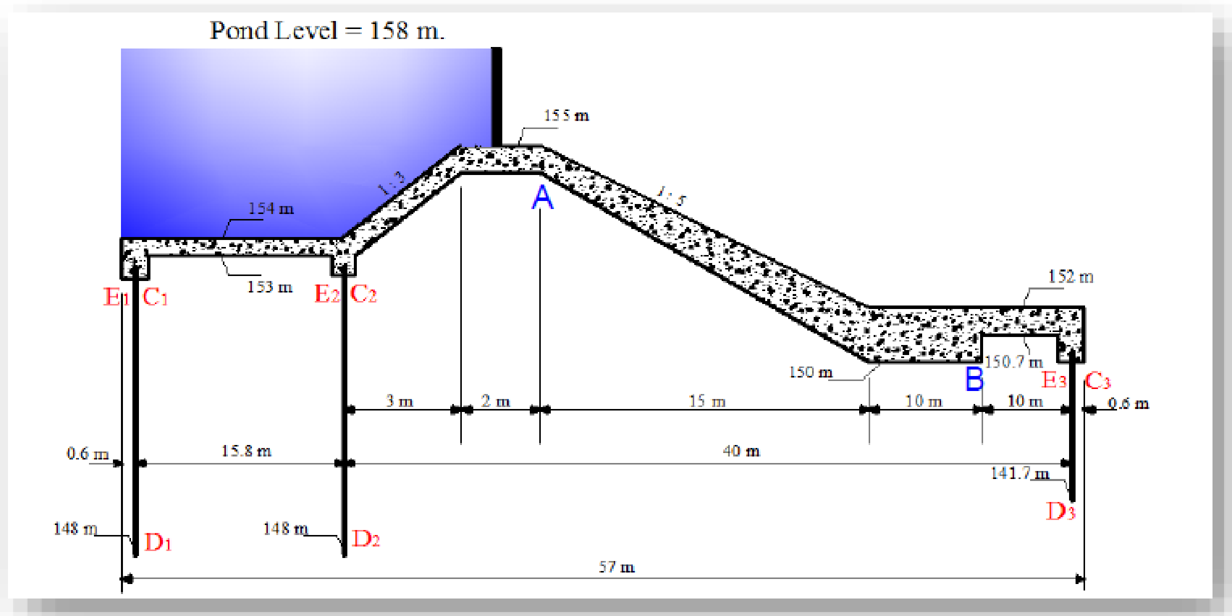
**Example ①:** Determine the percentage pressures at various key points in figure below. Also determine :

1) The exit gradient ;
2) Plot the hydraulic gradient line for pond level on U/S and no flow on D/S.
3) Whether the section provided safe against uplift pressure at points ( A , B ) and piping if it is founded on fine sand with permissible exit gradient of (1/6).




**Solution :-**

1) **Pile No.1 at the u/s**

$d_1 = d = 154 - 148 = 6m$

$b = 57m$

$$\emptyset_{C1} = \frac{100}{\pi}\cos^{-1}\left[\frac{2-\lambda}{\lambda}\right]$$

$$\emptyset_{D} = \frac{100}{\pi}\cos^{-1}\left[\frac{1-\lambda}{\lambda}\right]$$

$$\emptyset_{E} = 100\%$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} \quad , \qquad \alpha = \left(\frac{b}{d}\right) = \left(\frac{57}{6}\right) = 9.5$$

$\lambda = 5.27$

$\therefore \emptyset_{C1} = 71.3\%$ ,,, $\emptyset_{D1} = 80\%$ ,,, $\emptyset_{E1} = 100\%$

- Correction for floor thickness

  $t_1$ = 154-153=1m ,,, d=6m

  $$\text{Correction for } C_1 = \frac{\emptyset_D - \emptyset_C}{d_1} \times t_1 = \frac{80-71.3}{6} \times 1 = 1.45\%(+)$$

- Correction for interference of pile No.2

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

Here d = 153-148= 5m ; $b'$ = 15.8m ; b=57m ; D=153 – 148 = 5m

$$C = 19\sqrt{\frac{5}{15.8}}\left(\frac{5+5}{57}\right) = 1.87\%(+)$$

$\therefore$ ***Corrected pressures are*** $\boldsymbol{\emptyset_{C1} = 71.3 + 1.45 + 1.87 = 74.62\%}$

2) **intermediate pile line**

   $d=d_2$=154 – 148 = 6m ; b =57m ; $b_1$= 15.8m ; $b_2$= 40m

$$\emptyset_E = \frac{100}{\pi}\cos^{-1}\left[\frac{\lambda_1 - 1}{\lambda}\right]$$

$$\emptyset_D = \frac{100}{\pi}\cos^{-1}\left[\frac{\lambda_1}{\lambda}\right]$$

$$\emptyset_C = \frac{100}{\pi}\cos^{-1}\left[\frac{\lambda_1 + 1}{\lambda}\right]$$

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

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

$$\alpha_1 = \left(\frac{b_1}{d}\right) = \left(\frac{15.8}{6}\right) = 2.63$$

$$\alpha_2 = \left(\frac{b_2}{d}\right) = \left(\frac{40}{6}\right) = 6.66$$

$\lambda$ =4.77 ; $\lambda_1 = -1.96$

$\therefore\ \emptyset_{E2}$ =71.3% ,,, $\emptyset_{D2} = 63.4\%$ ,,, $\emptyset_{C2} = 56.4\%$

- Correction for floor thickness

t = 154-153=1m ,,, d=6m

Correction for $\emptyset_{C2} = \frac{\emptyset_D - \emptyset_C}{d_2} \times t_1 = \frac{63.4 - 56.4}{6} \times 1 = 1.16\ \ \%(+)$

Correction for $\emptyset_{E2} = \frac{\emptyset_E - \emptyset_D}{d_2} \times t_1 = \frac{71.3 - 63.4}{6} \times 1 = 1.31\ \ \%(-)$

- Correction for interference of pile No.1

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

Here d = 153-148= 5m ; $b' = 15.8$m ; b=57m ; D=153 – 148 = 5m

$$C = 19\sqrt{\frac{5}{15.8}}\left(\frac{5+5}{57}\right) = 1.87\%(-)$$

- Correction for interference of pile No.3

Here d = 153-148= 5m ; $\mathrm{b}' = 40\mathrm{m}$ ; b=57m ; D=153-141.7= 11.3m

$$Corr.for\ c2 = 19\sqrt{\frac{11.3}{40}}\left(\frac{5+11.3}{57}\right) = 2.88\%(+)$$

- Correction due to slope of 3:1

Correction from table = 4.5%

Correction is negative due to upward slope in the direction of flow

Horizontal length of the slope = 3 m

∴Actual correction , to be applied to $\emptyset_{C2} = 4.5 \times \frac{3}{40} = 0.3\%(-ve)$

Hence the corrected pressure are

$\boldsymbol{\emptyset_{E2} = 71.3 - 1.31 - 1.87 = 68.12\%}$

$\boldsymbol{\emptyset_{C2} = 56.4 + 1.16 + 2.88 - 0.3 = 60.14\%}$

3) **Pile No.3 at the d/s**

$d_3$= d= 152–141.7= 10.3m

b =57m

$$\emptyset_E = \frac{100}{\pi}\cos^{-1}\left[\frac{\lambda-2}{\lambda}\right]$$

$$\emptyset_D = \frac{100}{\pi}\cos^{-1}\left[\frac{\lambda-1}{\lambda}\right]$$

$$\emptyset_C = \frac{P_{C1}}{H} \times 100 = 0$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}\ , \qquad \alpha = \left(\frac{b}{d}\right) = \left(\frac{57}{10.3}\right) = 5.53$$

$\lambda = 3.3$

$\therefore \emptyset_{C3} = 0\%$ ,,, $\emptyset_{D3} = 25.45\%$ ,,, $\emptyset_{E3} = 37.11\%$

- Correction for floor thickness

t = 152–150.7 =1.3m ,,, d=152–141.7= 10.3m

$$\text{Correction for } \emptyset_{\text{E3}} = \frac{\emptyset_{\text{E}} - \emptyset_{\text{D}}}{\text{d}_3} \times \text{t}_3 = \frac{37.11 - 25.45}{10.3} \times 1.3 = 1.47 \quad \%(-)$$

- Correction for interference of pile No.2

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

Here d=152–141.7= 10.3m ; $\text{b}' = 40\text{m}$ ; b=57m ; D=150.7 – 148 = 2.7m

$$Corr.forE = 19\sqrt{\frac{2.7}{40}}\left(\frac{10.3+2.7}{57}\right) = 1.125\%(-)$$

$\therefore$ ***Corrected pressures are*** $\boldsymbol{\emptyset_{E3} = 37.11 - 1.47 - 1.12 = 34.52\%}$

Total head = 158 – 152=6 m

| Point | % pressure (∅) | Pressure head ($P$)m |
|---|---|---|
| $E_1$ | 100 | 6 |
| $C_1$ | 74.62 | 4.47 |
| $E_2$ | 68.12 | 4.08 |
| $C_2$ | 60.14 | 3.6 |
| $E_3$ | 34.52 | 2.07 |
| $C_3$ | 0 | 0 |

**1) Exit Gradient**

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

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}, \qquad \alpha = \left(\frac{b}{d}\right) = \left(\frac{57}{10.3}\right) = 5.53$$

$\lambda = 3.3 \quad , H = 6m \quad , d_3 = 10.3$

$G_E$ =0.1 < permissible exit gradient (1/6) $\therefore$ o.k. safe against piping

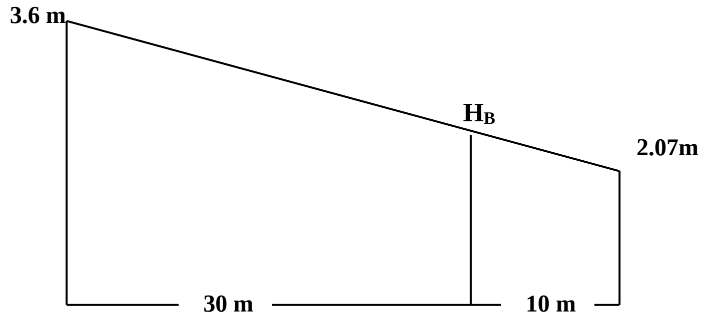
**2) Thickness at point B**

$$* \ t = \frac{H_p}{G_S - 1} \qquad \boldsymbol{, G_s = 2.24}$$

- First find Hp at point B by using interpolation between point $C_2$ and $E_3$

**$H_{C2}$= 3.6 m**

**$H_{E3}$= 2.07 m**

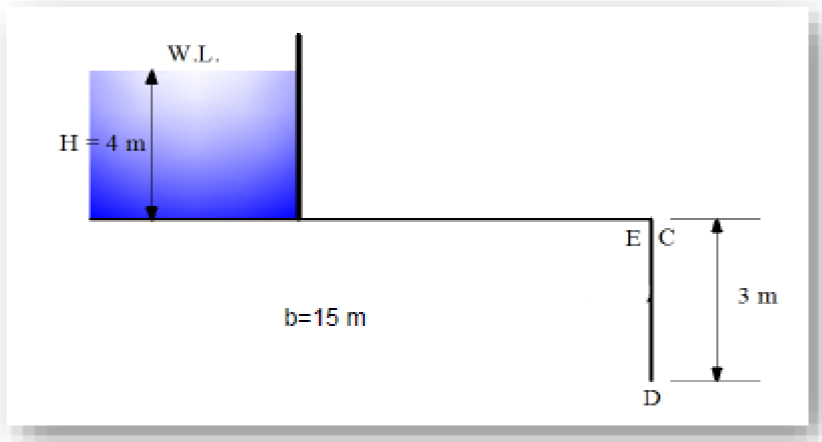



$$\frac{3.6-2.07}{40}=\frac{\mathrm{H_p}-2.07}{10}\rightarrow\ \ \mathrm{H_P}=2.45\mathrm{m\ of\ water}$$

$$-\,t_B=\frac{2.45}{2.24-1}=1.97\mathrm{m}<thickness\ at\ this\ point\ (152-150)$$

$$=2m\ ok.\,and\ \text{safe against uplift pressure}\ at\ this\ point$$

**H.W. ①:** A hydraulic structure with length of horizontal floor in alluvial soil 15 m and 3m deep vertical sheet pile is attached at its downstream end and the head of water is 4.0m. Find the thickness of the floor (using Khosla's theory). Is the structure safe against the exit gradient? (F = 1/8 , G = 2.45 ).




**Solution :-**

**Pile No.3 at the d/s**

$d_3$= d= 3m

b =15m

$$\emptyset_E = \frac{100}{\pi}\cos^{-1}\left[\frac{\lambda-2}{\lambda}\right]$$

$$\emptyset_D = \frac{100}{\pi}\cos^{-1}\left[\frac{\lambda-1}{\lambda}\right]$$

$$\emptyset_C = \frac{P_{C1}}{H}\times 100 = 0$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}\ , \qquad \alpha = \left(\frac{b}{d}\right) = \left(\frac{15}{3}\right) = 5$$

$$\lambda = 3.05$$

$\therefore \emptyset_{C3} = 0\%$ ,,, $\emptyset_{D3} = 26.53\%$ ,,, $\emptyset_{E3} = 38.81\%$

**لايوجد تصحيح للسمك لعدم وجود قيمه لسمك الأرضية ولايوجد تاثير ركيزه اخرى وبالتالي راح نعتمد فقط على ضغط في E**

## 1) Thickness at point B

$$* \ t = \frac{H_p}{G_S - 1} \qquad , \boldsymbol{G_s = 2.24}$$

Hp=4*0.3881=1.55m

$$* \ \mathrm{t} = \frac{1.55}{\mathrm{G_S} - 1} \qquad = 1.25\mathrm{m}$$

## 2) Exit Gradient

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

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}, \qquad \alpha = \left(\frac{b}{d}\right) = \left(\frac{15}{3}\right) = 5$$

$\lambda = 3.05$ $\quad , \mathrm{H} = 4\mathrm{m}$ $\quad , \mathrm{d}_3 = 3$

$G_E$ =0.24 > permissible exit gradient (1/8) $\therefore$ $\text{not o.k. ( not safe against piping)}$

# Introduction

## 1) Introduction :

A hydraulic structure is a structure submerged or partially submerged in any body of water, which disrupts the natural flow of water. They can be used to divert, disrupt or completely stop the flow. An example of a hydraulic structure would be a dam, which slows the normal flow rate of the river in order to power turbines. A hydraulic structure can be built in rivers, a sea, or any body of water where there is a need for a change in the natural flow of water.

Hydraulic structures may also be used to measure the flow of water. When used to measure the flow of water, hydraulic structures are defined as a class of specially shaped, static devices over or through which water is directed in such a way that under free-flow conditions at a specified location (point of measurement) a known level to flow relationship exists. Hydraulic structures of this type can generally be divided into two categories: flumes and weirs

## 2) Types of Hydraulic Structures :

According to the purpose of its function

1- **Storage Structures:** the function is to store water such as Dams & Tanks.
2- **Conveyance Structures:** the function is to convey water from place to another such as pipelines, Siphons, Culverts, Tunnels, aqueducts and open channels.
3- **Flow Diversion Structures:** the function is to regulate and divert the quantities of flow to another structures or canals such as barrages and regulators.

4- **Flow measurement structures:** the function is to measure the flow passing through it such as Weirs, Orifices, nozzles, Venturi .
5- **Energy dissipation structures:** the function is to protect the floor of hydraulic structure from erosion and damage due to severe waves which impact with the body of structure such as Stilling basins, Surge tanks, Check dams and vertical drop.
6- **Power Stations:** the function of these structure is to convert energy from a case to another such as Pumps, Turbines & Rams.
7- **Sediment and Chemical Control Structures:** the function is to control or remove sediments and other pollutants such as Sedimentation tanks, Screens, Traps, Filters & mixing basins.
8- **River Training and Waterway Stabilization Structures:** the function is to maintain river channel and water transportation such as Levees, Cutoffs, Locks, Piers Dikes, Groins, Jetties and Revetments

**Remark:** For any hydraulic structure to design, we must study the following:

1- Hydrologic studies.
2- Hydraulic studies.
3- Structural studies.

## 3) Steps for Design of Hydraulic Structures:

To construct any hydraulic structure, the following steps must be considered:

**1.** Prepare information for design.

a) The precise function of design.

b) Discharge (Max. & Min.) Use 1.2 Q max. discharge & 0.7 Q for min. discharge.

c) Head loss.

d) U/S & D/s canal.

**2.** Determine the best location of the structure.

**3.** The shape of approach and the other components of the structure.

**4.** The requirements of water-way.

**5.** Protection against scouring.

**6.** The best method of dissipation energy.

**7.** Forces acting on various parts of the structure, Hydraulic forces (hydrostatic pressure, dynamic forces) & other forces, live loads, dead loads, earth pressure.

## 4) Site Conditions:

In design of any structure, site condition have be taken into accounts:-

1. Soil properties.
2. Ground water.
3. Soil strength parameter.
4. Permissible bearing pressure.
5. Permeability.
6. Mineral content (especially sulphates) to both soil & ground water.

## 5) Structures on Gypsum Soils:

Regardless of the mode occurrence, the effect of saturation of the pore space with relatively fresh water is that gypsum as taken into solution. Permeability is increased with consequent increase in seepage rate, soil strength is reduced, cavities are formed in the soil structure and foundation failure by piping or undermining may occur. Where site investigation shows significant gypsum concentration in the underlying soil strata, every efforts should be made to relocate structures to more favorable locations. Channels U/S & D/S of the structure should be lined and particular attention paid to joints to ensure that water tights is maintained.

