UNIT-Ist

CANAL REGULATION WORKS AND CROSS STRUCTURES

Introduction → The structures (or Masonry works) constructed on a canal to control and to regulate discharge velocity, depth etc. are known as canal regulation works. These structures are required for proper and efficient functioning of an irrigation canal system. The water enters the main canal through a head regulator installed at the canal headworks is distributed into different branches and distributaries. To distribute the water effectively, the discharge is regulated in these smaller channels.

* Types of canal falls →

The following are the different types of canal falls that may be adopted according to the site condition →

① ogee fall ② Rapid fall ③ Stepped falls.
④ Trapezoidal Notch fall. ⑤ Vertical drop fall.
⑥ Straight glacis falls. ⑦ Modern glacis fall
⑧ Inglis fall ⑨ Montague fall.
⑩ Sarda fall.

* canals falls → The canals falls are required when the natural slope of the ground along the channel alignment is steeper than the bed slope of the channel. The difference in slopes of ground and canal is adjusted by providing falls in the bed of the channel at suitable points. The exact locations of a fall depend upon a number of factores. The following factors should be considered while deciding the location of the fall.

① The site for the fall in the case of main canals and branches, from which no direct irrigation is done, is usually selected from the consideration of economy of earthwork. As far as possible, the canal should be kept in the balanced depth of cutting. If the fall is not provided, the canal would go in excessive filling which is not desirable from the consideration of the economy of earthwork and the maintenance of the canal.

② The site for the fall in the case of distributaries from which direct irrigation is done is usually decided in such a way that the command is not sacrificed in the process of lowering of the water level.

③ For locating the fall, it is first necessary to fix the F.S.L required at the head of all offtake channels and outlets and mark them on the L-Section of the canal on which the fall is to be located. The F.S.L of the canal is then marked so that it covers all the commanded points and allows for a minimum working head of about 0.3m for the regulators of the offtaking channels and 0.15m for all outlets. The falls are then located at the points wherever actual F.S.L of the canal is much greater than the F.S.L required.

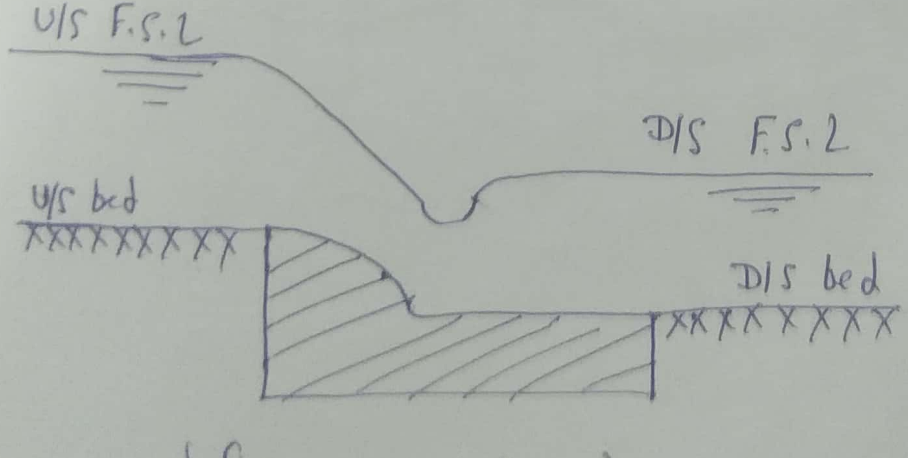
④ The location of a fall may also be decided from the consideration of the possibility of combining it with a cross - regulator, a road bridge or any other masonry work to effect economy and to have better regulation.

⑤ Relative economy should be considered while deciding the number of falls and the drop in each fall. In a given reach, if the drop of each fall is increased, the number of falls is decreased and vice versa. generally the provision of a large number of small falls results in economy of earthwork but the cost of fall structures is increased. on the other hand the provision of a small number of large falls results in extra cost of earthwork, but the cost of fall structures is decreased. The combination which gives the minimum overall cost subject to the condition that the command is not reduced should be selected.

⑥ Sometimes it may be necessary to provide fewer falls of large drops to enable hydropower generation at these falls.

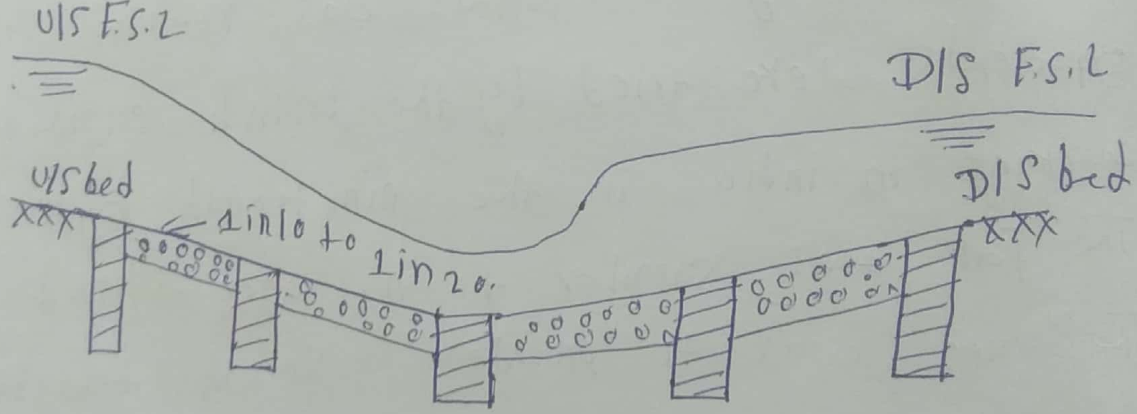
(a) <u>ogee-type fall</u> →

When the necessity of providing falls on irrigation canals was realised different expedients were tried by the british engineers working in india in the nineteenth century. The fall was similar to an ogee-shaped spillway. The aim was to provide a smooth transition from the U/P to D/S water levels and to avoid disturbance and to reduce impact.



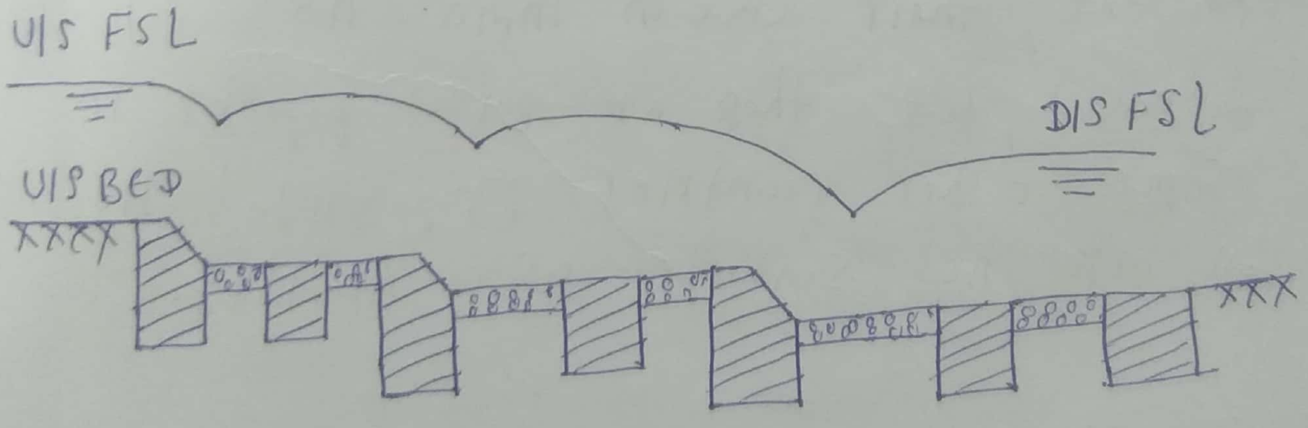
(fig - ogee-fall)

② **Rapid fall** → The rapid fall were provided with long sloping floors (called glacis) having gentle slopes in the range of 1 in 10 to 1 in 20. The long glacis ensured the formation of a hydraulic jump for the dissipation of energy. However, the falls could not become popular because of their high cost of construction.



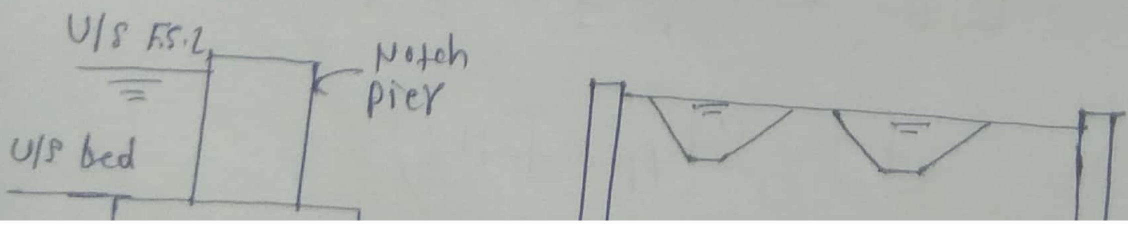
(fig – Rapid fall)

Stepped falls → The stepped falls were the modified forms of the rapid fall in which the long glacis was replaced by a long stepped floor. This type of fall also could not become popular because of high cost of construction. After the development of stepped falls it was recognised that the dissipation of energy can be achieved through vertical impact of the falling jet of water on the floor.

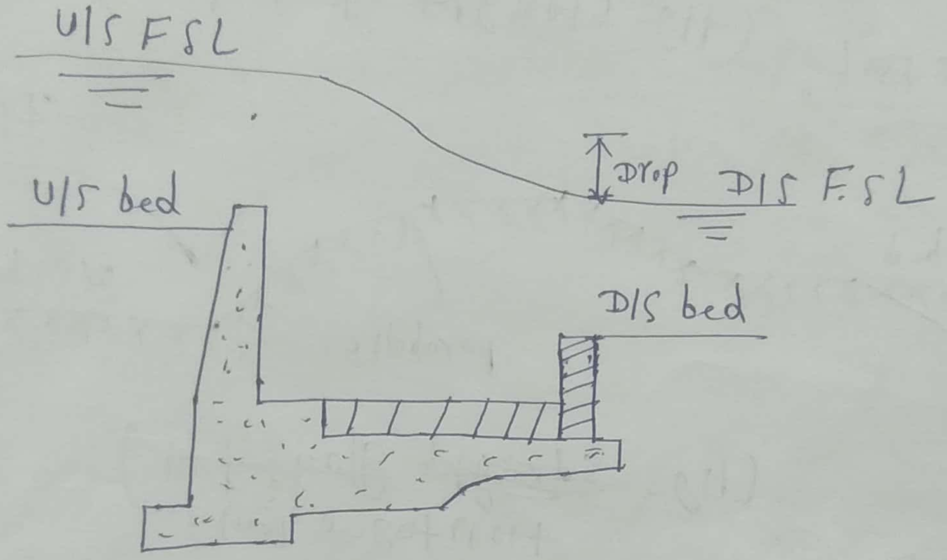


(fig - Stepped falls)

(4) Trapezoidal notch fall → A trapezoidal notch fall consists of a number of trapezoidal notches in a high breast wall, called notch pier, constructed across the channel. There is a smooth entrance to the notches. The notches of the falls were designed to maintain the normal depth of flow in the channel upstream of the fall at any two values of the discharge. The trapezoidal notch falls become quite popular in india and abroad. However, since the development of other types of falls which are more economical and more efficient, their use in india has considerably decreased, but these are quite popular in some other countries. Trapezoidal notch fall are not suitable as a matter.



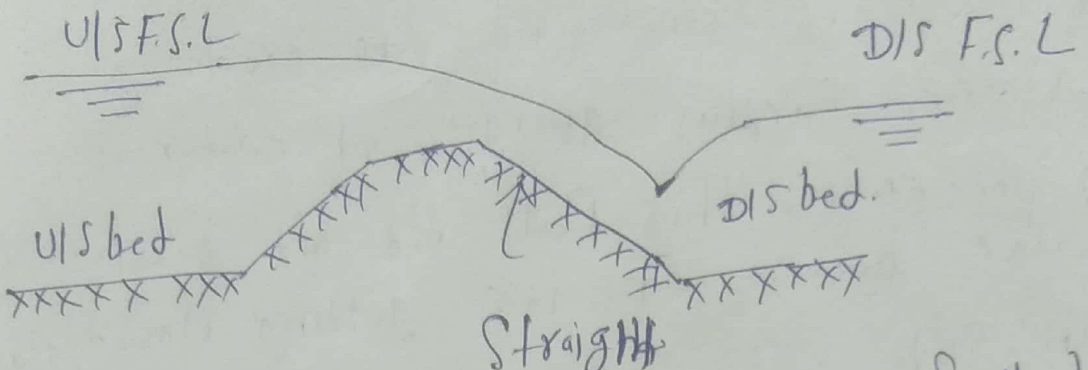
(5) Vertical drop fall → In a vertical drop fall, a crest wall is constructed to create a Vertical drop Surplus energy of water leaving the crest. This fall did not become popular because of its getting closed/clogged with silt.



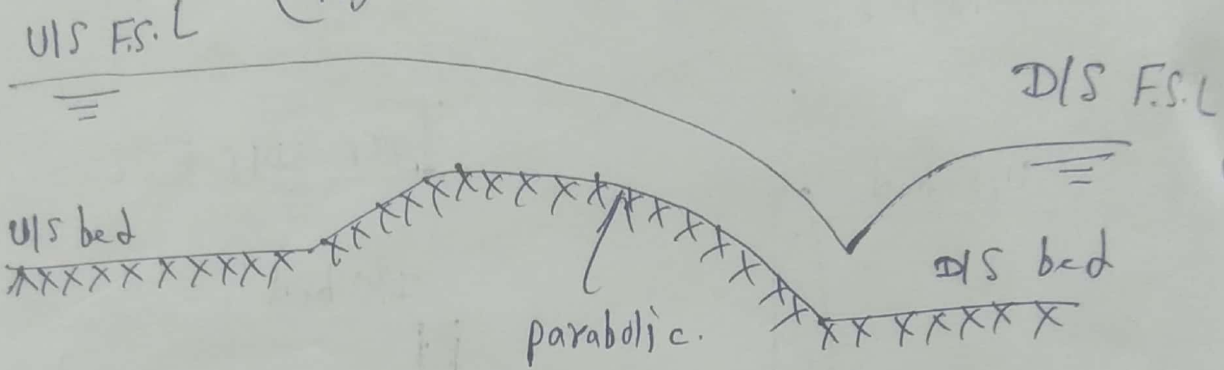
(fig - Vertical drop fall).

(6) Straight glacis fall → After the first world war, a large number of Major canal projects were taken up in india. The need was felt to develop new types of falls for large discharge and large drops. It utilises the formation of a hydraulic jump for the dissipation of energy. However it is not possible to dissipate the energy and considerable surplus energy is still left even after the formation of the hydraulic jump.

⑫



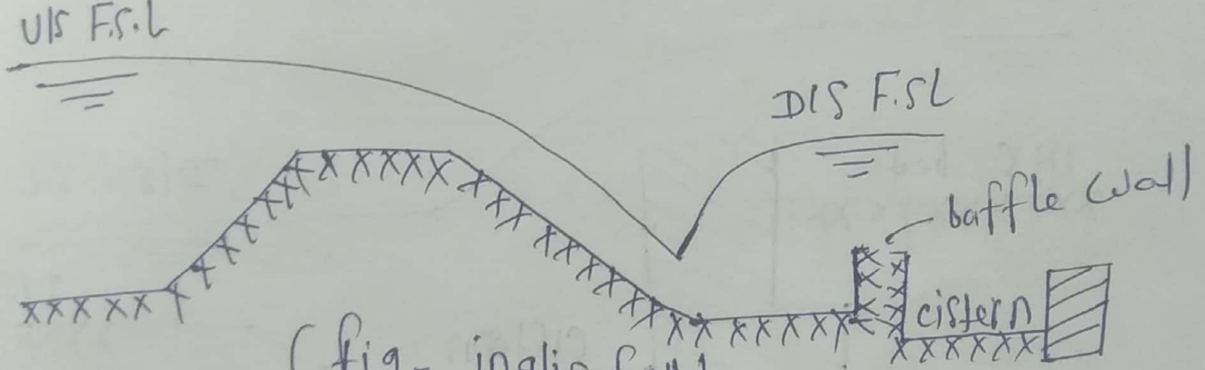
(fig- straight glacis fall)



(fig. ~~Straight glacis fall~~ Montague fall)

⑦ Modern glacis ~~Montague~~ fall → The old glacis type fall has been considerably modified in recent times in respect of slopes of glacis and spacing location and design of friction blocks and chute blocks for more efficient dissipation of energy. These falls are commonly used in practice, especially when the drop is large.

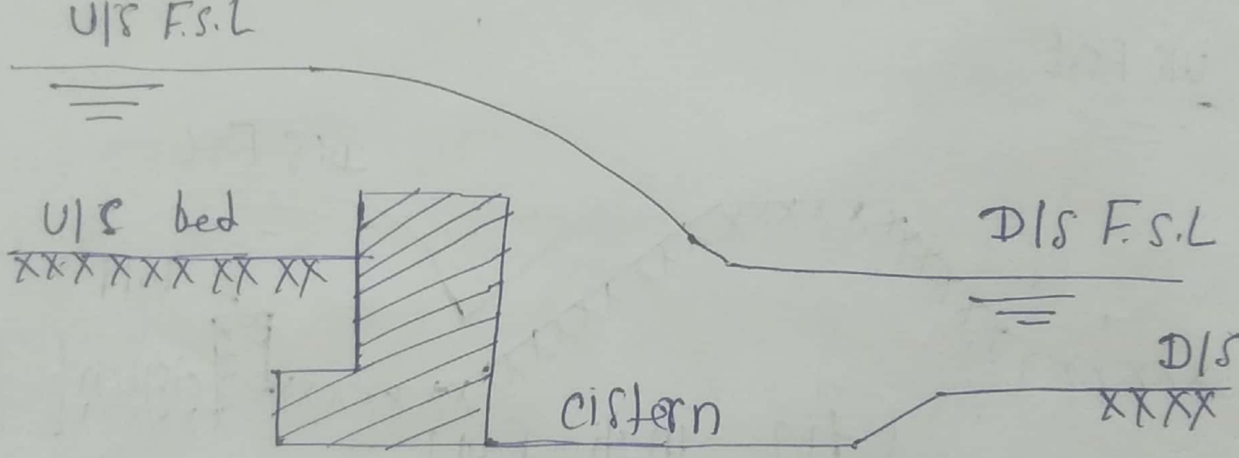
(6) <u>Inglis fall</u> → The inglis fall is also a modified form of the straight glacis fall. In this type of fall a baffle wall of a certain height is provided at some distance d/s of the toe of the glacis. The baffle wall ensures the formation of the hydraulic Jump on the baffle platform and effective dissipation of energy.



(fig - inglis fall)

(7) <u>Montague fall</u> → The Montague fall is a modified form of the glacis fall. In this type of fall a parabolic glacis, known as the Montague profile is provided. This shape gives the maximum horizontal acceleration to the jet of water in a given length of the glacis. However, this fall did not become popular because of the problems encountered in the construction of the curved glacis.

(10) Sarda fall → on the sarda canal in U.P. the raised crest fall, now known as sarda type fall, was evoled. The type of fall is quite suitable for small drops. In this type of fall, the nappe leaving the crest wall impinges into water in the cistern, resulting in the destruction of energy.



(fig- Sarda type fall).

* Functions of Regulator →

The regulators are required on a channel to regulate and control the supply of water. The functions of the distributary head regulator and cross-regulator are below →

* Functions of a distributary head regulator →

A distributary head regulator serves the following main purposes →

1) It regulates the supply of water from the parent channel to the offtaking channel.
2) It controls the entry of silt into the offtaking channel.
3) It can serve as a meter for the measurement of discharge.
4) It is used for shutting off the supply into the offtaking channel; when water is not needed or when the offtaking channel is required to be closed for repairs or maintenance.

functions of cross regulator's →

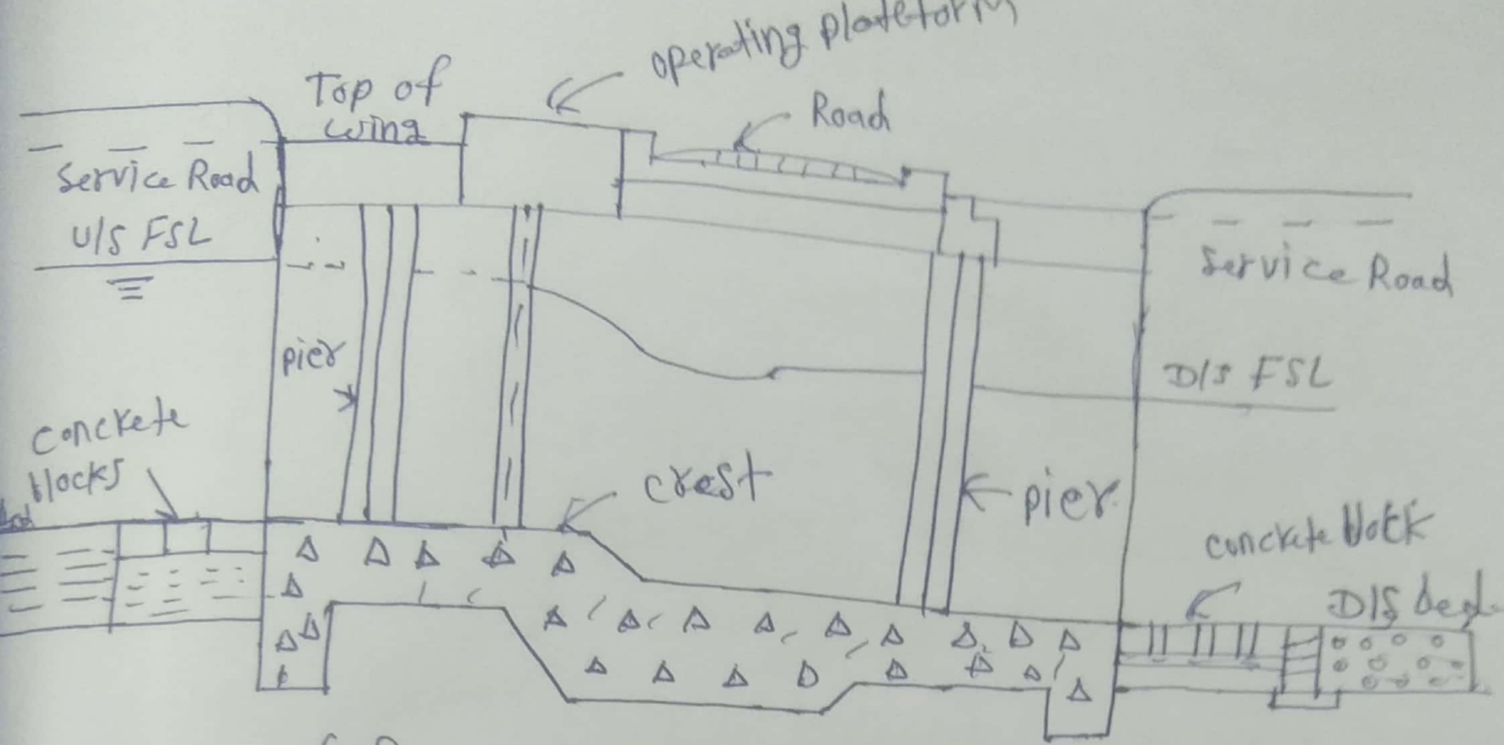
1. The main functions of a cross regulator is to raise the water level in the parent channel on the upstream so that the offtaking channel can take its full supply even when the water level in the parent channel is lower than F.S.L.

2. It is also used to close the supply in the parent channel on its D/S. The supply in that case is usually directed to other channels. If an escape is also provided in conjunction with a cross-regulator, the water can be diverted to the escape channel.

