3.4. CREEP'S THEORY

Bligh's Theory

Concept of the Theory:

Bligh assumed that the water which percolates into the foundation creeps through the joint between the profile of the base of weir and the subsoil. Of course water also percolates into the subsoil. He then stated that this percolating water loses its head en-route. The seeping water finally comes out at the downstream end. According to Bligh water travels along vertical, horizontal or inclined path without making any distinction.

The total length covered by the percolating water till it emerges out at the downstream end is called a creep length. It is clear from the knowledge of hydraulics that the head of water lost in the path of percolation is the difference of water levels on the upstream and the downstream ends. Also, an imaginary line which joins the water levels on the upstream and the downstream end is called a hydraulic gradient line. Figure 19.3 (a, b) gives the lull explanation of Bligh's theory.

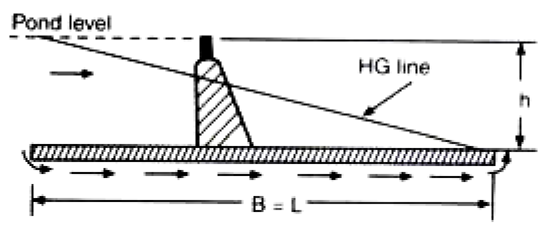


Fig. 19.3. (a) Bligh's creep theory

In Fig. 19.3 (a) arrows show the path followed by the creeping water.
B = L = total creep length and h/L is the head lost in creeping.
Loss of head per unit creep length will be h/L and it is hydraulic gradient.
To increase the path of percolation vertical cutoffs or sheet piles can be provided. Fig. 19.3 (b).

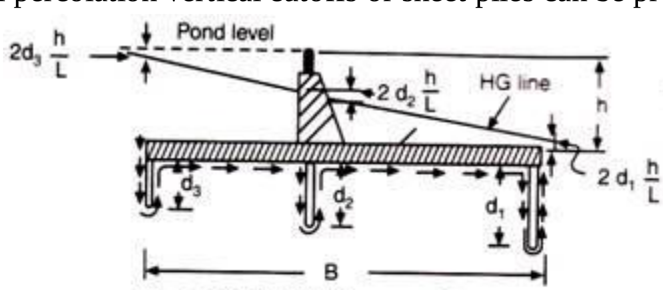


Fig. 19.3. (b) Bligh's creep theory

Bligh took vertical and horizontal path of percolation in the same sense. So now

Total creep length,

$$L = B + 2d_1 + 2d_2 + 2d_3$$

$$\therefore \quad HG = h/L$$

$$\text{or} \quad = \frac{h}{B + 2d_1 + 2d_2 + 2d_3}$$

When the water follows a vertical path the loss takes place in a vertical plane at same section. This loss is proportional to the length of the vertical path. For example, for cutoff d1, loss will be h/L x2d1 and it takes place in its plane. Loss of head at other cutoffs may be calculated in the same way.

Bligh gave the criteria for the safety of a weir against piping and uplift separately and is as follows:

The structure is safe against piping when the percolating water retains negligible upward pressure when it emerges out at the downstream end of the weir. Obviously the path of percolation should be sufficiently long to provide safe hydraulic gradient. It depends on the soil type.

This condition is provided by equation

L = CH

where L is creep length or path of percolation;
C is Bligh's creep co-efficient for soil; and
H is head of water against the weir.

Table 19.1 gives values of C for various soil types:

Table 19.1.

Soil Type	*Value of C*
Silt and sandy clay	18
Micaceous sand (North Indian river)	15
Coarse grained sand	12
Sand mixed with boulder, gravel and shingle	5 to 9
Loamy soil	5 to 9
Gravel	5

Obviously for no piping

$$\frac{H}{L} \leq \frac{1}{C} \quad \text{or}$$

$$\text{Hydraulic gradient} \leq \frac{1}{C}$$

To make the apron floor safe against the uplift pressure Bligh gives following criteria: From Fig. 19.4 it is clear that the uplift pressure at any point is represented by the ordinate between the bottom of apron floor and the hydraulic gradient line.

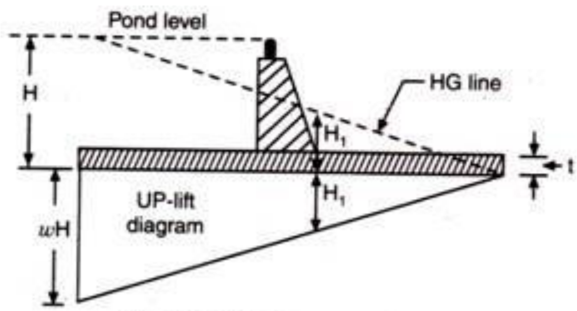


Fig. 19.4. Bligh's creep theory

Hence, uplift pressure = wH_1

where w is the density of water and H_1 is the ordinate between HG line and bottom of the apron floor.