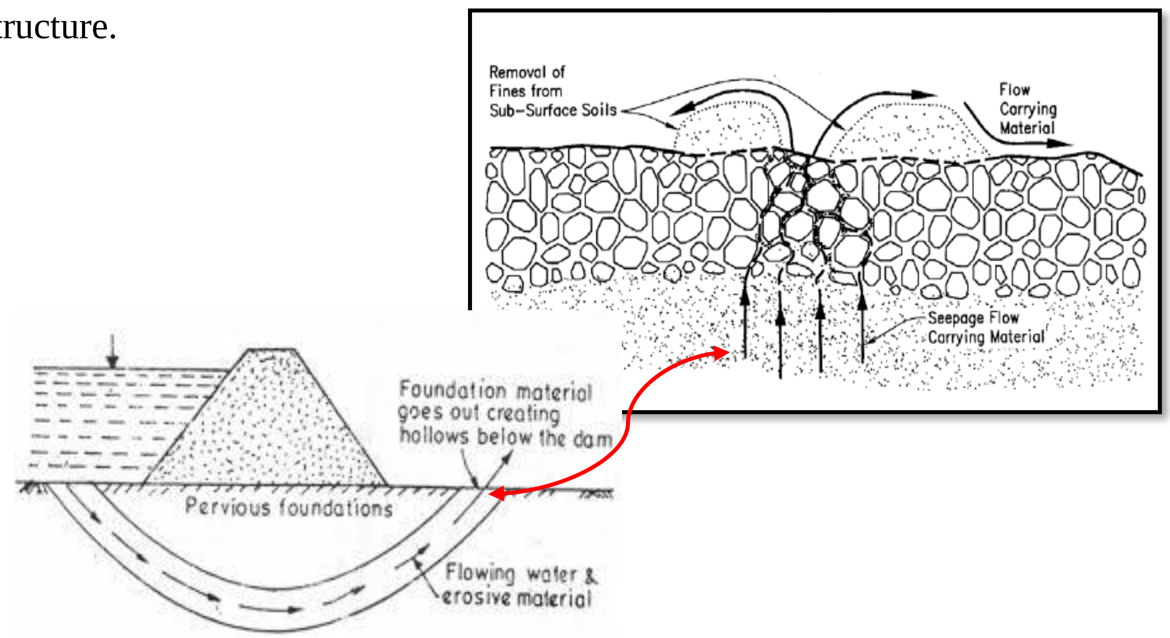
## 6) Percolation beneath Heading up of Hydraulic Structures:

The hydraulic Structures such as barrages, regulators, culverts, etc…, may either founded on an impervious solid rock foundation or a pervious foundation. It is subjected to seepage of water beneath the structure in addition to all other forces to which will be subjected. When founded on un impervious rock foundation, the water seeping below the body of the hydraulic Structure, endangers the stability of the structures may cause its failure.

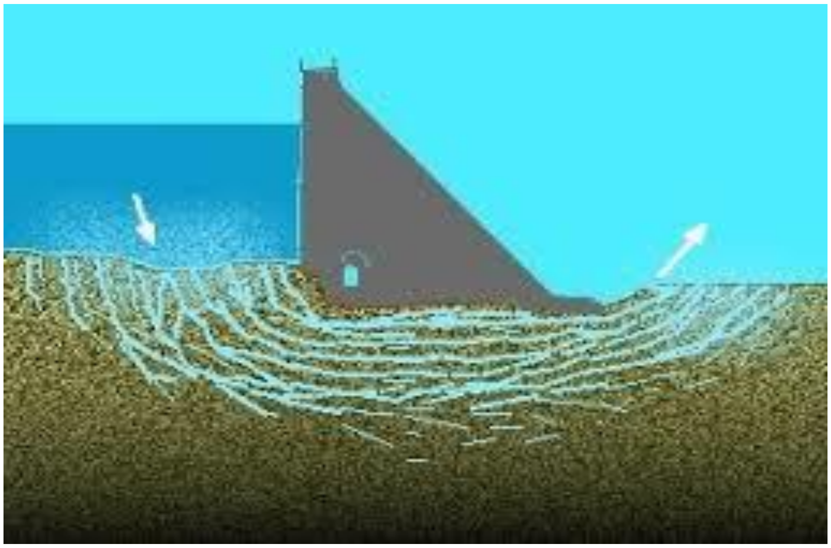
# 7) Causes of Failure of Hydraulic Structures Founded on Pervious Foundations:

## 7-1) Failure by Piping or Undermining

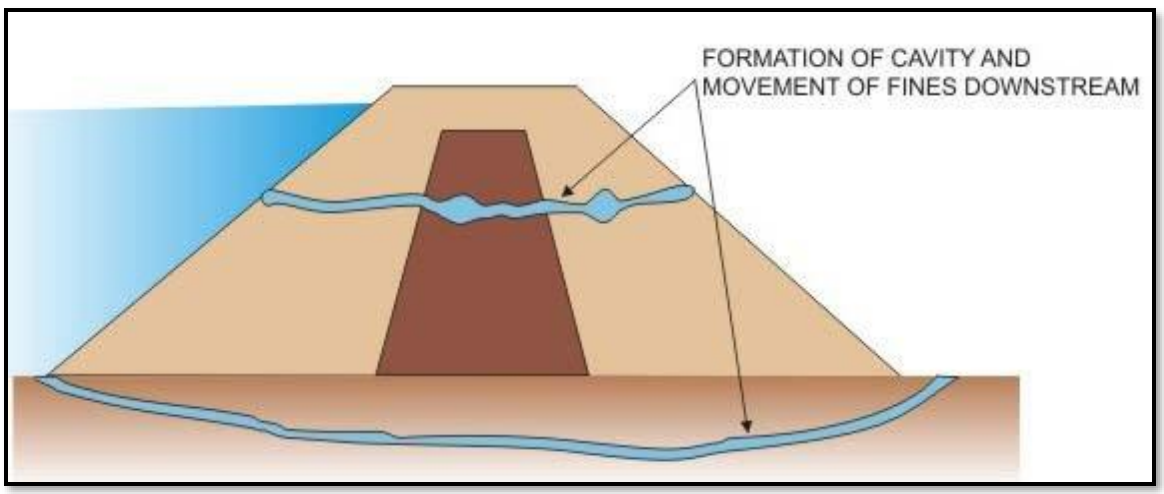
Water starts seeping under the base of hydraulic structure. It starts from U/S side and tries to exit at the D/S end of the impervious floor. At the point of the exit, the exit gradient may become more than the critical gradient, in which cause, the water starts dislodging the soil particles & carrying it away with it causing formulation a hole in the subsoil. So, formed resulting in the failure of the structure.



## 1) Seepage under the base of concert dams



## 2) Seepage through and under the base of earth dam



## Piping can have prevented by the following methods:

**a. By providing sufficiently long impervious floor**

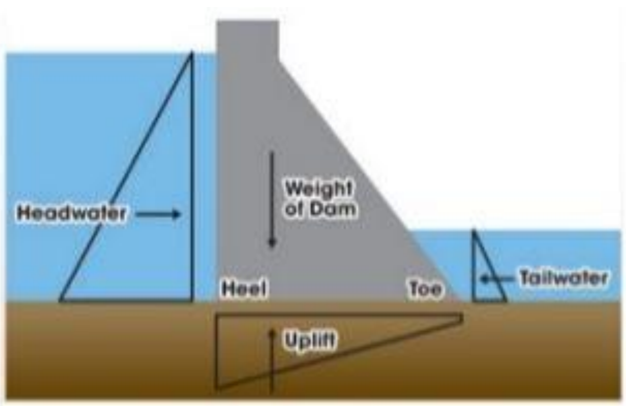
This long length will reduce the exit velocity & exit gradient. As the water has to travel along distance beneath the floor, its head will sufficiently have lost before it exits & its velocity will be such that it has cannot wash away any soil or sand particles.

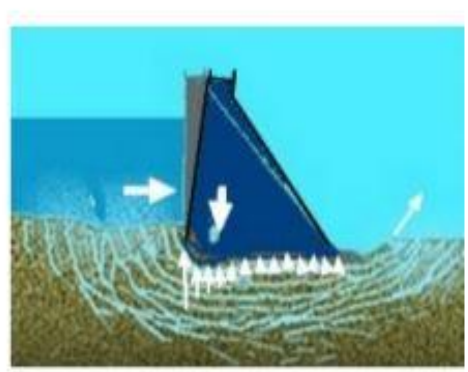
**b. By providing piles at both U/S and D/S ends**

This measure also results is increasing the path of the travel of seepage water & hence it decreases its exit velocity & exit gradient.

## 7-2) Failure by Direct Uplift

The water seeping below the structure exerts on uplift pressure on the floor of the structure if this pressure is not counter balance by the weight of concrete or masonry floor. The structure will fail by a rupture of a part of the floor. The pervious concept of the hydraulic structure due to subsurface flow where introduce by many engineers on the bases of experiments & the research work.



# Khosla's Theory

## 1) Introduction:

In 1926-1927 , some siphons on Upper Chenab canal, designed on Bligh's theory , gave trouble . Actual pressure measurement made with the help of pipes inserted in the floors of two of these siphons didn't show any relationship with the pressure calculated on the basis of Bligh's theory . This led to the following provisional conclusions by Khosla:

1) Outer faces of end sheet piles were much more effective than the inner ones and the horizontal length of the floor.
2) Intermediated piles of smaller length were ineffective except for local redistribution of pressure.
3) Undermining of floor started from tail end.
4) It was absolutely essential to have a reasonably deep vertical cut off at the downstream end to prevent undermining.
5) Khosla and his associates took into account the flow pattern below the impermeable base of hydraulic structure to calculate uplift pressure and exit gradient.
6) Starting with a simple case of horizontal flow with negligibly small thickness. (various cases were analyzed mathematically.)
7) Seeping water below a hydraulic structure does not follow the bottom profile of the impervious floor as stated by Bligh's theory but each particle traces its path along a series of streamlines.

## 2) Specific Cases

To complete his analyzed , Khosla and his associates consider the following specific causes of the general form shown in figures (1) ,(2) and (3) :

a) A straight horizontal floor of negligible thickness with a sheet pile at either ends. Fig.(1) , (2)

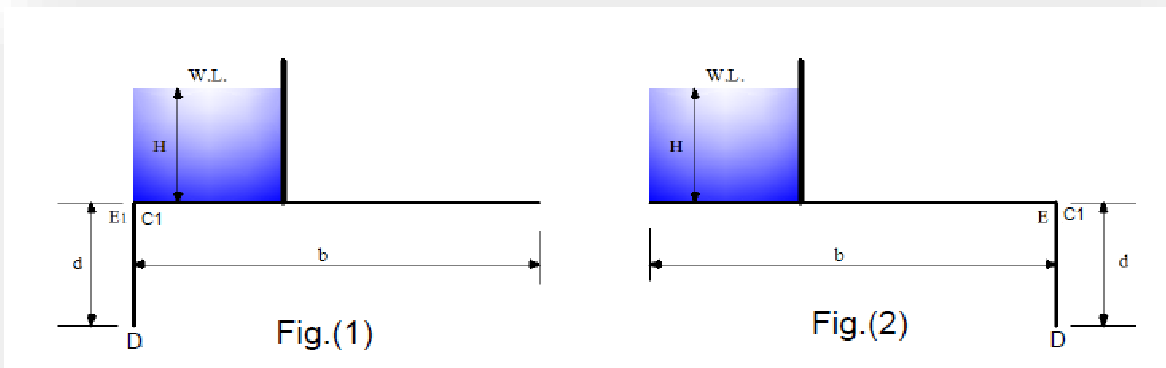



Fig.(1) Fig.(2)

b) A straight horizontal floor thickness with a sheet pile at some intermediate position. . Fig.(3)

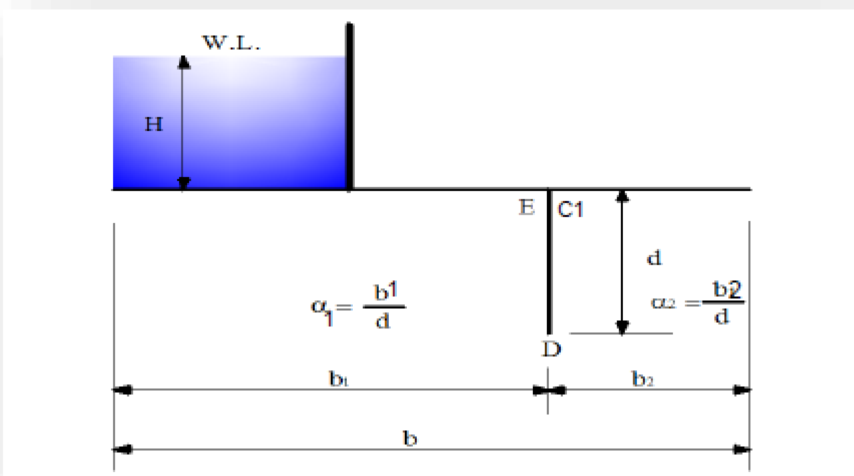



Fig.(3)

**Case ①: Pile at the upstream end ( Fig.1)**

If , however , the pile is provided at the upstream end ( and not at the downstream end ) , the uplift pressure $P_{E1}$ , $P_D$ , and $P_C$ or the percentage uplift pressures at the "key points" in $E_1$ , $D_1$ and $C_1$ are given by the following equation.

$$P_{E1} = H$$

$$P_D = \frac{H}{\pi}\cos^{-1}\left[\frac{1-\lambda}{\lambda}\right] = (H \times \emptyset_D) \rightarrow \emptyset_D = \frac{100}{\pi}\cos^{-1}\left[\frac{1-\lambda}{\lambda}\right]$$

$$P_{C1} = \frac{H}{\pi}\cos^{-1}\left[\frac{2-\lambda}{\lambda}\right] = (H \times \emptyset_{C1}) \rightarrow \emptyset_{C1} = \frac{100}{\pi}\cos^{-1}\left[\frac{2-\lambda}{\lambda}\right]$$

Where:

$\lambda = \dfrac{1+\sqrt{1+\alpha^2}}{2}$ **and** $\alpha = ( b / d )$

b:total length of the impervious floor

d: depth of sheet pile at upstream

## **Case ②: Pile at the downstream end ( Fig.2)**

The uplift pressure or the percentage uplift pressures at the "key points" in $E$ , $D$ and $C$ are given by the following equation.

$$P_E = \frac{H}{\pi}\cos^{-1}\left[\frac{\lambda-2}{\lambda}\right] = (H \times \emptyset_E) \rightarrow \quad \emptyset_E = \frac{100}{\pi}\cos^{-1}\left[\frac{\lambda-2}{\lambda}\right]$$

$$P_D = \frac{H}{\pi}\cos^{-1}\left[\frac{\lambda-1}{\lambda}\right] = (H \times \emptyset_D) \rightarrow \quad \emptyset_D = \frac{100}{\pi}\cos^{-1}\left[\frac{\lambda-1}{\lambda}\right]$$

$$P_{C1} = \frac{H}{\pi}\cos^{-1}\left[\frac{\lambda}{\lambda}\right] = 0 \quad \rightarrow \quad \emptyset_C = \frac{P_{C1}}{H} \times 100 = 0$$

Where:

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} \quad \textbf{and} \quad \alpha = (\,b\,/\,d\,)$$

b:total length of the impervious floor

d: depth of sheet pile at downstream

---

## **Case ③: Pile at some intermediate point ( Fig.3)**

The uplift pressure $P_E$ , $P_D$ , and $P_C$ or the percentage uplift pressures at the "key points" in $E$ , $D$ and $C$ are given by the following equation.

$$P_E = \frac{H}{\pi}\cos^{-1}\left[\frac{\lambda_1 - 1}{\lambda}\right] = (H \times \phi_{E1}) \rightarrow \phi_E = \frac{100}{\pi}\cos^{-1}\left[\frac{\lambda_1 - 1}{\lambda}\right]$$

$$P_D = \frac{H}{\pi}\cos^{-1}\left[\frac{\lambda_1}{\lambda}\right] = (H \times \phi_D) \rightarrow \phi_D = \frac{100}{\pi}\cos^{-1}\left[\frac{\lambda_1}{\lambda}\right]$$

$$P_{C1} = \frac{H}{\pi}\cos^{-1}\left[\frac{\lambda_1 + 1}{\lambda}\right] = (H \times \phi_{C1}) \rightarrow \phi_C = \frac{100}{\pi}\cos^{-1}\left[\frac{\lambda_1 + 1}{\lambda}\right]$$

Where:

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

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

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

$b_1$:horzointail length of the impervious floor at the left pile

$b_2$:horzointail length of the impervious floor at the right pile

d: depth of sheet pile at some intermediate point

**Hint :**

The "key points" are the junctions of the pile is at the bottom points ( E and C ) of the floor

## 3) Exit Gradient

For the case of horizontal impervious floor with cutoff at the downstream end ( Fig.2) the exit gradient ( $G_E$) is given by the following expression :

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

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

Where:

$G_E$: exit gradient

H: maximum static head

d: depth of d/s cutoff .

b: length of floor (horizontal).

# Theories of Creep

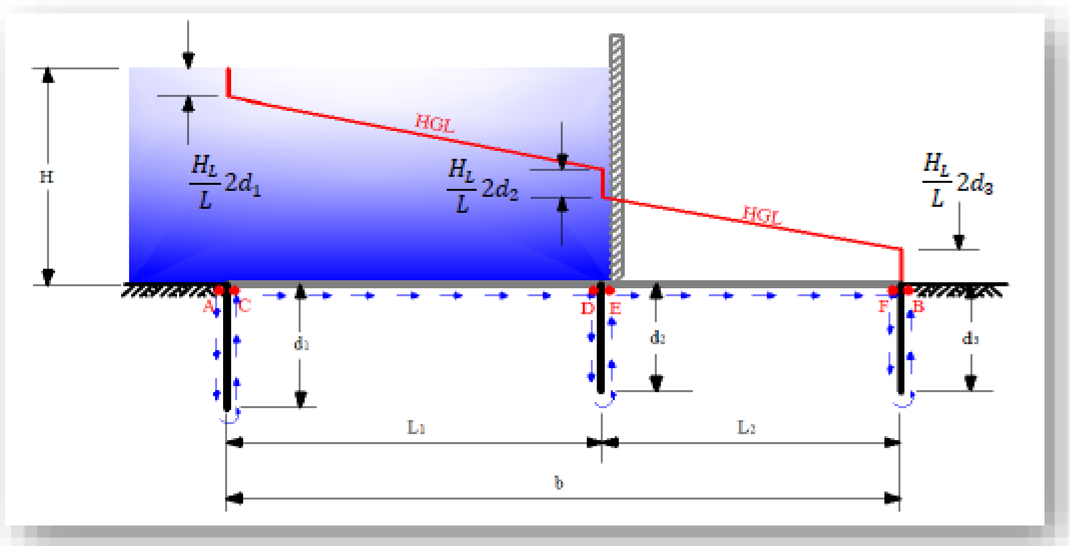
## 1) Bligh's Creep Theory

It is directly depended on the possibilities of percolation in porous soil on which the floor (apron) is built. Water from upstream percolates and creeps (or travel) slowly through weir base and the subsoil below it. The head lost by the creeping water is proportional to the distance it travels (***creep length***) along the base of the weir profile. The creep length must be made as big as possible so as to prevent the piping action. This can be achieved by providing deep vertical cut-offs or sheet piles.