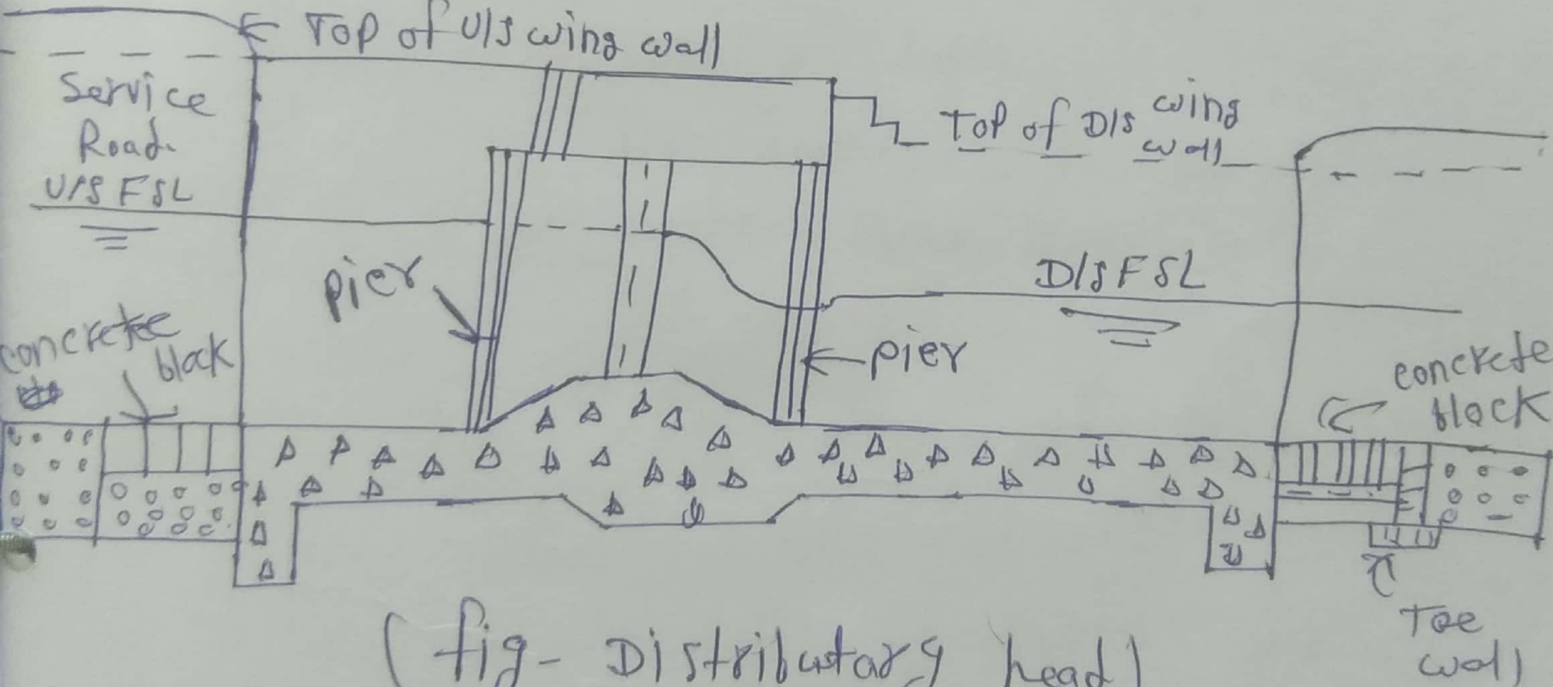
3. There is usually a bridge on the cross-regulator, which provides a means of communication.

4. It helps absorb fluctuations in the various sections of the canal system and thus prevents breaches in the tail reaches.

5. cross regulator's are useful for effective regulation of the entire canal system. In a good canal system, a large number of cross-regulators are usually provided.



(fig. cross-Regulator).



(fig- Distributary head Regulator).

* **cross-drainage work** → A cross-drainage work (also called CD-work) is a structure built on a canal where it is crosses a natural drainage. Such a stream or a river. Sometimes a cross-drainage work is required when the canal crosses another canal. The cross-drainage work is required to dispose of the drainage water so that the canal supply remains uninterrupted. A cross drainage work is also called as drainage crossing. The canal at a cross-drainage work is generally taken either over or below the drainage. However, it can also be at the same level as the drainage.

* Selection of a suitable type of cross-drainage work → The following factor's should be considered while selecting the most suitable type of the cross-drainage work →

① **performance** → As far as possible, the structure having an open channel flow should be preffered to the structure having a pipe flow.

(2) provision of road → A aqueduct is better than a Superpassage because in the former, a road bridge can easily be provided along with the canal trough at a small extra cost, whereas in the latter, a ~~large~~ separate road bridge is required.

(3) size of drainage → when the drainage is of small size, a syphon aqueduct will be preffered to an aqueduct as the latter involves high banks and long approaches However, if the drainage is of large size an aqueduct is prefffered.

(4) cost of earth work → The type of cross-drainage work which does not involve a large quantity of earth work of the canal should be preffferred.

(5) Foundation → The type of cross-drainage work should be selected depending upon the foundation able at the site of work.

6) **Material of construction** → Suitable type of material of construction in sufficient quantity should be available near the site for the type of cross-drainage work selected. Moreover, the soil in sufficient quantity should be available for constructing the canal banks if the structure requires long and high canal banks.

7) **Cost of construction** → The cost of construction of cross-drainage work should not be excessive.

8) **Overall cost** → The overall cost of the canal banks and the cross-drainage work including maintenance cost should be a minimum.

9) **Permissible loss of head** → Sometimes, the type of cross-drainage is selected considering the permissible loss of head. For example if the head loss cannot be permitted in a canal at the site of cross-drainage, a canal syphon is ruled out.

10) **Subsoil water table** → If the subsoil water table is high, the types of cross-drainage which requires excessive excavation should be avoided, as it would involve dewatering

* Types of cross-drainage works →

Depending upon the relative positions of the can[al] and the drainage, the cross-drainage works may be broadly classified into 3 categories.

① Canal over the drainage
(i) Aqueduct (ii) Syphon aqueduct.

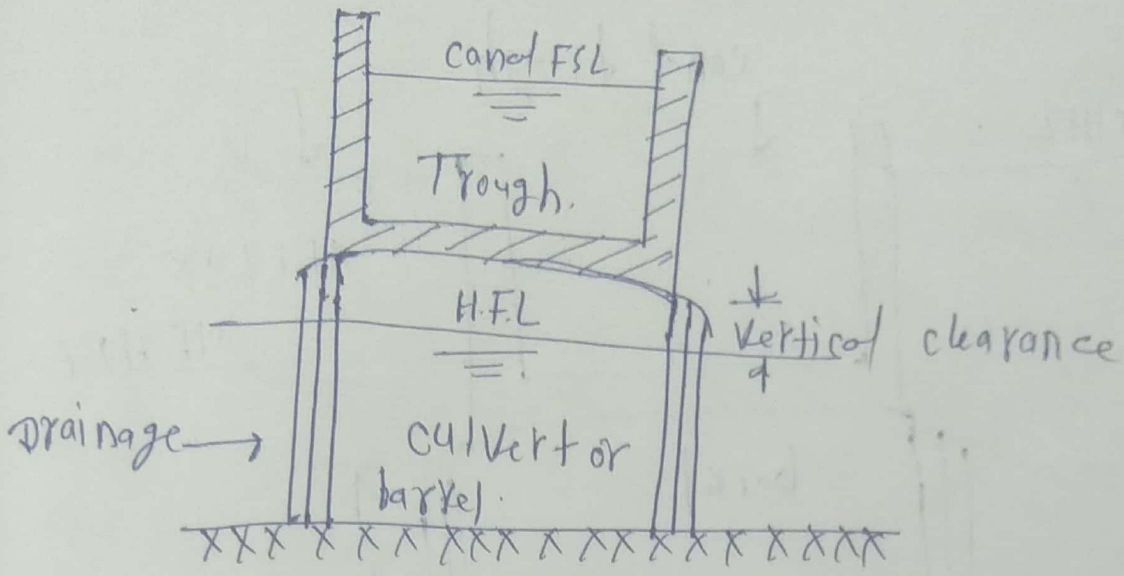
② canal below the drainage.
(i) Superpassage (ii) canal syphon.

③ canal at the same level as drainage
(i) Level crossing (ii) Inlet (iii) Inlet and outlet.

(ii)

① canal over the drainage →

(i) Aqueduct → An aqueduct (also called an ordinary aqueduct) is a structure in whic[h] the canal flows, over the drainage and the flow of the drainage in the barrel is open (channel flow). An aqueduct is similar [to] an ~~drainage~~ ordinary road bridge (or railway bridge) across a drainage, but in this case, the canal is taken over canal is taken over the drainage in a trough supported over the piers constructed on the drainage bed. An aqueduct is provided when the canal bed level is higher than H.F.L of the drainage.

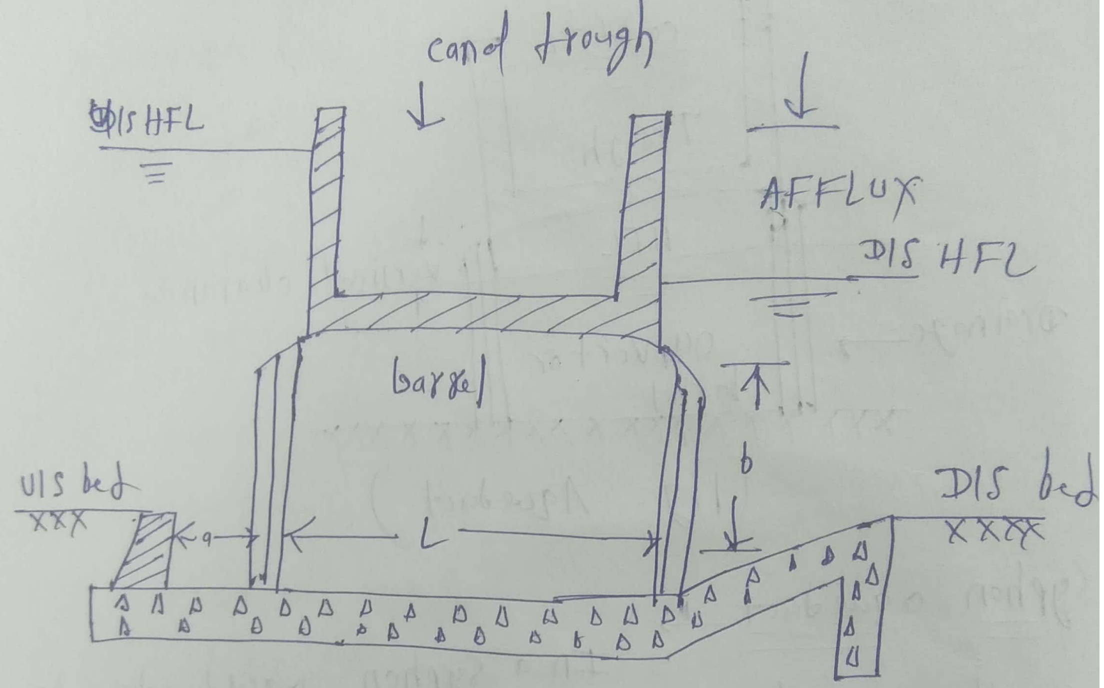


(fig. Aqueduct).

(ii) Syphon aqueduct →

In a Syphon aqueduct also the canal is taken over the drainage, but the flow in the barrel of the drainage is pipe flow. A Syphon aqueduct is constructed when the H.F.L of the drainage is higher than the canal bed level.

When sufficient level difference is not available b/w the canal bed and the H.F.L of the drainage to pass the drainage water the bed of the drainage may be depressed below its normal bed level. The drainage is provided with an impervious floor at the crossing and thus a barrel is formed b/w the piers to pass the drainage water under pressure. These barrels actually form an inverted syphon and not syphon. However, in the common usage the term Syphon is generally used



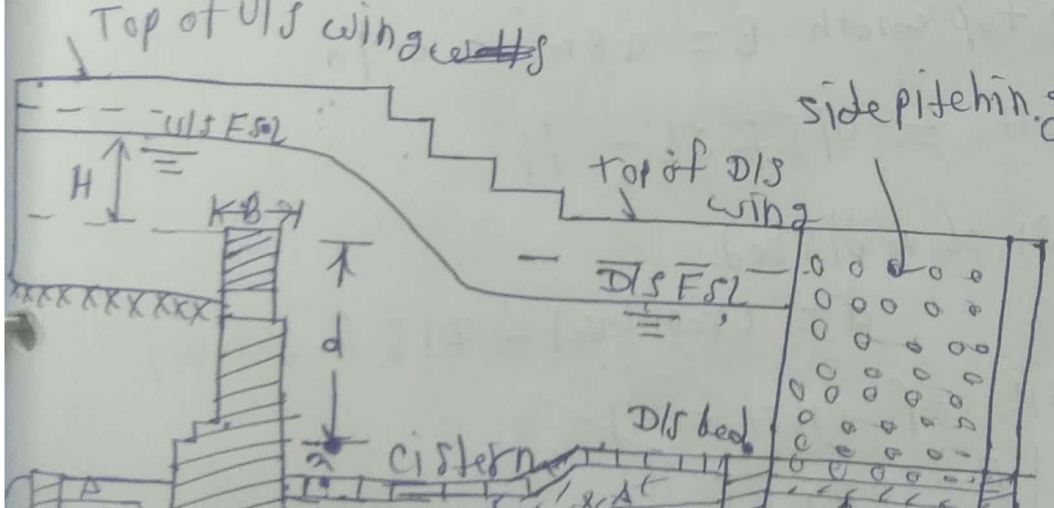
(fig - syphon aqueduct).

(2) canal below the drainage

* Design of Sarda type falls →

The complete design of the Sarda type fall consists mainly of the design of the following components →

1. crest wall
2. cistern
3. Impervious floor
4. D/S bed protection works.
5. U/S wings walls.
6. D/S wing walls.
7. D/S side protection.



Design Steps –

① Step → crest wall → length of crest wall →

There are two types of crest wall are used → (a) Rectangular crest wall → $Q \leq 14$ cu

(b) Trapezoidal crest wall → $Q > 14$ cu

L = Bed width of the channel U/S.

Discharge for Rectangular crest wall

$$Q = 1.835 L H^{3/2} (H/B)^{1/6}$$

Discharge for Trapezoidal crest wall

$$Q = 1.99 L H^{3/2} (H/B)^{1/6}$$

Assuming top width $B = 0.8$ m to 0.90

R.L of crest = U/S FSL − H.

Height of crest above d/s bed

d = crest level − D/S bed level.

Top width of crest wall.

$$B = 0.55\sqrt{d}$$

Assume top width is correct

$H + d = D_2 + H_2 =$

Bottom width $B_1 = \frac{H+d}{G}$

G = Specific gravity of the material of the crest wall = 2.24 to 2.30

Step ② **Cistern** → Depth of cistern →

$$n = \frac{1}{4}(H \times H_L)^{2/3}$$

R.L of cistern = D/S bed level − n

Length of cistern $L_c = 5\sqrt{H \times H_L}$

Step ③ **Impervious floor** →

Maximum seepage head H_s = Crest level − D/S bed level

Total creep length $L = C \times H_s$

C = Bligh creep coefficient given

Depth of U/S cutoff = $D_1/3 + 0.6$

Depth of D/S cutoff = $D_2/2 + 0.6$

Vertical creep = $2 \times$ (Depth of U/S cutoff + Depth of D/S cutoff)

Length of impervious Floor =

b = Total creep length − Vertical creep

Minimum length of D/S Floor

$$L_d = 2(D_2 + 1.2) + H_L$$

Length of U/S Floor = Length of impervious floor (b) − minimum length of D/S floor (L_d)

* Thickness of impervious floor → The thickness of the D/S concrete floor is calculated from the residual uplift pressure (h_r) found from the subsoil H.G.L using Bligh's theory.

$$t = \left(\frac{h_r}{G-1}\right)$$

A minimum thickness of concrete floor of 0.3m to 0.4m for small falls and 0.4 to 0.6m for large falls is usually provided.

Step-④ U/S wing walls →
Provide U/S wing walls splayed at an angle of 45°. (i.e slope 1:1) starting from the U/S edge of the crest wall:

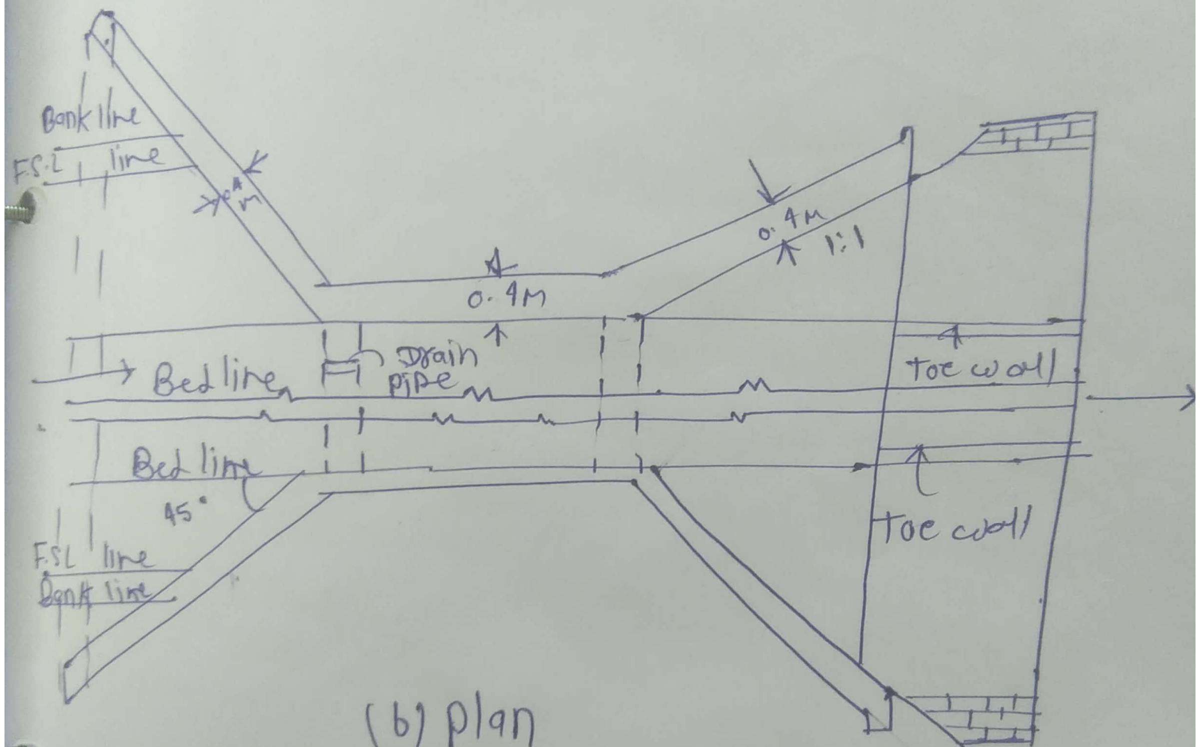
Top level of U/S wing walls = (U/S FSL + free board

Step ⑤ D/S wing walls → Length of D/S wing walls with a vertical face, measured from the crest wall.

$$= 6\sqrt{H \times H_2}$$

Step-⑥ D/S bed pitching →

Length of pitching $= 6 + 2H_2$



Bank line
F.S.L line
0.4 M
0.4 M
0.4 M
1:1
Bed line
Drain pipe
Bed line
45°
F.S.L line
Bank line
Toe wall
Toe wall

(b) plan

Step ⑦ D/S side pitching →

provide 20cm thick side pitching consisting of one brick on edge over one flat brick. The side pitching is warped from a slope of 1:1 to 1.5:1.

Q1 Design a sarda type fall for a canal from the following data:.

(i) Discharge $\frac{U/S}{D/S} = \frac{10 \text{ cumecs}}{10 \text{ cumecs}}$ (ii) Full supply level $\frac{U/S}{D/S} = \frac{201.50}{200.25}$

(iii) Drop = 1.25M (iv) Bed Level $\frac{U/S}{D/S} = \frac{200.00}{198.75}$

(v) Bed width $\frac{U/S}{D/S} = \frac{9.0M}{9.0M}$ (vi) Full supply depth $\frac{U/S}{D/S} = \frac{1.50M}{1.50M}$

(vii) Bligh's creep coefficient = 8.

Neglect the velocity of approach. Use Bligh's theory.

Step-Ist → Crest wall → As the discharge is less than 14 cumecs, a rectangular crest wall is provided.

Length of crest = bedwidth = 9.0M

Let us assume the top width B as 0.8M.

discharge $Q = 1.835 LH^{3/2} (H/B)^{1/6}$

$10 = 1.835 \times 9 \times H^{3/2} \times (H/0.8)^{1/6}$

$H^{5/3} = 0.588$ or $H = 0.72M$

R.L of crest = U/S FSL − H

= 201.50 − 0.72

R.L of crest = 200.78 M

Height of crest above D/S bed

$d =$ crest level − D/S bed level

$d = 200.78 - 198.75$

$d = 2.03M$

Top width of crest wall $= B = 0.55 \times \sqrt{d}$

$B = 0.55 \times \sqrt{2.03}$

$B = 0.78M$ say $B = 0.80M$

Therefore, the assumed top width is correct.

Now $H + d = D_2 + H_L = 1.50 + 1.25$

$H + d = 2.75M.$

Bottom width $= B_1 = \frac{H+d}{G}$

$B_1 = \frac{2.75}{2.24} = 1.22M$ say $1.25M$

provide U/S pitching for a length of 3M at a slope of 1 in 10 just U/S of the crest wall.

Step-II nd → cistern → The depth of cistern →

$x = \frac{1}{4}(H \times H_L)^{2/3} = \frac{1}{4} \times (0.72 \times 1.25)^{2/3}$

$x = 0.23M$ say $x = 0.25M$

R.L of cistern = D/S bed level − η.
= 198.75 − 0.25

R.L of cistern = 198.50

Length of cistern $L_c = 5 \times \sqrt{H \times H_L}$

$L_c = 5 \times \sqrt{0.72 \times 1.25}$

$L_c = 4.74$ m Say $L_c = 4.75$ M

Step-III^rd **Impervious floor** →

Maximum seepage head

H_s = crest level − D/S bed level

H_s = 200.~~__~~ − 198.75

H_s = 2.03 m

Total creep length = $L = C \times H_s$

$L = 8 \times 2.03$

$L = 16.24$ m Say $L = 16.25$ m.

Depth of U/S cutoff = $D/3 + 0.6$

$= \frac{1.50}{3} + 0.6 = 1.1$ M

Depth of D/S cutoff = $D/2 + 0.6 = \frac{1.50}{2} + 0.6$

= 1.35 m Say = 1.5 m

Width of U/S and D/S cutoff = 0.3 m

Vertical creep = 2 × (1.1 + 1.50) = 5.20 m.

Length of impervious floor =

$(b) = 16.25 - 5.20$

$b = 11.05\,M$

minimum length of d/s floor

$l_d = 2(D_2 + 1.2) + H_L$

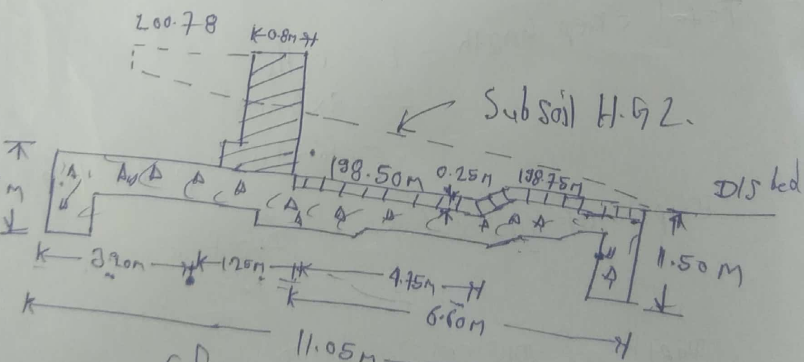
$l_d = 2 \times (1.5 + 1.2) + 1.25$

$l_d = 6.60\,M$

Length of U/S Floor $= 11.05 - 6.6 - 1.25$

$= 3.20\,M$

fig-a shows the details of the floor.



(fig-a)

(3)

c) Residual head at 4.75 m from the toe of the crest wall →

$$h_r = 0.25 + \left(\frac{200.78 - 198.75}{16.25}\right) \times (1.85 + 3.00)$$

$h_r = 0.86 m$

$$\text{Thickness of floor} = \frac{0.86}{2.24 - 1} = 0.70 m$$

provide 0.2m thick brick on edge over 0.70m thick concrete.

d) Residual head at end of d/s floor →

$$h_r = \left(\frac{200.78 - 198.75}{16.25}\right) \times 3.0$$

$h_r = 0.37 m$

$$\text{Thickness of floor} = \frac{0.37}{(2.24 - 1)} = 0.3m$$

provide 0.2m thick brick on edge over 0.4m thick concrete.

* U/s floor → The uplift pressure on the U/s floor will be counter-balanced by the weight of water and soil. provide a nominal thickness of 0.40m.

Scanned by CamScanner

* Thickness of impervious floor →

(a) Residual head at the d/s toe of the crest wall

~~hr = 0.25 +~~

$$h_r = 0.25 + \left(\frac{200.78 - 198.75}{16.25}\right) \times (6.60 + 3)$$

$$h_r = 1.45\,m$$

Thickness of floor $t = \frac{h_r}{(G-1)}$

$$t = \frac{1.45}{(2.24-1)}$$

$t = 1.17\,m$ say $1.20\,m$

provide 0.2m brick on edge over 1.20m thick concrete.
The brick on edge are in cement mortar and are therefore impervious.

