end{array} \right\} \left[\because d \geq 11 R S \right]$$

$$\left[\tau_c' = 0.75 \gamma_w R S \right]_{\text{slope.}}$$

Thus

$$\left\{ \tau_c' \leq k \tau_c \right\}_{\text{cond.}}$$

$$\text{or, } 0.75 \gamma_w R S \leq 0.556 \times \gamma_w \left(\frac{d}{11} \right) \quad \left(\text{for wide channel } R = D \right)$$

$$\text{or, } R \times \frac{1}{600} \leq \frac{0.556 \times 0.05}{0.75 \times 11}$$

$$\Rightarrow R \leq 2.02 \approx 2 \text{ m (say)}$$

For wide channel, $R = D$, taking 20% safety factor, $D = 0.80 \times R = 1.60 \text{ m} \leftarrow \text{Full supply depth}$
 $\therefore D = 1.60 \text{ m}$

N = rugosity coeff. is given by Strickler's formula

$$N = \frac{1}{24} (d)^{1/6} = \frac{1}{24} (0.05)^{1/6} = 0.025$$

(d in meter)

Manning's velocity of flow, $V = \frac{1}{N} R^{2/3} S^{1/2}$

$$V = \frac{1}{0.025} \times R^{2/3} \times \left(\frac{1}{600}\right)^{1/2}$$

Don't take R of before

$$[V = 1.63 R^{2/3}] - (I)$$

Side slope, $\tan 30^\circ = \frac{1}{n}$
 $\left[\frac{1}{\sqrt{3}} = \frac{1}{n}\right]$
 $\left[n = \sqrt{3}\right] \leftarrow$ side slope

or $\frac{1}{\sqrt{3}} = \frac{1}{n}$
 $\left[n = \sqrt{3}\right]$

or $\frac{1}{\sqrt{3}} = \frac{1}{n}$
 $\left[n = \sqrt{3}\right]$

$$[A = BD + nD^2 = 1.6B + 4.43] - (II)$$

$$[P = B + 2D \sqrt{1+n^2} = B + 6.40] - (III)$$

Taking trial value of B for adjusting Q

Width B (m)	Area, A (m ²)	P (m)	R (m)	V (m/s)	Q (m ³ /s)
100	164.43	106.40	1.55	2.18	358.48
150	244.43	156.40	1.56	2.19	535.75
165	268.43	171.40	1.566	2.20	590.55
167.5	272.43	173.90	1.57	2.20	599.34
167.75	272.83	174.15	1.57	2.20	600.2262

600
okay

So, width of canal, $B = 167.75m$ and depth, $D = 1.60m$.

⑦ Canal is designed to carry a discharge of $400m^3/s$, bed slope of 1 in 1200, soil is coarse alluvium having diameter of 5cm. Assume canal is trapezium to be unlined with protected banks. Determine suitable cross section of the channel & for soil may be taken as 37° .

→ Solution:

$$Q = 400m^3/s$$

$$d = 5cm = 0.05m$$

$$\phi = 37^\circ$$

$$S = 1/1200$$

Since, the canal banks have protected bank slopes.

The diameter of bed particles given by the relation

$$[d \geq 11RS]$$

$$0.05 \geq 11 \times R \times \frac{1}{1200}$$

$$[\therefore R_{max} = \frac{60}{11} = 5.45m]$$

Provide depth of flow, $D = R$

Taking 20% FOS, so, $D = 0.80R = 4.36 \approx 4.40m$

Assume side slope, of canal $\theta < \phi$ i.e. $\theta = 30^\circ$

$$n = \sqrt{3}$$

$$N = \frac{1}{24} (d)^{1/6} = \frac{1}{24} (0.05)^{1/6} = 0.025$$

For: $D = 400\text{m}$, $m = 1.73$, $N = 0.085$.

$S = 1/2000$, $K = 5.45\text{m}$ (km)

$$V_{max} = \frac{1}{N} \frac{K^{0.33} S^{0.33}}{0.045} = \frac{1}{0.085} \times 5.45^{0.33} \times \left(\frac{1}{2000}\right)^{0.33}$$

$$= 3.57\text{m/s}$$

Area of Section, $A_{min} = \frac{Q}{V_{max}} = \frac{400}{3.57} = 111.79\text{m}^2$

Now, Area of Trapezoidal section,

$$A_{min} = BD + mD^2$$

$$111.79 + 44.84 + 33.49$$

$$\Rightarrow B_{min} = 17.75 \approx 17.80\text{m}$$

- ⑧ Water flows in a wide stream at a depth of 1m having bed slope of 1 in 2000. The mean diameter of the sand particles is 2mm. determine whether the soil grains are stationary or moving and make your comment on scouring or non-scouring of its bed.

→ Solution:

Since, dia of particle is less than 6mm Reynold's no. will be less than 400. Shields' method cannot be applied.

Using Mittal and Swamy's equation for finding critical shear stress.

$$\tau_c = 0.155 + \left[\frac{0.409 d_{mm}^2}{\sqrt{1 + 0.177 d_{mm}^2}} \right] \left\{ \begin{array}{l} \text{water for all} \\ \text{dmm} \end{array} \right\}$$

$$= 0.155 + \left[\frac{0.409 \times 2^2}{\sqrt{1 + 0.177 \times 2^2}} \right]$$

$$= 1.267\text{N/m}^2$$

Again, shear stress produced on bed,

$$\tau_0 = \gamma_w R S = \gamma_w D S = 9810 \times 1 \times \frac{1}{2000}$$

$$\tau_0 = 4.905\text{N/m}^2$$

Here,

$$\tau_0 \gg \tau_c$$

Since, bed shear stress is greater than the critical shear stress, so bed particles would not be stationary but moving and also scouring as well as sediment transportation will occur.

Q) A concrete lined canal has a bed slope of 1 in 5000; carry the discharge of 50 cumecs. Side slope of canal is 1V: 0.75H. Calculate dimension of lined canal section.