According to Bligh's theory, the total creep length,

$L = b + 2(d_1 + d_2 + d_3)$ ,

where , $b = L_1+L_2$. If H is the total loss of head, then the loss of head per unit length of the creep shall be:



$$C = \frac{H}{L} = \frac{H}{b+2(d1+d2+d3)} \qquad \text{.............................(1)}$$

$$H = W.L(u.s) - W.L(d.s) \qquad \text{...........................(2)}$$

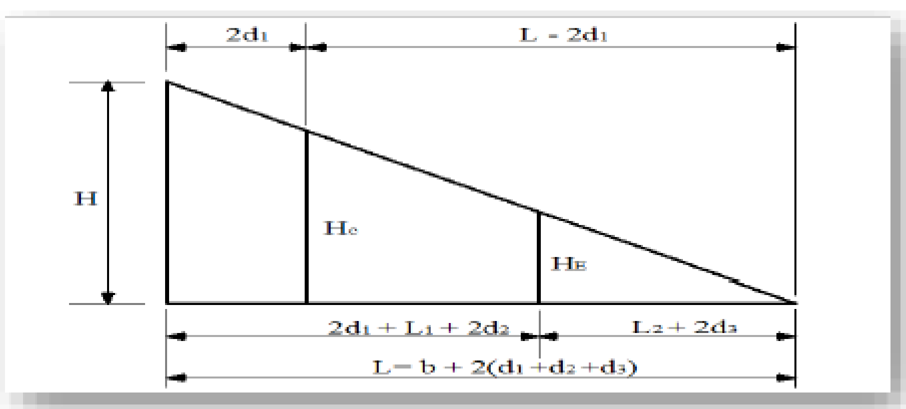
Bligh called the loss of head per unit length of creep as *Percolation coefficient* or *hydraulic gradient (C)*. The reciprocal, (L / H) of the percolation coefficient is known as the *coefficient of creep(C')*.

## 1-1) Assumptions

1- Hydraulic slope or gradient is constant throughout the impervious length of the apron.

2- The percolating water creep along the contact of the base profile of the apron with the sub soil losing head enroute, proportional to length of its travel. The length is called ***creep length***. It is the sum of horizontal and vertical creep.

3- Stoppage of percolation by cut off (pile) possible only if it extends up to impermeable soil strata.



For more explaining:

Total creep length (L) = $2d_1 + L_1 + 2d_2 + L_2 + 2d_3 = (L_1 + L_2) + 2(d_1 + d_2 + d_3)$

Head loss per unit length (hydraulic gradient)

$$C = \frac{H}{L} = \frac{H}{(L_1 + L_2) + 2(d_1 + d_2 + d_3)}$$

- Head loss occurs on upstream cutoff $= \frac{H_L}{L} \times 2d_1$
- Head loss occurs on intermediate cutoff $= \frac{H_L}{L} \times 2d_2$
- Head loss occurs on downstream cutoff $= \frac{H_L}{L} \times 2d_3$
- Head at point C ($H_C$)

$$H_C = H - \frac{H}{L} \times 2d_1$$

Hydraulic gradient drop at upstream cutoff = $H - H_C$

$= H - \left(H - \frac{2d_1}{L} \times H\right) = \frac{H_L}{L} \times 2d_1$

$\therefore$ uplift pressures at any point $H = H - \frac{H}{L}.\bar{L}_{at\ any\ point}$ ......................(3)

## 1-2) Limitations of Bligh's theory

1) This theory made no distinction between horizontal and vertical creep.
2) Did not explain the idea of exit gradient . The safety against undermining cannot simply be obtained by considering a flat average gradient but by keeping this gradient will be low critical.
3) No distinction between outer and inner faces of sheet piles or the intermediate sheet piles, whereas from investigation it is clear, that the outer faces of the end sheet piles are much more effective than inner ones.

4) Losses of head does not take place in the same proportions as the creep length. Also the uplift pressure distribution is not linear but follow a sine curve.
5) In case of two piles the horizontal distance between the pile line is greater than twice depth .

## 1-3) Safety against Piping or Undermining

Safety against piping can be ensured by providing sufficient creep length given by:

$L = C \times H$ ………………………….. (4)

C: Bligh's coefficient for the soil.

$$\frac{H}{L} = \frac{1}{C}$$

**Table (1): Recommended Safe Hydraulic Gradients**

| No. | Type of soil | Value of C | Safe exit gradient less than |
|---|---|---|---|
| 1 | Light sand and mud | 18 | $\frac{1}{18}$ |
| 2 | Fine sand, alluvial soil | 15 | $\frac{1}{15}$ |
| 3 | Coarse grained sand | 12 | $\frac{1}{12}$ |
| 4 | Sand mixed with boulders and gravel | 9-5 | $\frac{1}{9}$ to $\frac{1}{5}$ |

## 1-4) Safety against Uplift Pressure

The ordinate of hydraulic grade line above the bottom of the floor represent the residual uplift water head at each point. Say for example; if at any point the ordinate of H.G.L above the bottom of the floor is ***1m***, then ***1m*** head of water will acts as uplift at this point. If the uplift head at any point is ($H_p$), then water pressure equal to ($H_p$) meters will acts at this point and has to be counter balanced by the weight of the floor of thickness say ($t_p$).

Uplift pressure = $\gamma_w \times Hp$

Where$\gamma_w$ is the unit weight of water $\gamma_w = \rho . g$

Downward pressure $= (\gamma_w . Gs)t_p - \gamma_w . t_p$

Where Gs is the specific gravity of the floor material.

- Gs is approximately taken as 2.24 to 2.4
- For equilibrium condition:

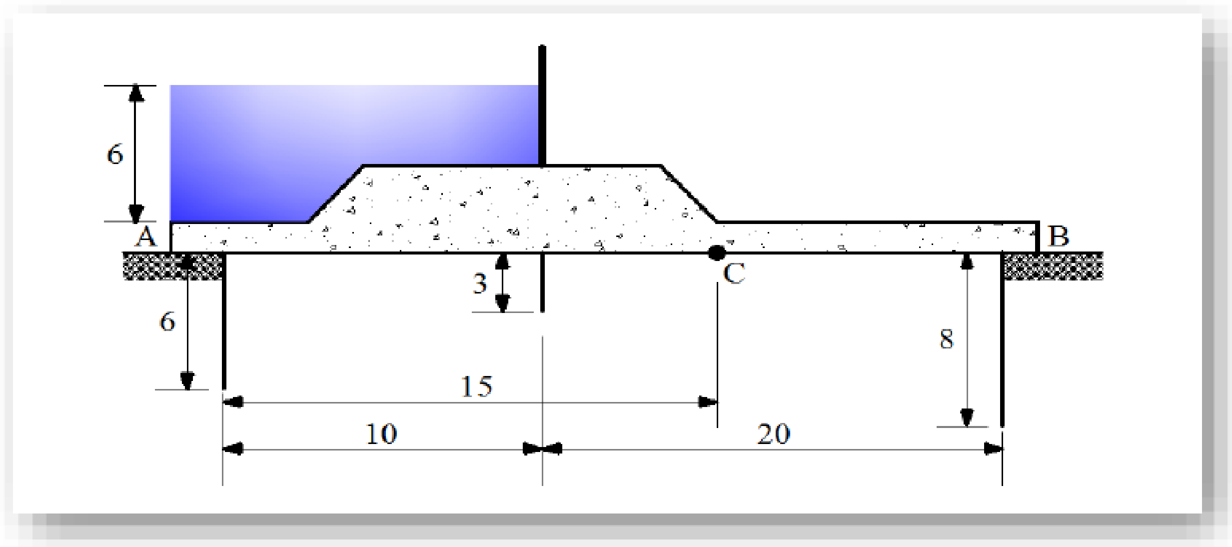
$$\gamma_w . H_p = (\gamma_w . G_S) \times t_p - \gamma_w \times t_p$$

$$H_p = G_S \times t_p - t_p = t_p(G_S - 1)$$

$$t = \frac{H_p}{G_S - 1}$$

---

**Example (1):** Find the hydraulic gradient and uplift pressure and the thickness of floor at a point 15 m from the upstream end of the floor in the figure below.




## Solution:

Water percolates at point *A* and emerges at point *B*

- Total creep length (L) = 2×6+10+ 2×3+ 20+ 2×8 = 64*m*
- Head of water on structure (H) = 6 *m*
- Hydraulic gradient = H/L = 6 /64 = 1/C = 1/10.67

According to Bligh's theory, the structure would be safe on sand mixed with boulders & Gravel

- Creep length up to point *C* = $\bar{L}$ = 2×6+ 2×3+15 = 33*m*

The residual uplift pressure at the point C under consideration

$$H_C = H - \frac{H}{L} \times \bar{L}$$

$$H_C = 6 - \frac{6}{64} \times 33 = 2.91m$$

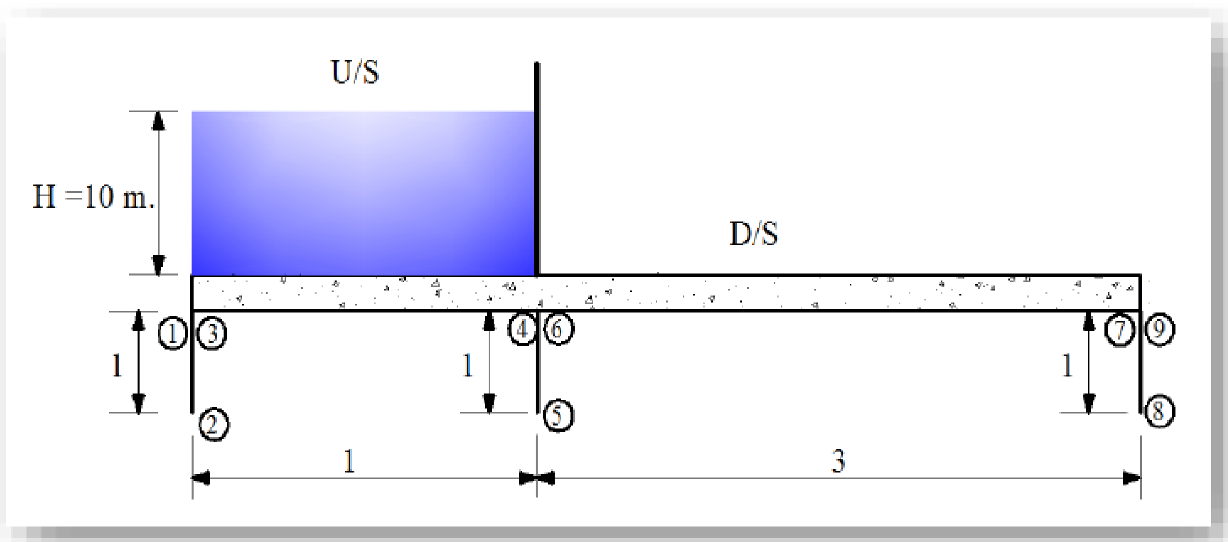
- The thickness of floor at C

$$t_C = \frac{H_C}{G_S - 1} = \frac{2.91}{2.24 - 1} = 2.08m$$

**Homework No. 1:** For the hydraulic structure shown below:

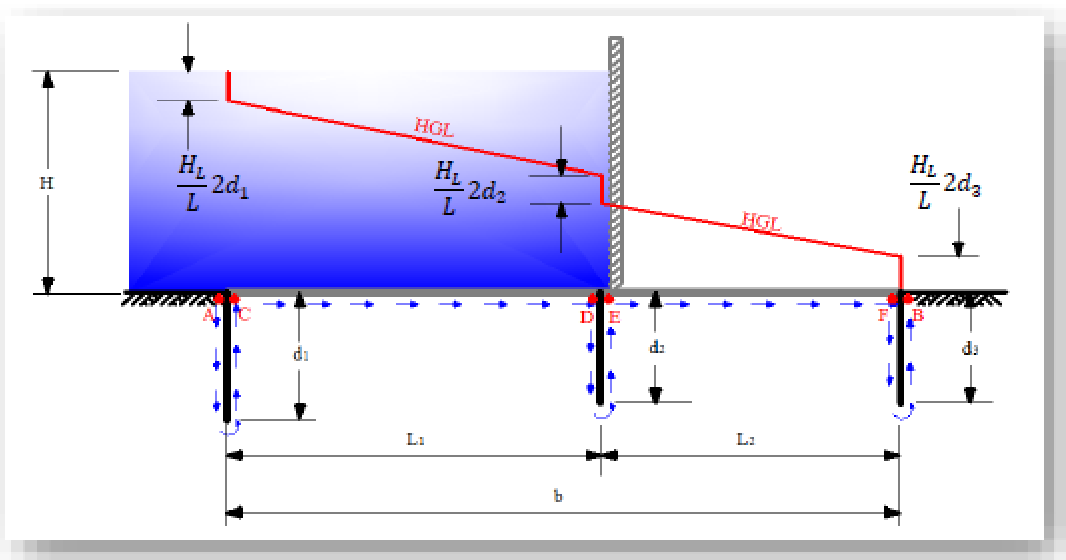
1) Sketch the H.G.L from U/S to D/S 2. Find the uplift pressure at key points ④&⑦.

2) Find the thickness of floor at key point ⑥



## 2) Lane's Weighted Creep Theory

Bligh, in his theory, had calculated the length of the creep, by simply adding the horizontal creep length and the vertical creep length, thereby making no distinction between the two creeps. However, Lane, on the basis of his analysis carried out on about 200 dams all over the world, stipulated that the horizontal creep is less effective in reducing uplift (or in causing loss of head) than the vertical creep. He, therefore, suggested a weight age factor of 1/3 for the horizontal creep, as against 1.0 for the vertical creep.




The total Lane's creep length ($L_l$) is given by:

$L_w = (d_1+d_1) + (1/3)L_1 + (d_2+d_2) + (1/3)L_2 + (d_3+d_3) = (1/3)b + 2(d_1+d_2+d_3)$

To ensure safety against piping, according to this theory, the creep length $L_w$ must not be less than $C_1H_L$.

Where $H_L$ is the head causing flow, and $C_1$ is Lane's creep coefficient given in table 2.

To ensure safety against piping

$L_w > C_1 \times H$

Hydraulic gradient $\frac{H}{L_w}$ should be less than $\frac{1}{C1}$

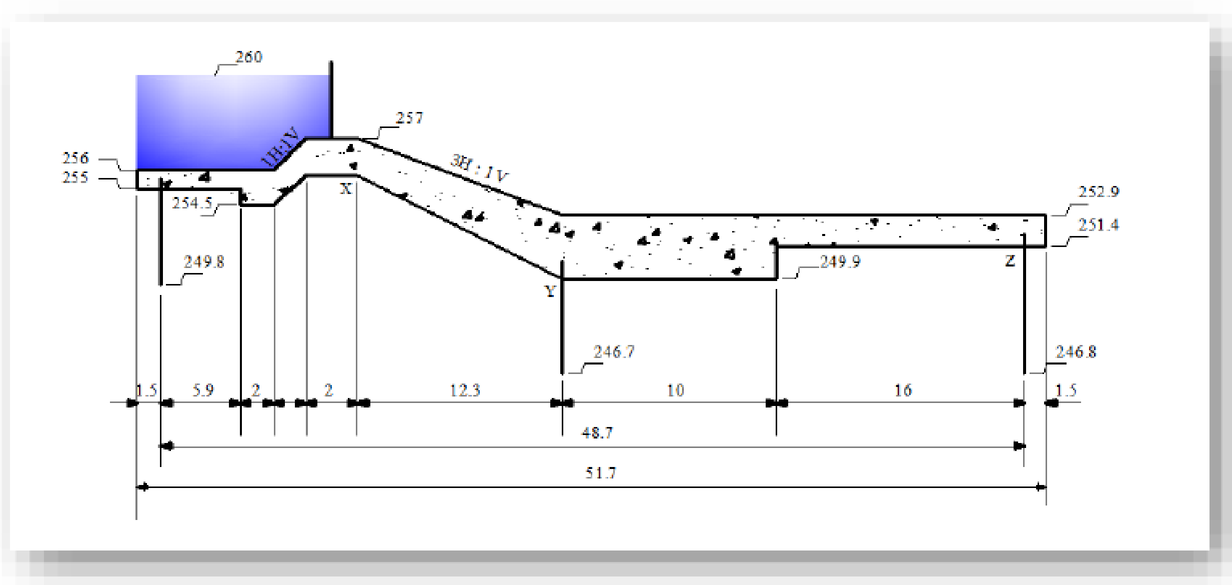
$\frac{H}{L_w} < \frac{1}{C1}$

**Table (2): Values of Lane's Safe Hydraulic Gradient for different types of Soils**

| No. | Type of soil | $C_1$ | Safe exit gradient less than |
|---|---|---|---|
| 1 | Very fine sand or silt | 8.5 | $\frac{1}{8.5}$ |
| 2 | Fine sand | 7.0 | $\frac{1}{7.0}$ |
| 3 | Coarse sand | 5.0 | $\frac{1}{5.0}$ |
| 4 | Gravel & sand | 3.5 to 3.0 | $\frac{1}{3.5}$ to $\frac{1}{3.0}$ |
| 5 | Boulders, gravel and sand | 3.0 to 2.5 | $\frac{1}{3.0}$ to $\frac{1}{2.5}$ |
| 6 | Clayey soil | 3.0 to 1.6 | $\frac{1}{3.0}$ to $\frac{1}{1.6}$ |

---

**Example (2):** You are working as a consultant for an engineering company, and you have received a design of a barrage structure on a river shown in figure below. It is required to check if the thickness at points X, Y and Z is sufficient to counteract the uplift pressure (G = 2.4), and check safety against piping if the soil type is coarse sand ($C_1$ =5).