(b) Residual head at 2m from the toe of the crest wall →

$$h_r = 0.25 + \left(\frac{200.78 - 198.75}{(16.25)}\right)(4.6 + 3)$$

$h_r = 1.20\,m$

Thickness of floor $= \frac{1.20}{(2.24-1)} = 0.97\,m$ say $1.0\,m$

provide 0.2m thick brick on edge over 1.00m thick concrete.

Step-IV th → U/S wing walls → provide U/S wing walls Splayed at an angle of 45° (i.e slope 1:1) Starting from the U/S edge of the crest wall. continue these walls into banks of the canal till they are embedded for a minimum distance of 1m from the F.S.L.

Top level of U/S wing walls = U/S F.S.L + free board = 201.50 + 0.50

= 202.00

Step-V th → D/S wing walls → Length of d/s wing walls with a vertical face, measured from the crest wall,

$= 6\sqrt{H \times H_L} = 6 \times \sqrt{0.72 \times 1.25}$

= 5.70 m

Keep it 6.60m i.e upto the end of impervious floor.

The d/s wing walls are then warped from the vertical face to 1:1 slope on a splay of 3:1.

Length of warped wall = 3 (1.5 + 0.5) = 6.00 M

Total Length of d/s wing wall = 6.6 + 6.00 = 12.6 M

Top level of D/S wing walls = 200.25 + 0.50
= 200.75

Step-VI th → D/S bed pitching → From table

Head over crest H (cm)		Length of d/s pitching (cm)
(i)	upto 0.30	3.0
(ii)	0.30 to 0.45	$3.0 + 2H_L$
(iii)	0.45 to 0.60	$4.5 + 2H_L$
(iv)	0.60 + 0.75	$6.0 + 2H_L$
(v)	0.75 + 0.90	$9 + 2H_L$
(vi)	0.90 + 1.05	$13.5 + 2H_L$
(vii)	1.05 + 1.20	$180 + 2H_L$
(viii)	1.20 + 1.50	$22.5 + 2H_L$

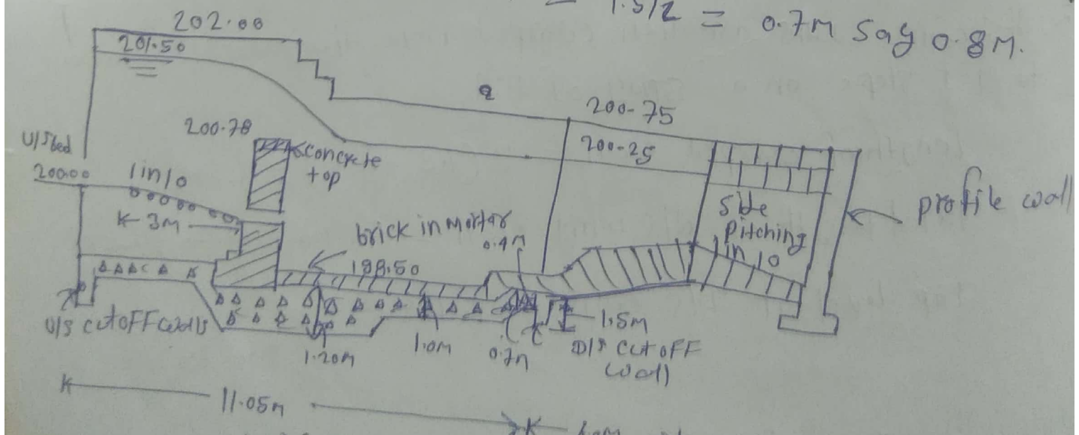
Length of pitching $= 6 + 2H_L = 6 + 2 \times 1.25 = 8.50M$

The bed pitching is provided horizontal upto the end of d/s wing f 6.0M and the remaining 2.50M slopping at 1 in 10. one curtain wall 0.3M thick and 0.6m deep is provided at the end of bed pitching.

Step-VII th → D/S Side pitching → provide 20cm thick side pitching consisting of one brick on edge over one flat brick. The side pitching is warped from a slope of 1:1 to 1.5:1.

It is supported on a toe wall 0.4M thick.

Depth of toe wall $= 1.5/2 = 0.7M$ say 0.8M.



(a) (L-Section).

* Design Steps For Sarda type Fall by khosla.

Theory →

Step-I[st] → crest Wall → As the discharge is More than 14 cumecs, provide a trapezoidal crest wall.

$$Q = 1.99\, L H^{3/2} \left(\frac{H}{B}\right)^{1/6}$$

Less than 14 cumecs Rectangular crest wall provide.
$Q = 1.835\, L H^{3/2} (H/B)^{1/6}$

Note $H+d = H_L + D_2$

Top width $B = 0.55 \times \sqrt{H+d}$

Velocity approach $V_a = \frac{Q}{a}$

Head due to Velocity of approach

$$h_a = \frac{V_a^2}{2g}$$

D/S TEL = D/S FSL + h_a

Again Discharge Formula.

$$Q = 1.99\, L \times E^{3/2} \left(\frac{E}{B}\right)^{1/6}$$

crest level = U/S TEL − E

height of crest above D/S bed =

d = crest level − D/S FSL + FSL Depth (D/S)

height of crest above U/S bed.

d = crest level − U/S FSL + FSL Depth (U/S)

provide a trapezoidal crest wall U/S slope of (1:3) and D/S slope of 1:8.

Step-II[nd] → cistern → Length of cistern →

$$L_c = 5 \times (E H_L)^{2/3}$$

Depth of cistern (x)

$$x = \frac{1}{4} (E H_L)^{2/3}$$

H_L = Drop given in questions.

R.L of cistern = D/S bed level − x

hight of crest wall above cistern level.

= crest level − R.L of cistern

Bottom width of crest wall →

$$B_1 = 1.10 + \left(\frac{1}{3} \times H.C.W.A.C.L\right) + \left(\frac{1}{8} \times H.C.W.A.C.L\right)$$

U/s pitching → provide U/s bed pitching for l[ength] equal to $D_1 = 2.0m$. The pitching should be slope down towards the crest at a slope of 1 in 10.

Step-III[rd] → Design of Impervious floor →

Depth of U/S cut off $\; d_1 = \left(\frac{D}{3}\right) + 0.60$

Depth of D/S cut off −

$$d_2 = \left(\frac{D}{2}\right) + 0.60$$

* Now

$$G_E = \left[\frac{H_s}{d_2 \pi \sqrt{\lambda}}\right]$$

H_s = Seepage head = crest level − D/S bed level.

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

Total length $(b) = \alpha d_2$

Minimum length of D/S impervious Floor-

$L_d = 2 \times (D + 1.2) + H_2$

Length of U/S Floor

L_u = Total length − bottom width of crest wall − L_d

* uplift pressure calculations →

(a) U/S cut-off wall →

$\alpha = \frac{b}{d_1}$ λ = Now Findout (From above equation)

$$\phi_E = \frac{100}{\pi} \cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$$

ϕ_{E_1} =

$\phi_{E_1} = 100 - \phi_E$

$$\phi_D = \frac{100}{\pi} \cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right) \qquad \boxed{\phi_{D_1} = 100 - \phi_D}$$

Assume the thickness of U/S floor as 0.4 m.

$$\text{correction for thickness} = +\left(\frac{\phi_{D_1} - \phi_{E_1}}{\text{Depth of U/S cutoff}}\right) \times \text{thick}$$

Correction for mutual interference.

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

Assume $d = 0.9$, $D = 0.9$ } for U/S case.

corrected $\phi_{E_1} = \phi_{E_1} +$ correction for thickness + correction for mutual interference.

(b) D/S cut-off wall →

$\alpha = \frac{b}{d_2}$ $\quad \lambda =$ from equation.

$$\phi_E = \frac{100}{\pi} \cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$$

$$\phi_D = \frac{100}{\pi} \cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right)$$

Assume the thickness at the D/S floor as 0.8 m.

$$\text{correction for thickness} = +\left(\frac{\phi_E - \phi_D}{d_2}\right) \times \text{thickness}$$

correction due to mutual interference

$$= -\left(\frac{d+D}{b}\right) \times \sqrt{\frac{D}{b}}$$

Now Assume

$d = 1.0$
$D = 1.0$ } in D/S case.

corrected $\phi_{D_1} = \phi_E$ + correction for thickness + correction for mutual interference.

* Thickness of Floor →

(a) At D/S toe of crest wall →

uplift pressure (%) = corrected ϕ_{D_1} + $\left(\frac{\text{corrected } \phi_{E_1} + \text{corrected } \phi_{D_1}}{b}\right) \times u_d$

Residual head = n + uplift pressure × correction for thickness

Thickness of Floor

$$t = \frac{\text{Residual head}}{G - 1}$$

G = Specific gravity of material

2.24 to 2.30

~~(b) At~~ Remaining uplift pressure can be calculated from different length of crest wall.

* Design of U/S wing walls →

Radius = 5 to 6 times of (H).

So ~~R~~

provide U/S wing walls having segmental portion with suitable Radius and subtending an angle of [illegible] at the centre and starting from the U/S edge of the crest wall.

Top level of U/S wing wall

= FSL (U/S) + Free board.

* D/S wing walls → length of D/S wing wall with a vertical face.

$= 6 \times \sqrt{E \times H_2}$

The wing walls are then warped to a slope of 1:1 at a splay of 3:1.

Top level of wall = D/S FSL + free board

height of wall above bed = Top level − D/S [illegible]

length of warped wing with a splay of 1 in 3.

Total length of D/S wingwall = length of wingwall + length of warped wall

D/S bed pitching → from which, length of D/S bed pitching

$= 9 + 2H_L$

provide horizontal pitching upto the end of Masonry wing walls for a length of 6.3m and then ~~stopping~~ sloping at 1 in 10 for a length of 6m. provide 0.2m thick dry brick pitching on 0.1m thick ballast.

(1)

* Design and Sketch a Sarda type fall for a channel from the following data.

(i) Full supply discharge $\frac{U/S}{D/S} = \frac{52 \text{ cumecs}}{52 \text{ cumecs}}$

(ii) Full supply level $\frac{U/S}{D/S} = \frac{203.50}{202.00}$

(iii) Drop = 1.50 M.

(iv) Bed width $\frac{U/S}{D/S} = \frac{38M}{38M}$.

(v) Full supply depth $\frac{U/S}{D/S} = \frac{2.0M}{2.0M}$.

(vi) Side slope = 1:1

(vii) Safe exit gradient = 1/6

(viii) Use Khosla theory.

Step-Ist → crest wall → As the discharge more than 14 cumecs provide a trapezoidal crest wall.

Length of crest wall = bed width = 38 M.

$$Q = 1.99 L H^{3/2} \times \left(\frac{H}{B}\right)^{1/6}$$

$$H+d = H_L + D_2 = 1.50 + 2 = 3.50M$$

Top width $B = 0.55 \times \sqrt{H+d}$

$= 0.55 \times \sqrt{3.50}$

$B = 1.03M$ say $\boxed{B = 1.10M}$

$L = 35M$, $B = 1.10M$ and $Q = 52$ cumecs.

So.

$$52 = 1.99 \times 35 \times H^{3/2} \times (H/1.10)^{1/6}$$

$\boxed{H = 0.80M}$

Velocity approach $V_a = \frac{Q}{A} = \frac{52}{2 \times (38+2)} = 0.65$

Head due to velocity approach.

$$h_a = \frac{V_a^2}{2g} = \frac{(0.65)^2}{2 \times 9.81} = 0.02 M$$

$$D/S\ TEL = D/S\ FSL + h_a$$
$$= 202 + 0.02$$
$$= 202.02 M$$

$$Q = 1.99 \times 38 \times E^{3/2} \times \left[\frac{E}{1.10}\right]^{1/6}$$

$$52 = 1.99 \times 38 \times E^{3/2} \times \left[E/1.10\right]^{1/6}$$

$$\boxed{E = 0.80 M}$$

Crest level = U/S TEL − E
= 203.50 − 0.80
= 202.7 M

Height of crest above U/s bed
= 202.70 − 203.50 + 2
= 1.20 M

provide a trapezoidal crest wall top width 1.10 M and U/s slope of 1:3 and D/S slope of 1:8.
The top surface of the crest wall, top width with 20cm thick concrete.

Step IInd → cistern — Length of cistern →

$$L_c = 5 \times (E \times H_L)^{2/3}$$
$$= 5 \times (0.80 \times 1.50)^{2/3}$$

$$\boxed{L_c = 5.65\,M}$$

Depth of cistern.

$$x = \frac{1}{4}\,[E \times H_L]^{2/3}$$
$$x = \frac{1}{4} \times [0.80 \times 1.50]^{2/3}$$
$$x = 0.282\,m \quad \text{say} \quad \boxed{x = 0.30\,M}$$

R.L of cistern = D/S bed level − x

$$= 200 - 0.30$$
$$= 199.70\,M.$$

Height of crest wall above cistern level.

$$= 202.70 - 199.70$$
$$= 3.0\,M.$$

Bottom width of crest wall.

$$B_1 = 1.10 + \left(\frac{1}{3} \times 3\right) + \left(\frac{1}{8} \times 3\right)$$

$$B_1 = 2.475\,m \quad \text{say} \quad \boxed{B_1 = 2.50\,M}$$

U/s pitching → provide U/s bed pitching for length equal to $D_1 = 20\,M$. The pitching should be slope down towards the crest at a slope of 1 in 10.

Step-III → Impervious Floor →

Depth of U/S cut off

$d_1 = \frac{D}{3} + 0.6$

$= \frac{2}{3} + 0.6$

$d_1 = 1.26\,M$ say $d_1 = 1.30\,M$

Depth of D/S cut off.

$d_2 = \frac{D}{2} + 0.6$

$= \frac{2}{2} + 0.60$

$d_2 = 1.60\,M$

provide U/S cut off wall, with 1.30 M depth and 0.30 M width.

provide D/S cut off wall with 1.60 M depth and 0.30 M width.

Now $G_E = \left[\frac{H_s}{d_2 \pi \sqrt{\lambda}}\right]$

H_s = Seepage head = crest level − D/S bed level

$H_s = 202.70 - 200$

$H_s = 2.70\,M$

$\frac{1}{5} = \frac{2.70}{1.60 \times \pi \times \sqrt{\lambda}}$

$\lambda = 7.21$

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

$$7.21 = \frac{1+\sqrt{1+\alpha^2}}{2}$$

$$\boxed{\alpha = 13.39}$$

Total length $b = \alpha \times d_2$

$b = 13.39 \times 1.60$

$b = 21.42$ m

Say $b = 21.45$ m

minimum length of D/S impervious Floor $= 2 \times (D + 1.2) + H_2$

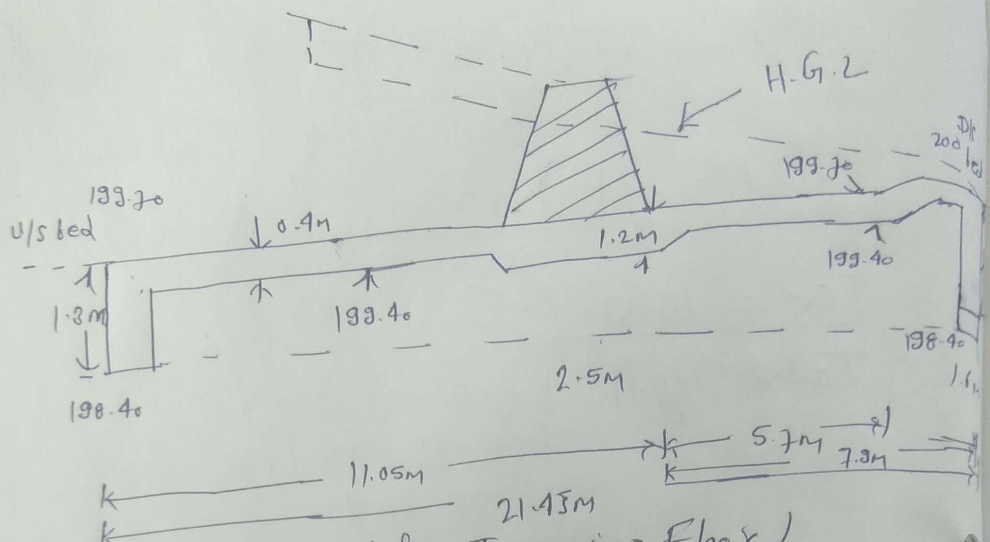
$l_d = 2 \times (2 + 1.2) + 1.50$

$$\boxed{l_d = 7.9\,m}$$

Length of U/S Floor.

$l_u = 21.45 - 2.50 - 7.9$

$$\boxed{l_u = 11.05\,m}$$



(fig - Impervious Floor)

* uplift pressure calculations ⟶

(a) U/S cut-off Wall ⟶

$d_1 = 1.30M$, $b = 21.45M$

$\alpha = \frac{21.45}{1.30} = 16.50 \quad \lambda = 8.77$

$\phi_E = \frac{100}{\pi} \cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right) = 21.9\%$

$\phi_{E_1} = 100 - \phi_{E_1} = 78.1\%$

$\phi_D = \frac{100}{\pi} \cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right) = 15.3\%$

$\phi_{D_1} = 100 - 15.3 = 84.7\%$

Assume the thickness of U/S floor as 0.4M.

Correction for thickness $= +\left(\frac{84.7 - 78.1}{1.30}\right) \times 0.4$

$= +2$

Correction for Mutual Interefrence

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

$$= +19\left(\frac{0.9+0.9}{21.45}\right)\times\sqrt{\frac{0.9}{21.45}}$$

$$= +0.3$$

Corrected $= \phi_{E_1} = 78.1 + 2 + 0.3$

$$\phi_{E_1} = 80.4\,\%$$

(b) D/S cut-off wall →

$d_2 = 1.60$, $b = 21.45$, $\alpha = 13.41$, $d = 7.22$

$$\phi_E = \frac{100}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right) = 24.3\,\%$$

$$\phi_D = \frac{100}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right) = 17.0\,\%$$

Assume the thickness at the end of D/S Floor as 0.60 m.

Correction for thickness $= \left(-\frac{24.3-17}{1.6}\right)\times 0.60$

$$= -2.7\,\%$$

Correction due to mutual inference

$$= -\left(\frac{d+D}{b}\right)\times\sqrt{\frac{D}{b}}$$

Corrected $\phi_E = 24.3 - 2.7 - 0.4$

$= 21.2\,\%$

* Thickness of Floor →

(a) At D/S toe of crest wall →

uplift pressure $= 21.2 + \left(\frac{80.4 - 21.2}{21.45}\right) \times 7.9$

$= 43\%$

Residual head $= 0.3 + 0.43 \times 2.7$

$= 1.46$ m

Thickness of floor $= \frac{1.46}{2.24 - 1} = 1.18$ m

Say 1.20 m.

provide 0.2m thickness on edge over 1.0m thick concrete total 1.20m thick.

(b) At 3m from D/S toe crest wall →

uplift pressure $= 21.2 \left(\frac{80.4 - 21.2}{21.45}\right) \times 4.9$

$= 34.7\%$

Residual head $= 0.3 + 0.347 \times 2.7$

$= 1.23$ m

Thickness of Floor $= \frac{1.23}{2.24 - 1}$

$= 1.00$ m

provide 0.2m thick brick on edge over ~~0.80~~ 1.00 m thick concrete.

(5)

(c) At 5.70m from D/S toe of crest wall →

uplift pressure $= 21.2 + \left(\frac{80.4 - 21.2}{21.45}\right) \times 2.3$

$= 27.5\%$

Residual head $= 0.3 + 0.275 \times 2.7$

$= 1.04M$

Thickness of Floor $= \frac{1.04}{1.24} = 0.84M$

Say 0.85M

provide 0.2M thick brick on edge 0.85M thick concrete.

(d) At end of D/S floor uplift pressure

$= 21.2\%$

Residual head $= 0.212 \times 2.7 = 0.57M$

Thickness of Floor $= \frac{0.57}{1.24}$

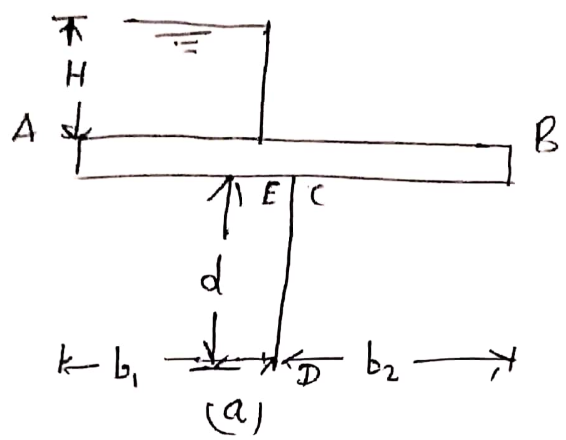
$= 0.46M$ Say 0.5M

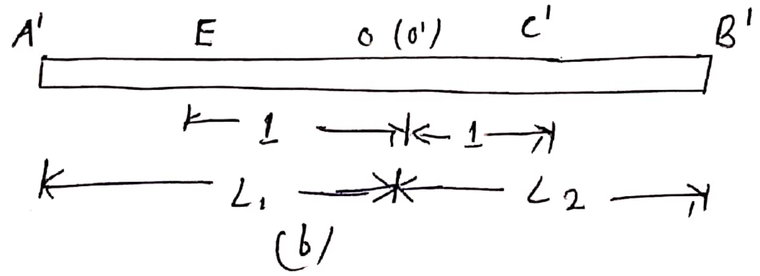
provide 0.2M thick brick on edge over 0.5M thick concrete.

UNIT- 2nd

Numerical portion

* Impervious Floor with an Intermediate pile →





The uplift pressure at salient points E, D and C are given by the following equations —

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

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

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

where

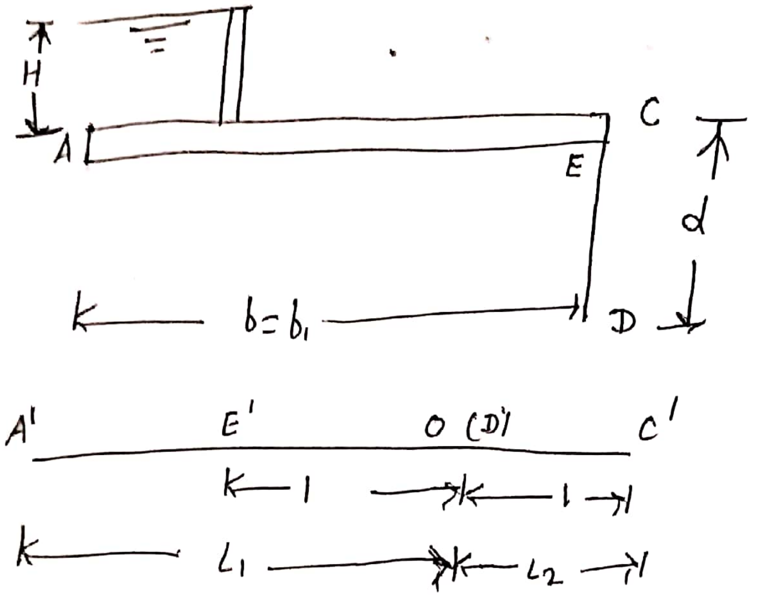
$$\lambda = \frac{L_1 + L_2}{2} = \frac{\sqrt{1+\alpha_1^2} + \sqrt{1+\alpha_2^2}}{2}$$

$$\lambda_1 = \frac{L_1 - L_2}{2} = \frac{\sqrt{1+\alpha_1^2} - \sqrt{1+\alpha_2^2}}{2}$$

In these equations

$\alpha_1 = b_1/d$ and $\alpha_2 = b_2/d$

* Impervious Floor with a D/s pile →



The uplift pressure at the salient points E, D and C are given by the following equations.