→ Since, the discharge of canal is 50 cumecs < 100 cumecs so, triangular section of lined canal has to be designed.

$$\text{Side slope of canal} = \tan \theta = \frac{1}{0.75}$$

$$\therefore \theta = 0.927^\circ$$

$$\text{Area of section, } A = y^2 (\theta + \cot \theta)$$

Wetted perimeter,

$$P = 2y (\theta + \cot \theta)$$

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

$$V = \frac{1}{N} R^{2/3} S^{1/2}$$

$$= \frac{1}{0.015} \times \left(\frac{y}{2}\right)^{2/3} \left(\frac{1}{5000}\right)^{1/2} \quad [\text{for concrete, } N = 0.015]$$

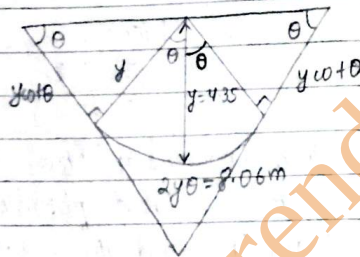
$$[V = 0.593 y^{2/3}]$$

$$A = \frac{Q}{V}; y^2 (\theta + \cot \theta) = \frac{50}{0.593 y^{2/3}}$$

$$\text{or } y^2 (0.927 + \cot 0.927) = 84.32 y^{-2/3}$$

$$\Rightarrow [y = 4.344 \text{ m} \approx 4.35 \text{ m}]$$

(multiple of 5 cm)



Q) Designed a lined canal to carry a discharge of 250 m³/s having longitudinal slope of bed as 1 in 4000, side slope of 1:1 (H:V), $n = 0.015$ for the lining material.

→ Solution, (85)

$Q = 250 \text{ m}^3/\text{s} > 100 \text{ m}^3/\text{s}$, so trapezoidal lined canal has to be designed.

$$\tan \theta = 1/1$$

$$\theta = 0.785^\circ$$

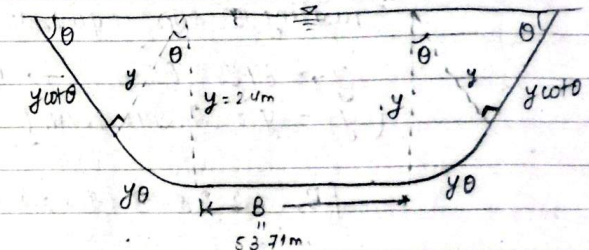
Limiting velocity	P.C.C.	2-2.5 m/s
	Burst clay tile	1.8 m/s
	Boulder/Stone	1.5 m/s

$$A = By + y^2 (\cot \theta + \theta)$$

$$A = By + 1.785 y^2 \quad \text{--- (I)}$$

$$P = B + 2y (\cot \theta + \theta)$$

$$P = B + 3.57 y \quad \text{--- (II)}$$



Trapezoidal lined canal.

Assume limiting velocity in canal be 1.8 m/s for burst clay tile lining.

$$Q = A \times V$$

$$250 = A \times 1.8$$

$$\therefore A = 138.88 \text{ m}^2$$

$$\text{Again, } V = \frac{1}{N} R^{2/3} S^{1/2}$$

$$1.8 = \frac{1}{0.015} \times R^{2/3} \left(\frac{1}{4000}\right)^{1/2}$$

$$\Rightarrow [R = 2.23 \text{ m}]$$

Putting value of A in eqⁿ ①

$$138.88 = 8y + 1.785y^2$$

$$138.88 = 8y + 1.785y^2 \quad \text{--- (3)}$$

$$[P = A/P \Rightarrow P = \frac{A}{P}]$$

$$\therefore P = \frac{138.88}{2.23} = 62.28 \text{ m}$$

Thus from eqⁿ ②

$$62.28 = B + 3.57y$$

$$B = 62.28 - 3.57y \quad \text{--- (4)}$$

Equating ③ & ④, (quadratic in y)

$$y_1 = 64.99 \approx 65 \text{ m} \rightarrow \text{not possible}$$

$$[y_2 = y = 2.394 \approx 2.4 \text{ m}] \rightarrow \text{possible}$$

(+FB)

$$[B = 62.28 - 3.57 \times 2.4 = 53.71 \text{ m}]$$

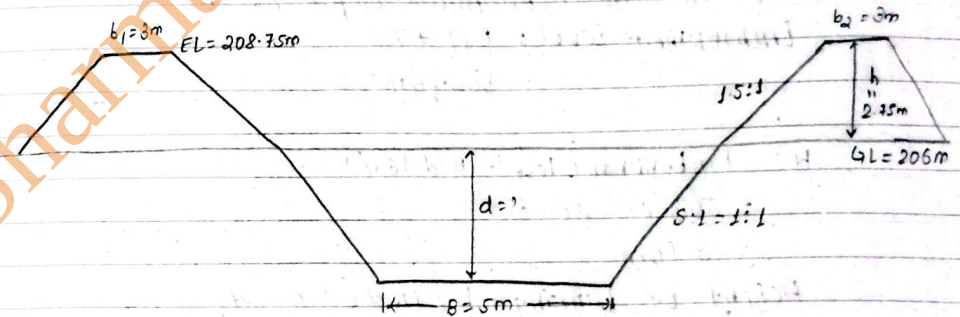
① calculate the economical depth or balancing depth of cut of canal from the following data provided.

Top of Embankment level: 208.75 m

Ground level: 206 m

Top width of embankment for both right & left banks: 3 m
side slopes (H:V) for cutting and banking: $1:1$ and $1\frac{1}{2}:1$ respectively

→ Solⁿ:-



$$\text{Here, } A_{\text{cut}} = Bd + sd^2 = 5 \times d + 1 \times d^2 = 5d + d^2$$

$$\begin{aligned} A_{\text{fill}} &= (b_1 + b_2) \times h + 2S_1 h^2 \\ &= (3 + 3) \times 2.75 + 2 \times 1.5 \times 2.75^2 \\ &= 39.1875 \text{ m}^2 \end{aligned}$$

For balancing depth of cut,

$$A_{\text{cut}} = A_{\text{fill}}$$

$$\text{on } 5d + d^2 = 39.1875$$

$$\Rightarrow [d_1 = -9.24 \text{ m}] \rightarrow \text{not possible}$$

$$[d_2 = 4.24 \text{ m}] \rightarrow \text{possible}$$

$$\therefore \text{Balancing depth of canal} = \underline{4.24 \text{ m}}$$

(12) What will be the possible economical depth of cut of an irrigation canal if bed width is 8m and side slopes 1.5:1 and 2:1 for cutting and banking respectively. The bed level of canal is at 500m, FSL is at 503m and free board is of 0.60m. Take embankment width for left and right banks as 2 and 3m respectively.