## Solution:

ملاحظة :

أذا كانت الزاوية للضلع المائل في الأرضية اقل من( $\theta < 45$ ) نعتبر الضلع أفقي ويقسم على 3

$$L_W = \frac{1}{3}b + d$$

Where :

b = horizontal distance

d = vertical distance

$$\mathbf{L_W = 1 + \frac{1.5}{3} + (2 * 5.2) + \frac{5.9}{3} + 0.5 + \frac{2}{3} + \sqrt{0.5^2 + 0.5^2} + \frac{2}{3} + \left(\frac{\sqrt{5.1^2 + 12.3^2}}{3}\right) + (2 * 3.2) + \frac{10}{3} + 1.5 + \frac{16}{3}(2 * 4.6) + \frac{1.5}{3} + 1.5 = 48.6\mathit{m}}$$

H = 260- 252.9 =7.1 m

$$* \ \frac{\mathrm{H}}{\mathrm{L_w}} = \frac{7.1}{48.6} = \frac{1}{6.84} < \frac{1}{5} \quad \mathrm{O.k.} \ \boldsymbol{The\ structure\ is\ safe\ against\ piping}$$

$$* \ t = \frac{H_p}{G_S - 1}$$

$$* \ \text{uplift pressures at any point } \mathrm{H_P} = \mathrm{H} - \frac{H}{L} . \bar{L}_{at\ any\ point}$$

$L_x = 16.4$ m, $L_Y = 20.83$ m, $L_Z = 37.4$ m

$$\mathrm{H_X} = 7.1 - \frac{7.1}{48.6} . 16.4 = 4.7\mathrm{m\ of\ water}$$

$$\mathrm{H_Y} = 7.1 - \frac{7.1}{48.6} . 20.83 = 4.05\mathrm{m\ of\ water}$$

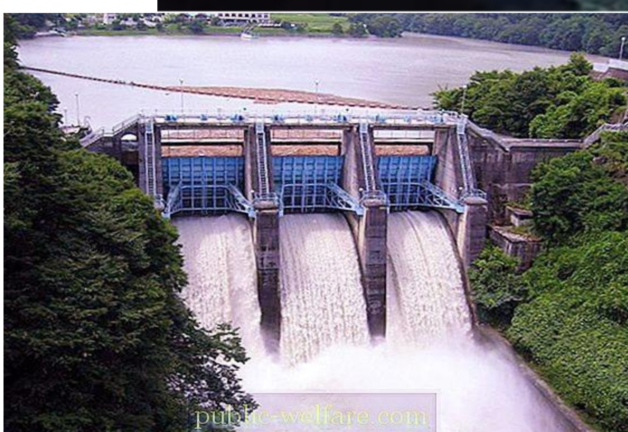
$$\mathrm{H_z} = 7.1 - \frac{7.1}{48.6} . 37.4 = 1.63\mathrm{m\ of\ water}$$

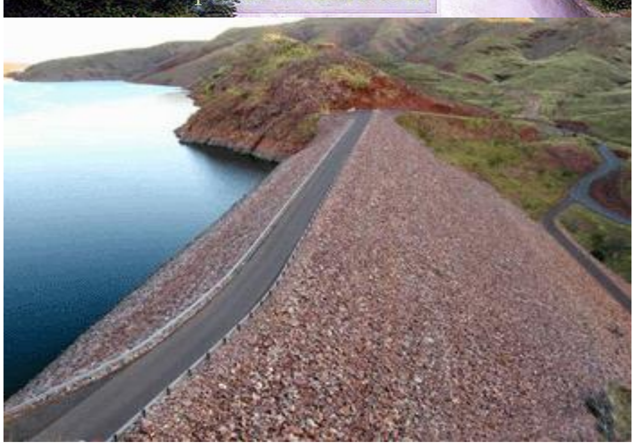
$$- \ t_X = \frac{4.7}{2.4 - 1} = 3.36 > 2m\ not\ ok.$$

$$- \ t_Y = \frac{4.05}{2.4 - 1} = 2.89\mathrm{m} < 3\mathrm{m\ Ok.}$$

$$- \ t_Z = \frac{4.7}{2.4 - 1} = 1.63\mathrm{m} < 1.5\mathrm{m\ \ ok.}$$

# Dams



# Dams

## 1. Introduction

The dam is a barrier constructed across the river to store water on its upstream side due to construction of a dam the water level of the river on its upstream is very much raised. Due to rise in water level large areas lying upstream of the dam get submerged.

Dams are constructed to store the river water in form of on artificial lake or reservoir. The stored water can be utilized for generation of hydro-electric power, water supply, and irrigation or for any other purpose.

## 2. Classification of Dams :

Dams may be classified in several way as follows:

### 1. Classification Based on Materials of Construction

a) Earth fill dams
b) Rock fill dams
c) Concrete dams
d) Masonry dams
e) Steel dams
f) Timber Dams

### 2. Classification Based on Flow over its Top

a) Over flow dams
b) Non over flow dams

### 3. Classification Based on the Use of the Dam

a) Storage dams
b) Diversion dams

c) Detention dams

d) Multi-purposes dams

### 4. Classification Based on the Mode or Resistance Offered by the Dams against External Forces

a) Gravity dams

b) Buttress dams

c) Arch dams

### 5. Classification Based on Rigidity of the Dams

a) Rigid dams

b) Non- rigid dam

## 3. Advantages and Disadvantages of Gravity Dams

- **Advantages**

1) Maintenance cost is negligible.
2) They are especially suitable for deep steep valley conditions where no other dam is possible.
3) If suitable foundation is available, such dams can be constructed for very large heights.
4) Because they can be constructed in very large heights, they can store more amount of water.
5) If suitable separate place is not available for installation of spillways, they can be installed in the dam section itself.
6) This dam gives prior indication of instability. If remedial measures are taken in time unsafe dams may even be rendered safe. Even if they cannot be made safe they give sufficient time for the people to move out the area likely to be submerged due to failure of the dam.

7) Silting rate of the reservoir can be reduced considerably by installing under sluices in the dam near the bed of the reservoir. Sluices can be operated from time to time and silt may be scoured out of the reservoir.
8) They are not affected by very heavy rainfall. Earth dams cannot sustain very heavy rainfall because of heavy erosion.

- **Disadvantages**

1) They are very costly in initial construction.
2) They take lot of time to construct.
3) They require skilled laborer for construction.
4) Such dams can be constructed only on good foundation.
5) If height of the dam is to be raised, it cannot be done unless provision for it had been made in the construction of the lower part of the dam.

## 4. Advantages and Disadvantages of Earth Fill and Rock Fill Dams

- **Advantage**

1) They can be constructed on any type of foundation.
2) They can be constructed in comparatively less time.
3) They do not require skilled labor.
4) Initial cost of construction is low as locally available soils, and rock boulders are normally used.
5) Their height can be increased without any difficulty.
6) They are especially suitable for condition where slopes of river banks are very flat. Gravity dams under such conditions are not found suitable.

- **Disadvantage**

1) The fail all of a sudden without giving any per-warning
2) Flood water affect the dam safety

3) Spillways have to be located independent of the dam
4) They cannot be constructed as over flow dams
5) They require continuous maintenance
6) They cannot be constructed in narrow steep valleys
7) They cannot with stand heavy rains unless properly protected
8) They cannot be constructed in large height. The usual height is 30 m for which most of the earthen dams

## 5. Factors Governing Selection Types of Dams

### 1) Topography

a) V-shape nor row valley select arch dam
b) Narrow U-shape valley indicates choice of over flow concrete dam
c) A low, rolling plan suggest earth dam

### 2) Geology & Foundation

a) Solid rock-foundation: select any type
b) Gravel &coarse sand foundation: select earth dam or Rock fill dam
c) Silt & fine sand foundation: select earth dam or low concrete dam up to 8m
d) Clay foundation: select earth dam with special treatment

### 3) Availability of Materials of Construction

a) If sand, gravel and stone is available , concrete gravity dam may be suitable
b) If coarse and fine grained soils are available, an earth dam may be suitable

### 4) Length and Height of Dam

If the length of the dam is very long and its height is low, an earth dam would be abetter choice. If the length is small and height is more, gravity dam is preferred.

### 5) Spillways

a) Separate spillway lead to constructing earth dam

b) Large spillway with dam lead to concrete gravity dam and no separate

### 6) Road Way over the Dam

We can construct earth or gravity dam

### 7) Generation of Hydro-Electric Power

Concrete or masonry gravity dams because it can be constructed at height level and develop sufficient head for running the turbines.

## 6. Concrete Dams

Is a structure which is designed in such a way that its weight resist the force exerted up on it. It may constructed of concrete or masonry.

# Forces Acting on Gravity Dams

## 1) Forces Acting on Gravity Dams

Following are the forces acting on a gravity dam.

1) Water pressure
2) Uplift pressure
3) Silt pressure
4) Wave pressure
5) Pressure due to earth quake force
6) Ice pressure
7) Weight of the dam

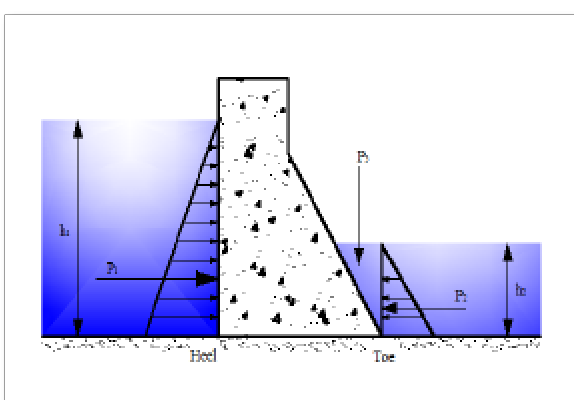
### 1) Water pressure

This is 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 zero intensity at the water surface , to a value $(\gamma \times h)$ at any depth $h$ below water surface , as shown in Figure below

Thus , horizontal force ,

$\rightarrow P_h = \frac{1}{2} \times \gamma \times h^2$ , this acts at a height $(h/3)$ from the base of the dam.



- If the upstream face is partly vertical and partly inclined , as shown in Figure , the resultant water pressure can be resolved in two components .

**i)** Horizontal component $P_h$

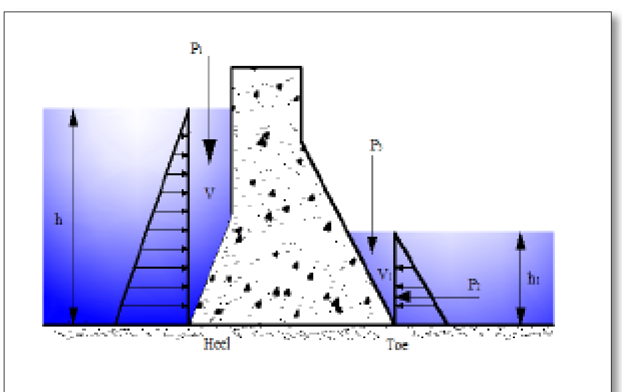
Thus , horizontal force ,

$\rightarrow P_h = \frac{1}{2} \times \gamma \times h^2$ , this acts at a height ( $h / 3$ ) from the base of the dam.

**ii)** Vertical component $P_1$ due to weight of water supported by the inclined face .

vertical force $P_1$ =( $\gamma \times h_V$ ) ↓ weight of water contained by shaded column and acting at the C.G . of the area .

- Similarly , if there is tail water of height $h_1$ on the downstream side , it exerts both horizontal pressure $\leftarrow P_h = \frac{1}{2} \times \gamma \times h_1^2$ as well as vertical pressure (↓$P_1$ =( $\gamma \times h_{V1}$ )



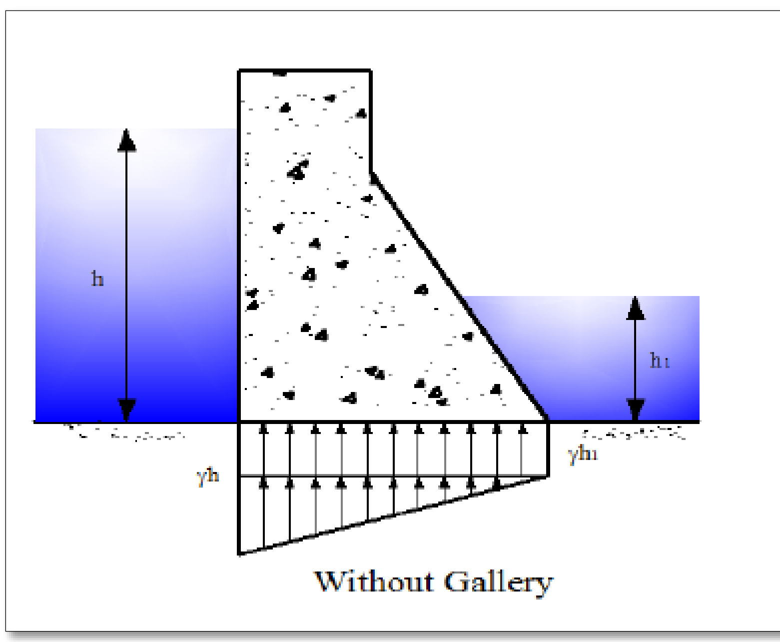
## 2) Uplift pressure

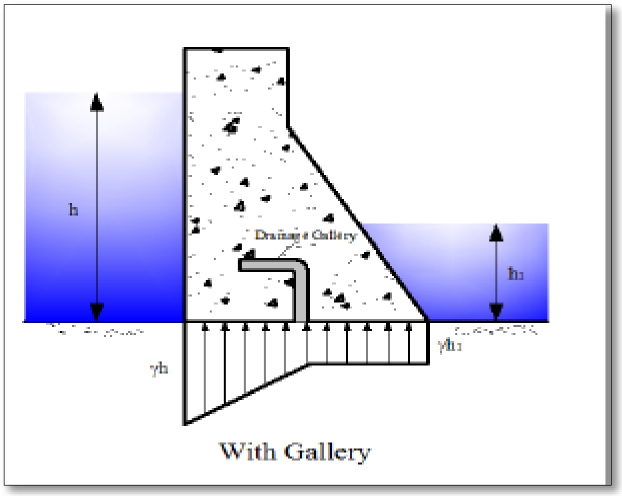
Water has a tendency to seep through the pores and fissures of the foundation material . It also seeps through the joints between the body of the dam and its foundation at the base , and through the pores of the material in the body of the dam . The seeping water exerts pressure and must be accounted for in the stability calculations.

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 . A portion of the weight of the dam will be supported on the upward pressure of water ; hence net foundation reaction due to vertical force will reduce .

The uplift pressure intensities equal to the hydrostatic pressure of water at the toe and heel joined by a straight line in between .

$$\gamma \times h_1 + \frac{1}{3}(\gamma \times h - \gamma \times h_1) = \gamma(\frac{(2h_1 + h)}{3})$$

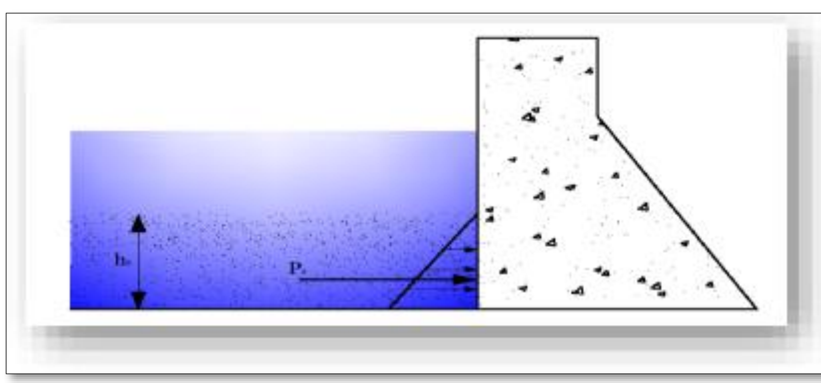





## 3) Pressure due to silt deposited on U/S face

The river brings silt and debris along with it . The silt load gets deposited to an appreciable extent when dam is constructed .The dam is , therefore , subjected to silt pressure in addition to the water pressure . If ( $\gamma_s$ ) is the submerged unit weight of silt and ( $\Phi$ ) is the angle of internal friction and ( $\boldsymbol{h_s}$ ) is the height to which the silt is deposited . the silt pressure is given by

$$P_S = \frac{1}{2}\gamma_S \times h_S^2 \times k_a \qquad \text{Where,} \quad K_a = \frac{1-\sin\phi}{1+\sin\phi}$$

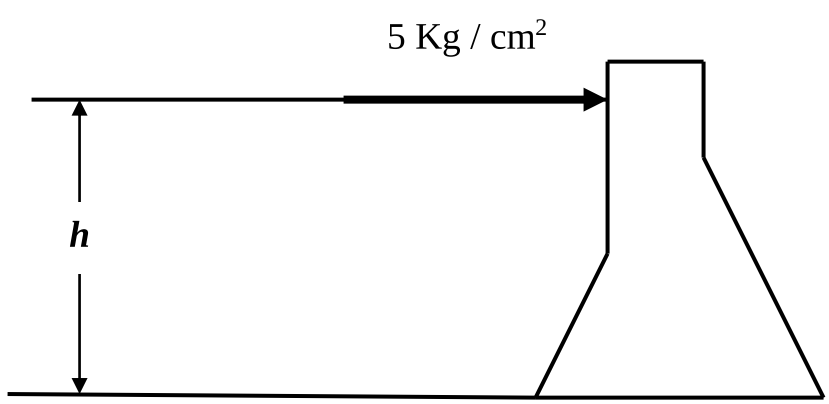


- If the upstream face is inclined , the vertical weight of silt supported on the slope also acts as vertical force .

## 4) Ice pressure

The ice pressure is more important for dams constructed in to 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. The coefficient of thermal expansion of ice being five times more than that of concrete , the dam face has to resist the force due to expansion of ice . This force acts linearly along the length of the dam , at the reservoir level .

The average value of ( 5 Kg / cm$^2$) or ( 50 Ton / m$^2$ ) may be taken as an ice force .

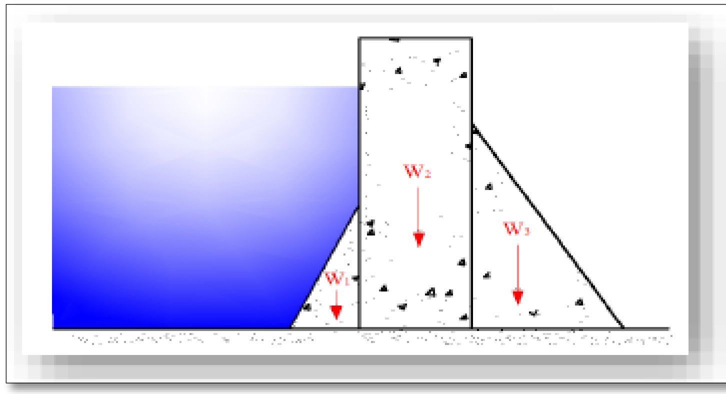



## 5) Weight of the dam

The weight of the dam is major resisting force . For analysis purposes , generally , unit length of the dam is considered .The cross – section of the dam may divided in to several triangles and rectangles , and weights $W_1$, $W_2$, $W_3$, ect. of each of these may be computed conveniently , along with determination of their lines of action. The total weight W of the dam acts at the C.G. of its section .

$\downarrow \boldsymbol{W_1} = \gamma_{con} \times V_1$ , $\downarrow \boldsymbol{W_2} = \gamma_{con} \times V_2$

$\therefore W_{dam} = \sum W = W_1 + W_2 + W_3 + \ldots\ldots$




## 6) Earth quake force

### a) Effect of vertical acceleration

- When the acceleration is vertically upward the inertia force ; ***F= w.k*** ; ( where ***w*** = weight of the dam and ***k*** = coff. of earth quake of vertical direction ) acts vertically downwards , these increasing the downwards weight.

- When the acceleration is vertically downward the inertia force acts upward and decrease the downwards weight.