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

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

$$P_C = 0$$

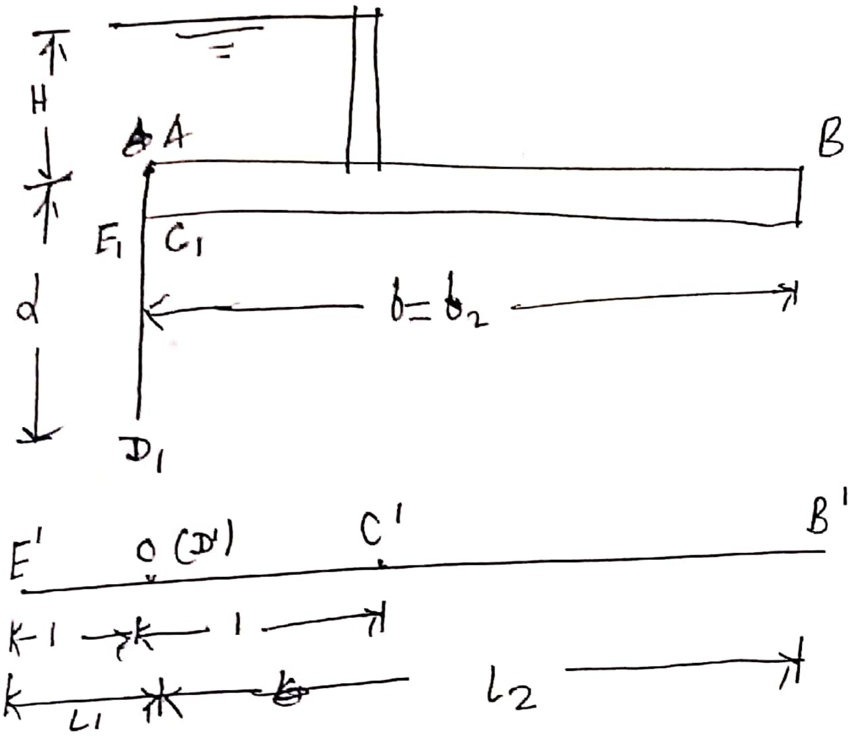
where $\lambda = \dfrac{1 + \sqrt{1 + \alpha^2}}{2}$

in which $\alpha = b/d$

Exit gradient (G_E) is given by.

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

* Impervious Floor with an U/P pile →



The uplift pressure at the salient points E_1, D_1 and C_1 are given by the following equations.

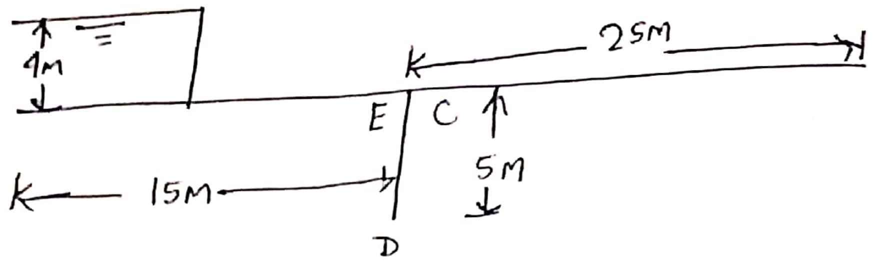
$$P_{E_1} = H$$

$$P_{D_1} = \frac{H}{\pi} \cos^{-1}\left(\frac{1-\lambda}{\lambda}\right)$$

$$P_{C_1} = \frac{H}{\pi} \cos^{-1}\left(\frac{2-\lambda}{\lambda}\right)$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} \quad \text{and} \quad \alpha = b/d$$

① Determine the uplift pressure at the Salient points E, D and C of the intermediate pile as shown in fig.



In this case, $b_1 = 15m$, $b_2 = 25m$, $b = 40m$

$d = 5m$ and $H = 4m$

from eq $\quad \lambda = \dfrac{\sqrt{1+\alpha_1^2} + \sqrt{1+\alpha_2^2}}{2}$

where $\alpha_1 = b_1/d = 15/5 = 3$ and $\alpha_2 = b_2/d = 25/5 = 5$

Therefore $\quad \lambda = \dfrac{\sqrt{1+(3)^2} + \sqrt{1+(5)^2}}{2}$

$\lambda = \dfrac{3.16 + 5.10}{2} = 4.13$

$\lambda_1 = \dfrac{\sqrt{1+(3)^2} - \sqrt{1+(5)^2}}{2} = \dfrac{3.16 - 5.10}{2}$

$\boxed{\lambda_1 = -0.97}$

from eq $\quad P_E = \dfrac{H}{\pi} \cos^{-1}\left(\dfrac{\lambda_1 - 1}{\lambda}\right) = \dfrac{4}{\pi} \cos^{-1}\left(\dfrac{-0.97-1}{4.13}\right)$

$P_E = \dfrac{4}{\pi} \times \left(\dfrac{118.489^\circ}{180} \times \pi\right) = 2.633m$

from eq

$$P_c = \frac{H}{\pi}\cos^{-1}\left(\frac{\lambda_1+1}{\lambda}\right) = \frac{4}{\pi}\cos^{-1}\left(\frac{-0.97+1}{4.13}\right)$$

$$P_c = \frac{4}{\pi} \times \frac{89.584}{180} \times \pi$$

$$\boxed{P_c = 1.991\ m}$$

and

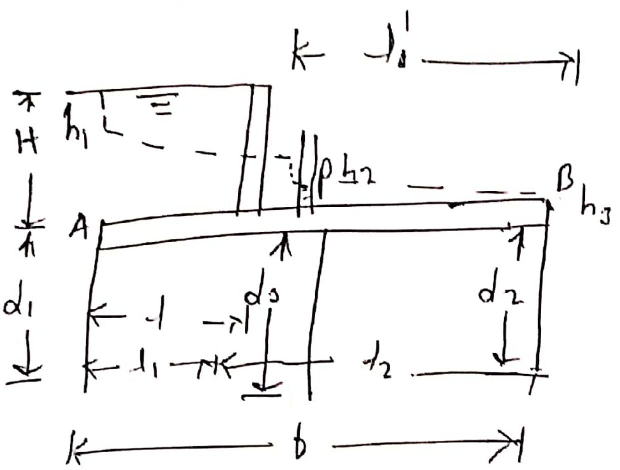
$$P_D = \frac{H}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$$

$$P_D = \frac{4}{\pi}\cos^{-1}\left(\frac{-0.97}{4.13}\right)$$

$$P_D = \frac{4}{\pi} \times \frac{103.584}{180} \times \pi$$

$$\boxed{P_D = 2.302\ m}$$

* Bligh creep theory →



Total creep length

$$L = 2d_1 + l_1 + 2d_3 + l_2 + 2d_2$$

where d_1, d_2 and d_3 are the depth of U/S and D/S intermediate piles. d_1 and d_2 are the length of the U/S and D/S floors.

The subsoil hydraulic gradient is given by

$$i = H/L$$

Uplift pressure Formula

$$h = H - \left(\frac{H}{L}\right) \times d$$

where d is the horizontal length from the entry point A to point P.

thickness of Floor

$$t = \frac{4}{3}\left(\frac{h}{G-1}\right)$$

G = specific gravity of the Floor material. G usually varies from 2 to ~~8~~ 2.30.

At point B $= 1 - \frac{1}{15.75}(2\times 6 + 25) = 1.65m$

At point C $= 1 - \frac{1}{15.75}(2\times 6 + 35) = 1.02m$

* Thickness of Floor →

$$t = \frac{4}{3}\left(\frac{h}{G-1}\right)$$

point A $= \frac{4}{3}\times\left(\frac{2.29}{2.24-1}\right) = 2.46m$

At point B

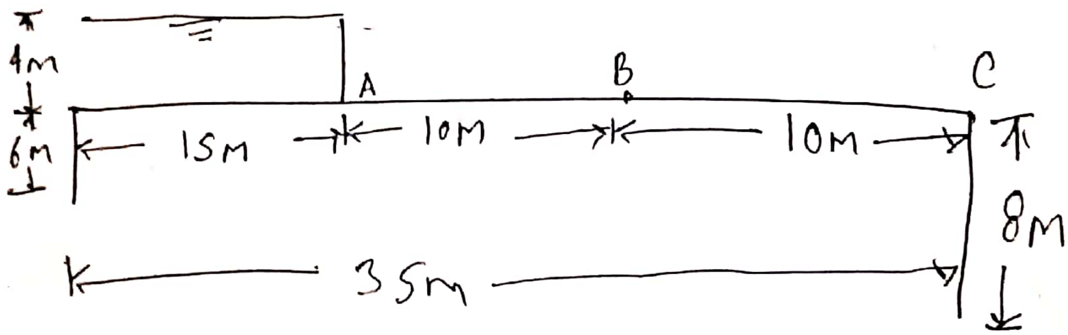
$$t = \frac{4}{3}\left(\frac{1.65}{2.24-1}\right) = 1.77m$$

At point C $t = \frac{4}{3}\left(\frac{1.02}{2.24-1}\right) = 1.10m$

Scanned by CamScanner

~~[illegible]~~ (4)

Example (1) fig shows a hydraulic structures built on fine sand (c = 15). Determine uplift pressure at point A, B, and C at distance 15, 25 and 35 m from the u/s end. and also calculate thickness of floor. Use bligh creep theory. G = 2.24.



Creep length

$L = 2\times 6 + 35 + 2\times 8 = 63\,m$

hydraulic gradient $i = H/L$

$i = 4/63 = \frac{1}{15.75} < \frac{1}{15}$ (Safe)

uplift pressure head ⟶ $h = H - \left(\frac{H}{L}\right)L$

At point $A = 4 - \frac{1}{15.75}(2\times 6 + 15)$

$= 2.29\,m$

* EMBANKMENT DAMS *

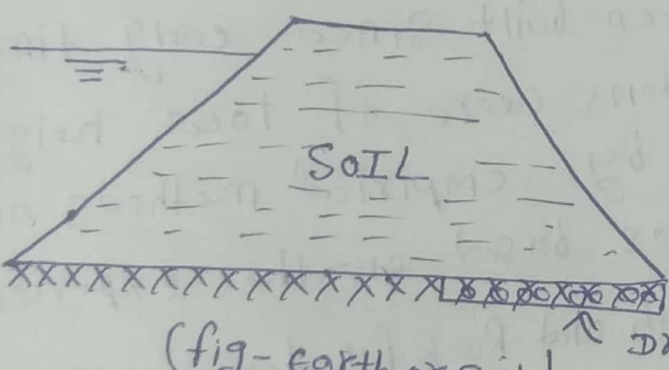
UNIT - III rd

*Definition → These dams are built of Soil or Rockfill or both. As the soil and Rockfill are non-Rigid materials the embankment dams are also called non rigid dams.

=> The embankment dams are broadly classified as follows →

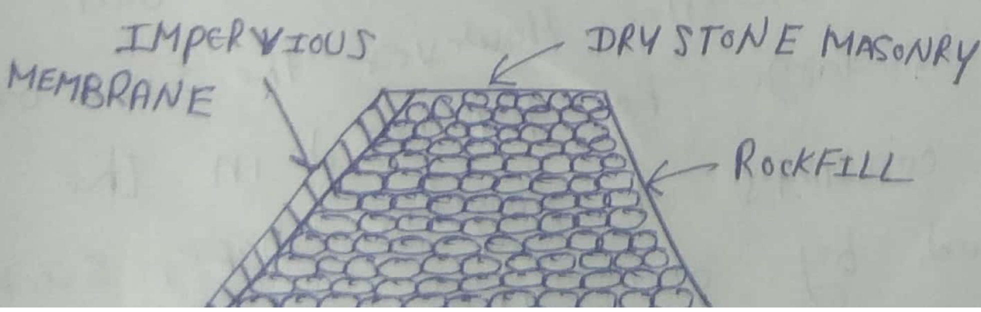
(1) Earth DaMS (2) Rockfill DaMS

(1) Earth DaMS → These dams are constructed mainly from earth or soil.

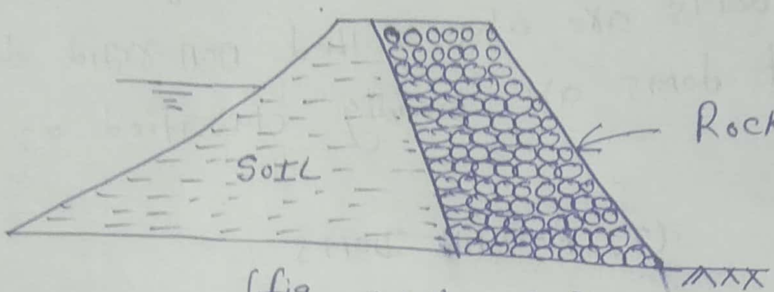


(fig - earth or soil DaM)

(2) Rockfill DaMS → These dams are constructed mainly from Rockfill or pieces of Rocks.



(3) composite earth and Rockfill dam → These dams are constructed from both earth and Rockfill.



(fig- Earth and Rockfill Dam)

* Earth Dams → Earth Dams For the storage of water For Irrigation have been built since early time. However, early earth dams were of low height, as these were designed by empirical methods and their construction was based on the experience.

=> Some of the high earth and Rockfill dams are. MICA DAM, canada (244M), Oroville Dam USA (235M). and Tehri Dam, India (261M).

* Rockfill Dam → A rock-fill dam, like an earth dam is composed of Material. However the size of the Material is large. Rockfill used in the Rockfill dam consists of Angular shape, such as those produced by quarrying Rocks or those occuring naturally such as coarse gravels, cobbles and bulders etc.

* Types of EARTH DAMS Based on methods of construction →

On the basis of the methods of construction the earth dams may be classified into the following types →

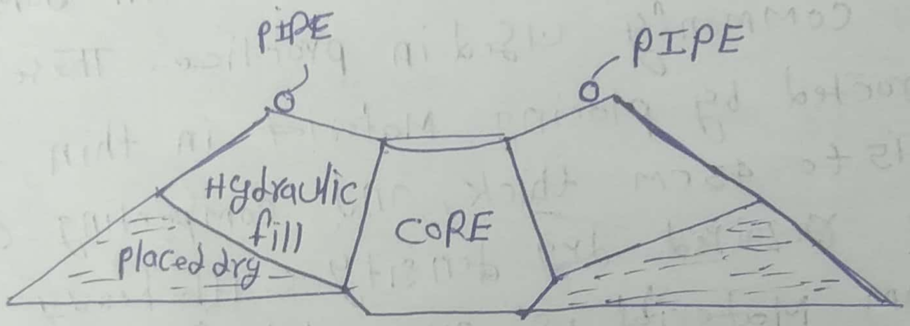
(1) Rolled-fill earth dam (2) Hydraulic-fill earth dam

(3) Semi hydraulic earth dam

(1) Rolled fill earth dam → Rolled fill dams are most commonly used in practice. These dams are constructed by placing material in thin layers about 15 to 45 cm thick, and compacting each layer to the required dry density with heavy Rollers. The dam material is excavated from the borrow pits located near the dam site. Generally, heavy earth moving machines. Such as dragline and power shovels, are used. Scrapers and dumpers are used for transporting the materials to the dam site. The material is then spread by bull dozers to form layers of the required thickness. Each layer is properly compacted at the required water content to the desired maximum dry density.

Generally sheep foot rollers and pneumatic tired rollers are used to compacting the soil.

(2) Hydraulic-fill earth dam → In a hydraulic fill earth dam, water is used for transporting and placing the materials. No roller is required for compaction. The material at the borrow pits is mixed with a large quantity of water required. This water is transported through flumes or pipes and discharged along the outside edges of the fill of the earth dam.



(fig - Hydraulic fill Earth dam)

(3) SEMI-HYDRAULIC-FILL DAMS → In semi-hydraulic-fill dams the coarse material is damped from tracks into the required position to form shell. This method of constructions requires carefull controll to achieve a satisfactory embankment section. In the case of a hydraulic fill dam, no compaction is required. And the material is transported from the borrow pits to the dam site and damped dry within the proposed section of the dam without the use of water.

* CAUSES OF FAILURE OF EARTH DAMS →

Earth dam failures are mainly caused by improper design, lack of through investigation inadequate case in construction and poor maintenance. failures can be divided into three categories →

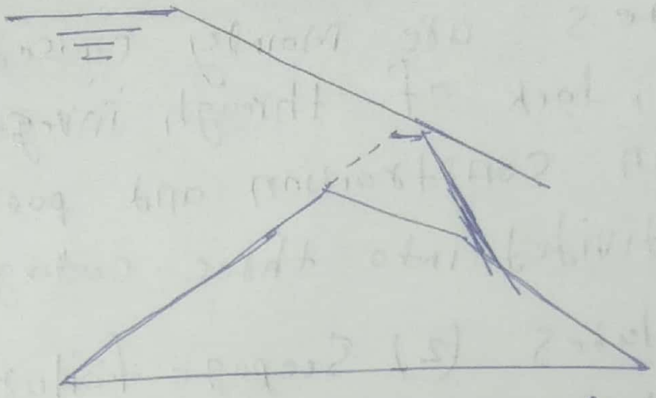
(1) Hydraulic failures (2) Seepage failures.
(3) Structural failures

(1) Hydraulic failures → The hydraulic failures may occur due to following causes.

a (i) overtopping. (ii) Erosion of U/S face
(iii) Erosion of D/S Face (iv) Erosion of D/S toe.
(v) Frost action.

(i) overtopping → An earth dam fails as soon as its overtopping occurs. overtopping is the most common causes of the failure of an earth dam. An earth dam cannot withstand the erosive action of the overflowing water and its fails suddenly. overtopping of the earth ~~following causes~~ dams occurs if.

(a) The spillway capacity is not adequate.
(b) The spillway gates are not properly operated.
(c) The free board is not sufficient.



(fig-a-overtopping failure)

(ii) Erossion of u/s Face → About 5% of the failures of earth dams in the past have been caused by the erosion of the upstream faces by waves. Generally the upstream face is provided with a rip-rap to safe guard against wave erosion.

(iii) Erosion of down stream face → Erosion of D/S face may occur due to rains. Sometimes erosion of the down stream face also occurs because of high winds. Heavy rains falling directly over the D/S face may lead to gully formation resulting in partial failure of slopes.

(iv) Erosion of D/S Toe → Erossion of the D/S toe of an earth dam may occur due to the following two reasons-

(a) cross-currents that originate from the spillway bucket, if a spillway is provided along with the dam.

(b) Waves developed in the tail water.

(v) frost Action → If the earth dam is located at a place where the temperature falls bellow the freezing point, frost may from in the pores of the soil in the earth dam. where there is heaving, the cracks may form in the soil. This may lead to dangerous seepage and consequent failure.

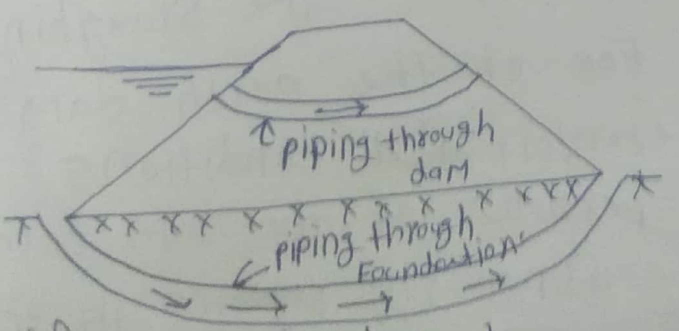
*2 Seepage failure → Seepage failure may occures due to the following causes.

(i) piping through the dam (ii) piping through the foundation.

(iii) conduit leakage (iv) sloughing of D/S toe.

(i) piping through the dam → piping through the dam

Some seepage is inevitable through all earth dams. If the seepage is suitably controlled it does not cause any harm other than some loss of water. However, if the seepage is uncontrolled and concentrated, it may lead to piping and the subsequent failure of the earth dam. About 23% of the total failures occured in the past were piping failure.



(fig - piping failure)

* piping in the dam may occur due to following causes →

(a) poor construction
(b) Differential settlement.
(c) Burrowing Animals.
(d) Surface cracks.
(e) persence of Roots.

(ii) piping through the foundations →

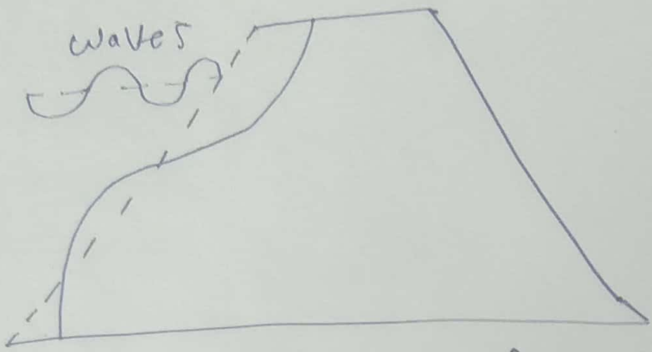
piping through the foundation occurs when the rate of pressure drop (i.e hydraulic gradient) resulting from seepage through the foundation exceeds the resistance of the soil particles. piping in the foundations may also occur when there are pockets of loose soil in the foundation.

(iii) conduit leakage → Sometimes outlet (conduits) are provided through the earth dam. cracks may develop in these conduits due to foundations settlements or due to the deterioration of the condait itself. Leakage occurs through these cracks, which may lead to the failure of the dam.

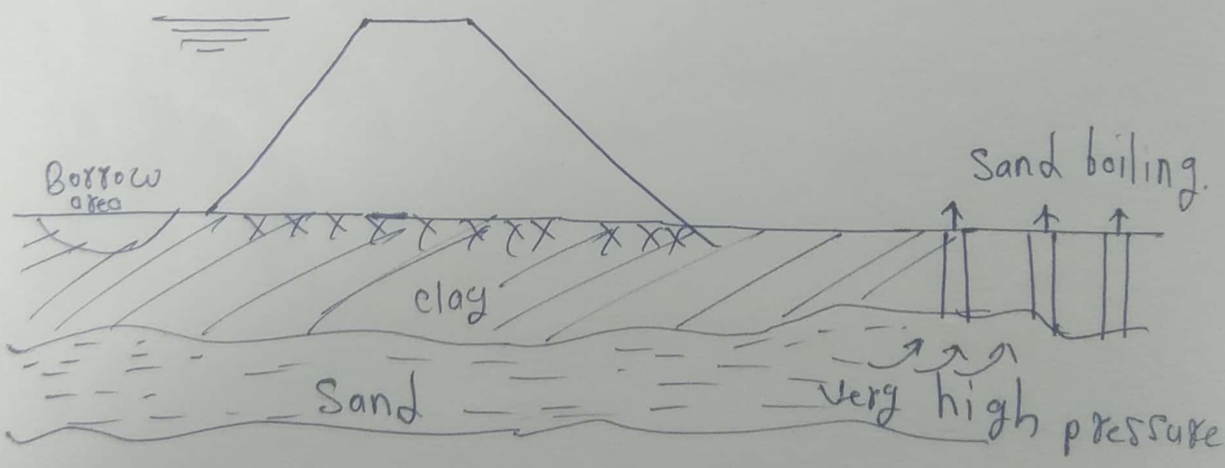
(iv) sloughing of D/s toe → The sloughing of the D/s toe of the earth dam occurs under the Reservoir full conditions when the D/S portions of the dam becomes saturated and contineously remains in the same state, causing the softening and weakening of the soil Mass.

3) **Structural failures** → Structural failures in earth dams are generally shear failures leading to sliding of the embankments or the foundations. Structural failures in the earth dams are ~~generally~~ of the following types.

(i) slides in embankments.
(ii) foundation slides.
(iii) failures by spreading.
(iv) failures due to earthquakes.
(v) failures due to holes caused by burrowing animals.



(fig - erossion of U/S face)



(fig - Seepage failures)

* **Site Selection for a dam** → There are following site selections for a dam →

(i) Topography → up stream to down stream
(ii) Suitable foundation.
(iii) Availability of materials.
(iv) Water tightness of Reservoir.
(v) Small submerged area.
(vi) Accessibility → Roads, Railway, labour's materials.
(vii) Healthy surroundings.
(viii) Development of backward areas
(ix) Minimum overall cost

Gravity Dam and embankment dam:

Advantages

(i) gravity dams can be constructed to very great heights, provided good Rock foundations are available.
(ii) The maintaenance cost of a gravity dam is very low.
(iii) The gravity dam does not fail suddenly to take warning and for saving human life.
(iv) gravity dam can be constructed during all types of climatic conditions.

(V) Gravity dams are specially suited to the area of very downpoor.

Earth dams →

Advantages

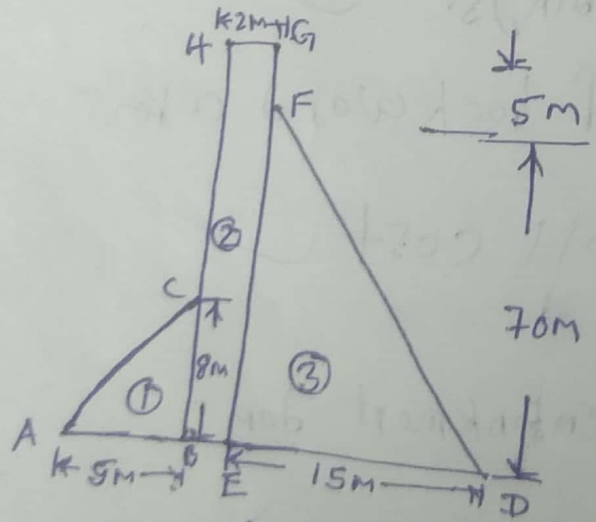
(i) It is a cheaper than gravity dam.