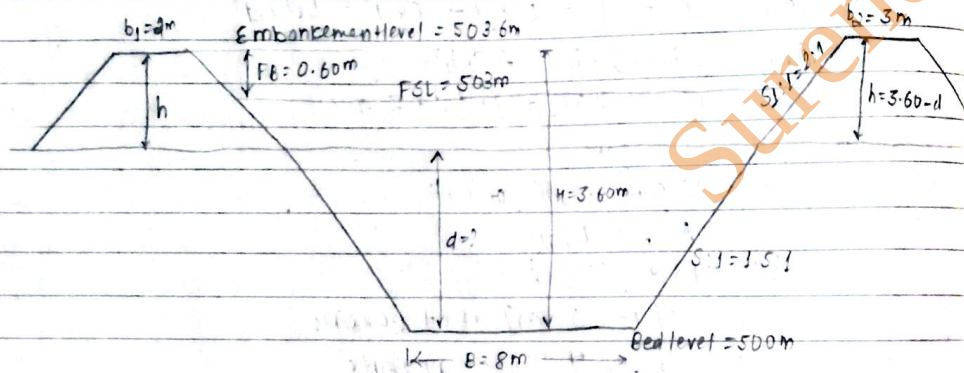
→ Solution,

Let, d be the economical depth of cut

$$\begin{aligned}\text{Embankment level} &= \text{FSL} + \text{FB} \\ &= 503.60 \text{ m}\end{aligned}$$

$$\begin{aligned}H &= \text{Embankment level} - \text{Bed level} \\ &= 503.60 - 500.0 \\ &= 3.60 \text{ m}\end{aligned}$$

$$\text{Height of banking} = h = H - d = 3.60 - d.$$



From the figure above,

$$\text{Area of cut} = Bd + Sd^2 = 8d + 1.5d^2 \quad \text{--- (I)}$$

$$\begin{aligned}\text{Area of fill} &= (b_1 + b_2)h + 2S_2 h^2 \\ &= (2 + 3) \times (3.6 - d) + 2 \times 2 \times (3.6 - d)^2 \\ &= 4d^2 - 33.8d + 69.84 \quad \text{--- (II)}\end{aligned}$$

Equation (I) & (II).

$$d^2 - 16.72d + 27.94 = 0$$

quadratic ind,

$$d_1 = 14.837 \text{ m} > H \rightarrow \text{not possible}$$

$$d_2 = 1.883 \text{ m} \rightarrow \text{possible}$$

Hence, Economical depth of cut = $d = 1.883 \text{ m}$

Chapter 7: Regulating Structures:

Surendra Sharma

Design of distributory Head Regulator & cross-regulator

(a) Crest level

1. Crest level of ^{crest} regulator is kept at u/s bed level of the parent canal.
2. Crest level of distributory head regulator is provided at 0.3m to 1m above the crest of cross regulator.

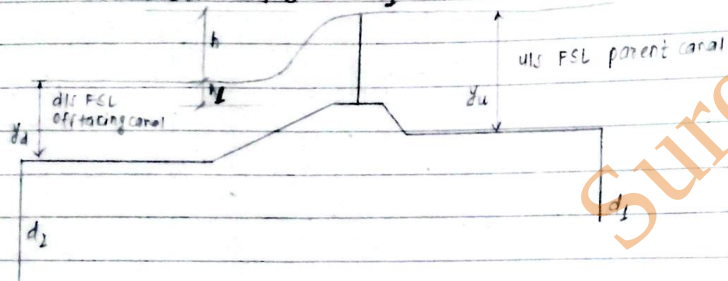
(b) Water way can be worked out using drowned weir formula:

$$Q = \frac{2}{3} C_{d1} \sqrt{2g} B (h + h_v)^{3/2} + C_{d2} B h_1 \sqrt{2g} (h + h_v) \quad *$$

Neglecting velocity of approach, i.e. $V_a = 0$, $h_v = 0$

$$C_{d1} = 0.577 \text{ \& } C_{d2} = 0.80$$

$$\Rightarrow Q = 1.7 B h^{3/2} + 3.54 B h_1 h^{1/2}$$



(c) Vertical cutoff-

$$\text{u/s, } d_1 = \frac{y_2}{3} + 0.60$$

$$\text{d/s, } d_2 = \frac{y_1}{2} + 0.60$$

$\frac{Q}{\text{clearwayway}}$
 $\frac{Q \& H_L}{\text{blend curve}}$

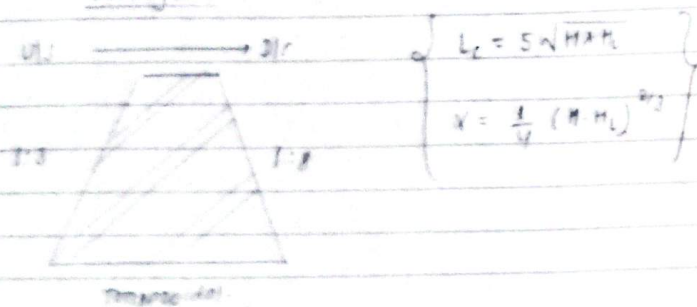
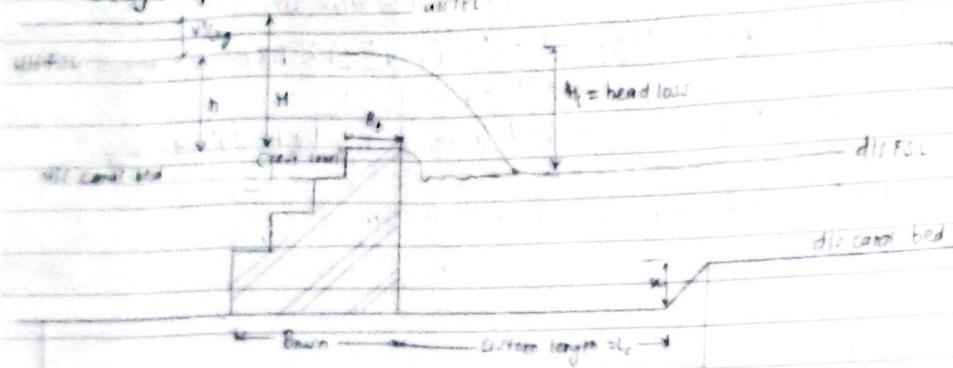
$$\textcircled{a} \text{ Downstream flood level} = \text{d/s FSL} - \textcircled{E f_2} \quad \text{or, d/s FSL} - \text{depth of d/s water} \quad \text{Min}$$

$$\textcircled{b} \text{ d/s floor length} = 5(y_2 - y_1); \frac{2}{3} \text{ of } b \text{ whichever is larger.} \quad \textcircled{b} \rightarrow \frac{4E}{w}$$

d/s floor thickness is calculated by using Khosla's theory.