Net weight = $W(1 \pm K_V)$

+ for acceleration is vertically upward

_ for acceleration is vertically downward

### b) Effect of Horizontal acceleration

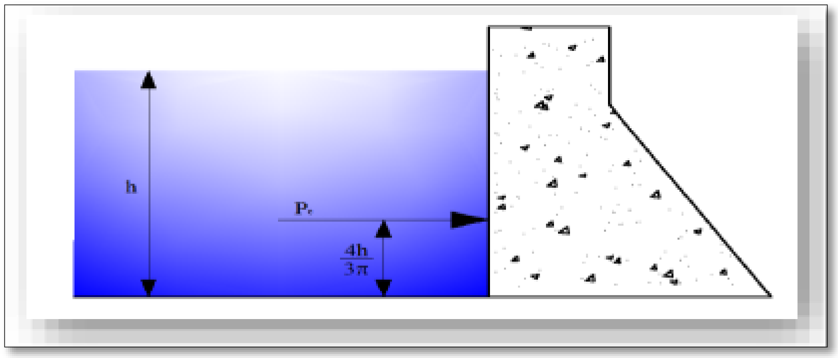
#### i) Hydrodynamic pressure

The horizontal acceleration of the dam and foundation towards the reservoir causes a momentary increase in water pressure .

The increase in water pressure ( Pe ) is given by :

$P_e = 0.55 K_h \times \gamma \times h^2$ acts at a height $\frac{4h}{3\Pi}$ from the base of the dam

Where $K_h$ = coff. of earth quake of horizontal direction




### ii) Horizontal Inertia force

The inertia force acts in direction opposite to the acceleration imparted by the earth quake force $F_h = W. K_h$

This force can be considered at the centre of gravity of the mass

Where :

$K_h$ = coff. of earth quake of horizontal direction

W = weight of the dam (This force can be considered at the center of gravity of the mass)

## 7) Wave pressure

Wave pressure depends on the height of the wave ( $h_w$ ) developed .

$$h_W = 0.032\sqrt{V \times F} + 0.763 - 0.27 \times \sqrt[4]{F} \qquad \text{......... for } F \leq 32 \text{ Km}$$

$$h_W = 0.032\sqrt{V \times F} \qquad \text{......... for } F \geq 32 \text{ Km}$$

Where : $h_w$ = height of the wave in ( meter )

$V$ = wind velocity in ( Km / hr )

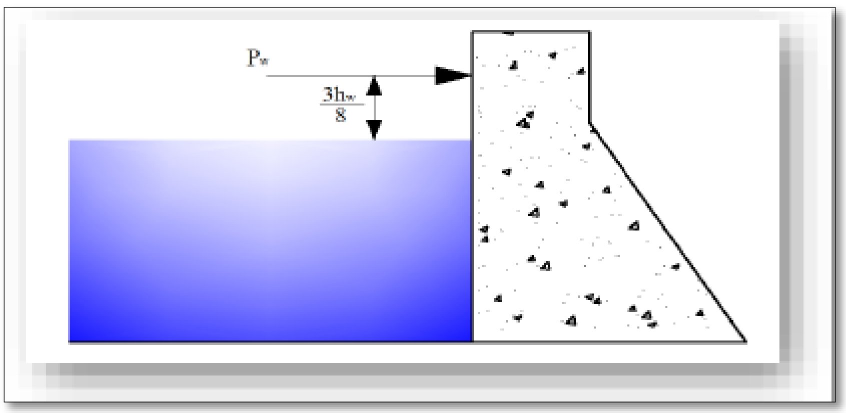
$F$ = straight length of water expanse in ( Km )

$\therefore$ wave pressure is

$P_w = 2000\ \gamma\ h^2_w$ Kg / m

$= 2\ \gamma\ h^2_w$ Ton / m

This force acts at distance ( $3\ h_w$ / 8 ) above the reservoir surface .



# pressure correction for floor with complex profile

## Method of independent variables

Fig.(1) , (2) and (3) shows some specific cases only . In actual practice ,we may have a number of piles , ( i.e. at u/s , d/s and intermediate points) , and the floor also has some thickness . Khosla solved the "actual problem " by an empirical method known as the method of independent variables .This consists of breaking up a complex profile in to a number of simple profile ( such as shown in figures (1) ,(2) and (3) ) , each of which is independently amenable to mathematical treatment , and then applying the correction s due to the mutual interference of pile and due to the thickness and slope of floor.

Thus , to analyze the flow for the floor , the complex profile can be broken up into the following simple profile , and pressures at " key points" can be obtained :

i) Straight floor of negligible thickness with pile at u/s end .

ii) Straight floor of negligible thickness with pile at intermediate point .

iii) Straight floor of negligible thickness with pile at d/s end .

The percentage uplift pressures obtained at the "key points" ( the key points are the junctions of the pile is at the bottom points ( E and C ) of the floor) by considering the simple profile are then corrected for the following :

1) Correction for the thickness of floor.
2) Correction for the mutual interference of the piles.
3) Correction for the slope of the floor.

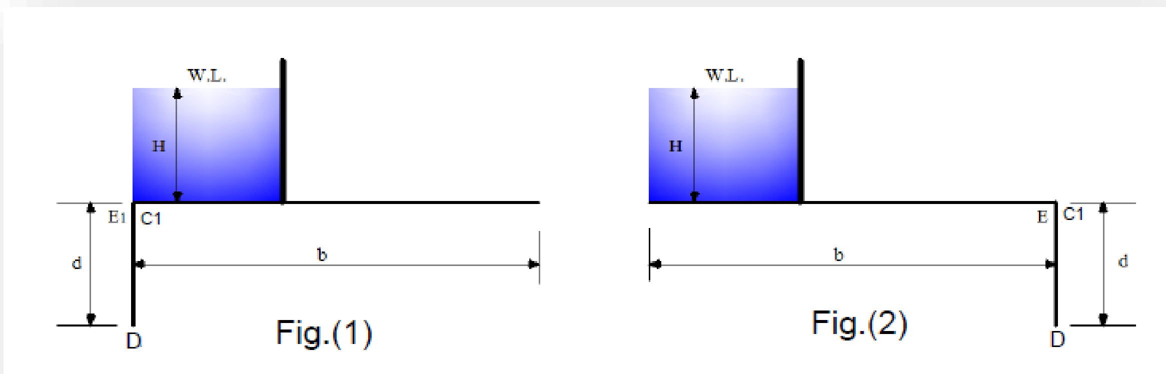



Fig.(1) Fig.(2)

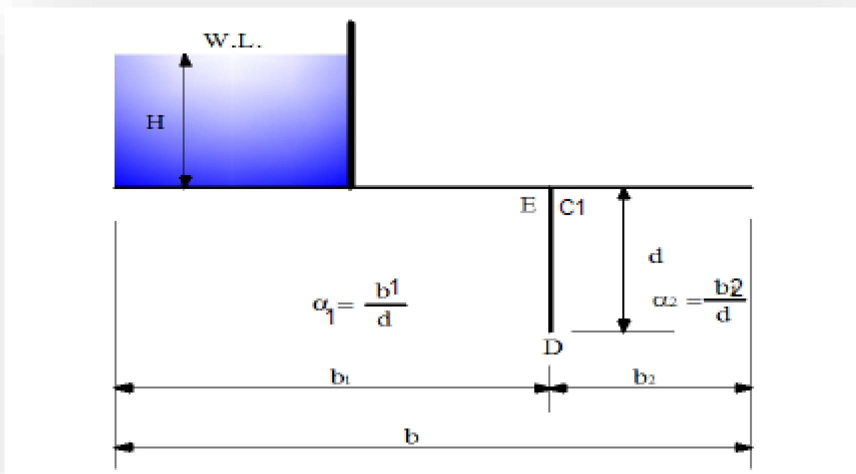
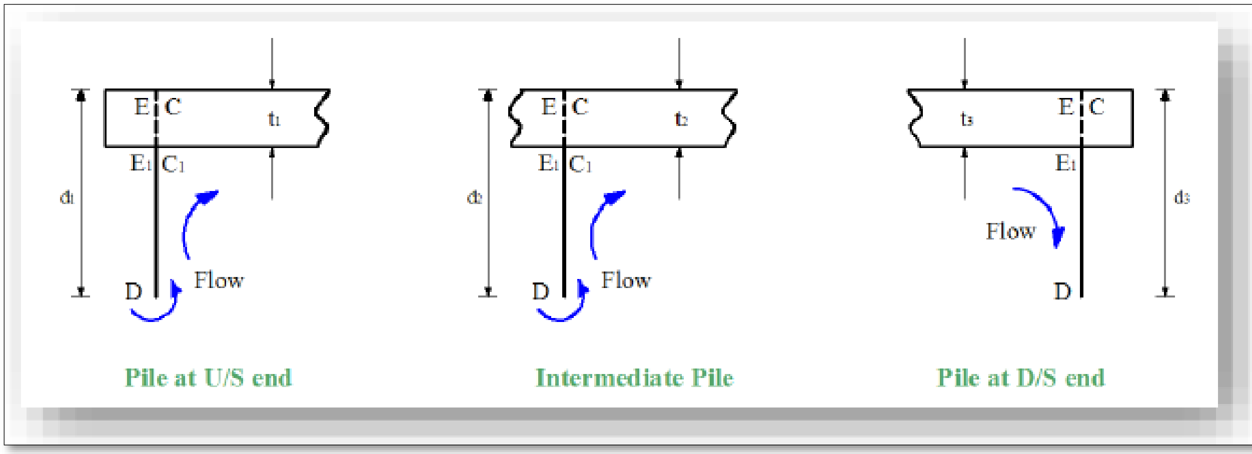



Fig.(3)

## 1) Correction for the Thickness of Floor

For the figures shown below, give pressure at key points assuming thickness of the floor to be negligibly small .Thus , the pressure at the key points E and C pertain to the level at the top of the floor , while actually the junction of the pile is at the bottom ( points $E_1$ and $C_1$) of the floor . The pressure at actual points $E_1$ and $C_1$ are computed by considering liner variation of pressure between point D and the hypothetical points E and C




Thus, for different locations of piles, the corrections to be applied are as follows:

a) **Correction for u/s pile**

$$\text{Correction for } C_1 = \frac{\emptyset_D - \emptyset_C}{d_1} \times t_1 \qquad \text{(additive)}$$

$$\therefore \ \text{pressure at point } C_1 = \emptyset_{C1} = \emptyset_C + \frac{\emptyset_D - \emptyset_C}{d_1} \times t_1$$

Where:

$t_1$= floor thickness at upstream pile ;

d1 = depth of u/s pile

## b) Correction for intermediate pile

Correction for $E_1 = \frac{\emptyset_E - \emptyset_D}{d} \times t$ (subractive)

$\therefore$ pressure at $E_1 = \emptyset_{E1} = \emptyset_E - \frac{\emptyset_E - \emptyset_D}{d} \times t$

**********

Correction for $C_1 = \frac{\emptyset_D - \emptyset_C}{d_1} \times t_1$ (additive)

$\therefore$ pressure at $C_1 = \emptyset_{C1} = \emptyset_C + \frac{\emptyset_D - \emptyset_C}{d_1} \times t_1$

$t_2 =$ floor thickness at intermediate pile ;

$d_2 =$ depth of intermediate pile

## c) Correction for d/s pile

Correction for

$E_1 = \frac{\emptyset_E - \emptyset_D}{d_2} \times t_2$ (subractive)

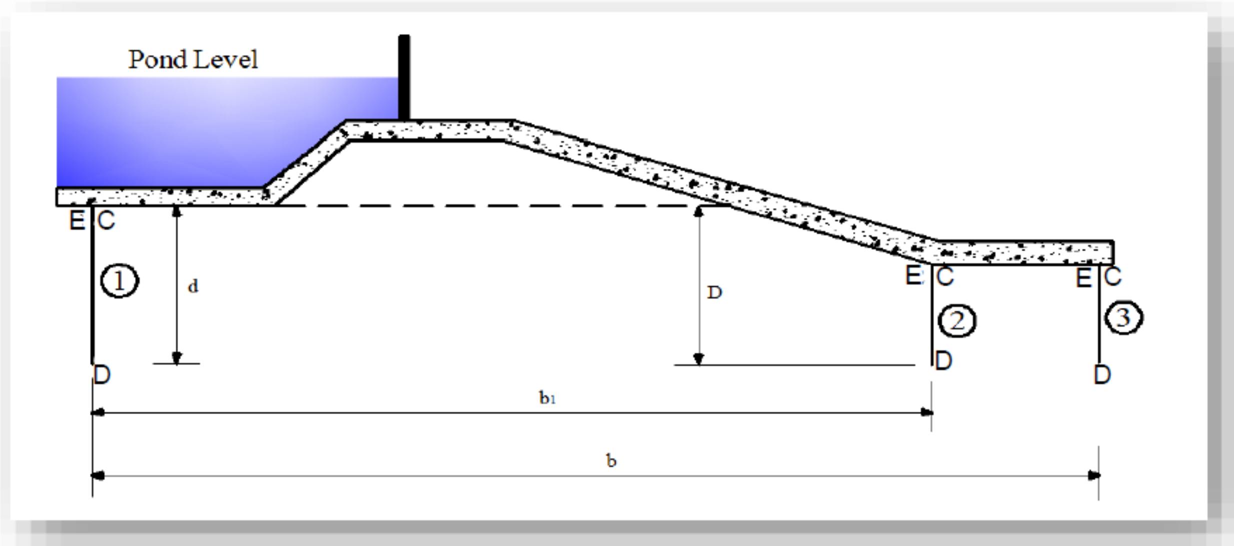
$\therefore$ pressure at E1 $= \emptyset_{E1} = \emptyset_E - \frac{\emptyset_E - \emptyset_D}{d_2} \times t_2$

$t_3 =$ floor thickness at downstream pile ;

$d_3 =$ depth of d/s pile

## 2) The Correction for the Mutual Interference of Piles

For the figure shown below.




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

<u>Where:</u>

C : percentage correction to be applied to the pressure head .

b : total floor length of the impervious floor .

b' : distance between two piles.

d : length of the pile on which the effect of another piles of depth D is required to be determined

D : depth of pile whose effect is required to be determined on the neighboring pile of depth (d)

- Both d and D are measured below the level at which interference is required to be determined .

## Notes:

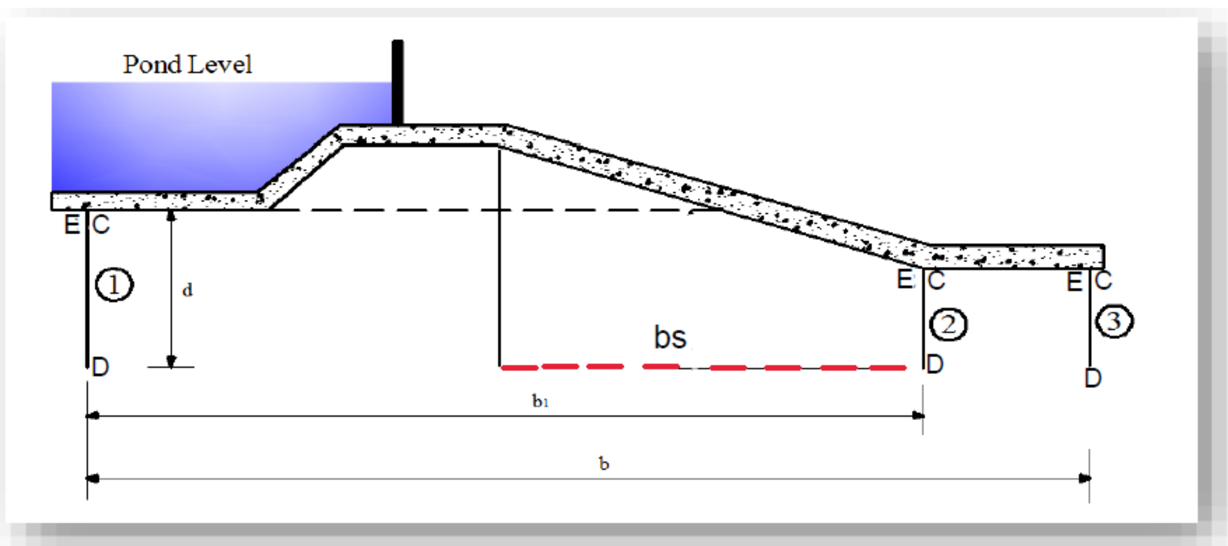
1) The correction is positive (+ve) for points in the rear or back water and negative (- ve) for points forward in the direction of flow. i.e.

- **Effective of D/S pile on U/S pile (+ve).**
- **Effective of U/S pile on D/S pile (–ve).**

2) This equation does not apply to the effect of an outer pile on the intermediate pile if the latter (intermediate pile ) is equal to or smaller than the outer pile and is at a distance less than twice the length of the outer line.

## 3) Correction for the Slope of the Floor

The percentage pressure under a floor sloping down or sloping up the direction of flow are respectively greater or less than those under horizontal floor for the same base ratio . Thus , for sloping floor a suitable percentage correction is to be applied.



$$C_S = \frac{b_s}{\bar{b}} \ . \mathrm{C}$$

$C_s$: slope correction.

C: coefficient due to slope from table (1)

$b_s$: horizontal length of slope.

$\bar{b}$: distance between two piles which the sloping floor is located.

**Notes:**

1) The correction is ***plus*** for the down slopes and ***minus*** for the up slopes following the direction of flow.
2) The slope correction is applicable to the key point of pile line which is fixed at the ***beginning*** or ***the end*** of the slope.

**Table (1): Khosla's Theory Slope Corrections**

| Slope (V:H) | 1:1 | 1:2 | 1:3 | 1:4 | 1:5 | 1:6 | 1:7 | 1:8 |
|---|---|---|---|---|---|---|---|---|
| % Correction | 11.2 | 6.5 | 4.5 | 3.3 | 2.8 | 2.5 | 2.3 | 2.0 |

# Hydraulics and Theories of Weirs

## 1. Rectangular Sharp – Crested Weirs

To find the discharge over rectangular weir, consider an elementary horizontal strip of water thickness dh and length ( $l$ )at a depth ( h ) from the water surface.

Consider a rectangular notch shown in Fig. 9.2.

Let $l$ = Length of the notch

$H$ = Head of water over the crest of the notch.

Consider an elemental horizontal strip of water of length $l$ and thickness $dh$, at a depth $h$ below the free surface of water.

Theoretical velocity of water flowing through the elemental strip $= \sqrt{2gh}$

$\therefore$ Theoretical discharge through the elemental strip $= l\,dh\sqrt{2gh}$

$$\therefore \text{Total theoretical discharge} = Q = l\sqrt{2g}\int_0^H h^{1/2}\,dh = \frac{2}{3} l\sqrt{2g}H^{3/2}$$

$$\text{Actual discharge, } = q = \frac{2}{3} C_d l\sqrt{2g}H^{3/2}$$

Where $= C_d$ = Coefficient of discharge.