(ii) It is constructed all types of foundation.

(iii) It is constructed in short period.

(iv) Not skilled labour are required.

(V) Earth dams are more earthquake resistant than gravity dams.



unit weight of concrete. $\gamma = 24 KN/m^3$

To calculate the weight of the dam

* Topography → As far as possible, the dam should be located where the river has a narrow gorge which opens out upstream to create a large reservoir. In that case the length of the dam would be small and the capacity of the reservoir on its upstream would be large.

* Suitable foundation → Suitable foundation should exist at the site for the particular type of dam. If suitable foundation is not available but it can be improved by adopting various measures, the site may be considered for selection.

* Availability of Materials → The dam requires a large quantity of materials for its construction. Suitable type of material in sufficient quantity should be available at or near the dam site to reduce the cost.

* Accessibility → The site should be easily accessible. It should be preferably well connected by a road or a railway line. This would facilitate transportation of labour, materials and machinery.

* Healthy surrounding — The surrounding of the site should be healthy and free from mosquitoes so that the labores can comfortably live in colonies constructed near the dam site.

* Development of backward areas →

For the development of a particular backward area, the dam may be constructed in the region.

* minimum overall cost → The site should be such that the minimum overall cost of the project including subsequent maintenance etc. Are to be manage.

Sources of water
Dam, River, well tubewell
canal etc.

~~Embankment Dams~~

Gravity and Embankments Dams

* **DAM** → Dam is a hydraulic structure. which is built in a River.

purposes ⇒ made in electricity, to distribute the water for Irrigation purposes.

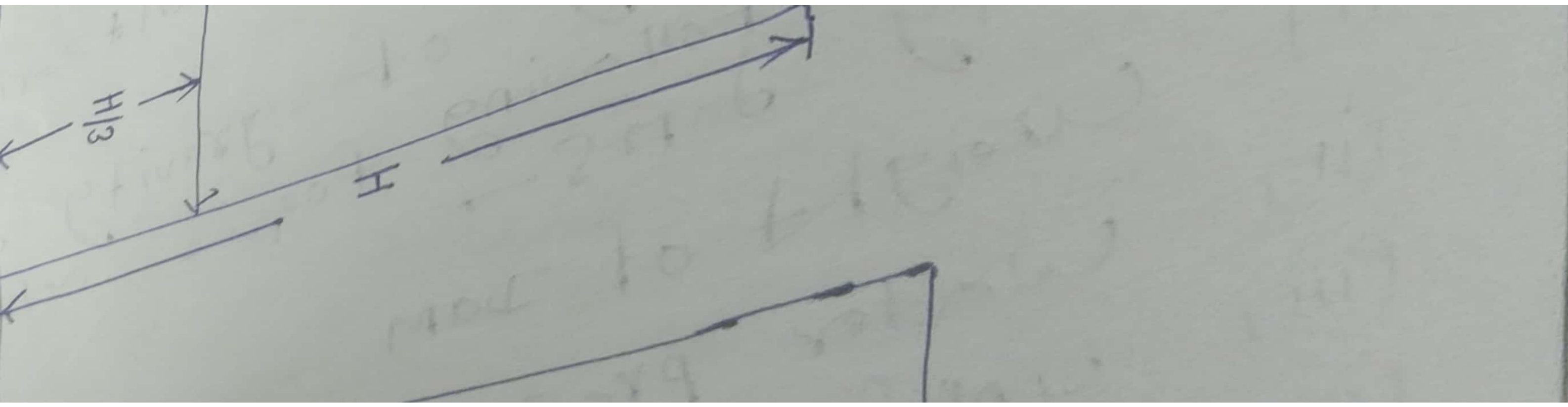
* Embankment dams → This dams are built of soil or rock fill or both non rigid materials.

* **Gravity dams** → The gravity dams is a ~~dam~~ Solid Structure made of concrete or masonary, constructed across a river to create a reservoir on its upstreams side.

highest gravity dam in Switzerland which is 285m high.

* Forces acting of gravity dams →
There are following forces acting on gravity dams →

(i) weight of DAM
(ii) water pressure.
(iii) uplift pressure
(iv) Wave pressure.
(v) Silt pressure.
(vi) Ice pressure
(vii) wind pressure.
(viii) Earthquake forces


H/3
H

* (1) Weight of the Dam → The weight of the dam is major resisting force. The cross-section of the dam may be divided into several triangles, and the weights W_1 W_2 W_3 etc. of each of these may be computed along with determination of their lines of action. The total weight W of the dam acts at the CG of its section.

* (2) Water pressure → This is the major external force acting on dam. When the upstream face of the dam is vertical, the water pressure acts horizontally. The intensity of pressure varies triangularly with a zero intensity at the water surface, to a value wh at any depth (h) below water surface.

* (3) Uplift pressure → The uplift pressure is defined as the upward pressure of water as it flows or seeps through the body of the dam or its foundation.

(4) Wave pressure → Wave are generated on the Reservoir surface because of the wind blowing over it. Wave pressure depends on the height of the wave developed.

(5) Silt pressure → The river brings silt and debries along with it. The silt load gets deposited to an appreciable extent when dam is constructed.

*(6) Ice pressure → The ice pressure is more important for dams constructed in cold countries, or at higher elevations. The ice formed on the water surface of the Reservoir is subjected to expansion and contraction due to temperature variations. This force acts linearly along the length of the dam, at the reservoir level.

(*) (7) wind pressure → It is a minor force and need hardly be taken into account for the design of the dams. wind pressure is required to be considered only on that portion of the super structure which is exposed to the action of wind. Normally wind pressure is taken as 1 to 1.5 kN/m^2 for the area exposed to the wind pressure.

* (8) Earthquake force → An earthquake can be defined as a vibration of surface of earth caused by a disturbance of the rocks beneath the surface. Damage to the dam during earthquake is mostly due to vibrations in earth in horizontal directions when the ground beneath the dam is suddenly moved to one side the dam structure tends to remain in its original position due to its inertia.

* failures of gravity dams → There are following modes of failures of a gravity dam.

① overturning ② sliding.
③ compression or crushing. ④ Tension.

① overturning → The overturning of the dam section takes place when the Resultant force at any section cuts the base of the dam downstream of the toe.
For stability requirements the dam must be safe against overturning.
The factor of safety against overturning are

$$F.S = \frac{\Sigma M_B}{\Sigma M_o} = \frac{\Sigma \text{Rightning Moment}}{\Sigma \text{overturning moment}}$$

② sliding → A dam will be fail in sliding at the base or at any other level if the horizontal forces causing sliding are more than the resistance available to it at that level.
The factor of safety against sliding are.

$$F.S.S = \frac{\mu \times \Sigma V}{\Sigma H}$$

where μ = coefficient of friction.

③ compression or crushing → In order to calculate the normal stress distribution at the base or at any section let (H) be the total horizontal force, (V) be the total vertical force and (R) be the resultant forces cutting the base at any eccentricity (e) from the centre of the base of width (b).

Than. Normal stress at the toe is.

$$(P_n)_{toe} = \frac{V}{b}\left(1 + \frac{6e}{b}\right)$$

And

Normal stress at the heel.

$$(P_n)_{heel} = \frac{V}{b}\left(1 - \frac{6e}{b}\right)$$

④ Tension → In the case of extra high dams, 230 to 260m small tension within the permissible limits is generally permitted for comparatively small periods of loading such as heavy flood or earthquake.

* Instrumentation in gravity Dams →

Instrumentation in gravity dams is generally required to study the structural behaviour and to assess its safety when subjected to different loads.

There are basically two types of instruments used in gravity dams →

① Internal instruments.
② Surveying Instruments.

① Internal instruments → These instruments are placed inside the body of the dam.

(i) Strain meters → The strain meters are used to measure strains at various points in the body of the dam.

(ii) Stress meters → The stress meters directly measures the compressive stress in the concrete. These are basically pressure which directly measure the normal stresses.

(iii) pressure meter → The pressure meter are used to measure hydrostatic pressure developed in the pores of concrete. These also used to measures the uplift pressure at various points.

(iv) Resistance thermometers → Resistance are used to measures the Reservoir water temperature on the upstream and to measure the temperature of concrete.

(v) Displacement meter → The displacement meter are used to measure relative displacements b/w adjacent blocks of concrete.

(vi) ~~Lead~~ & water level meters → Water level meter's are used to measure the water level in the reservoir.

② Surveying instruments → precise surveying instruments are generally used to take measurements on the targets fixed on the top of the dam, on the abutments and on the downstream face. Electronic theodolites and precise levelling instruments are used for taking the measurements. Total Station instruments are quite convenient for this purpose.

$$F.S = \frac{\tan\phi \, \Sigma N' + c \times La}{\Sigma T}$$

~~$$F.S = \frac{\tan 25 \times 14 + 20 \times 78.53}{4.2}$$~~

~~F.S =~~

$$F_S = \frac{\tan 25 \times 396.30 + 20 \times 78.53}{2205}$$

F.S = 1.54. Safe

Scanned by CamScanner

factor of safety

$$F_S = \frac{\tan\phi \; \Sigma(N-U) + c L_a}{\Sigma T}$$

$$= \frac{\tan 25 \times (6500 - 735.75) + 20 \times 78.53}{3300}$$

$F.S = 1.29 < 1.50$ (safe)

* Stability of U/S Slope under Sudden drawdown

$a_N = 14.0\ cm^2$

$a_T = 4.2\ cm^2$

Scale $1cm = 5m$

$\Sigma N = a_N \times m^2 \times \gamma'$ $\quad (\gamma_{sub} = \gamma_{sat} - \gamma_w)$

$\Sigma N = 14 \times (5)^2 \times (21 - 9.8)$

$\boxed{\Sigma N = 3916.80 KN}$

$\Sigma T = a_T \, m^2 \times \gamma_{sat}$

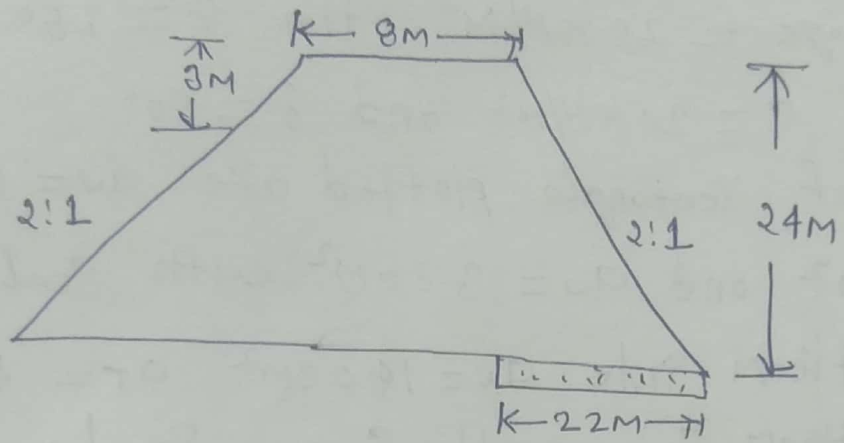
$\Sigma T = 4.2 \times (5)^2 \times 21$

$\Sigma T = 2205 KN$

$$L_a = \frac{2\pi r \delta}{360^\circ} = \frac{2 \times \pi \times 75 \times 60^\circ}{360} = 78.53M$$

Scale = 1 cm = 5 M

Radius r = 75 cm



* Stability of D/S Slope under Steady Seepage →

$$\Sigma N = (a_N \times n^2) \times \gamma = (13 \times (5)^2) \times 20\,kN/m^3$$

$$\Sigma N = 6500\,kN$$

$$\Sigma T = a_T \times n^2 \times \gamma = (6.6 \times (5)^2) \times 20$$

$$\boxed{\Sigma T = 3300\,kN}$$

$$\Sigma U = a_U \times n^2 \times \gamma_w = (3 \times (5)^2 \times 9.81)$$

$$\boxed{\Sigma U = 735.75\,kN}$$

$$L_a = \frac{2\pi r \theta}{360^\circ} = \frac{2 \times \pi \times 75 \times 60}{360}$$

~~[illegible]~~

$$L_a = 78.53\,M$$

Numericals → Based on Stability Analysis →

① Check the stability of U/S and D/S slopes of the given earth dam. Assume the saturated unit weight = 21 KN/M³ and unit weight under steady seepage = 20 KN/M³ the $\phi = 25°$ and under $c = 20\ KN/M^2$ and $\delta = 60°$. The area of rectangle plotted are $a_N = 13\ cM^2$, $a_T = 6.6\ cM^2$ and $a_u = 3.0\ cM^2$ with scale 1 cM = 5 M for downstream side $a_N = 14.0\ cM^2$, $a_T = 4.2\ cM^2$ for up stream side with same scale. The Radius r of the slip circle = 75 cM in both cases.

Solution →

given data →

Saturated unit weight

$\gamma_{sat} = 21\ KN/M^3$

unit weight steady seepage = $20\ KN/M^3$

$\phi = 25°$

Cohesion $c = 20\ KN/M^2$

$\delta = 60°$

Area of U/S side

$a_N = 13\ cM^2$

$a_T = 6.6\ cM^2$

$a_u = 3.0\ cM^2$

Area of D/S side

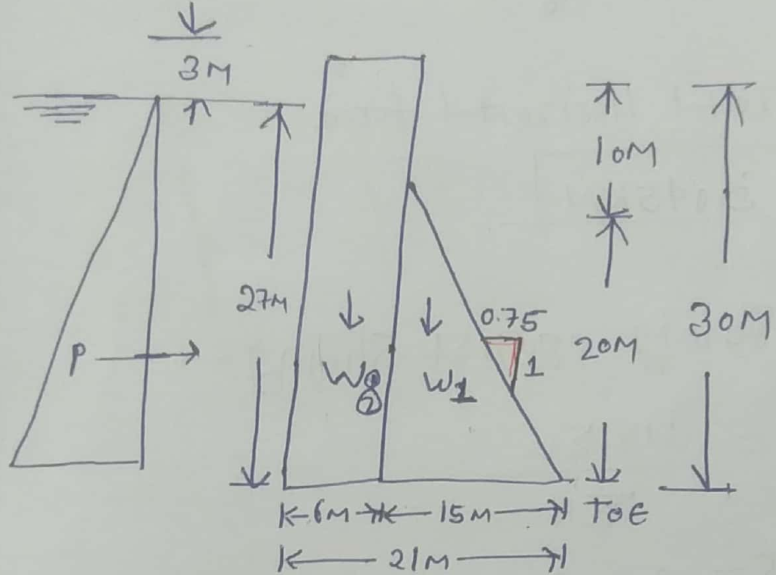
$a_N = 14.0\ cM^2$

② check the Stability of the gravity dam shown in figure. For Reservoir full condition.

Assume $\mu = 0.70$ Unit weight of concrete $w_c = 24 KN/M^3$

unit wt of water $w = 10 KN/M^3$

And $q = 1400 KN/M^2$.



Step ① weight of the Dam →

$$W = W_1 + W_2$$

$W_1 \Rightarrow \frac{1}{2} \times 20 \times 15 \times 24$

$W_1 = 3600 KN$

$C.G = \frac{2}{3} \times 15 = 10M$ from toe

$W_2 = 6 \times 30 \times 24 = 4320 KN$

$C.G = 15 + \frac{6}{2} = 15 + \frac{6}{2} = 18M$ from toe

ΣV = Total Vertical force

$\Sigma V = 3600 + 4320 = 7920 KN$

water pressure →

$$P = \frac{WH^2}{2}$$

$$= \frac{10 \times (27)^2}{2}$$

$$\boxed{P = 3645 KN}$$

$C.G = H/3 = \frac{27}{3} = 9m$ from base.

ΣH = Total Horizontal force

$$\boxed{\Sigma H = 3645 KN}$$

(a) factor of safety against sliding →

$$F_S = \frac{\mu \times \Sigma V}{\Sigma H}$$

$$F_S = \frac{0.7 \times 7920}{3645}$$

$$\boxed{F.S = 1.52}$$

(b) Shear friction factor →

$$S.F.F = \frac{\mu \Sigma V + b \times q}{\Sigma H}$$

$$= \frac{0.7 \times 7920 + 21 \times 1400}{3645}$$

$$\boxed{S.F.F = 9.58}$$

(e) factor of Safety against overturning →

$\Sigma M_R = 3600 \times 10 + 4320 \times 18$

$\Sigma M_R = 113760$ KN-M

And $\Sigma M_o = 3645 \times 9$

$\Sigma M_o = 32805$ KN-M.

$F_o = \frac{\Sigma M_R}{\Sigma M_o} = \frac{113760}{32805}$

$F_o = 3.47$

(d) Maximum Stress

Reservoir full condition → Taking the moments of all the forces about toe.

$\Sigma M = 3600 \times 10 + 4320 \times 18 - 3645 \times 9$

$\Sigma M = 80955$ KN-M

And $\Sigma M = \Sigma V \times \bar{x}$

$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{80955}{7920}$

$\bar{x} = 10.22$ M

eccentricity $e = \frac{B}{2} - \bar{x}$

$e = \frac{21}{2} - 10.22$

$e = 0.28M$

Vertical stress at

toe = f_{yd} or $(P_n)_{toe}$

$(P_n)_{toe} = \frac{\Sigma V}{b}\left(1 + \frac{6e}{b}\right)$

$= \frac{7920}{21} \times \left(1 + \frac{6 \times 0.28}{21}\right)$

$(P_n)_{toe} = 407.31 KN/M2$

Principal stress at toe

$\sigma_d = (P_n)_{toe} \times Sec^2 \phi_d$

$\sigma_d = 407.31 \times (1 + \tan^2 \phi_d)$

$\sigma_d = 407.31 \times (1 + (0.75)^2) = 636.42 KN/M^2$

Shear stress at toe.

$\tau_d = (P_n)_{toe} \tan\phi_d = 407.31 \times 0.75$

$\tau_d = 305.48 KN/M^2$

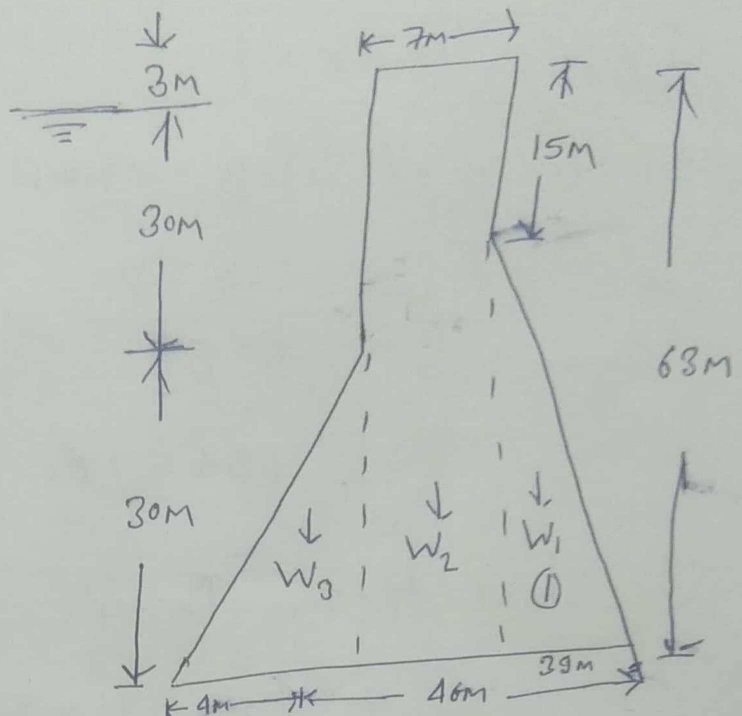
* Stability Analysis of a Gravity Dam →

given

$W_c = 24 kN/m^3$

$q = 1.4$ mpa

$W = 10 kN/m^3$



Step-I st → Weight of the Dam →

$W_1 = \frac{1}{2} \times 39 \times 48 \times 24 \times 1$

$W_1 = 22464$ KN — $C.G = \frac{2}{3} \times 39 = 26m$ from toe

$W_2 = 7 \times 63 \times 24 \times 1$

$W_2 = 10584$ KN — $C.G = 39 + \frac{7}{2} = 42.5$ m from toe

$W_3 = \frac{1}{2} \times 4 \times 30 \times 24 \times 1$

$W_3 = 1440$ KN — $C.G = 46 + \frac{2}{3} \times 4 = 48.67$ m from toe

Step-II nd → Water pressure → (Horizontal Force

$P = \frac{WH^2}{2} = \frac{10 \times 60 \times 60}{2}$ $\quad$ $CG = H/3 = 60/3$
$= 20m$ from base

$P = 18000$ KN

ΣV = Total Vertical Force

$\Sigma V = 22464 + 10584 + 1440 = 34488$ KN

ΣH = Total horizontal force

$\boxed{\Sigma H = 18000 \text{ KN}}$

(a) factor of safety against sliding →

$$F_S = \frac{u \times \Sigma V}{\Sigma H} = \frac{0.7 \times 34488}{18000}$$

$F_S = 1.34 < 1.50$ OK

(b) Shear friction Factor

$$S.F.F = \frac{u \times \Sigma v + b \times 2}{\Sigma H}$$

$$= \frac{0.7 \times 34488 + 50 \times 1400}{18000}$$

$\boxed{S.F.F = 5.23}$

(c) factor of safety against overturning →

$\Sigma M_{R_1} = 22464 \times 26 = 584064$ KN-M

$\Sigma M_{R_2} = 10584 \times 42.5 = 449820$ KN-M

$\Sigma M_{R_3} = 1440 \times 48.67 = 70084.8$ KN-M

$\Sigma M_R = \Sigma M_{R_1} + \Sigma M_{R_2} + \Sigma M_{R_3}$

$\boxed{\Sigma M_R = 1103968.8 \text{ KN-m}}$

and $\Sigma M_0 = 18000 \times 20 = 360000$ KN-m

$$F_0 = \frac{\Sigma M_R}{\Sigma M_0} = \frac{1103968.8}{\ldots} = 3.06$$

(d) Maximum stress →
Reservoir empty condition →

$$\bar{x} = \frac{\Sigma M_R}{\Sigma V}$$

$$= \frac{1103968.8}{34488}$$

$$\boxed{\bar{x} = 32.01\ m}$$

eccentricity $e = \frac{B}{2} - \bar{x}$

$$= \frac{50}{2} - 32.01$$

$$\boxed{e = -7.01\ m}$$

Negative value of (e) indicates that the eccentricity is towards the upstream of the centre of the base

$$e < \frac{B}{6}$$

$$\boxed{e < 8.33}$$

vertical stress at toe

$$(P_n)_{toe} = \frac{\Sigma V}{b}\left(1 + \frac{6e}{b}\right)$$

$$= \frac{34488}{50}\left(1 + \frac{6\times(-7.01)}{50}\right)$$

= ~~79.78 KN/m²~~
109.53 KN/m² (compression)

at heel

$$(P_n)_{heel} = \frac{\Sigma V}{b}\left(1 - \frac{6e}{b}\right) = \frac{34488}{50}\left(1 - \frac{(6\times(-7.01))}{50}\right)$$

= ~~[illegible]~~ 1269.98 KN/m²

principal Stress at toe

$\sigma_d = (P_n)_{toe} \times \sec^2 \phi_d$

$= ~~79.78~~ \; 109.53 \times (1 + \tan^2 \phi_d)$

$= ~~79.78~~ \times (1 + (0.81)^2)$

$\sigma_d = ~~132.12~~ \; kN/m^2 \quad 181.39 \; kN/m^2$

Shear stres at toe

$\tau_d = P_{n\,toe} \times \tan \phi_d$

$= ~~79.78~~ \times 0.81$

$= 109.53 \times 0.81$

$= 88.71 \; kN/m^2$

principal Stress at heel

$\sigma_u = (P_n)_{heel} \times \sec^2 \phi_d$

$= 1269.98 \times (1 + 0.81^2)$

$= 2103.21 \; kN/m^2$

shear Stress $\tau_u = -P_{n\,heel} \times \tan \phi_u$

$= -1269.98 \times 0.81$

$= -1028.68 \; kN/m^2$ (towards upstream)

UNIT IV *

SPILLWAYS AND HYDRO POWER PLANT

INTRODUCTION → A spillway is a structure constructed at or near the dam site to dispose of surplus water from the reservoir to the channel downstream. Spillways are provided for all dams as a safety measure against overtopping and the consequent damages and failure. A spillway acts as a safety valve for the dam, because as soon as the water level in the reservoir rises above a predetermined level excess water is discharged safety to the d/s channel, and the dam is not damaged.

* **Essential Requirements of a spillway** → The essential requirements of a spillway are as follows →

(1) It must have adequate discharge capacity.