Min^m floor thickness u/s 0.30 to 0.50m

Design of vertical drop fall / Sarada type fall



S.N.	Particulars	Rectangular crest	Trapezoidal crest
1.	Discharge	upto $14 \text{ m}^3/\text{s}$	Greater than 14 cumecs .
2.	$Q = C_d \sqrt{2g} L H^{3/2} \left(\frac{H}{B_c} \right)^{1/6}$	$C_d = 0.415$, $Q = 1.84 L H^{3/2} \left(\frac{H}{B_c} \right)^{1/6}$	$C_d = 0.45$, $Q = 1.99 L H^{3/2} \left(\frac{H}{B_c} \right)^{1/6}$
3.	Top width of crest wall (B_c)	$0.55 \sqrt{d_c}$ where, d_c = crest ht.	$0.55 \sqrt{(H + d)}$
4.	Base width (Minimum)	$(h + d) / G$; $G = 2$ for masonry $G = 1$ for bare material	$(h + d) / G$; $G = \text{sp. gr. of material}$
5.	Side slope of crest wall	Both faces vertical	Inclined faces; $u/s \rightarrow 1:3$ & $d/s \rightarrow 1:8$
6.	Wing wall u/s	Straight wing wall placed at an angle of 45° from the u/s edge of crest and embedded into berm.	Segmental wing wall with radius, $R = 5$ to 6 times H and subtending angle of 60° at center and carried tangentially into the berm.
7.	Wing wall d/s	Straight up to end of d/s floor and joined to return wall	Straight up to distance 5 to 8 times $\sqrt{H \cdot H_L}$ and warped in a side slope of canal.
8.	Depth of curtain wall	(i) For u/s: $1/3$ of depth of flow (ii) For d/s: $1/2$ of depth of flow	
9.	Length of pitching	(i) For d/s: 3 times depth of d/s water (ii) For u/s: depth of u/s water	
10.	Length of abutment	$L_c = 5 \sqrt{H \cdot H_L}$	
11.	Depth of abutment	$x = \frac{1}{4} (H \cdot H_L)^{2/3}$	
12.	Min. d/s floor length	$L_d = 2(\text{water depth} + 1.2) + H_L$	

+ Floor thickness & Total depth of flow (from crest)

cross regulator $\rightarrow$ parent canal.

Tutorial 7: Canal regulation works:

1. Design a cross regulator for a canal with the data given below.

Discharge of parent canal = $100 \text{ m}^3/\text{s}$

FSL of parent canal u/s & d/s = $195 \text{ m} / 194.7 \text{ m}$

Bed width of parent canal u/s & d/s = $35 \text{ m} / 28 \text{ m}$

Water depth in parent canal u/s & d/s = 2.60 m

Silt factor = 1

Safe exit gradient = $1/6$

$\rightarrow$ Solution:-

$\rightarrow$ (Refer fig at last)

(a) Crest level of cross regulator:

Provide crest level of cross regulator at the u/s bed level of canal = u/s FSL - Depth of flow

$$= 195.0 - 2.60$$

$$= 192.40 \text{ m}$$

(b) Calculation of waterway

discharge passing the cross regulator:

$$Q = 1.7 B h^{3/2} + 3.54 B h_1 h^{1/2}$$

$$\left\{ \begin{array}{l} h_1 = 194.7 - 192.40 = 2.3 \\ h = 195.0 - 194.7 = 0.30 \end{array} \right\}$$

$$\text{or, } 100 = 1.7 B \times (0.30)^{3/2} + 3.54 \times B \times (2.3) \times (0.30)^{1/2}$$

$$\therefore B = \frac{100}{4.739} = 21.10 \text{ m}$$

Provide 3 bays of 8 m each with a clear water way
 $= 3 \times 8 = 24 \text{ m}$

Provide 2 piers of 2 m each = $2 \times 2 = 4 \text{ m}$

Thus, the overall waterway = $24 + 4 = 28 \text{ m}$

(c) Calculation of downstream floor level:
 discharge intensity = $\frac{100}{24} = 4.17 \text{ cumec/m}$

head loss, $H_L = h = 0.30 \text{ m}$

From the Blench curve for $q = 4.17$ and
 $H_L = 0.30$; $Ef_2 = 2.05 \text{ m}$

D/S floor level = d/s FSL - $Ef_2 = 194.70 - 2.05$
 $= 192.65 \text{ m}$

But, d/s bed level = d/s FSL - depth of flow
 $= 194.7 - 2.60$
 $= 192.10 \text{ m}$ which is lower than
 the calculated d/s floor level.

Hence, provide d/s floor at 192.10 m .

(d) Calculation of d/s floor length
 $= 5(y_2 - y_1)$
 $= 5(1.76 - 0.74)$
 $= 5.10 \text{ m}$

for $Ef_2 = 2.05$;

$$Ef_2 = y_2 + \frac{v_2^2}{2g}, \text{ but, } v_2 = q/y_2$$

$$2.05 = y_2 + \frac{(4.17)^2}{y_2^2 \times 2 \times 9.81}$$

$$\Rightarrow [y_2 = 1.76 \text{ m}] \text{ (lower of all)}$$

$$Ef_1 = Ef_2 + H_L = 2.05 + 0.30 = 2.35 \text{ m}$$

y_1 corresponding to Ef_1 is calculated.

$$Ef_1 = y_1 + \frac{v_1^2}{2g} \text{ but, } v_1 = \frac{q}{y_1}$$

$$2.35 = y_1 + \frac{4.17^2}{19.62 \times y_1^2}$$

$$\Rightarrow [y_1 = 0.74 \text{ m}]$$

(e) Vertical cut-off

Provide u/s cutoff for a depth = $\frac{y_2}{3} + 0.6$ u/s water depth.

$$d_1 = \frac{2.6}{3} + 0.6$$

$$= 1.46 \text{ m} \approx 1.50 \text{ m}$$

below the u/s floor or
 bed level of canal

Provide d/s cutoff for a depth = $\frac{y_2}{2} + 0.6 = \frac{2.6}{2} + 0.6$

$$= 1.90 \text{ m}$$

below the d/s floor or
 bed level of canal.