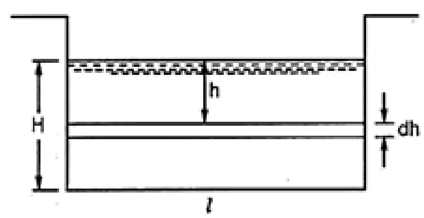



**Fig. 9.2.**

## Francis Formula—End Contractions:

We know for a rectangular notch of length ( $l$ ) the discharge over the notch is given by,

$$q = \frac{2}{3} C_d l\sqrt{2g}H^{3/2}$$

When the length of the weir is less than the width of the stream, we find there will be a lateral contraction at each end such a weir is called a contracted weir.

According to Francis each lateral contraction (also called end contraction) is equal to 0.1 H.

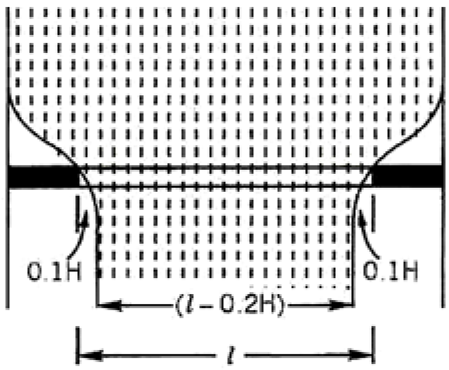
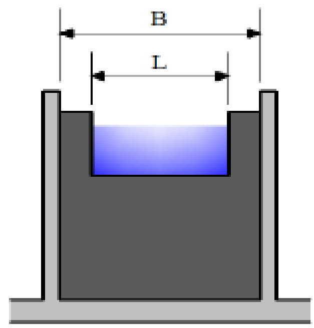



**Fig. 9.16.**




Contracted

If the actual length of the weir is $l$ , then the effective length of the weir will be ( $l$ – 0.2 H). In some cases there may be intermediate obstacles like piers over the weir. In such a case if $l$ is the length of the weir after making deductions for the widths of the obstacles and if there are n lateral contractions the effective length of the weir will be ($l$ – 0.1 nH).

Hence the discharge over the weir will be :

$$q = \frac{2}{3} C_d \sqrt{2g} (l - 0.1nH) H^{3/2}$$

Taking $C_d = 0.623$, we have

$$q = \frac{2}{3} \times 0.623 \sqrt{2 \times 9.81} (l - 0.1nH) H^{3/2}$$

$$= 1.84\ (l - nH)\ H^{3/2}$$

The above formula is called Francis formula.

* The experimental work of Rehbock led to an empirical formula for **$C_d$** of well ventilated sharp- crested rectangular weir:

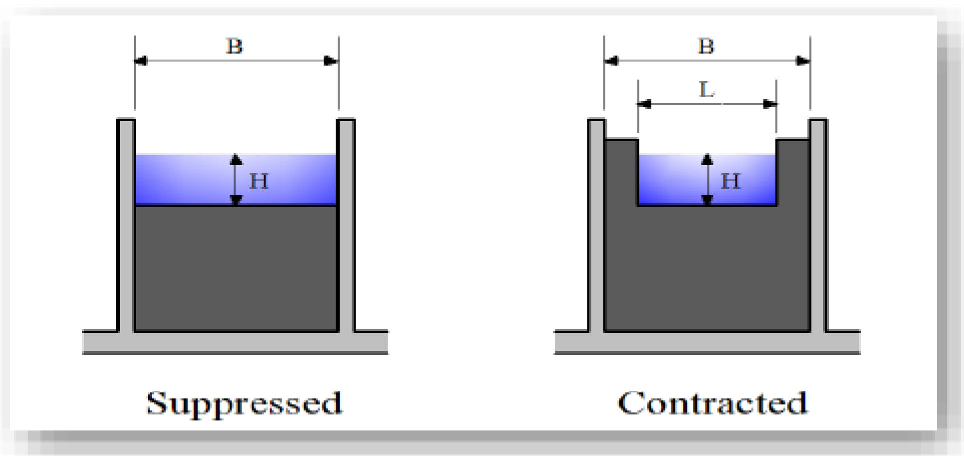
$$C_d = 0.611 + 0.08\frac{H}{P} \qquad \text{if } \left(\frac{H}{P} \leq 5\right)$$

The effective width (b) is considered as:

1) b = B  (for suppressed rectangular weir)
2) b = (B – n× 0.1 H)  (for contracted rectangular weir)

<u>where :</u>

n : number of contractions (usually one to each side)



## 2. Triangular Weir (V- notch)

Fig. 9.3 shown a triangular notch.

Let $H$ = head of water over the apex

$\theta$ = Angle of the notch

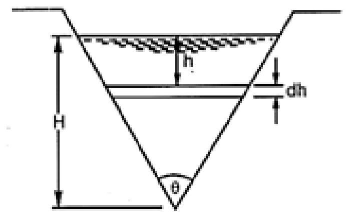



*Fig. 9.3.*

Width of the notch at any depth $h$

$$= 2(H-h)\tan\frac{\theta}{2}$$

Consider an elemental horizontal strip of the opening at depth $h$ and having a height $dh$. The theoretical velocity of flow through the strip $= \sqrt{2gh}$

$\therefore$ Theoretical discharge through the strip

$$= 2(H-h)\tan\frac{\theta}{2}\,dh\sqrt{2gh}$$

$$\text{Total discharge } = Q = \int_0^H 2\sqrt{2g}\tan\frac{\theta}{2}(H-h)h^{1/2}\,dh$$

$$= 2\sqrt{2g}\tan\frac{\theta}{2}\left[H\frac{2}{3}H^{3/2} - \frac{2}{5}H^{5/2}\right] = \frac{8}{15}\sqrt{2g}\tan\frac{\theta}{2}H^{5/2}$$

$$\text{Actual discharge } = q = \frac{8}{15}C_d\sqrt{2g}\tan\frac{\theta}{2}H^{5/2}$$

where $C_d$ = Coefficient of discharge

The vertex angle for a triangular notch may be from 25° to 90°. A vertex angle of 90° is commonly adopted. The coefficient of discharge is found to depend on the vertex angle. At lower heads and lower vertex angles the values of $C_d$ are found to be higher. This may be due to a lesser degree of contraction of the nappe.

$$\text{For a 90° notch } \tan\frac{\theta}{2} = 1$$

$$\text{and the discharge } = q = \frac{8}{15}C_d\sqrt{2g}H^{5/2}$$

Taking $C_d = 0.6$, we have

$$\frac{8}{15}C_d\sqrt{2g} = \frac{8}{15}\times 0.6\sqrt{2\times 9.81} = 1.47$$

and accordingly $q = 1.417\,H^{5/2}$.

Coefficient ($C_d$) for Lenz is:

$$C_d = 0.56 + \frac{0.7}{R_e^{\,0.165}\, W^{0.17}}$$

Where:

$R_e$: Reynolds No.

W: Surface tension

The conditions of ($C_d$) for Lenz:

1) $H > 0.06$ m 2) $R_e > 300$ 3) $W > 300$

**Note:** $C_d \approx 0.59$ for weir of ($2\alpha = 90°$)

---

**Example ( 1) :** Determine the discharge over a sharp crested weir 4.5m long with no lateral constrictions (suppressed). The measured head over the crest being 0.45m and the sill height of the weir is 1m.

**Solution:**

$$Q = C\frac{2}{3}\, b\, \sqrt{2g}\, H^{3/2}$$

$$\frac{H}{P} = \frac{0.45}{1.0} = 0.45 < 5$$

$$Cd = 0.611 + 0.08\,\frac{H}{P} = 0.611 + 0.08\,\frac{0.45}{1} = 0.647$$

$$Q = 0.647 * \frac{2}{3} * 4.5\,\sqrt{2 * 9.81}\,(0.45)^{3/2} = 2.61\ m^3/sec$$

**Example ( 2):** A 6m long weir was measured to carry a 1.4 $m^3$/ sec discharge when the crest is over topped by 0.2m of water. Determine the discharge coefficient of the weir?

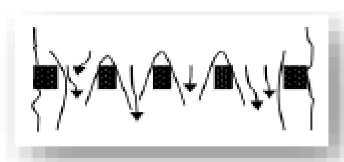
**Solution:**

$$C_d = \frac{Q}{\frac{2}{3} b \sqrt{2g}\, H^{3/2}} = \frac{1.4}{\frac{2}{3}*6*\sqrt{2*9.81}*(0.2)^{3/2}} = 0.883$$

---

**Example (3 ) :** A 30 m long weir is divided into 10 equal bays by vertical posts each 0.6 m wide. Calculate the discharge over the weir under an effective head of 1m? $C_d = 0.623$

**Solution:** Sometimes the total length of a weir is divided into a number of bars or span by vertical posts in such case, the number of bays or spans, into which the weir is divided.



No. of bays = 10 (30 m length of weir)

Width of each post = 0.6 m

Effective length L = (30 - 9* 0.6) = 24.6m

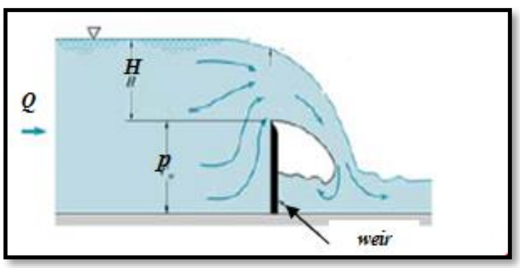
No. of end contractions, n = 2*10 = (one bay has two end contraction)

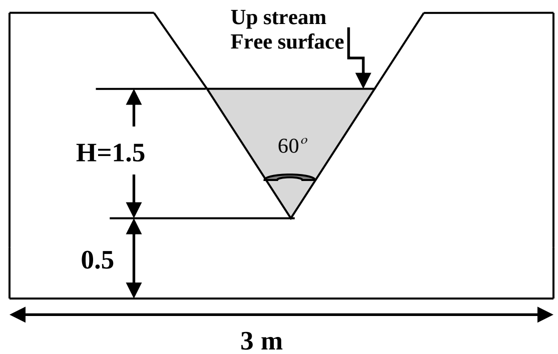
$$Q = \frac{2}{3}\, Cd\, (L - 0.1nH)\sqrt{2g}\; H^{3/2}$$

$$= 2/3 * 0.623 * \sqrt{2g}\,(24.6 - 0.1*20*1) * 1^{3/2} = 41.6\ m^3/sec$$

## H.W 1:-

The flow rate of water flowing in a (3 m ) wide channel is to be measured with a sharp crested triangular weir (0.5 m ) above the channel bottom with a notch angle of ( $60^o$ ) . if the flow depth upstream from the weir is (1.5 m ) , determine the flow rate of water through the channel . Take the weir discharge coefficient Cd = 0.6




## H.W 2:-

Water flows over a triangular right-angled weir with a depth is (0.375m) and afterwards passes through a rectangular weir ( 1m) wide . Find the depth of water through the rectangular weir .The value of Cd for triangular and rectangular weirs are 0.59 and 0.6 respectively .

# Hydraulics and Theories of Weirs

## 3. Rectangular Sharp – Crested Weirs

To find the discharge over rectangular weir, consider an elementary horizontal strip of water thickness dh and length ( $l$ )at a depth ( h ) from the water surface.

Consider a rectangular notch shown in Fig. 9.2.

Let $l$ = Length of the notch

$H$ = Head of water over the crest of the notch.

Consider an elemental horizontal strip of water of length $l$ and thickness $dh$, at a depth $h$ below the free surface of water.

Theoretical velocity of water flowing through the elemental strip $= \sqrt{2gh}$

$\therefore$ Theoretical discharge through the elemental strip $= l\,dh\sqrt{2gh}$

$$\therefore \text{Total theoretical discharge} = Q = l\sqrt{2g}\int_0^H h^{1/2}\,dh = \frac{2}{3} l\sqrt{2g}H^{3/2}$$

$$\text{Actual discharge, } = q = \frac{2}{3} C_d l\sqrt{2g}H^{3/2}$$

Where $= C_d$ = Coefficient of discharge.

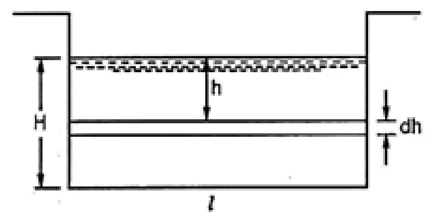



**Fig. 9.2.**

## Francis Formula—End Contractions:

We know for a rectangular notch of length ( $l$ ) the discharge over the notch is given by,

$$q = \frac{2}{3} C_d l\sqrt{2g}H^{3/2}$$

When the length of the weir is less than the width of the stream, we find there will be a lateral contraction at each end such a weir is called a contracted weir.

According to Francis each lateral contraction (also called end contraction) is equal to 0.1 H.

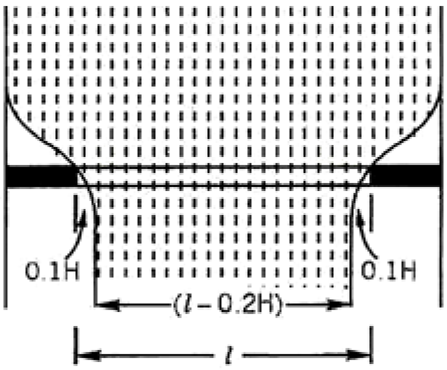
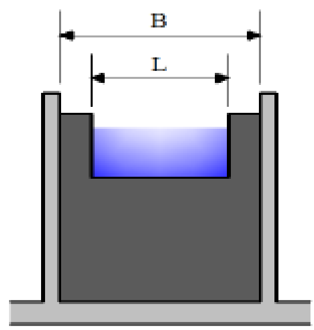



**Fig. 9.16.**




Contracted

If the actual length of the weir is $l$ , then the effective length of the weir will be ( $l$ – 0.2 H). In some cases there may be intermediate obstacles like piers over the weir. In such a case if $l$ is the length of the weir after making deductions for the widths of the obstacles and if there are n lateral contractions the effective length of the weir will be ($l$ – 0.1 nH).

Hence the discharge over the weir will be :

$$q = \frac{2}{3} C_d \sqrt{2g}(l - 0.1nH)H^{3/2}$$

Taking $C_d = 0.623$, we have

$$q = \frac{2}{3} \times 0.623\sqrt{2 \times 9.81}(l - 0.1nH)H^{3/2}$$

$$= 1.84\ (l - nH)\ H^{3/2}$$

The above formula is called Francis formula.

* The experimental work of Rehbock led to an empirical formula for **$C_d$** of well ventilated sharp- crested rectangular weir:

$$C_d = 0.611 + 0.08\frac{H}{P} \qquad \text{if} \quad \left(\frac{H}{P} \leq 5\right)$$

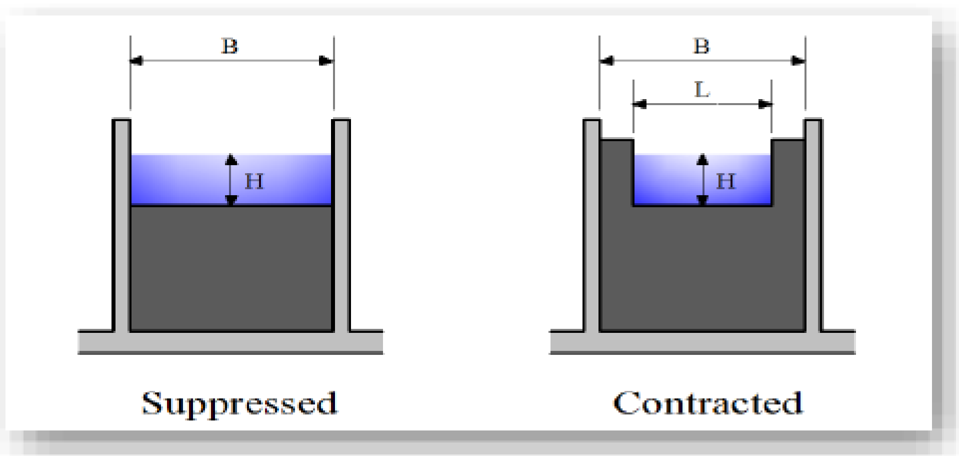
The effective width (b) is considered as:

3) b = B (for suppressed rectangular weir)

4) b = (B − n× 0.1 H) (for contracted rectangular weir)

where :

n : number of contractions (usually one to each side)



## 4. Triangular Weir (V- notch)

Fig. 9.3 shown a triangular notch.

Let $H$ = head of water over the apex

$\theta$ = Angle of the notch

Width of the notch at any depth $h$

$$= 2(H-h)\tan\frac{\theta}{2}$$

Consider an elemental horizontal strip of the opening at depth $h$ and having a height $dh$. The theoretical velocity of flow through the strip $= \sqrt{2gh}$

$\therefore$ Theoretical discharge through the strip

$$= 2(H-h)\tan\frac{\theta}{2}\, dh\sqrt{2gh}$$

$$\text{Total discharge } = Q = \int_0^H 2\sqrt{2g}\tan\frac{\theta}{2}(H-h)h^{1/2}\,dh$$

$$= 2\sqrt{2g}\tan\frac{\theta}{2}\left[H\frac{2}{3}H^{3/2} - \frac{2}{5}H^{5/2}\right] = \frac{8}{15}\sqrt{2g}\tan\frac{\theta}{2}H^{5/2}$$

$$\text{Actual discharge } = q = \frac{8}{15}C_d\sqrt{2g}\tan\frac{\theta}{2}H^{5/2}$$

where $C_d$ = Coefficient of discharge

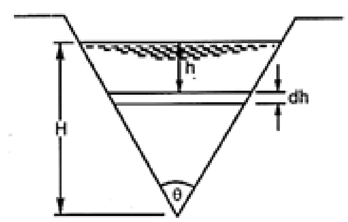



Fig. 9.3.

The vertex angle for a triangular notch may be from 25° to 90°. A vertex angle of 90° is commonly adopted. The coefficient of discharge is found to depend on the vertex angle. At lower heads and lower vertex angles the values of $C_d$ are found to be higher. This may be due to a lesser degree of contraction of the nappe.

$$\text{For a } 90° \text{ notch } \tan\frac{\theta}{2} = 1$$

$$\text{and the discharge } = q = \frac{8}{15}C_d\sqrt{2g}H^{5/2}$$

Taking $C_d = 0.6$, we have

$$\frac{8}{15}C_d\sqrt{2g} = \frac{8}{15}\times 0.6\sqrt{2\times 9.81} = 1.47$$

and accordingly $q = 1.417\, H^{5/2}$.

Coefficient ($C_d$) for Lenz is:

$$C_d = 0.56 + \frac{0.7}{R_e^{0.165}\, W^{0.17}}$$

Where:

$R_e$: Reynolds No.