(2) It must be hydraulically and structurally safe.

(3) The surface of the spillway must be erosion Resistant.

(4) The spillway must be so located that the spillway discharge does not erode or undermine the d/s toe of the dam.

(5) It should be provided with some device for the dissipation of excess energy.

(6) The spillway discharge should not exceed the safe discharge capacity of the d/s channel to avoid its flooding.

* **Required Spillway capacity** → The required spillway capacity is usually determined by flooding routing. The spillway capacity should be equal to the maximum outflow rate determined by flood routing.

⇒ The following data are required for the flood routing →

(1) Inflow flood hydrograph, indicating the rate of inflow with respect to time. It is the same as the design flood hydrograph of the spillway.

(2) Reservoir capacity curve, indicating the reservoir storage at different reservoir elevations.

(3) outflow discharge curve, indicating the rate of outflow through spillways at different reservoir elevations.

* **Factor's affecting the required Spillway capacity** →

The following factor's affect the spillway capacity →

① Inflow flood hydrograph ② Available storage capacity.

③ capacity of outlets ④ Gates of spillway

⑤ possible damage, if the capacity is exceeded.

① **Inflow flood hydrograph** → The inflow flood hydrograph should be selected according to the degree of protection that ought to be provided to the dam. It will depend upon the type and height of dam, its location with respect to inhabited and developed area, and consequences of its failures.

obviously, a high dam storing a large volume of water and located upstream of a town should have a much degree of protection as compared to that in the case of a small dam storing a small volume of water and on whose downstream the area is uninhabited.

② **Available Storage capacity** → If the available storage capacity of the reservoir is quite large as compared to the inflow, a spillway of smaller capacity will normally be required.

③ **capacity of outlets** → If the dam outlets can be used to discharge a portion of the flood, the spillway capacity can be correspondingly reduced.

④ Gates in spillway → If the spillway is gated its discharge capacity can be modified. For a gate controlled spillway, the water can be stored upto the top of the gates, whereas in the case of an ungated spillway, the water can be stored only upto the crest level. By operat of gates higher heads may be created above the crest so that greater outflow rate through the spillway is achived.

(5) possible damage → If there is a possibility of extensive damage on the downstream a large spillway capacity should be provided.

* CLASSIFICATION OF SPILLWAYS → The spillways can be classified into different types based on the vairous criteria, → (A) classification based on purpose →

① Main (or service) spillway ② Auxiliary spillway
③ Emergency spillway

(B) classification based on control →

① controlled (or gated) spillway ② uncontrolled (or ungated) spillway.

(C) classification based on prominent feature →

① Free overfall (or straight drop) spillway.
② overflow or ogee spillway.
③ chute (or open channel or trough spillway)
④ Side channel spillway.
⑤ shaft or morning glorey spillway.
⑥ Siphon spillway.
⑦ conduit (or tunnel) spillway.
⑧ cascade spillway

* (A) classification based on purpose →

(1) Main (or Service) Spillway → A main (or service) spillway is the spillway ~~is~~ designed to pass a prefixed or the design flood. This spillway is necessary for all dams and in most of the dams, it is the only spillway. Therefore, in general terms, the spillway means the main spillway.

(2) Auxiliary Spillway → In some dams, where the site conditions are favourable, an auxiliary spillway is usually constructed in conjunction with a main spillway. In such a case, the main spillway is usually designed to pass floods which are likely to occur more frequently.

(3) Emergency Spillway → An emergency spillway is sometimes provided in addition to the main spillway. It comes into operation only during an emergency which may arise at any time during the life of the dam. An emergency spillway is an additional safety valve of the dam.

* classification based on control →

(1) Controlled Spillway (gated spillway) → A controlled spillway is one which is provided with the grates over the crest to control the outflow from the reservoir. In the controlled spillway, the full reservoir level of the reservoir is usually kept at the top level

of the gates. The water can be stored up to the top level of the gates. The outflow from the reservoir can be varied by lifting the gates to different elevations.

2) uncontrolled spillway (corungated spillway) →

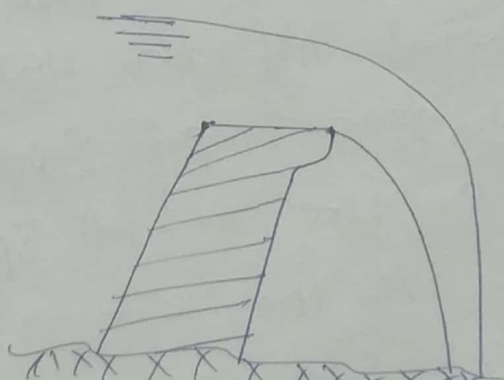
In an uncontrolled spillway, the gates are not provided over the crest to control the outflow from the reservoir. The full reservoir level is the crest level of the spillway. The water level escapes automatically when the water level arises above the crest level. The main advantage of an uncontrolled spillway is that it does not require the gates and the operator and lifting power to operate the gates.

(c) classification based on the perminent features →

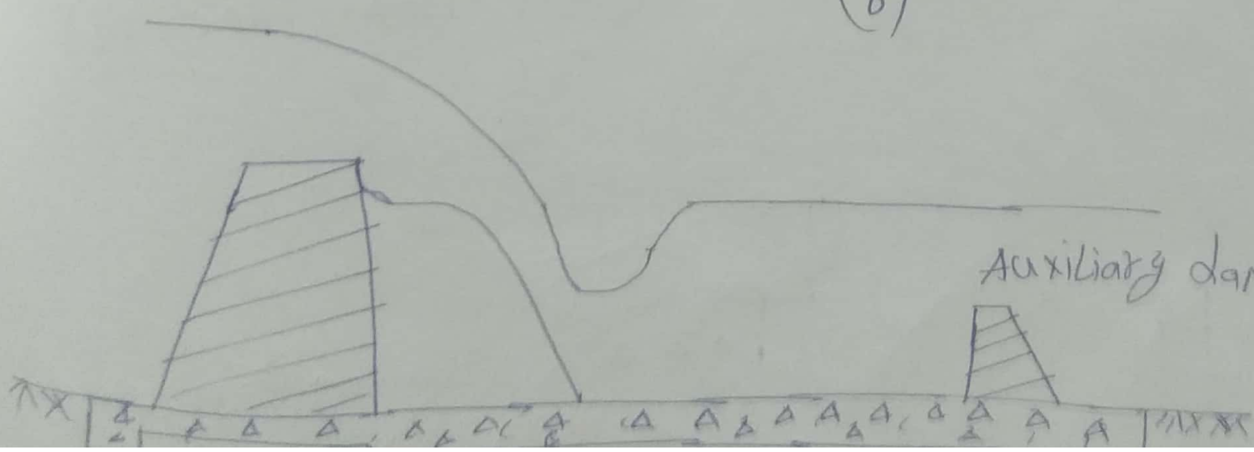
There are following types →

(1) Free overfall spillway → A free overfall spillway (or straight drop spillway) is a type of spillway in which the control structure consists of a low height, narrow crested weir and the D/S face is vertical or nearly vertical so that the water falls freely more or less vertical. The overflowing water may discharge as a free nappe, as in the case of a sharp crested weir, or it may be supported along the narrow section of the crest.

(a)



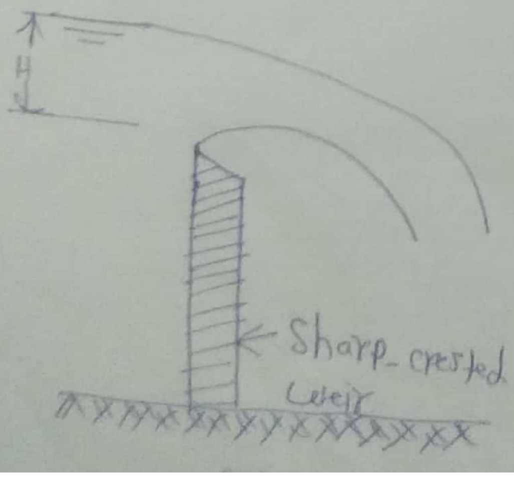
(b)

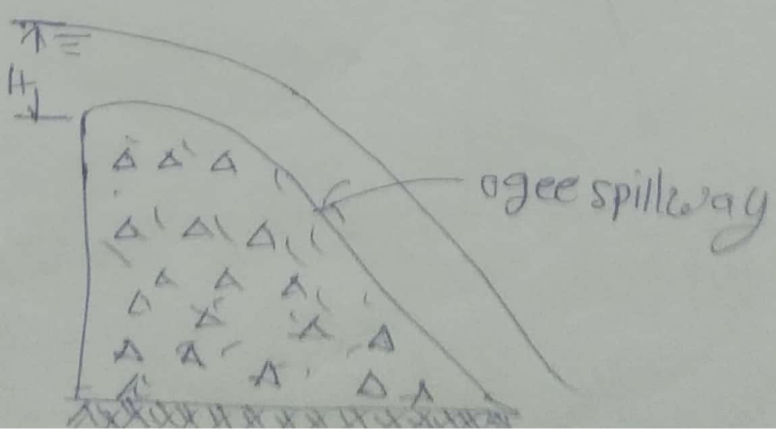


A free overfall spillway is commonly used for a low arch dam whose D/S face is almost vertical, This type of spillway is also used as a separate structure for low earth dams.

(2) Ogee shaped or overflow spillway →

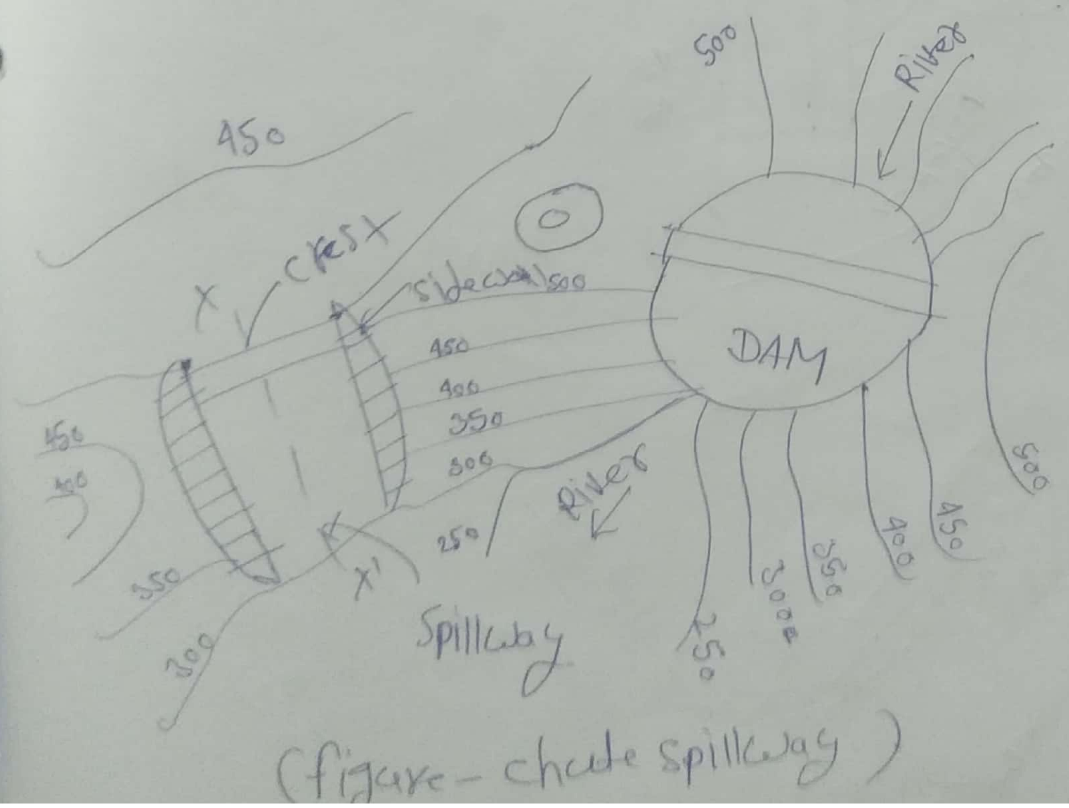
A ogee shaped (or overflow) spillway is the most commonly used spillway. It is widely used with gravity dams, earth dams etc. Several earth and rockfill dams are also provided with this type of spillway as a separate structure. An ogee shaped spillway is an improvement upon the free overfall spillway.
The essential difference b/w the free overfall spillway and the ogee-shaped spillway is that in the case of a free overfall spillway, the water flowing over the crest of the spillway drops vertically as a free jet clear from the d/s face.





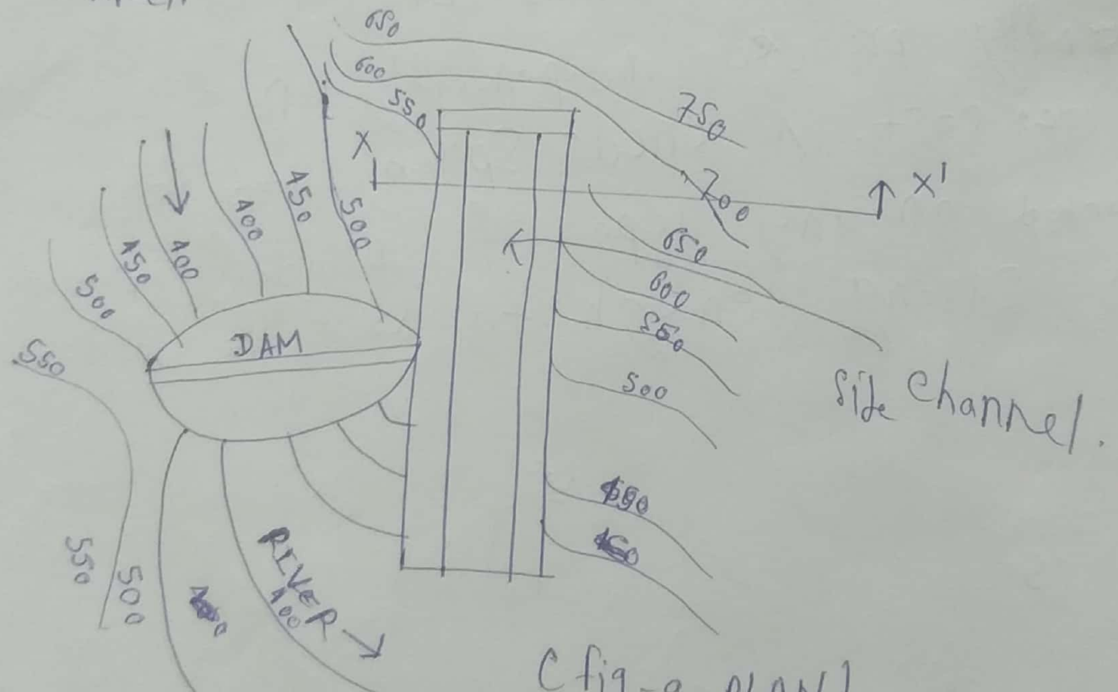
(5) chute or open channel Spillway → A chute or open chanel Spillway consists of a steep-sloped open channel called a chute or trough. Which carries the water passing over the crest of Spillway to the river downstream. For earth dams and rockfill dams a separate spillway is generally constructed in a flank or a saddle away from the dam if a suitable sites exists. The chute spillway is generally most suitable for such conditions. It can be conveniently provided independently in a saddle at a low cost. A chute Spillway may be constructed on any type of foundation provided it is strong enough to bear the load.

e

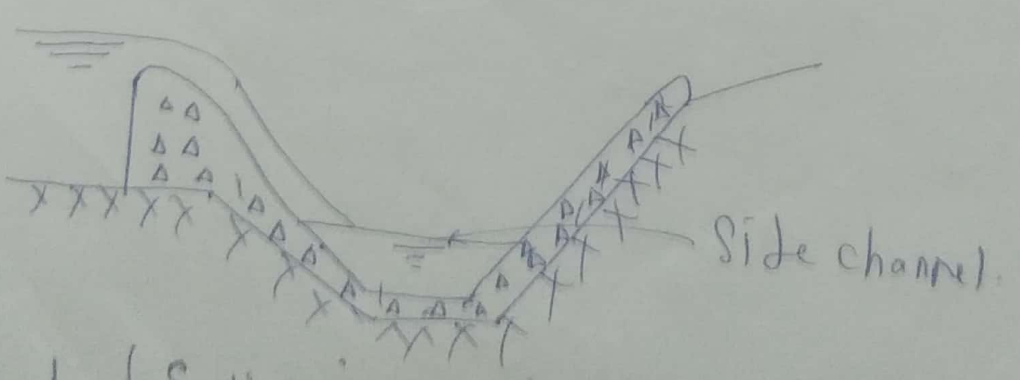


(figure - chute spillway)

*(4) Side channel Spillway → In a side channel Spillway, the crest of the control weir is placed along the side of the discharge channel. The crest is approximately parallel to the side channel at the entrance. Thus the flow after passing over the crest is carried in a discharge channel running parallel to the crest. water flows over the crest into the narrow trough of the discharge channel opposite the weir, it turns approximately at right angle and then continues in the discharge channel.

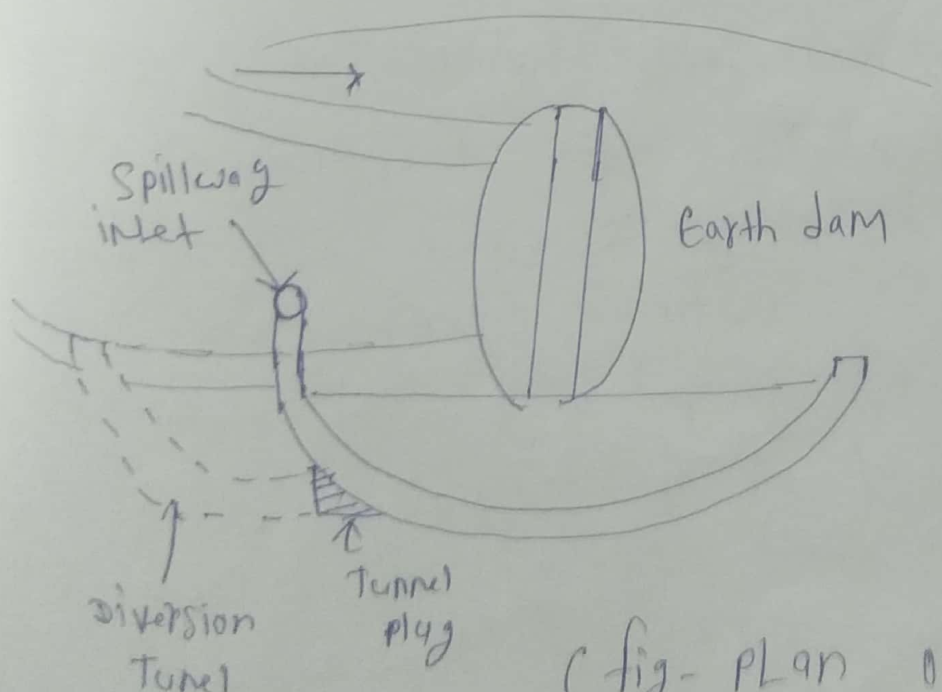


(fig-a PLAN)



(fig-b (Section x-x)).

5) **Shaft Spillway** → A Shaft (or Morning glory) Spillway consists of a large vertical funnel, with its top surface at the crest level of the spillway and its lower end connected to a vertical or (nearly vertical) Shaft. The other end of the vertical Shaft is connected to a horizontal or nearly horizontal conduit or tunnel, which extends through or around the dam and carries the water to the downstream. When the water level rises above the crest level, it starts overflowing the crest and drops from the rim of the ~~channel~~ funnel into the vertical Shaft and then flows in the horizontal conduit which conveys it past the dam.



(fig- PLan of Shaft Spillway)

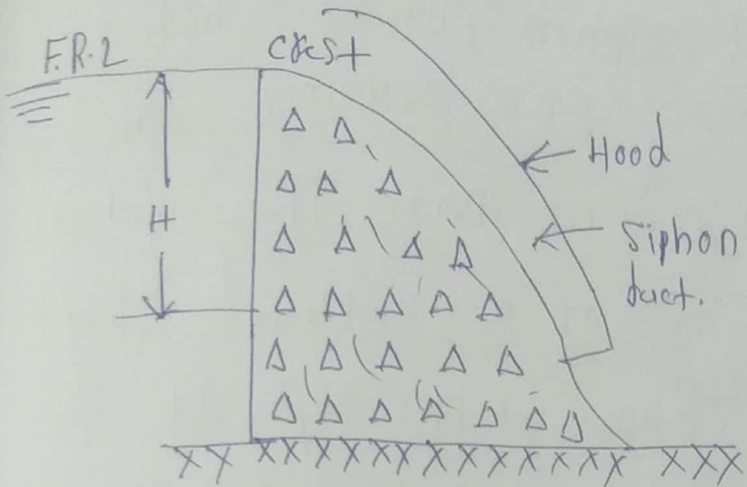
*(6) SIPHON SPILLWAYS → A Siphon Spill way operates on the principle of siphonic action. There are basically two types of Siphon Spillways.

(A) Saddle Siphon spillway (B) Volute Siphon Spillway.

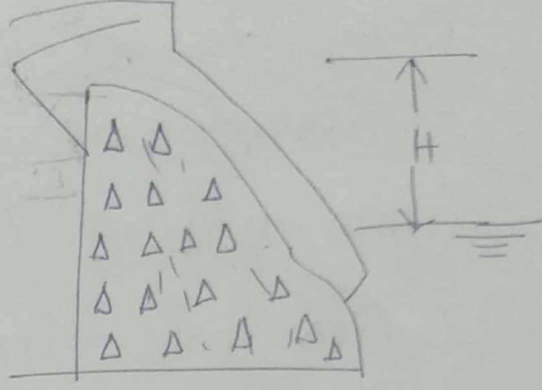
(A) Saddle Siphon Spillway → A Saddle Siphon spillway is a closed conduit of the shape of an inverted ~~U~~ with unequal legs. Saddle Siphon Spillway is commonly used in practice. Saddle Siphon Spillway are usually of two types.

(a) Hood type (b) Tilted outlet type.

(a) Hood Siphon Spillway → The various component parts of the hood Saddle Siphon Spillway are shown in fig-(a) This type of Spillway is also called the hood spillway. The siphon duct is formed by an air tight reinforced concrete cover, called hood, over an ogee-shaped body wall made of concrete. The top of the body wall forms the crest of the ~~[illegible]~~ spillway and is kept at the full Reservoir level (F.R.L) of the reservoir. The Top of the is called crown. The space b/w the crown and the crest is known as throat.