Hence, bottom level of d/s cutoff = $192.10 - 1.90$

$$= 190.20 \text{ m}$$

(f) Calculation of total floor length from exit gradient consideration:

$$q_E = \frac{H_s}{d} \times \frac{1}{\pi \sqrt{\lambda}}; \text{ max } \approx \text{ static head will occur when there is no flow in}$$

$$\frac{1}{6} = \frac{2.90}{1.90} \times \frac{1}{\pi \sqrt{\lambda}}$$

d/s max head (when CR closed & no water in ds)

$$\left\{ \begin{array}{l} H_s = \text{ULS HFL} - \text{d/s bed level} \\ = 195.0 - 192.10 \\ = 2.90 \text{ m} \\ d = d_2 = 1.90 \text{ m} \end{array} \right.$$

$$\Rightarrow \lambda = 8.5$$

$$\lambda = 8.5 = \frac{1 + \sqrt{1 + \alpha^2}}{2}$$

$$\Rightarrow \alpha = 15.97 = \frac{b}{d} < 1.90$$

$$\Rightarrow b = 30.34 \text{ m say } 30.50 \text{ m}$$

$$\text{d/s floor length} = \frac{2}{3} \times b = \frac{2}{3} \times 30.50 = 20.33 \text{ m}$$

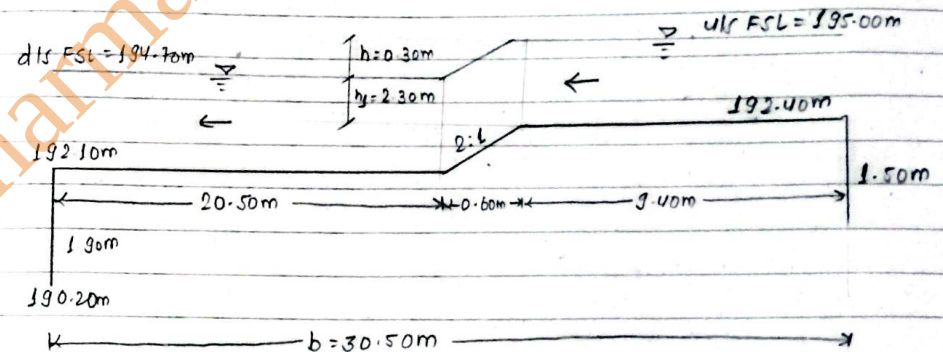
$$= 5(y_2 - y_1) = 5.10 \text{ m} \quad \left. \vphantom{\frac{2}{3} \times b} \right\} \text{ max.}$$

Thus provide d/s floor length of 20.50 m.

Now, let us provide slope of d/s glacis of 2:1.

$$\begin{aligned} \text{The H.L. length under glacis} &= 2 \times (\text{ULS bed level} - \text{d/s bed level}) \\ &= 2 \times (192.40 - 192.10) \\ &= 0.60 \text{ m} \end{aligned}$$

$$\text{The ULS balance for the floor} = 30.50 - 0.60 - 20.50 = 9.40 \text{ m}$$



The thickness of floor is calculated by considering the uplift pressure at key points of cutoff using Khosla's theory. First assume thickness of ULS floor taken as 0.50m and d/s floor as 0.80m and apply corrections for percentage prestresses of key points of cutoff. Plot the H.L. line considering total static head as no flow d/s condition. Find the uplift prestresses at points on floor. Determine concrete floor thickness using formula, $t = \frac{h}{q - 1}$. (Do you recall)

The protection works as inverted filter and launching apron as in previous ways.

$$W_a = 1.5D,$$

D = Scour depth for uls and dls respectively
for uls and dls protections.

$$\text{Thickness of apron} = \frac{2.25D}{W_a}$$

(Not designed distributory head regulator, difference?)

→ uls parent canal FSL & dls distalcing canal FSL

→ crest level different
(fig same as cross regulator)

Surendra Sharma

Sloping glaciers fall $\equiv$ sloping glaciers wear
 $\downarrow$
 on canal bed (FSL) } on river bed (LHFL, PL)

Q2) Design a vertical drop / Sarada fall with the following data provided.

Discharge of canal = $10 \text{ m}^3/\text{s}$,

u/s FSL of canal = 201.50m, u/s bed level = 200m

Drop ht. of canal bed = 1.25m,

$$\text{dis FSL of canal} = 200.95\text{m}$$

Bed width of canal = 9 m,

side slope of canal = 1:1,

Bligh's coefficient = 8.

→ solution:-

Since, discharge of canal is $10 \text{ m}^3/\text{s} < 14 \text{ m}^3/\text{s}$,
hence rectangular crest type fall can be designed.

Using discharge formula,

Using discharge formula,
 $Q = 1.49 L H^{3/2} (H/B_t)^{1/6}$ for rectangular crest

Length of coast, $L = \text{width of canal} = 9 \text{ m}$

Length of coart, $L = \text{width of canal} = 9\text{m}$
 Assuming, top width of coart (B_t) = 0.80m { later B_t checked with actual value }
{ 4% FSL = 201.50m }