W: Surface tension

The conditions of ($C_d$) for Lenz:

2) $H > 0.06$ m 2) $R_e > 300$ 3) $W > 300$

**Note:** $C_d \approx 0.59$ for weir of ($2\alpha = 90^\circ$)

---

**Example ( 1) :** Determine the discharge over a sharp crested weir 4.5m long with no lateral constrictions (suppressed). The measured head over the crest being 0.45m and the sill height of the weir is 1m.

**Solution:**

$$Q = C\frac{2}{3}\, b\sqrt{2g}\, H^{3/2}$$

$$\frac{H}{P} = \frac{0.45}{1.0} = 0.45 < 5$$

$$Cd = 0.611 + 0.08\frac{H}{P} = 0.611 + 0.08\frac{0.45}{1} = 0.647$$

$$Q = 0.647 * \frac{2}{3} * 4.5\sqrt{2 * 9.81}(0.45)^{3/2} = 2.61\ m^3/sec$$

**Example ( 2):** A 6m long weir was measured to carry a 1.4 $m^3$/ sec discharge when the crest is over topped by 0.2m of water. Determine the discharge coefficient of the weir?

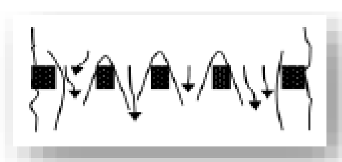
**Solution:**

$$C_d = \frac{Q}{\frac{2}{3} b \sqrt{2g}\, H^{3/2}} = \frac{1.4}{\frac{2}{3} * 6 * \sqrt{2 * 9.81} * (0.2)^{3/2}} = 0.883$$

---

**Example (3 ) :** A 30 m long weir is divided into 10 equal bays by vertical posts each 0.6 m wide. Calculate the discharge over the weir under an effective head of 1m? $C_d = 0.623$

**Solution:** Sometimes the total length of a weir is divided into a number of bars or span by vertical posts in such case, the number of bays or spans, into which the weir is divided.



No. of bays = 10 (30 m length of weir)

Width of each post = 0.6 m

Effective length L = (30 - 9* 0.6) = 24.6m

No. of end contractions, n = 2*10 = (one bay has two end contraction)

$$Q = \frac{2}{3}\, Cd\, (L - 0.1nH)\sqrt{2g}\ H^{3/2}$$

$$= 2/3 * 0.623 * \sqrt{2g}\, (24.6 - 0.1*20*1) * 1^{3/2} = 41.6\ m^3/sec$$

## 3. The Trapezoidal Notch:

Consider a trapezoidal notch whose crest length is $l$ and the sides are at θ with the vertical.

Let H be the head of water over the crest. In this case the notch may be taken to consist of a rectangular notch of length ( $l$ ) and a triangular notch subtending an angle 2θ

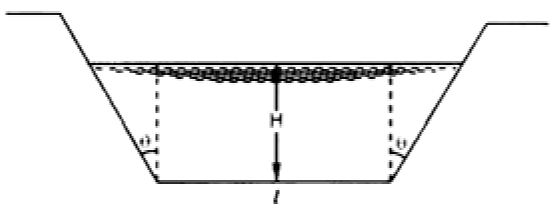



*Fig. 9.4. The trapezoidal notch.*

Total discharge = $q$ = discharge over the rectangular notch + discharge over the triangular notch.

$$q = \frac{2}{3} C_d l \sqrt{2g}\, H^{3/2} + \frac{8}{15} C_d \sqrt{2g} \tan\theta \cdot H^{5/2}$$

$$= C_d \sqrt{2g}\, H^{3/2} \left( \frac{2}{3} l + \frac{8}{15} \tan\theta . H \right)$$

## 4. The Cippoletti Weir:

We know for a rectangular weir with the two end contractions, the discharge is given by :

$$q = \frac{2}{3} C_d \sqrt{2g}(l - 0.2H)H^{3/2} = \frac{2}{3} C_d l \sqrt{2g} H^{3/2} - \frac{2}{15} C_d l \sqrt{2g} H^{5/2}$$

Hence we find that due to end contractions the discharging capacity of the weir is reduced by $\frac{2}{15} C_d \sqrt{2g} H^{5/2}$. This loss of discharging capacity can be made good by providing side slopes making the weir trapezoidal. The side slopes should be such that the increase in discharge through the additional triangular portion is exactly equal to the loss of flow due to end contractions. If θ be the slope of the sides with the vertical, the condition to be satisfied is,

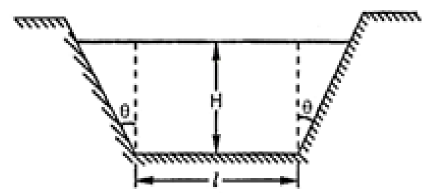



*Fig. 9.18.*

$$\frac{8}{15} C_d \sqrt{2g} \tan\theta\, H^{5/2} = \frac{2}{15} C_d \sqrt{2g} H^{5/2}$$

$$\therefore \qquad \tan\theta = 1/4$$

$$\therefore \qquad \theta = 14° 2'$$

A weir which is so compensated to make up for the loss of discharging capacity due to end contractions is called a *Cippoletti weir.*

When we use this type of weir , we can obtained W.L more stability than The type of rectangular weir because that ( $l$ ) increase with increase of the depth & give a greater discharge & keep the W.L At stable , therefore ; its use in the escape weir.

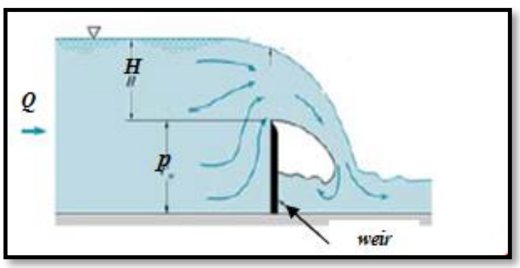
---

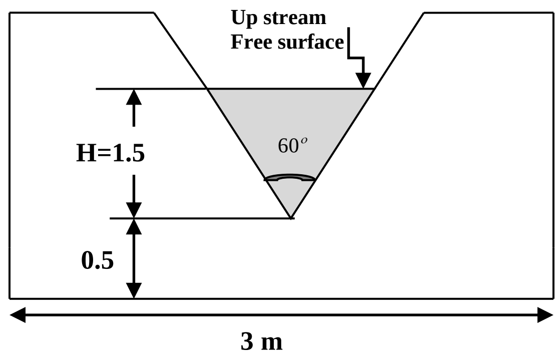
***H.W 1:-***

Water flows over a triangular right-angled weir with a depth is (0.375m) and afterwards passes through a rectangular weir ( 1m) wide . Find the depth of water through the rectangular weir .The value of Cd for triangular and rectangular weirs are 0.59 and 0.6 respectively .

## H.W 2:-

The flow rate of water flowing in a (3 m ) wide channel is to be measured with a sharp crested triangular weir (0.5 m ) above the channel bottom with a notch angle of ( $60^o$ ) . if the flow depth upstream from the weir is (1.5 m ) , determine the flow rate of water through the channel . Take the weir discharge coefficient Cd = 0.6



# Weirs

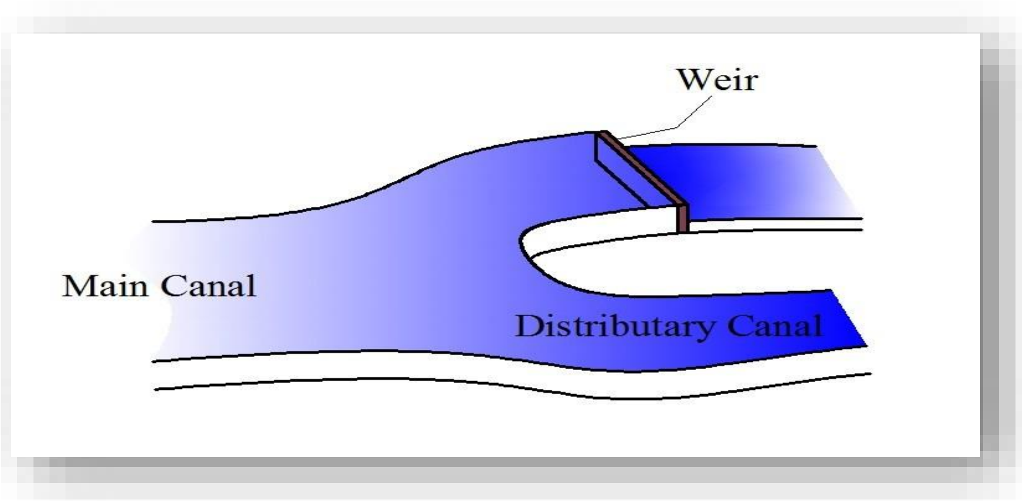
## 1. Definition:

An overflow structure designed to measure the discharge of water in a river or open channel. It is placed perpendicular to the flow direction of water. It is also used to prevent flooding or to make a river more navigable.

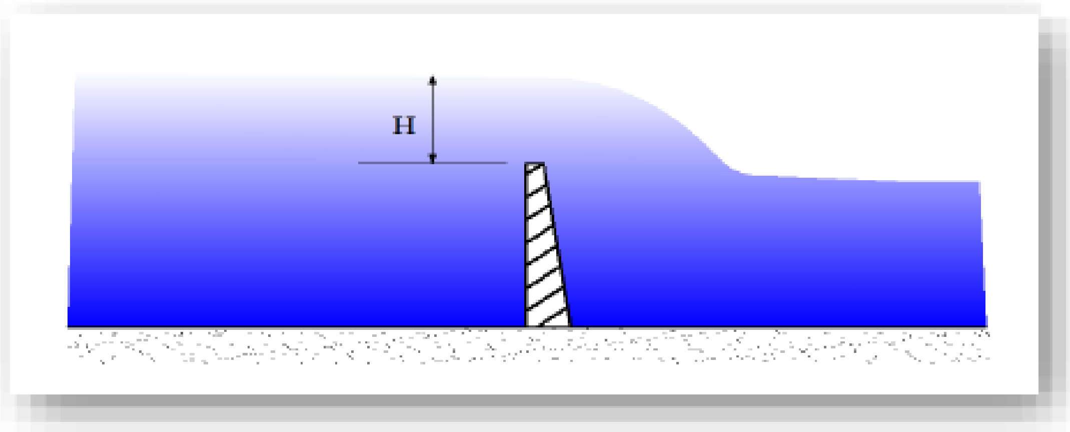
## 2. Practical Purposes of Weirs :

Weirs are used for the following purposes:

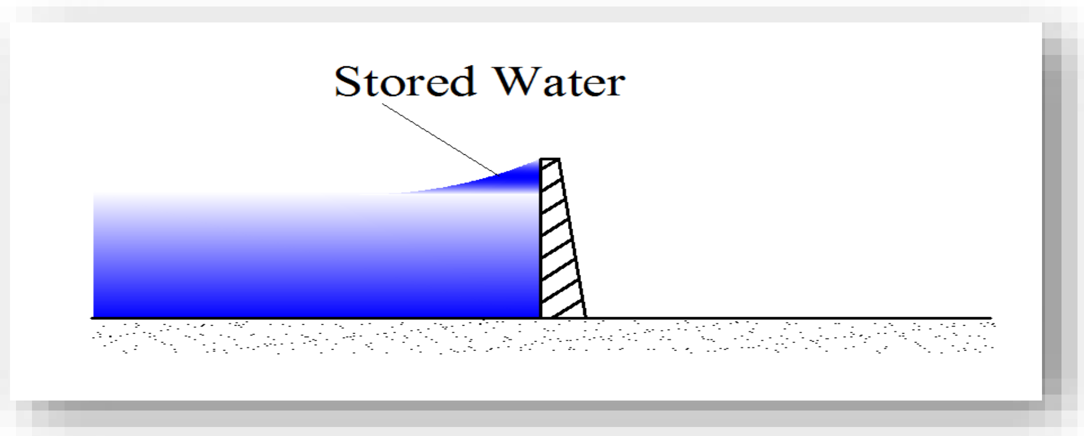
1. To maintain high water level in order to divert water into a diversion channel for irrigation or Power purpose.



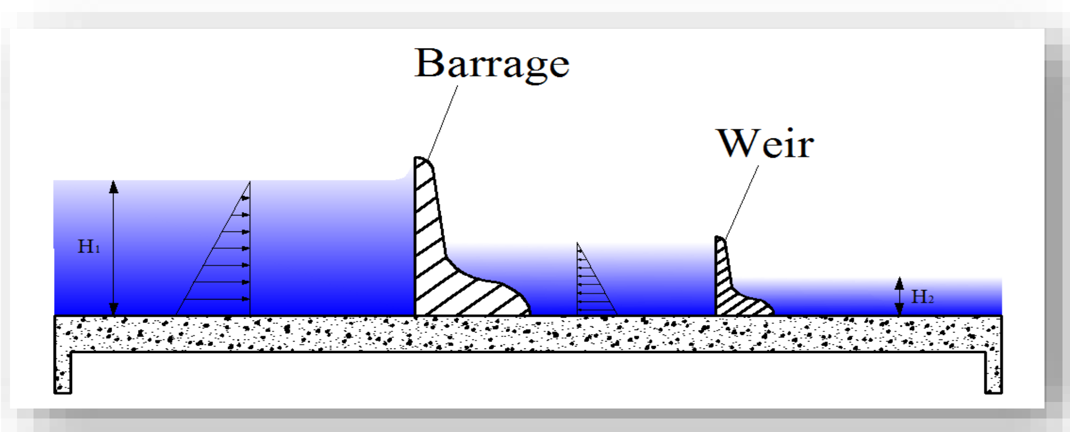

2. To gauge the discharge of branch channel at their intakes, the discharge of drains at their escape & the discharge of canals funding power houses.



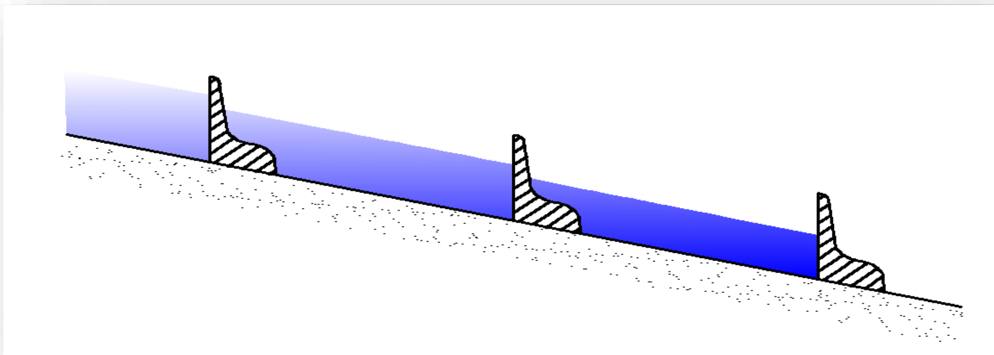

**3.** Water can be stored for a short period



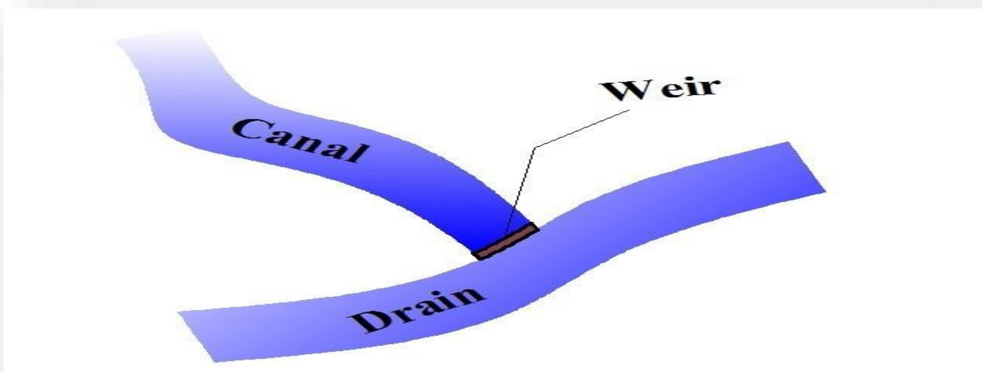

**4.** To reduce the head acting on a barrage.



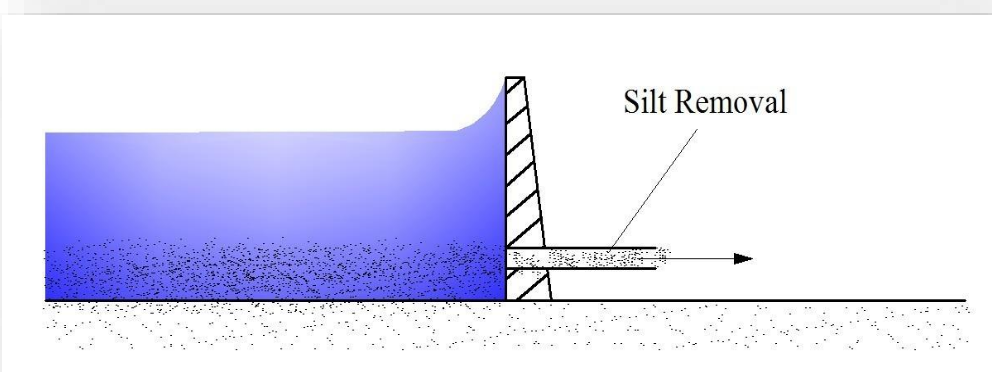
**5.** To reduce the water slope in case of a very steep land.



**6.** To escape the water in canal automatically.




**7.** To control silt movement into the canal system, we can use the weir to many purpose at the same time.



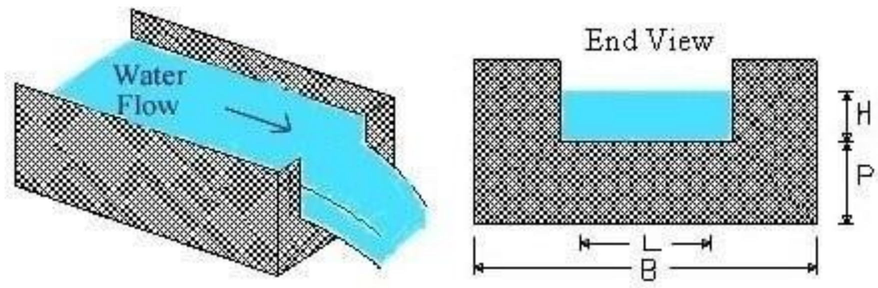
## 3. Classification or Types of Notches and Weirs :

Weirs are classified according to:

### 1) Types of Weirs based on Shape of the Opening

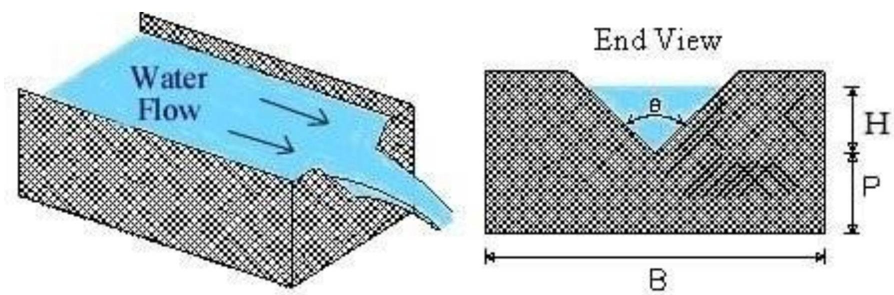
- **Rectangular weir**

- It is a standard shape of weir. The top edge of weir may be sharp crested or narrow crested.
- It is generally suitable for larger flowing channels.