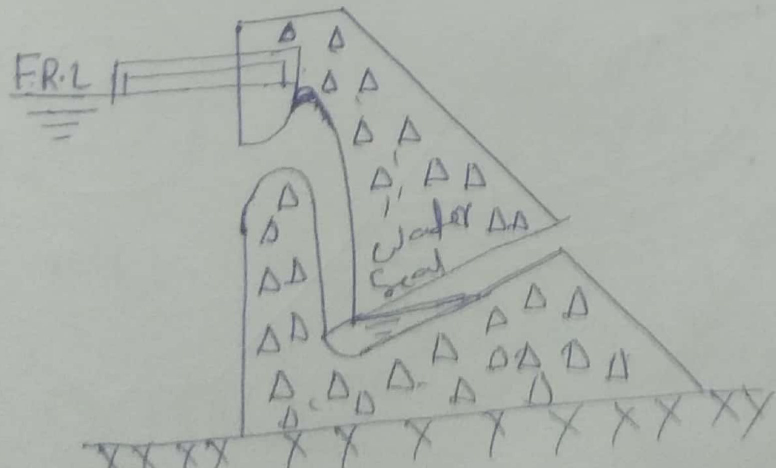


(free discharge (a))



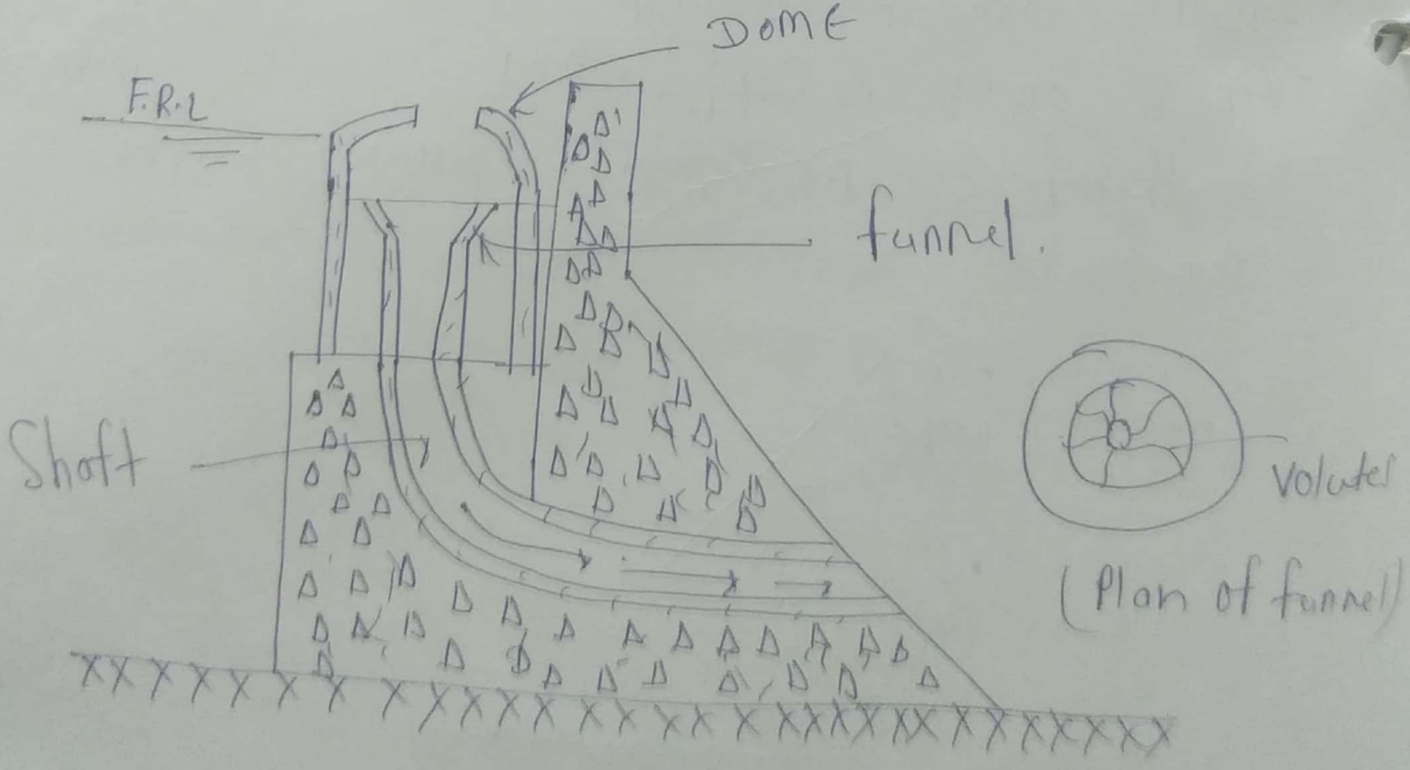
Submerged outlet (b)

(b) Tilted outlet type siphon spillway → Another type of saddle siphon spillway, called tilted outlet type. In this type of spillway, the siphon duct is formed within the body of the dam.
In this case, the draught of the water falling over the crest is sufficient to cause priming and therefore no separate priming device is required.



* Volute Siphon Spillway →

The volute siphon spillway is a special type of siphon spillway which makes use of volutes for priming. This type of spillway was designed in india. The Volute Siphon spillway consists of a vertical shaft which has funnel shape at its top. At the bottom end it is connected to a horizontal or nearly horizontal outlet conduit through a right angled bend. Which leads the water to the D/S channel. The top of the funnel is kept at the full reservoir level. The inner sloping surface of the funnel is provided with a number of volutes.



Even with a large increase in the water level in the reservoir, the increase in discharge is small.

* **Conduit or Tunnel Spillway** →

A conduit or tunnel spillway consists of a closed conduit to carry the flood discharge to the d/s channel. It is constructed in the abutment or under the dam.

The closed conduit may take the form of a vertical or inclined shaft a horizontal tunnel or a conduit constructed in an open cut and the covered. Such Spillway is suitable for dam sites in narrow canyons with steep abutments.

N
—
①

Advantages and disadvantages of a Siphon Spillway *

***Advantages →**

(i) It is automatic in action and does not have any moving part or mechanical device.

(ii) It can be used where a large discharge capacity is not required and the space is limited.

(iii) The discharge per unit length of the spillway is quite high because it has a high operating head.

(iv) The height of the dam required above the crest of the spillway is small.

(v) The operating cost is very low.

*** Disadvantages →**

(i) It requires a strong foundation to resist vibrations which are generally quite severe.

(ii) The spillway is unable to pass the ice and debris to the d/s.

(iii) It has high initial cost.

(iv) It is unable to handle discharge greater than the design discharge.

* **Spillway gates** → when the floods occur's the gates are opened (or lifted up) so that the full spillway capacity is available for discharging the flood. However during the periods of the low flows the gates can be kept closed and an increase in the reservoir level is permitted. These low flows if required to be discharged d/s, are passed through the dam outlets or sluiceways.

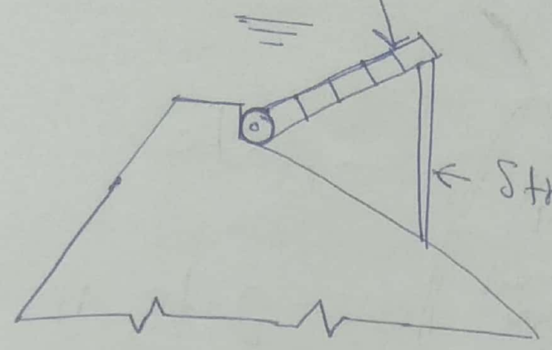
Gates installed on the spillways of the earth and rockfill dams require extra precaution, because any operational failure of the gates may lead to overtopping of the dam, resulting in its failure and catastrophe.

The following are some of the common types of gates used for spillways:→

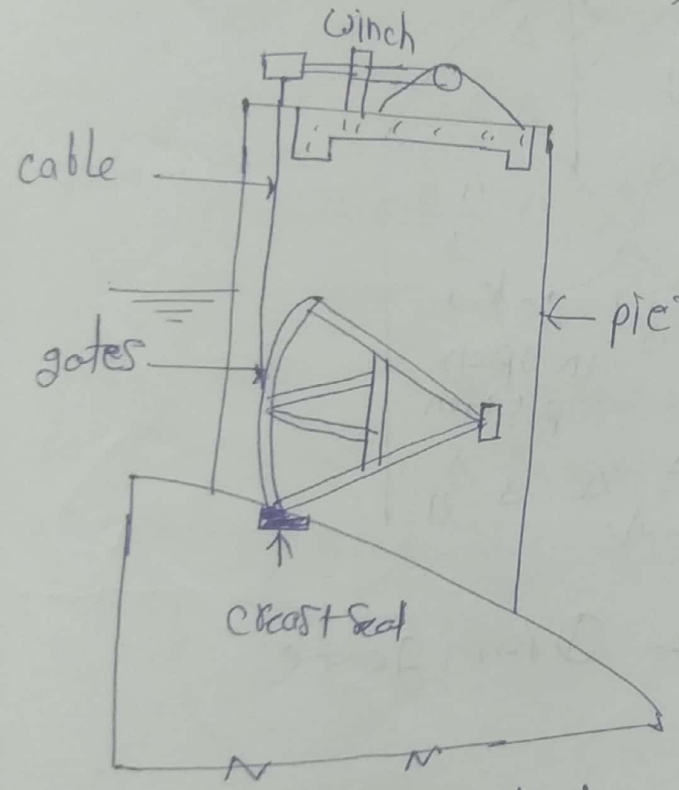
① Flash boards ② Radial gates.
③ Drum gates ④ Vertical lift gates.
⑤ Bear trap gates ⑥ Rolling gates.

① **flash boards** → Flash boards are wooden boards or panels, placed side by side, on the crest of the spillways to form a continuous shutter. These are the simplest and oldest type of gates. These are quite efficient and economical for small height where they can be readily handled by the available lifting arrangements.

② **Radial gates** → A Radial gates also known as a tainter gate has its water supporting face, made of steel plates, in the shape of sector of a circle, properly braced and hinged at the pivot. The gate can thus be made to rotate about fixed horizontal axis. The load of the gate and water etc. is carried on bearings, mounted on piers. The gate can be lifted by means of ropes and chain acting simultaneously at both ends or with the help of power driven winches.

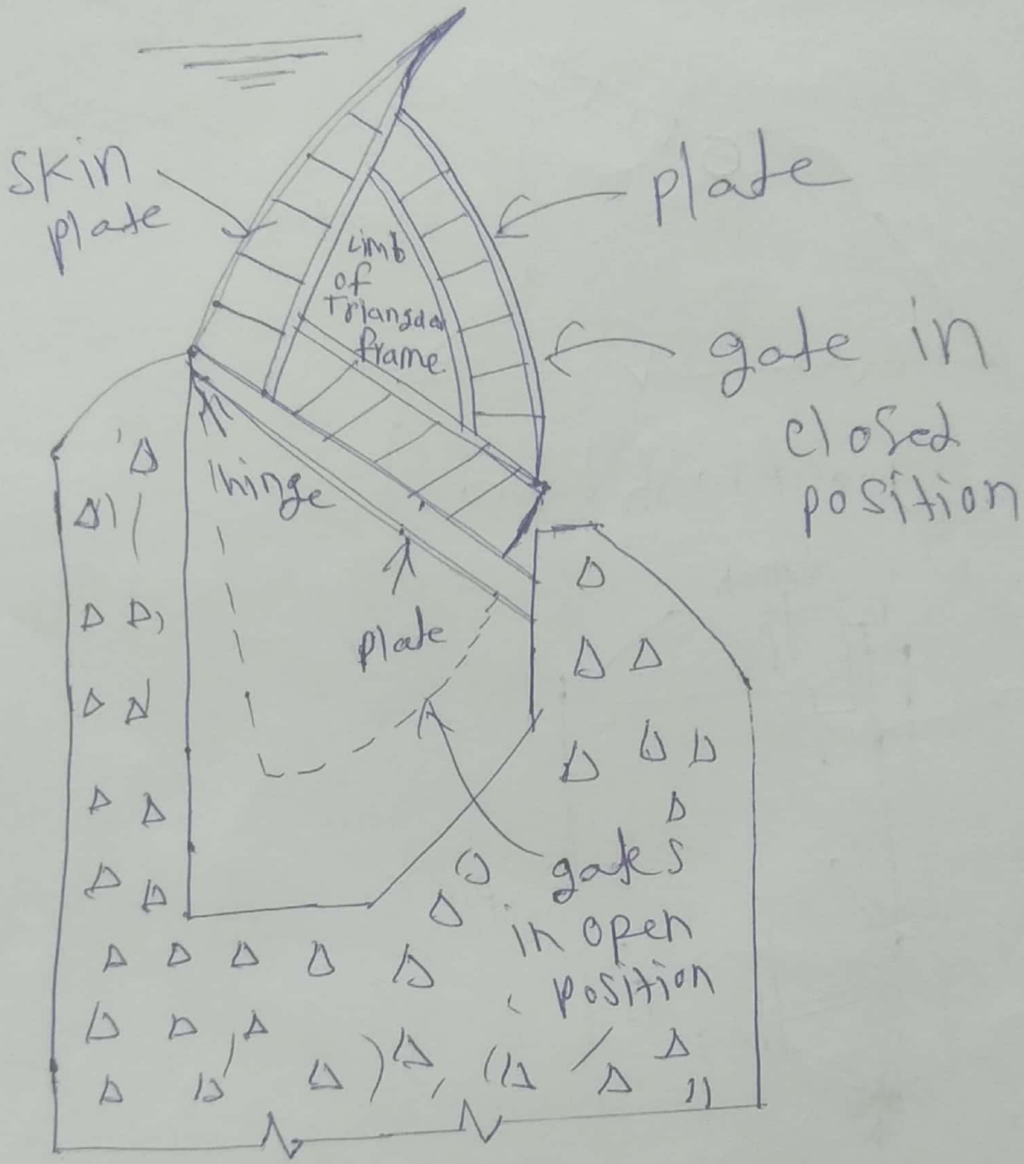


(fig - flash board).



(fig → Radial gate).

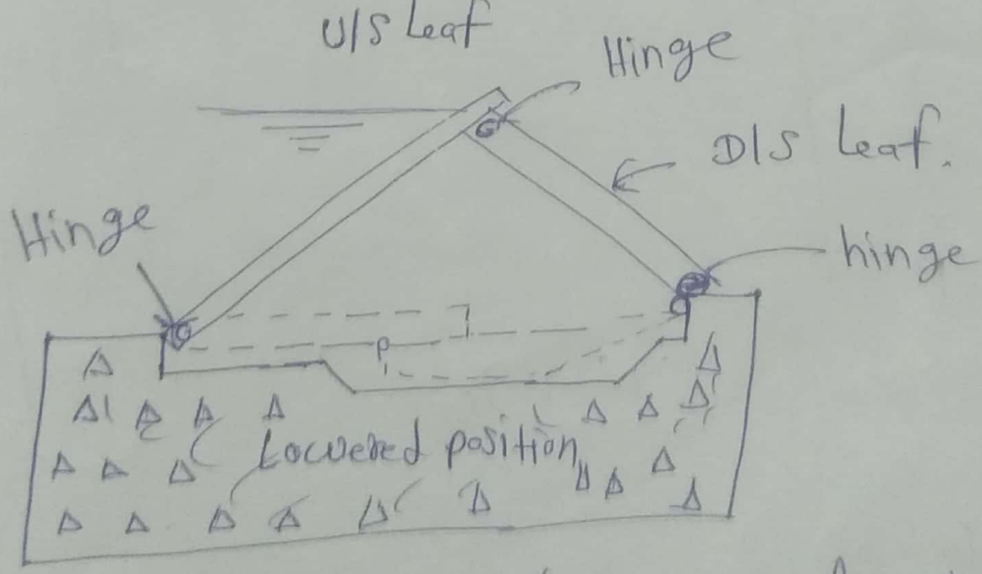
③ <u>Drum gates</u> → Drum gates are suitable for long spans with moderate heights. However, on account of a large recess (or cavity) required by the drum gates, these are not suitable for low dams where the height of spillway is small.



(fig- Drum gate)

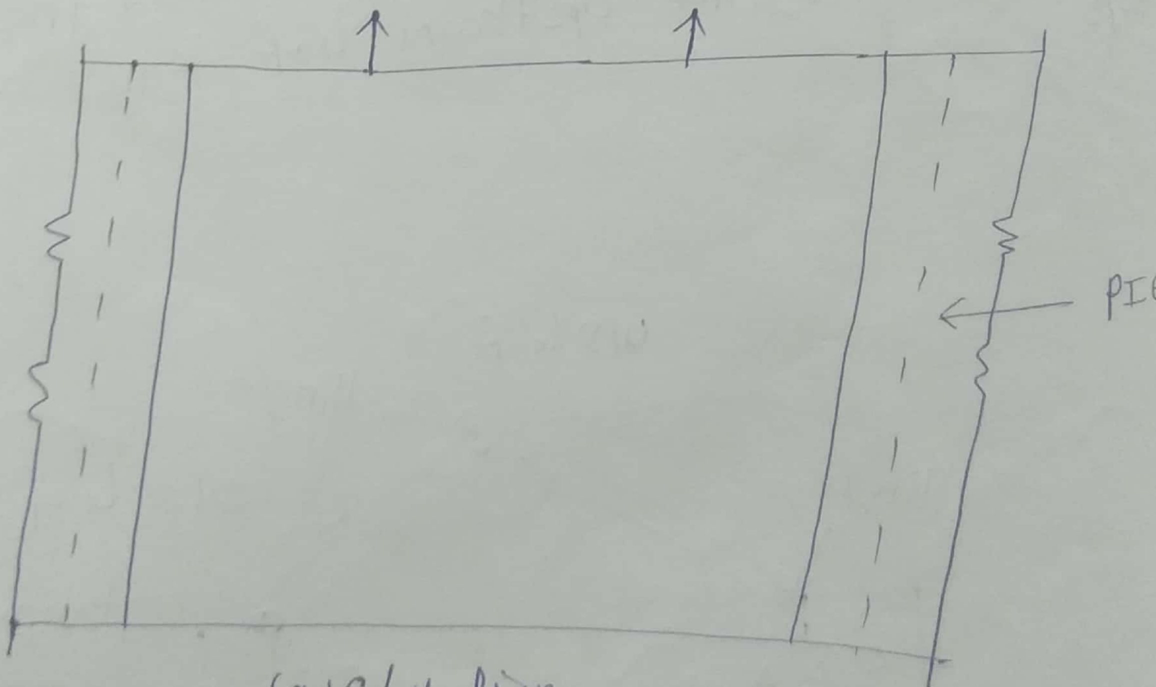
(5) Bear and Trap gates → Bear or trap gates are also known as Movable drum gates. A bear gate consists of two leaves of steel or timber, with one leaf hinged on the upstream side and the other on the downstream side over the crest.

When the water is admitted into the space below the leaves, they are forced upward and the gate is closed. When the water is removed, both the leaves lie flat (shown by dotted lines). The leaves are then in the horizontal position with the d/s leaf housed in the recess and the upstream leaf lying on its top.



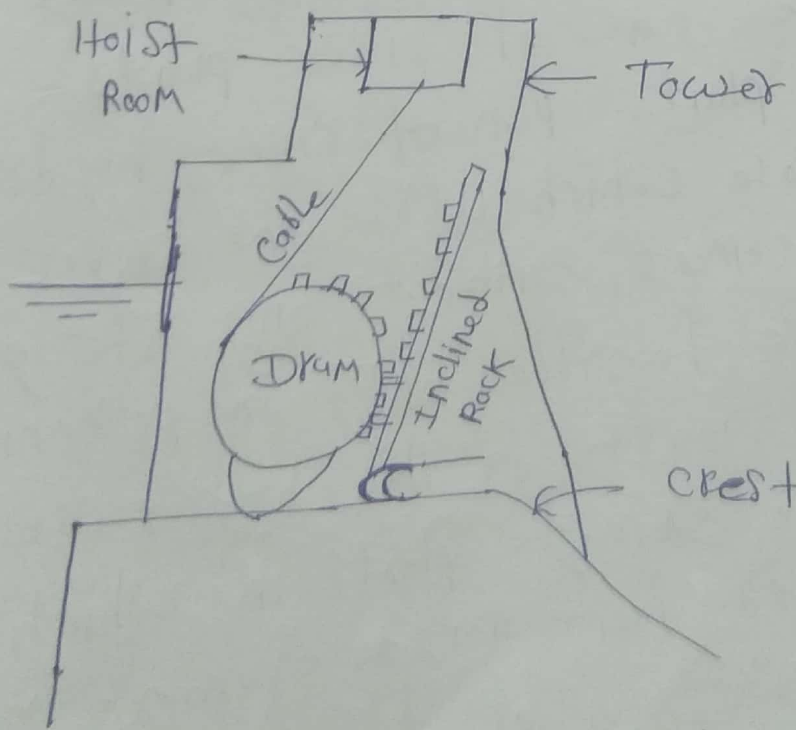
(fig - Bear and Trap gates).

④ Vertical lift gate → Vertical Rectangular gates are commonly used for spillways. A Vertical lift gate consists of a vertical framework fabricated of steel members. A steel skin plate is fixed on the upstream side of the steel framework. The vertical gate can move vertically on its own plane in the grooves provided in piers. The grooves are provided with a steel lining, which is usually a steel channel section of the required thickness and depth.



(a) Elevation

⑥ Rolling gate → It essentially consists of steel cylinder as large in diameter as the hight of opening and spanning b/w piers. A heavy annular rim having gear teeth at its periphery encircles each end of the cylinder. Each pier has an inclined rack which engages the gear teeth. The gate is rolled up the inclined rack by means of pull from the hoisting cable operated from the hoist room. A cylinderical segment, attached to the lower portion of the gate, makes contact with spillway.



(fig - Rolling gate)

* Hydropower → Water power (or hydropower) is generated by utilising the energy of water (or hydraulic energy). Hydropower is obtained from the generators coupled to water turbine which convert the hydraulic energy into the mechanical energy. High head required for running the turbines is created by constructing a dam across the river.

Most of the multipurpose scheme have hydropower as one of the major functions. Sometimes single-purpose projects only for hydropower are also undertaken if economically justified. Hydropower plants may be run-of river plants or storage plants. Run-of-river hydropower plants are those which utilise the river water as it comes, without any storage. These plants are feasible only on perennial rivers. In india, most of the hydropower plants are the storage plants in which water is supplied from large storage reservoir created by construction of dams across rivers.

In these reservoirs, the water available in the river during the floods is stored and later utilised for the generation of power and other purposes.

* general arrangement of a hydropower project →

The general arrangement of a hydropower project most suitable for a given site depends upon several factors such as head, available flow, general topography, location of the power house, the power potential and economy. A hydropower project generally includes the following components.

(1) Dam or weir → A dam is usually constructed across a river to create a storage reservoir on its upstream for storage plants. A weir is usually constructed across a river for raising the water level in the case of the run-of-river plants.

(2) Intakes → Intakes are constructed to draw the water stored in the reservoir or pond. Intakes may be integral parts of the dam or they may be separate independent structures.

If the power house is located away from the dam or weir an intake is also required at the upstream of the power house.

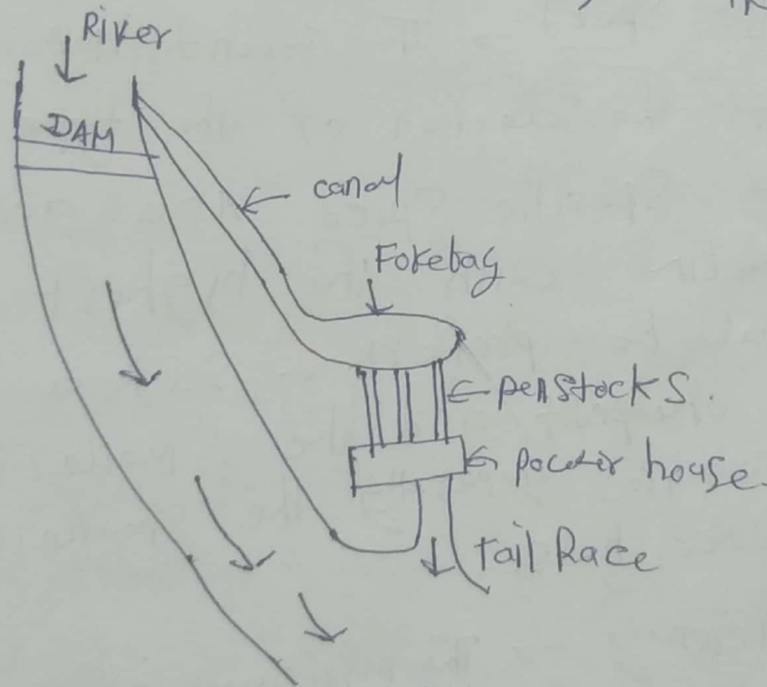
③ convegance system → The conveyance system is required to carry the water from the reservoir to the power house. It may includes canals, pipes, tunnels penstocks. (penstocks are high-pressure conduits).

④ forebay → A forebay is an enlarged body of water just upstream of the intake. It is usually the enlarged section of the power canal. A forebay is required only when the convenyance system has channel flow.

⑤ Surge tank → If the conveyance system consists of a penstock under a very high pressure a surge tank is usually required at the upstream of the power house to reduce the water hammer effect.

① Power house → A power house is a building for housing
the equipment required for generation of power
Such as turbines and generators. A power
house is generally constructed as integral part of
the dam near its toe. If it is located away
from the dam, it is an independent structure.

② Tail race → A tail race is the channel in
which the water after passing through the
turbines is discharged. In the case of reaction
turbine, the water of the turbine passes through a
draft tube before it is discharged into the
tail race.



(Layout of a plan).

* Selection of turbine → Turbines most
commonly used in practice are kaplan turbines,
Francis turbines and pelton wheel turbines.
The other types of turbines which are used
in some plants are Deriaz (or diagonal)

turbines and tabular turbines (or bulb turbines). Different types of turbines have their own specific characteristics and are therefore specially suited for specific conditions of head, power, and speed.

While selecting the turbine for a particular power plant, the following factors should be considered.

① **Head** → The head available at the plant is the most important factor governing the selection.

② **Specific Speed** → The second important factor which affects the selection of the type of turbine is the specific speed. As a general rule, a turbine with the highest specific speed should be preferred. Such a turbine usually the cheapest and the smallest in size. It requires generally the smallest generator and the power house.

③ **Efficiency** → The maximum efficiency varies with the type of turbine and the specific speed. Generally, the efficiency falls with an increase of specific speed both for the impulse and reaction turbines beyond the optimum value. The efficiency is also low at very low specific speed.

④ part load operation →

If the turbine is likely to run at part load. the turbine which has high efficiency at part load should be selected. The efficiency of the pelton wheel at part load is greater than that of a francis turbine.