Then

Then,

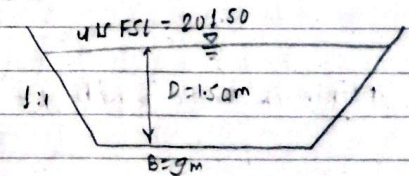
$$10 = 1.84 \times 9 \times H^{3/2} \times \left(\frac{H}{0.86} \right)^{1/6}$$

$$[H = 0.722 = 0.72 \text{ m}]$$

Crosssectional area of canal,

$$A = BD + nD^2$$

or $A = 9 \times 1.5 + 1 \times 1.5^2 = 15.75 \text{ m}^2$



Velocity of approach,

$$V_a = \frac{Q}{A} = \frac{50}{15.75} = 0.634 \text{ m/s.}$$

$$\text{velocity head} = \frac{V_a^2}{2g} = 0.02 \text{ m}$$

$$\text{EL of ulr TEL} = 201.50 + 0.02 = 201.52 \text{ m}$$

$$\begin{aligned} \text{EL of crest} &= \text{TEL} - H = 201.52 - 0.72 \\ &= 200.80 \text{ m} \end{aligned}$$

$$\begin{aligned} \text{dlr bed level} &= \text{ulr bed} - \text{draught} \\ &= 200 - 1.25 \\ &= 198.75 \text{ m} \end{aligned}$$

$$\begin{aligned} \text{Ht. between crest \& dlr bed} &= d = 200.80 - 198.75 \\ &= 2.05 \text{ m} \end{aligned}$$

Now,

$$\begin{aligned} \text{Top width of crest, } B_t &= 0.55\sqrt{H} = 0.55 \times \sqrt{2.05} \\ &= 0.787 \text{ m} \\ &\approx 0.8 \text{ m} \end{aligned}$$

Hence, provide crest width of 0.8 m,

$$\begin{aligned} \text{Ht. of water above crest (h)} &= \text{ulr FSL} - \text{crest level} \\ &= 201.50 - 200.80 \\ &= 0.70 \text{ m.} \end{aligned}$$

$$\begin{aligned} \text{Minimum base width of crest wall} &= \frac{(h+d)}{4} \\ &= \frac{(0.70 + 2.05)}{4} \\ &= 1.375 \text{ m.} \end{aligned}$$

hence, provide base width of 1.50 m.

Wing walls:

→ Provide 40 cm thick ulr wing walls straight at an angle of 45° from the ulr edge of crest wall and shall be embedded into the beam by 1 m.

→ Similarly, on dlr side 40 cm thick wing walls are kept straight and parallel upto end of the floor and joined to return wall and with a slope equal to canal side slope 1:1 towards water side.

ulr protection:-

(a) Brick pitching (ulr ulr ulr ulr depth E),

On ulr side brick pitching equal to depth of flow is laid on ulr bed i.e. 1.50 m length of pitching towards the crest wall at slope of 1:10 and provided with 3 drain pipes of 15 cm diameter so that draining of water during closure of the canal from the ulr bed.

(b) ulr curtain wall / cutoff wall:

$$\text{Minimum depth of ulr curtain wall} = \frac{D_y}{3} = \frac{1.50}{3} = 0.50 \text{ m}$$

Hence, provide 0.40 m thick and 0.80 m deep curtain wall on ulr side including over 30 cm CC bed.

Design of cistern:

$$\begin{aligned} \text{Length of cistern, } L_c &= 5\sqrt{H_1 - H_2} = 5\sqrt{0.72 + 1.25} = 4.74 \text{ m} \\ H_1 &= \text{ulr FSL} - \text{dlr FSL} \end{aligned}$$

Hence, provide cistern length of 5 m.

Depth of stream below dr bed = $x = \frac{1}{4} (H H_0)^{2/3}$
 $= \frac{1}{4} (0.92 \times 1.25)^{2/3}$
 $= 0.233 \text{ m}$

Hence, provide cistern depth of 0.25m.

Design of floor:

Maximum static head, $H_s = d = \text{crest level} - \text{dr bed level of canal}$
 $= 2.05 \text{ m.}$

(1c) Total floor length required = $C_b \times n_s \leftarrow \underline{\text{Bligh's}}$
 $= 8 \times 2.05$
 $= 16.40 \text{ m}$

Minimum d/c floor length required
 $= 2 \times (\text{water depth} + 1.2) + H_L$
 $= 2 \times (1.50 + 1.20) + 1.25$
 $= 6.65 \text{ m}$

✓ Provide dir floor of 7m and rest of 9.40m under and ult side of coast wall.

↳ Floor thickness:

Max^m unbalance uplift at the dis toe of crest

$$= x + \frac{\text{EL of crest} - \text{EL of distrib of canal}}{\text{total floor length}} \times \text{dist length of floor}$$

$$= 0.25 + \frac{2.05}{16.40} \times 7 = 1.125\text{m.}$$

3/ Thickness required = $(1.125) / (6.24 - 1) = 0.907m = 0.95m$ say.
 Provide concrete floor thickness of 0.95m including 0.20m of brick pitching over it.

4/ Unbalanced head at 3m dls of crest top = $0.25 + (2.05 / 6.40) \times 4 = 0.75m$
 Provide floor thickness of 0.95m including 0.20m brick pitching.
 Unbalanced head at 5m dls = $0.25 + (2.05 / 6.40) \times 2 = 0.50m$
 Provide floor thickness of 0.30m with brick pitching of 0.20m over it.

5/ Provide 4th floor of 30cm thick of 4th of crest wall.

($D_d \rightarrow$ length of dis. depth)