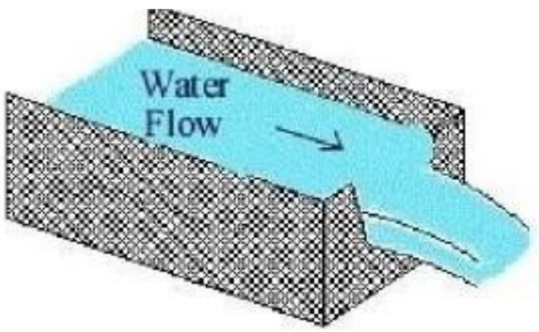

- **Triangular weir**

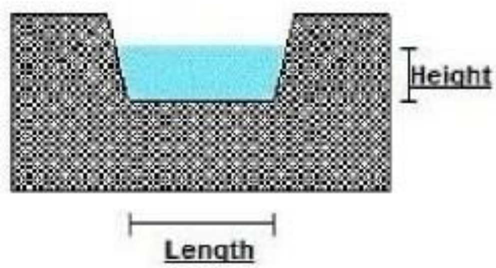
- The shape of the weir is actually reverse triangle like V. so. it is also called V-notch weir.
- This type of weirs are well suitable for measuring discharge over small flows with greater accuracy



- **Trapezoidal weir**

- In this case the notch may be taken to consist of a rectangular notch of length ( L ) and a triangular notch subtending an angle 2θ.
- Trapezoidal weir is also called as Cippoletti weir , if the sides are inclined outwards with a slope ( 1H : 4 V )

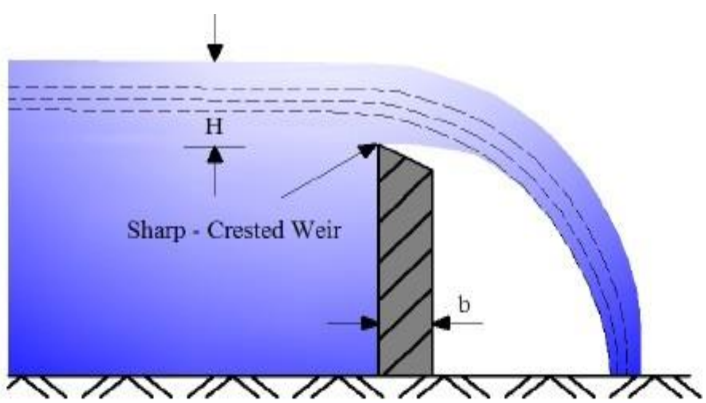



## 2) Types of Weirs based on Shape of the Crest

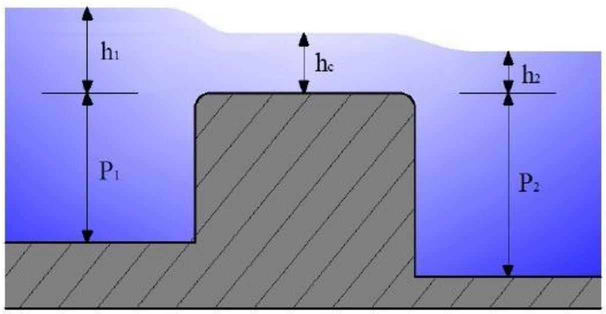
- **Sharp-crested weir**

- The crest of the weir is very sharp such that the water will springs clear of the crest.
- Flow over sharp-crested weir is similar as rectangular weir



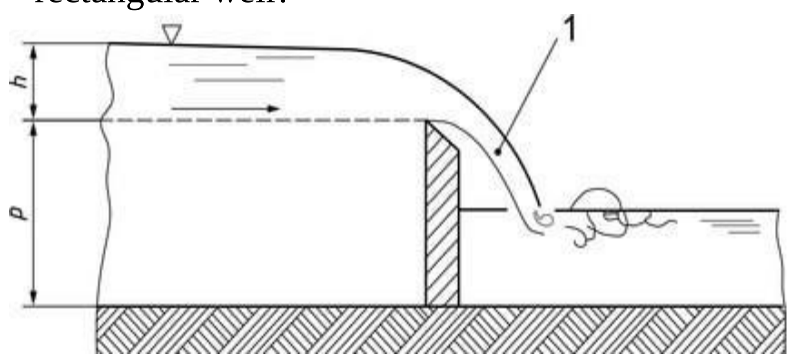
- **Broad- crested weir**

- These are constructed only in rectangular shape and are suitable for the larger flows.
- Head loss will be small in case of broad crested weir



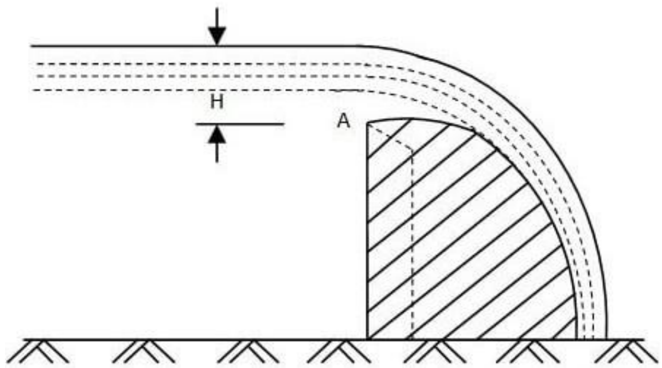

- **Narrow-crested weir**

- It is similar to rectangular weir with narrow shaped crest at the top.
- The discharge over narrow crested weir is similar to discharge over rectangular weir.



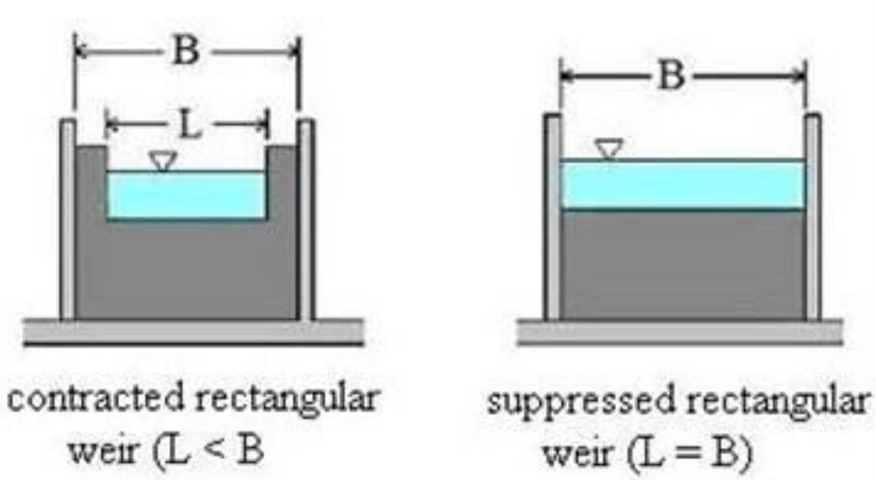
- **Ogee-shaped weir**

- Generally ogee shaped weirs are provided for the spillway of a storage dam.
- The crest of the 0266 weir is slightly rises and falls into parabolic form.
- Flow over ogee weir is also similar to flow over rectangular weir.

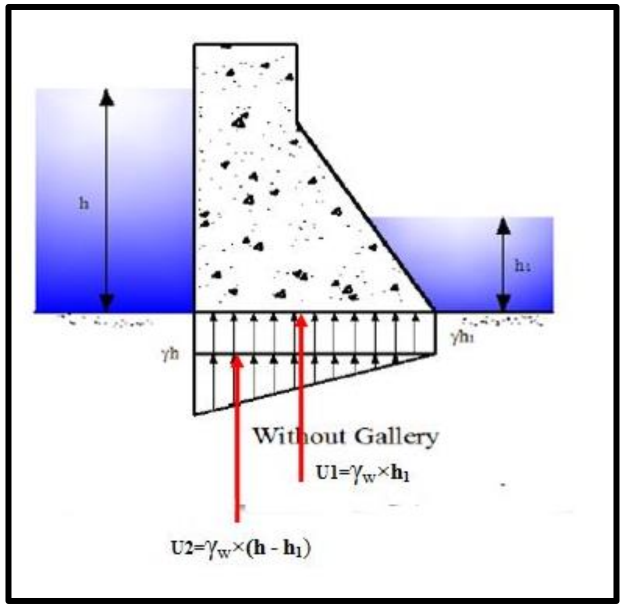



### 3) Types of weirs based on Effect of the sides on the emerging nappe

- Weir with end contraction (contracted weir)
- Weir without end contraction (suppressed weir)

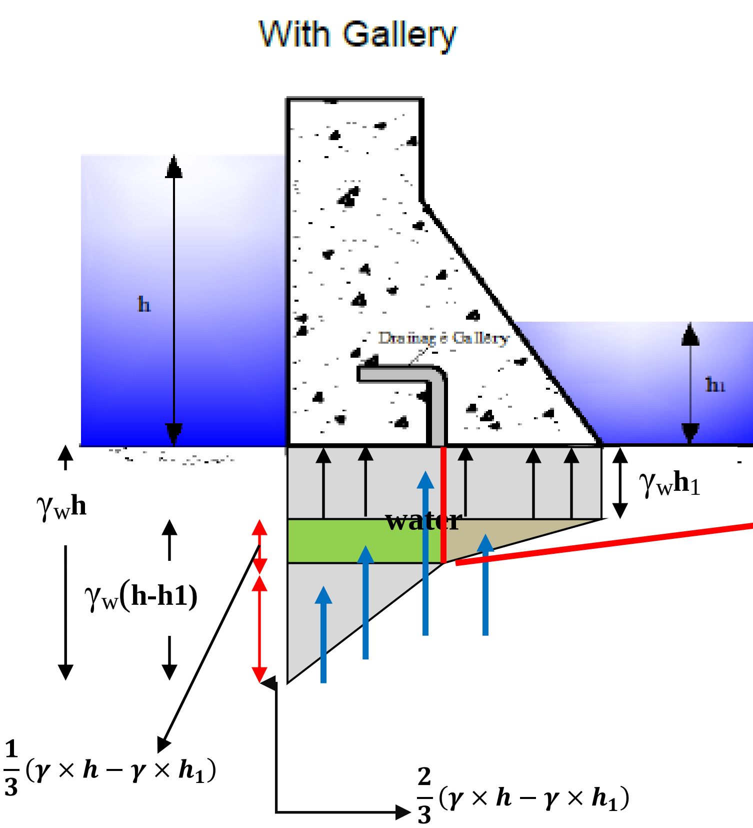


## 1) Calculate uplift pressure without gallery




## 2) Calculate uplift pressure with gallery

### i) There is tail water of height h1 on the downstream side



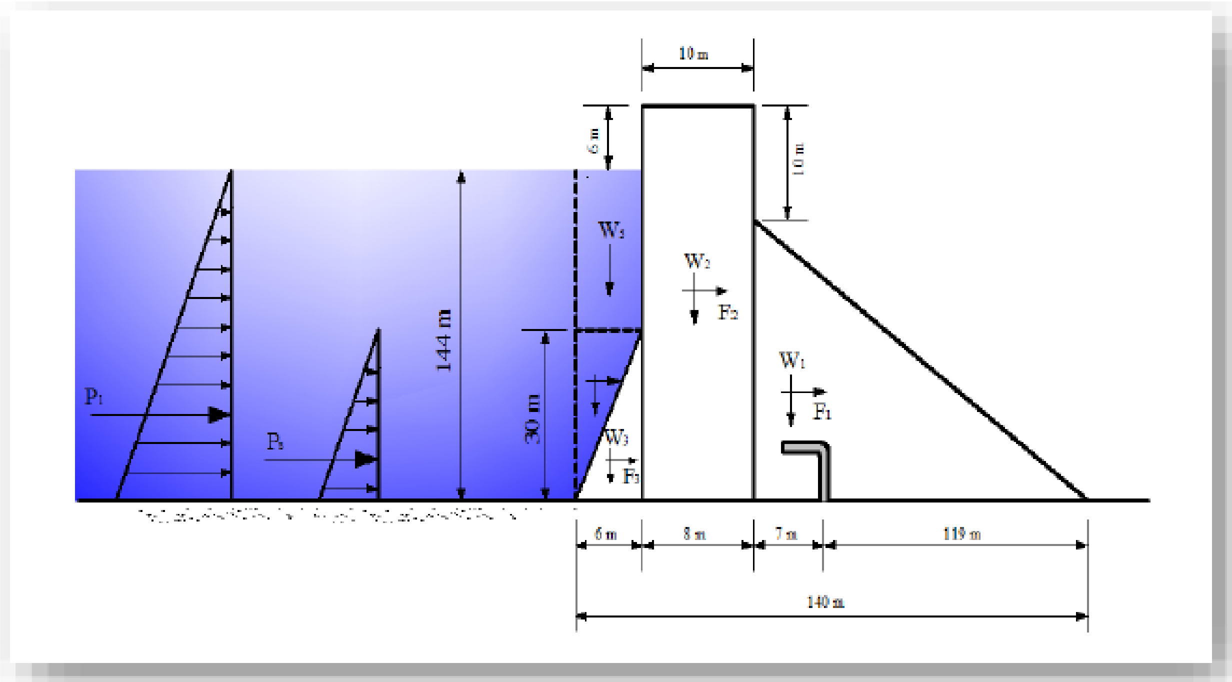

$$\gamma\times h_1+\frac{1}{3}(\gamma\times h-\gamma\times h_1)=\gamma(\frac{(2h_1+h)}{3})$$

**(H.W) draw the uplift pressure**

**ii) There is no tail water ( h1 =0 on the downstream side)**

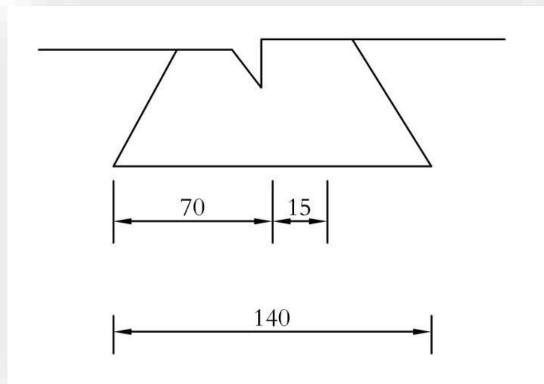
**Example :** Determine the heel and toe stresses and the factor of safeties for sliding and overturning for the gravity dam section shown in the figure below for the following loading conditions:

- Horizontal earthquake (Kh) = 0.1
- Normal uplift pressure with gallery drain working
- Silt deposit up to 30 m height
- No wave pressure and no ice pressure
- Unit weight of concrete = 2.4 Ton/$m^3$ and unit weight of silty water = 1.4 Ton/$m^3$
- Submerged weight of silt = 0.9 Ton/$m^3$
- Coefficient of friction = 0.65 and angle of repose = 25°




## Solution :

| Name of forces | Magnitude | $L_a$ (m) | Moment toe |
|---|---|---|---|
| **1- vertical forces** | | | |
| $W_{1(\text{weight of dam})}$ | (126+140) * 0.5 * 1*2.4 = 21168 +↓ | 84.3 | +1778112 |
| $W_{2(\text{weight of dam})}$ | 150 * 8 *1*2.4 = 2889+↓ | 130 | 374400 |
| $W_{3(\text{weight of dam})}$ | (30+6)*0.5*1*2.4 = 216 +↓ | 136 | 29376 |
| $W_{4(\text{weight of silt})}$ | (30+6)*0.5*1*1.4 = 126 +↓ | 138 | 17388 |
| $W_{5(\text{weight of water})}$ | 114 *6*1*1 = 684 +↓ | 137 | 93708 |
| **∑ W** | **+ 25074** | | **∑ M = 229284** |
| **2- Uplift Pressure↑(-)** | | | |
| $U_1$ | ⅓ (1)(144)(21)(1) = - 1008 ↑(-) | 129.5 | - 130536 |
| $U_2$ | ½ (⅓ *1*144)*119*1 = - 2856 ↑ | 79.33 | - 226576 |
| $U_3$ | ½ (⅔*1*144)*21*1 = -1008 ↑ | 133 | - 134065 |
| **∑ U↑(-)** | **- 4872** | | **∑ M = -491176** |
| **3- horizontal forces→(-)** | | | |
| $P_1 = \frac{1}{2}\gamma h^2$ | $0.5 * 144^2 * 1*1 = - 10368$ | 144/3 | - 497664 |
| $P_s = \frac{1}{2}\gamma_s h_s^2 k_a$ | $0.5 *0.9 * 30^2 *0.4058 = - 164.4$ | 30/3 | - 164.4 |
| **∑ H→(-)** | **- 10532.4** | | **- 499014** |
| **4- earth quake →(-)** | | | |
| $F_1 = W_1 * K_h$ | 21168 *0.1= - 2116.8 | 140/3 | - 98784 |
| $F_2 = W_2 * K_h$ | 2880*0.1 = - 288 | 150/2 | - 21600 |
| $F_3 = W_3 * K_h$ | 216 *0.1 = - 21.6 | 30/3 | - 216 |
| Hydrodynamic ($P_e$) ($0.55\, k_h \gamma h^2$) | $0.55 * 0.1 *1 * 144^2 = - 1150.848$ | $144/3\pi$ | - 703346 |
| **∑ E** | **- 3577.284** | | **∑ M = - 190934.6** |




$\sum M = +2292984 - 499014 - 190934.6 - 491176 = 1111859.4$ Ton. m

$\sum V = \sum W - \sum U = 25074 - 4872 = 20202$ Ton $e = (B/2) - X' = (B/2) - (\sum M / \sum V)$

$e = (140/2) - (1111859.4 / 20202) = 15$ m

$P_{max,\ min} = (\sum V / B)\ (1 \pm ((6e)/B)) = (20202/140)\ (1 \pm ((6*15)/140))$

$P_{max} = 237.06$ Ton / m2

$P_{min} = 51.54$ Ton / m2

$F.S_{sliding} = (\mu \sum (V-U)) / (\sum H) \quad > 1.0$

$= 0.933 < 1$ not o.k.

$(F.S)_{overturning} = (\sum M_R) / (\sum M_O) = 2292984 / (Mu + M_H + M_E)$