The efficiency of a kaplan turbine at part load is much greater than that of francis turbine. As far as possible a kaplan turbine should be selected for the best efficiency at part load if there is a choice.

⑤ Sediments → If the water contains a large amount of sand and other coarse materials, the reaction turbine should be avoided, since its runner would be able to withstand high erosive action. A pelton wheel should be preferred to a francis turbine such a case, if there is a choice.

⑥ Overall cost → The turbine which has the lowest overall cost, including the initial cost and the maintenance cost, should be selected as far as possible.

ke

(7) <u>cavitation</u> → while Selecting a reaction turbine its cavitation characterstics should be considered. The turbine which gives best cavitation free performance should be selected..

ft tube →

A draft tube of a reaction turbine discharges at a level below the river bed. The upslope of the tail race channel is usually kept 1 in 8 from the draft tube exit to the river bed. The width and depth of the tail race channel depend upon the number of turbine units, the design discharge, the width of the draft tube and the thickness of the piers b/w the adjacent draft tube.

cavitations → cavitation is defined as the phenomenon of formation of vapor bubbles of a flowing liquid in a region where the pressure of the liquid falls below its vapor pressure and the sudden collapsing of these vapor bubbles in a region of higher pressure. When the vapor bubbles collapse a very high pressure is created. The metallic surfaces, above which these vapor bubbles collapse, is subjected to these high pressure

ke

which cause pitting action on the surface. Thus cavities are formed on the Metallic Surface and also considerable noise and vibrations are produced.

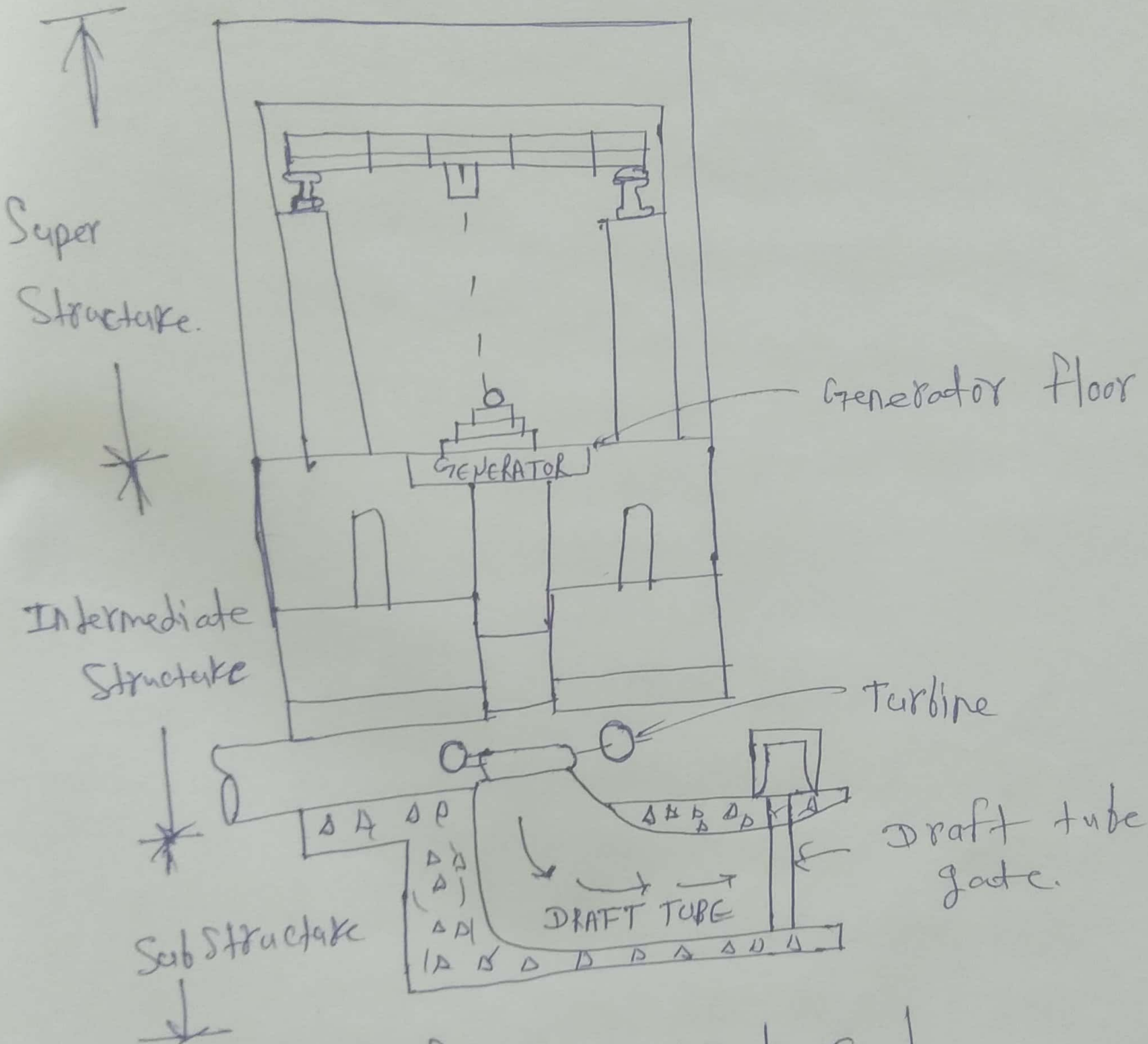
* **Power house** → The power house is a building constructed for housing and protection of the various hydraulic and electrical equipment required for the generation of power, such as spiral casing, turbines, governors, draft tube, generator etc. The power house consists of three main parts :-

1. Sub structure
2. Intermediate structure
3. Superstructure.

1. **Substructure** → The substructure of a power house consists of a large foundation block situated below the turbine level. It is usually monolithic concrete structure b/w the foundation bed and the turbine level with suitable waterways formed within it. The draft tubes are generally cast monolithically with the substructure when the concreting is done, with steel liners serving as form work.

② **Intermediate Structure** → The turbines including their casing, governor, gates val are housed in the intermediate structure. It extends from the top of the draft tube to the generator floor. The casing may be made of concrete for low-head units, but it consists of steel plates embedded in concrete for high-head units.

③ **Superstructure** → Superstructure is the main building of the power house which is constructed above the generator floor. It houses the generator the exciters, and other operating equipments. Vertical shaft turbines are placed immediately below the generators, whereas the horizontal shaft turbines are placed on the generator floor along side the generators.
The switch boards and the control room may be placed on the generator floor, but usually they are placed in a mezzanine floor.
All the generating units are placed in a row at right angles to the direction of flow of water through ...



(fig - A power house Structure)

UNIT- ~~IV~~ th

RESERVOIRS AND OPTIMIZATIONS

INTRODUCTION → A reservoir is a large artificial lake created by constructing a dam across a river. Broadly speaking any water pool or a lake may be termed ~~as~~ a Reservoir. However the term reservoir in water resources engineering is used in a restricted sense for a comparatively large body of water stored on the upstream of a dam constructed for this purpose.
Thus a dam and a reservoir exist together.

* **purpose of Reservoir** → The Storage Reservoir is formed for the following purposes →

① Flood control

② Irrigation.

③ water supply.

④ Hydro electric power generation.

⑤ Development of fishrey.

⑥ Navigation.

⑦ Soil conservation.

* classification of Reservoir → There are following classification of Reservoir →

① Storage Reservoir

② Flood control Reservoir

③ Retarding Reservoir.

④ Detenition Reservoir.

⑤ Distribution Reservoir.

① Storage Reservoir → The storage reservoir is formed by constructing a dam across a river valley. The idea of constructing such a reservoir is to store the excess water which flows through the river during the high floods or rainy season. This stored water is then utilised for vairous purposes. Such as irrigation, water supply, fishery, hydroelectr power generation etc.

Flood control Reservoir →

are formed by constructing dams at suitable places in the river valley to arrest flood water temporarily so that the D/S area may not get damaged by sudden high flood discharge. The arrested water is then allowed to flow or released gradually without causing any harm to the low lying areas on the D/S side. This type of Reservoir is designed as single purpose reservoir. The flood control Reservoir may be of two types.

(a) Retarding Reservoir → In this type of Reservoir are provided with the dam at such level and capacity so that the flood discharge through them is safe for the D/S areas. That means the high flood discharge is retarded and it takes long time for the flood water to flow completely towards the D/S area.

The discharge stops when the water level falls below the crest of the spillways.

(b) Detention Reservoir →

In this type of Reservoir the spillways with adjustable gates are provided with the dam so that the flood water may be detained for sometime and then released according to the situation of the D/S area by operating the gates of the spillways.

③ Distribution Reservoir →

The distribution reservoir is not formed by constructing a dam across a river valley or river. It is constructed by Masonry work or concrete work in the form of a rectangular or circular tank at suitable places near the town or city. The water from the river or lake is pumped into this reservoir and stored there for supplying to the consumers of the town or city. The water may be supplied to consumers by pumping system or gravity system.

Reservoir Sedimentation →

The Sediments are produced in the catchment of the river by erosion. Rivers carry a large amount of Sediments load along with river. These Sediments are deposited in the reservoir on the U/P of the dam because of reduction of velocity. Sedimentation reduces the available capacity of the reservoir. with continuous Sedimentation, the useful life of the reservoir goes on decreasing.

The Sediments load of a river depends upon the following factors →

(1) Nature of Soil in the catchment.

If the soil in the catchment is loose and easily erodible, the Sediment load is large. on the other hand. If the soil is hard and non erodible, the Sediments load is quite Small.

(2) Vegetal cover → If the catchment area has no vegetal cover, the soil is easily eroded and the Sediment load is large.

③ Topography of the catchment → In case of catchments having steep slopes, the sediment load is large because of high velocity of water.

④ Intensity of Rainfall → If the intensity of rainfall is high, the discharge in the river is increased and the sediment load is large.

* Measures to control Reservoir Sedimentation →

The following measures are usually taken to reduce the reservoir sedimentation →

① Selection of suitable site → The reservoir should be at the location where the sediment inflow is low. It should exclude the run-off from an easily erodible part of catchment.

② Proper design → The reservoir may be designed in such a way that its capacity increases in stages. Initially a reservoir of smaller capacity is created by constructing the dam to a lower height. After a portion of the reservoir gets filled up with sediments, the height of the ...

(4) creating large Reservoir → As far as possible large reservoir should be created. Therefore the useful life of a large reservoir is longer than that of a small reservoir if all other factors remain constant of course, the cost of larger reservoir will also be more.

(5) control of Sediment inflow → The inflow of Sediments to a reservoir can be controlled by the following methods →

(i) check dams → A check dam is a small dam constructed on a stream to trap the Sediments carried by the stream. check dams are constructed on the tributaries carrying large quantity of Sediments.

(ii) Vegetation screens → Vegetal covers on the catchment reduces the impact of rain drops and hense minimises erosion. Vegetal s screen is developed by promoting the growth of Vegetation.

* Computer Aided irrigation design →

In last 20 years computer aided designs have become more popular.

*** Advantages** → There are following advantages –

(i) Save time in designing and planning process of any irrigation project.

(ii) Solve complicated equations involving more than 10 variables.

(iii) Getting results with alternative is quite easy.

(iv) Designs are comparatively more accurate.

(v) Economical.

(vi) More useful even in future designs.

Various softwares are available in Market for design of hydraulic structures and saving time for different types of problem selected to irrigation, hydraulics etc. Some of these software are STORM, CIVIL STORM, STORM CAD, pondpade etc.

on the Shallow tube well or deep tube well for irrigation which works out to be expensive for them. So, the watershed management becomes very important in village area now-a-days. The water shed management may be done in two ~~days~~ ways.

(1) preventive measures → The following steps should be taken :→

(a) contour bunds or terrace bunds →

Along the slope of the hilly catchment area contour bunds or terrace bunds should be constructed at different levels. These bunds from the water pockets which arrest the sediments and serve as detention basins for the heavy run-off during the rainy season.

(b) Small dams → Small dams are constructed across the tributaries of a river and even on the river at the upstream region to form small reservoirs where sediments are arrested and the flood water is detained.

* Watershed Mangement → The area enclosed within the watershed area line is known as watershed area. The watershed area may consist of hilly area and plane area. In hilly area there is no possibility of water logging, but the sediments carried by the tributaries badly affect the D/S plane area. In the plane area the rivers, streams etc are silted up by the sediments and the water carrying capacity is reduced. Consequently, the surrounding areas may be submerged during the flood. Sometimes, due to lack of proper drainage system the vast area may get water logged.

The cultivation becomes impossible and it creates problems for the villagers. It is also found that the low lying areas of some villages get submerged due to heavy rainfall and the are remains under water for a long period. During this period, the agricultare in the area is totally stoped. But in dry season, when the whole area is dried up the villagers depend

(c) Soil conservation → The soil conservation methods should be ~~detected and effective methods~~ adopted in the catchment area. These methods include the method of afforestation, prevention of deforestation, control of grazing etc.

(d) Slip stabilisation → The slip or land slides in the catchment area should be detected and effective method are employed to stabilise them.

(e) Control of cultivation in catchment area → cultivation should be done in controlled way with shallow marginal bund along the boundary of specified land so that the loose soil is not washed out by rain water and carried to the D/S area.

Optimization Techniques →

optimization is the technique of obtaining best result under given circumstances. The optimization techniques also known as mathematical programming techniques' are the methods which gives the best results, under the given conditions to the given programming problems.

In water Resources planning optimization problems are encountered at three stages →

① Deals with individual features of the projects → given cost diameter and head loss-diameter function of a penestock is to determine accurately the optimum (least cost) solution.

② Involve Single project → To some extent sub optimization of project limits lead to optimization of the whole project.

③ Involve System of projects → multiple reservoirs, canals etc.

Different methods of optimization techniques are —

1. Linear programming (simplex algorithm)
2. Linear programming (graphical method)
3. Dynamic programming.
4. Lagrange multipliers.
5. Simulation
6. Hydrologic modellings.

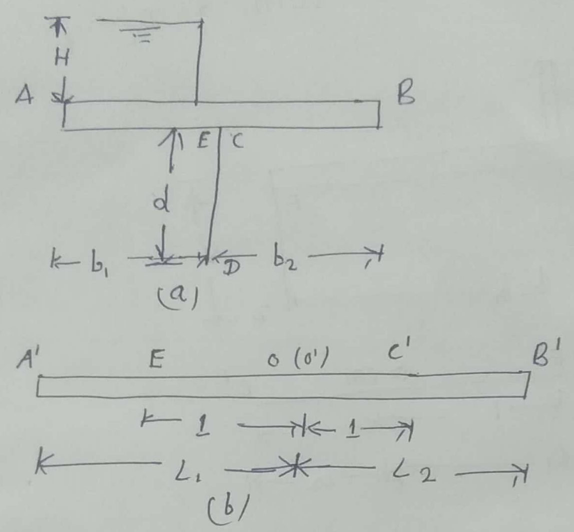
* G.I.S (geographic information system) → It is a system of hardware, software and procedures designed to support the capture, management manipulation analysis modelling and display of spatially referenced data for solving complex planning and management problems.

Geographical information system (GIS) today is an indispensable tool in the planning for tommorrow requirements, such as rail network, dam site location, urban development etc.

UNIT- 2nd

Numerical portion

* Impervious Floor With an Intermediate pile ⟶



The uplift pressure at salient points E, D and C are given by the following equations ⟶

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

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

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

where

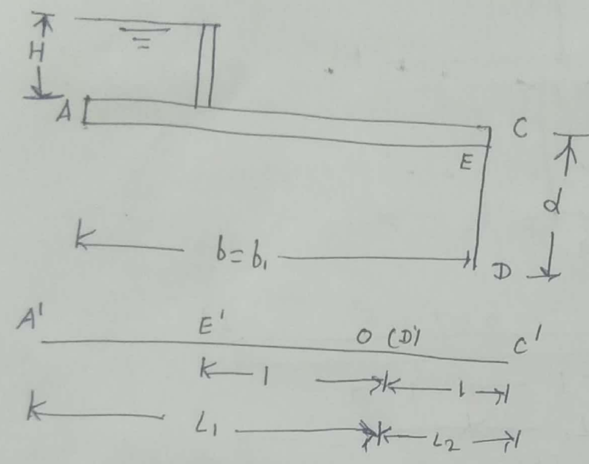
$$\lambda = \frac{L_1 + L_2}{2} = \frac{\sqrt{1+\alpha_1^2} + \sqrt{1+\alpha_2^2}}{2}$$

$$\lambda_1 = \frac{L_1 - L_2}{2} = \frac{\sqrt{1+\alpha_1^2} - \sqrt{1+\alpha_2^2}}{2}$$

In these equations

$\alpha_1 = b_1/d$ and $\alpha_2 = b_2/d$

* Impervious Floor With a D/S pile ⟶



The uplift pressure at the Salient points E, D and C are given by the following equations.

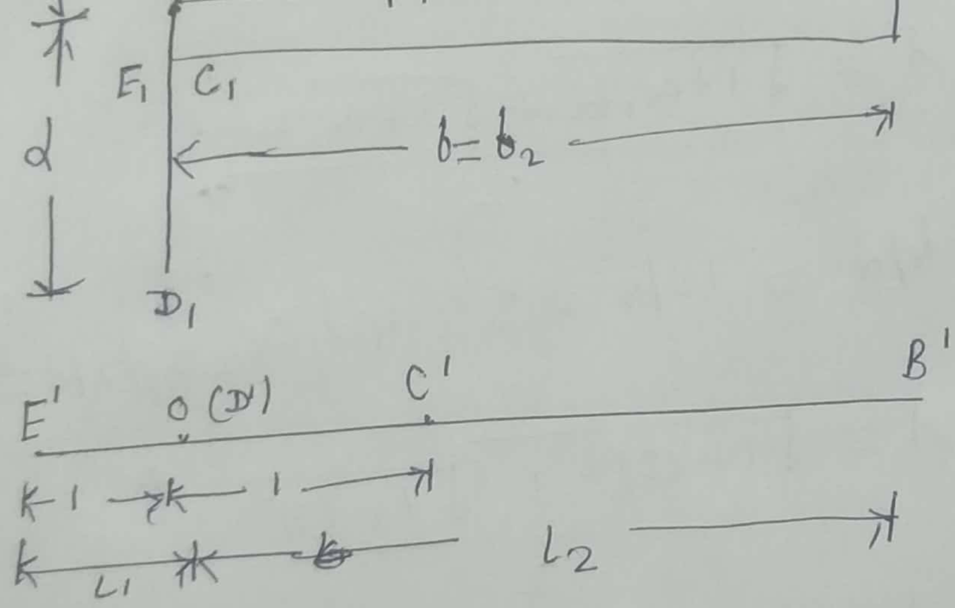
$$P_E = \frac{H}{\pi} \cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$$

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

$$P_C = 0$$

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

in which $\alpha = b/d$



The uplift pressure at the salient points E_1, D_1 and C_1 are given by the following equations.

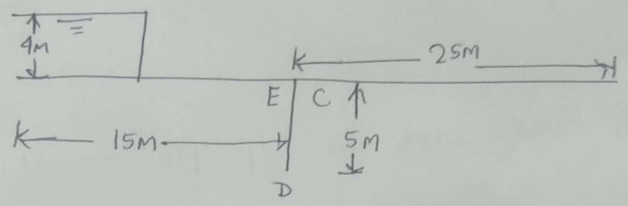
$$P_{E_1} = H$$

$$P_{D_1} = \frac{H}{\pi} \cos^{-1}\left(\frac{1-\lambda}{\lambda}\right)$$

$$P_{C_1} = \frac{H}{\pi} \cos^{-1}\left(\frac{2-\lambda}{\lambda}\right)$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$$ and $\alpha = b/d$

(1) Determine the uplift pressure at the Salient points E, D and C of the intermediate pile as shown in fig.



In this case, $b_1 = 15m$, $b_2 = 25m$, $b = 40m$

$d = 5m$ and $H = 4m$

from eq

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

where $\alpha_1 = b_1/d = 15/5 = 3$ and $\alpha_2 = b_2/d = 25/5 = 5$

Therefore

$$\lambda = \frac{\sqrt{1+(3)^2} + \sqrt{1+(5)^2}}{2}$$

$$\lambda = \frac{3.16 + 5.10}{2} = 4.13$$

$$\lambda_1 = \frac{\sqrt{1+(3)^2} - \sqrt{1+(5)^2}}{2} = \frac{3.16 - 5.10}{2}$$

$$\boxed{\lambda_1 = -0.97}$$

from eq

$$P_E = \frac{H}{\pi} \cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right) = \frac{4}{\pi} \cos^{-1}\left(\frac{-0.97-1}{4.13}\right)$$

$$P_E = \frac{4}{\pi} \times \left(\frac{118.489^\circ}{180} \times \pi\right) = 2.633m$$

from eq

$$P_C = \frac{H}{\pi}\cos^{-1}\left(\frac{\lambda_1+1}{\lambda}\right) = \frac{4}{\pi}\cos^{-1}\left(\frac{-0.97+1}{4.13}\right)$$

$$P_C = \frac{4}{\pi} \times \frac{89.584}{180} \times \pi$$

$$\boxed{P_C = 1.991\,m}$$

and

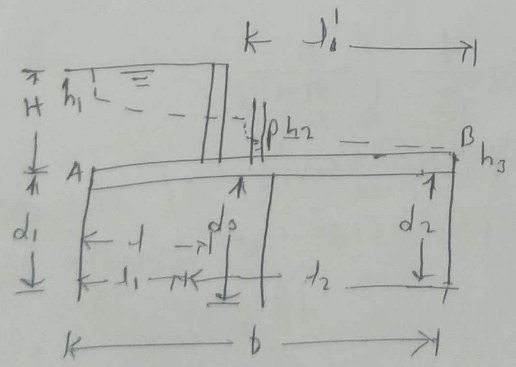
$$P_D = \frac{H}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$$

$$P_D = \frac{4}{\pi}\cos^{-1}\left(\frac{-0.97}{4.13}\right)$$

$$P_D = \frac{4}{\pi} \times \frac{103.584}{180} \times \pi$$

$$\boxed{P_D = 2.302\,m}$$

* Bligh creep theory →



Total creep length

$$L = 2d_1 + l_1 + 2d_3 + l_2 + 2d_2$$

where d_1, d_2 and d_3 are the depth of U/S and D/S intermediate piles. d_1 and d_2 are the length of the U/S and D/S floors.

The sub soil hydraulic gradient is given by

$$i = H/L$$

uplift pressure Formula

$$h = H - \left(\frac{H}{L}\right) \times l$$

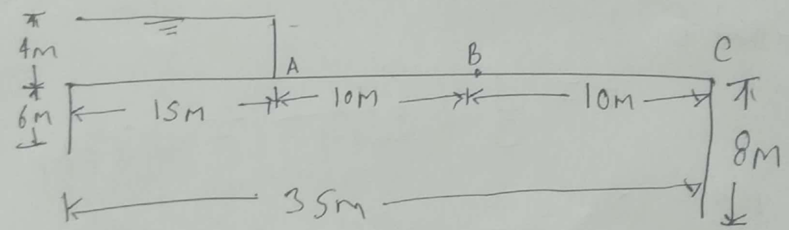
where l is the horizontal length from the entry point A to point P.

thickness of Floor

$$t = \frac{4}{3}\left(\frac{h}{G-1}\right)$$

G = specific gravity of the Floor material. G usually varies from 2 to 2.30.

Example ① fig shows a hydraulic structures built on fine sand ($c = 15$)
Determine uplift pressure at point A, B, and C at distance 15, 25 and 35m from the u/s end. and also calculate thickness of floor. Use bligh creep theory $G = 2.24$.



Creep length

$L = 2 \times 6 + 35 + 2 \times 8 = 63m$

hydraulic gradient $i = H/L$

$i = 4/63 = \frac{1}{15.75} < \frac{1}{15}$ (Safe)

uplift pressure head ⟶ $h = H - \left(\frac{H}{L}\right)L$

At point $A = 4 - \frac{1}{15.75}(2 \times 6 + 15)$

$= 2.29m$

At point B $= 4 - \frac{1}{15.75}(2\times 6 + 25) = 1.65m$

At point C $= 4 - \frac{1}{15.75}(2\times 6 + 35) = 1.02m$

* Thickness of Floor ⟶

$$t = \frac{4}{3}\left(\frac{h}{G-1}\right)$$

At point A $= \frac{4}{3} \times \left(\frac{2.29}{2.24-1}\right) = 2.46m$

At point B

$$t = \frac{4}{3}\left(\frac{1.15}{2.24-1}\right) = 1.77m$$

At point C $t = \frac{4}{3}\left(\frac{1.02}{2.24-1}\right) = 1.10m$