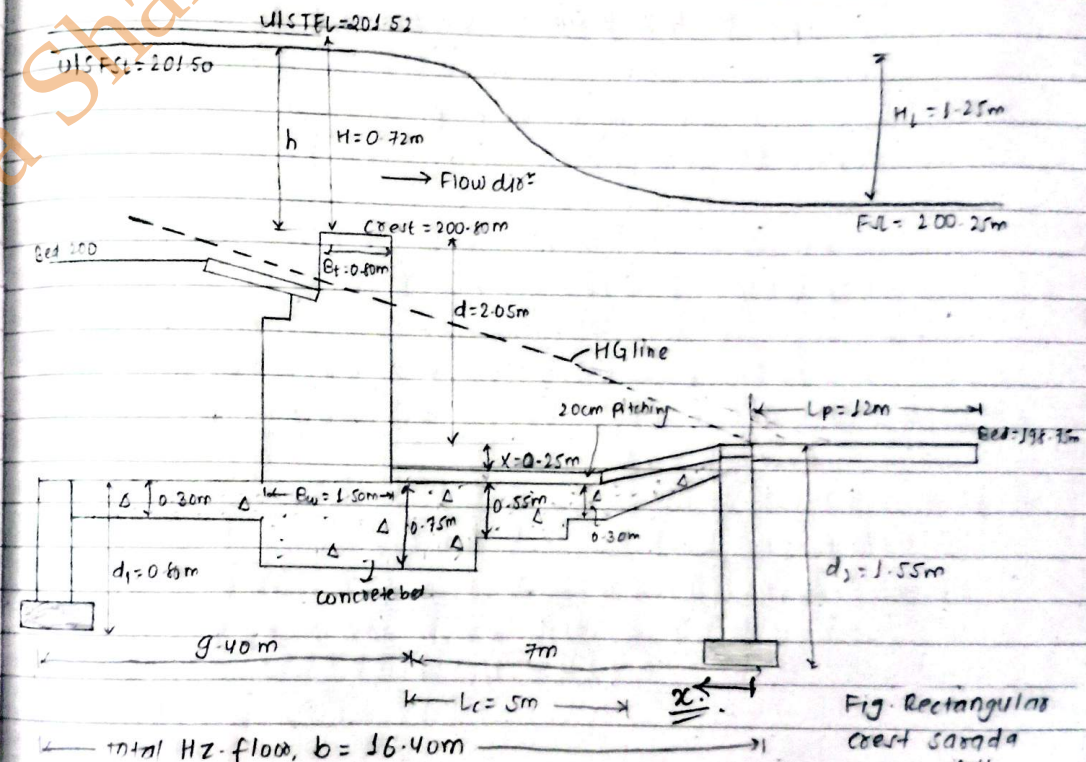


Fig. Rectangular
coast sand dune
fall.

$D_d \rightarrow$ depth of floor

$$\text{Depth of u/s curtain wall/cutoff wall} = \frac{D_d}{2} = \frac{1.50}{2} = 0.75 \text{ m}$$

Provide 0.40m thick * 1.55 deep curtain wall including 30cm cc base. $1.55 = 0.30 + 0.75 + 0.30 + 0.20$
(From table) - For $H: 0.7 \sim 0.8 \text{ m}$

$$\text{Total length of d/s pitching} = \underline{9 + 2 \times D_d} = 9 + 2 \times 1.5 = 12 \text{ m,}$$

which is aligned from the end of curtain wall after concrete floor d/s considering according to head over crest h , if table is not provided then,

$$L_p = 3 \times D_d = 4.5 \text{ m}$$

(83) Design a 1.5m Sabada fall for a canal carrying the discharge of $35 \text{ m}^3/\text{s}$ with the data provided below.

Bed level u/s = 500.50m ; side slope of canal = 1.5:1
Bed level d/s = 499.00m ; FSL u/s = 502.50m, FSL d/s = 501.00m
Berm level u/s = 503.00m ; canal bed width ~~at u/s & d/s~~ = 24m.
Safe exit gradient for Khosla theory = 1/5

→ Solution:-

(a) Design of crest wall (Refer table)

Length of crest, L = canal bed width = 24m

Since, discharge of flow is greater than $14 \text{ m}^3/\text{s}$, trapezoidal crest can be designed.

$$\text{Discharge } Q = 1.49 L H^{3/2} (H/B_t)^{1/6}$$

Assume,

top width of crest $B_t = 1 \text{ m}$

$$35 = 1.49 \times (24) \times H^{3/2} \times (H/1)^{1/6}$$

$$\Rightarrow [H = 0.83 \text{ m}]$$

$$\text{Velocity of approach, } V_a = \frac{Q}{A} = \frac{35}{(24 \times 2 + 1.5 \times 2^2)} = 0.65 \text{ m/s}$$

$$\text{Velocity head} = \frac{V_a^2}{2g} = \frac{0.65^2}{2 \times 9.81} = 0.022 \text{ m}$$

$$\text{u/s TEL} = \text{u/s FSL} + \text{velocity head} = 502.50 + 0.022 = 502.522 \text{ m}$$

$$\text{RL of crest} = \text{u/s TEL} - h = 502.522 - 0.83 = 501.692 \text{ m}$$

$$\begin{aligned} \text{Height of crest above d/s bed (d)} &= \text{Crest level} - \text{bed level d/s} \\ &= 501.692 - 499.00 \\ &= 2.692 \text{ m} \end{aligned}$$

(b) Wing wall

✓ U/s wing wall radius $R = 5H$ to $6H$
 $= 5 \times 0.83$ to 6×0.83
 $= 4.15 \text{ m to } 4.98 \text{ m}$

→ segmental wing wall of radius 4.50 m subtending angle of 60° at center which shall be carried tangentially into the beam 1 m can be provided at u/s side.

✓ → d/s wing wall shall be kept straight with a distance of $6\sqrt{H \times H_c} = 6\sqrt{0.83 \times 1.50}$
 $= 6.69 \approx 7 \text{ m}$.

then wrapped in a slope of $1.5:1$ and taken upto the end of concrete floor.

(c) u/s protection

✓ ⇒ Brick pitching of length equal to u/s depth of water $= 2 \text{ m}$ is laid in slope of $1:10$ towards the crest, provide 20 cm dia. drain pipes of 3 nos at u/s bed for draining u/s bed during the closure of canal.

⇒ Curtain wall

(1) u/s curtain wall shall be provided to a minimum depth of $1/3$ of u/s water depth
 $= 1/3 \times 2 = 0.67 \approx 0.7 \text{ m}$.

✓ Provide curtain wall of 0.40 m thick of 1 m deep laid over 0.30 m concrete foundation bed.

✓ (ii) Minimum depth d/s curtain wall $= 1/2$ of depth of water
 $= 2/2 = 1 \text{ m}$

Provide d/s curtain wall of $0.40 \text{ m} \times 1.80 \text{ m}$ including 0.30 m cement concrete foundation.

Total depth of d/s cutoff $d_2 = 1.10 \text{ m}$

$$= 1 + 0.20 + 0.30 + 0.30$$

Provide d/s brick pitching of $3 \times D_d = 3 \times 2 = 6 \text{ m}$.

(d) Design of cistern.

(i) Length of cistern, $L_c = 5\sqrt{H \times H_c} = 5\sqrt{0.83 \times 1.50}$
 $= 5.58 \text{ m}$

Provide cistern length of 5.60 m .

(ii) Depth of cistern, $X = 1/4 (H \times H_c)^{2/3} = 1/4 \times (0.83 \times 1.50)^{2/3}$
 $= 0.289$
 $\approx 0.30 \text{ m}$

(e) calculation of total floor length.

Exit gradient, $G_E = \frac{H_c}{d_2} \times \frac{1}{\pi \sqrt{\lambda}}$ ($d \text{ } H_c = d$)

$$1/5 = \frac{2.692}{1.80} \times \frac{1}{\pi \sqrt{\lambda}}$$

$$\Rightarrow \lambda = 5.67 = \frac{1 + \sqrt{1 + \alpha^2}}{2}$$

$$\Rightarrow \alpha = 10.29 = b/d_2 = 1 [b = 18.52 \text{ m}]$$

Provide total floor length of 19m.

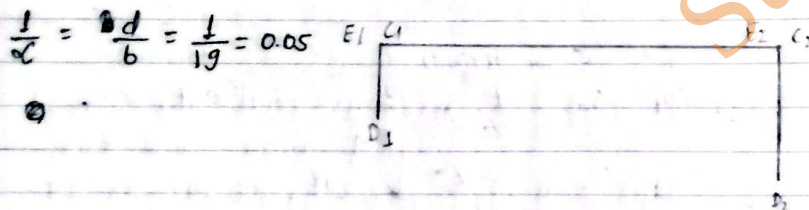
$$\begin{aligned}\text{Minimum floor length required on dlr side} &= 2(\text{water depth} + 1.2) + H_L \\ &= 2(2 + 1.2) + 1.50 \\ &= 7.90\text{m} \approx 8.0\text{m say.}\end{aligned}$$

Provide the balance of floor of 11m under and ulr of crest wall.

(f) Calculation of uplift pressures and thickness of floor.

Assuming ulr floor thickness of 0.50m and dlr floor thickness of 0.80m and floor thickness at toe be 1.50.

(i) For ulr wall: end pile condition
 $d = 1\text{m}$, $b = 19\text{m}$



From Kharla curve for end pile condition

$$\Phi_{E1} = 100\%$$

$$\Phi_{D1} = 17.5\%, \quad \Phi_{C1} = 100 - \Phi_{D1} = 82.5\%$$

Graph

** Assuming floor thickness of 0.50m at ulr cutoff and 0.80m at dlr cutoff.

$$\Phi_E = 27.5\%, \quad \Phi_{C1} = 100 - \Phi_E = 72.5\%$$

(ii) For dlr wall: end pile condition

$$d = 1.80\text{m}, \quad b = 19\text{m}, \quad \frac{1}{\alpha} = \frac{d}{b} = \frac{1.80}{19} = 0.09$$

$$\Phi_{E2} = \Phi_E = 27.5\%$$

$$\Phi_{D2} = \Phi_D = 18\%, \quad \Phi_{C2} = 0\%$$

} Graph

Since interference correction is very small and neglected.

Points	Ulr wall			Dlr wall		
	E1	D1	C1	E2	D2	C2
% Pressures	100	82.5	79	27.5	18	0
Thickness corr ⁿ	-	-	1.75	-4.22	-	-
Corrected % pressure	100	82.5	80.75	23.28	18	0
Static head = 2.692m	2.692	2.22	2.13	0.63	0.48	0
Head loss	0	0.472	0.562	2.062	2.212	2.692
Elevation of H _g line	501.692	501.22	501.13	499.63	499.48	499.00

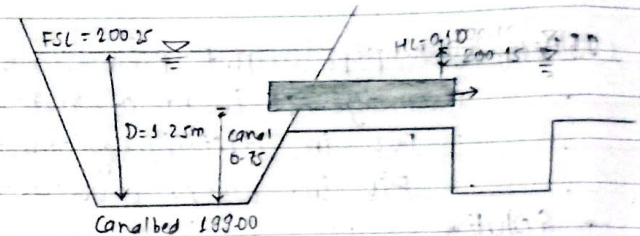
(Q4) Design a pipe outlet from the following data.

Discharge of outlet = 65 litres/s = 0.065 cumecs

FSL of canal = 200.25m, FSD of canal = 1.25m

Water level in water course = 200.15m.

→ Solution:-



The head loss b/w canal and water course = $200.25 - 200.15 = 0.10$
 which is less than 0.20 depth of flow = $0.20 \times 1.25 = 0.25m$ depth.

Hence, submerged pipe outlet has to be designed.

$$Q = C_d A \sqrt{2gH_L} \quad \left[\text{Min}^{\text{m}} \text{ head loss } 0.20D \text{ for proper functioning} \right]$$

$$0.065 = 0.80 \times A \times \sqrt{2 \times 9.8 \times 0.10}$$

$$\Rightarrow A = 0.058m^2$$

Let, diameter of pipe be d

$$A = \pi d^2/4$$

$$\Rightarrow d = 0.27m$$

Hence, provide pipe of diameter 30cm.

The pipe can be fixed at 20cm below FSL of canal. i.e. the sill level of pipe is fixed at $200.25 - 0.20 - 0.30 = 199.75\text{m}$
 i.e. $199.75 - 199.00 = 0.75\text{m}$ above bed level of canal.

(95) Design a pipe outlet from data provided.
 Discharge at head of water course = $0.10\text{m}^3/\text{s}$
 FSL in distributary canal = 503.00m
 FSL in water course = 502.00m

→ Solution.
 Available head across the outlet = $H_1 = 503 - 502 = 1.00\text{m}$

★ This head is sufficient to provide outlet discharge fully into the water course. ∴, dis end of the pipe can be fixed below the water level of the water course making it semi-module.

Discharge through pipe outlet = $Q = C_d A \sqrt{2g H_0}$

Assuming diameter of pipe be 25cm.

We have,

$$0.10 = 0.62 \times \frac{\pi \times 0.25^2}{4} \times \sqrt{2 \times 9.81 \times H_0}$$

$$(H_0 = 0.55\text{m})$$

↑
 working head.
 (pipe center below FSL of canal)

Now, RL of center of pipe outlet = $503.00 - 0.55 = 502.45\text{m}$

~~800~~ Sill level of pipe outlet = $502.45 - \frac{0.25}{2} = 502.325\text{m}$.

Hence, pipe of 25cm dia can be laid horizontally with its sill/bottom level at 502.325m .