Downward force exerted by the material in the apron is given by

$$t \cdot w \cdot \rho$$

where t is the thickness of apron floor and ρ is the specific gravity of material used in the apron.

The stable condition is attained when

$$wH_1 = t \cdot w \cdot \rho$$

or $$t = \frac{H_1}{\rho} \qquad \text{...(1)}$$

It is clear from Fig. 19.4 that H1 can only be known when t is known. Hence to determine 't' following algebraical manipulation may be done. From equation (1)

$$H_1 = t \cdot \rho$$

$$(H_1 - t) = t \cdot \rho - t = (\rho - 1) \cdot t$$

or $$\frac{(H_1 - t)}{\rho - 1} = t \qquad \text{...(2)}$$

Where (H, – t) is the ordinate between dotted HG line and top of the apron. It can be known easily and hence depth of apron can be calculated from equation (2). Now adding factor of safety of 4/3 to equation (2), the expression finally becomes

$$t = \frac{4}{3}\left(\frac{(H_1 - t)}{\rho - 1}\right) \qquad \text{...(3)}$$

For economy provide greater apron length on the upstream side which requires minimum practical thickness. Of course on the downstream side some minimum length of apron is required to protect the bed of the channel.

Limitations of Bligh's Theory:

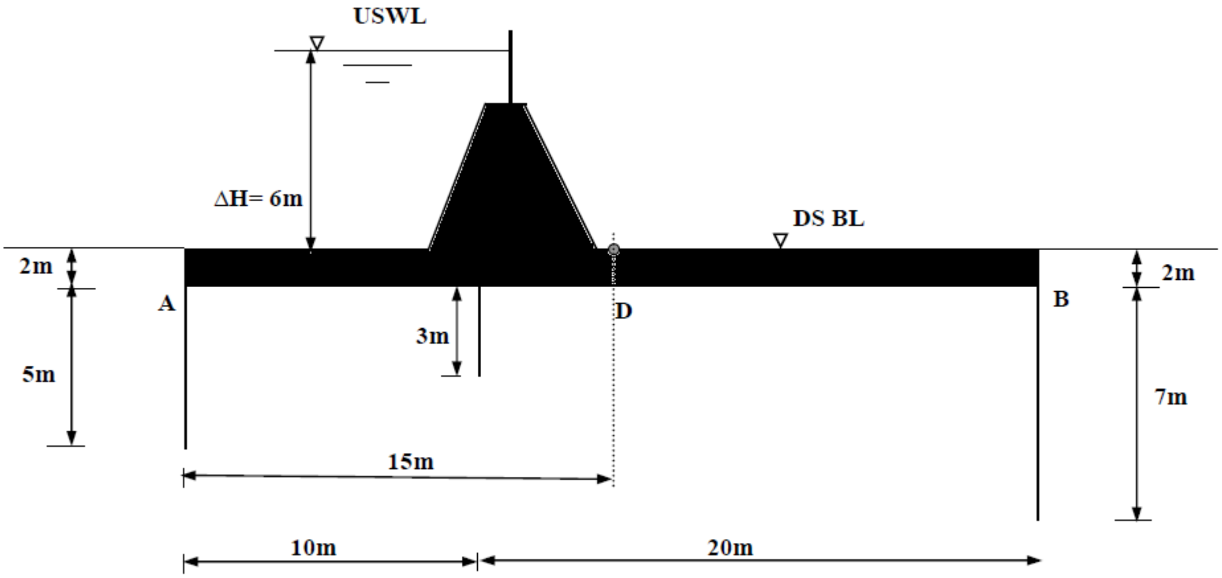
Bligh's theory has several limitations. They are:

- In his theory Bligh made no distinction between horizontal and vertical creep lengths.
- The idea of exit gradient has not been considered.
- The effect of varying lengths of sheet piles not considered.
- No distinction is made between inner or outer faces of the sheet piles.
- Loss of head is considered proportional to the creep length which in actual is not so.
- The uplift pressure distribution is not linear as assumed but in fact it follows a sine curve.

- Necessity of providing end sheet pile not appreciated.

Example: Find the hydraulic gradient and the head at point D of the following structure for Static condition.

The water percolates at A and exits at B.



Total creep length, Lc = 2 + 5*2 + 10 + 2*3 + 20 + 2*7 + 2 = 64m

Hydraulic gradient, i = H_L/Lc =6/64=1/10.66

According to the Bligh's table, the structure is safe on gravel and sand but not on coarse and fine sand.

Remember H_L/L =1/C !!!

Creep length up to point D (Lc_D) =2 + 5*2 + 15 + 2*3 = 33m

The residual uplift pressure head at D = U_D = H_L(1-Lc_D/Lc) = 6(1-33/64) = 2.9 m.

The thickness of floor at any point should be sufficient to resist the residual uplift pressure. If hD is the unbalanced head at point D, then

$$h_D = U_D - (\text{elevation of point D} - \text{elevation of DS floor}) = 2.9 - (0) = 2.9 \text{ m}$$

(Note: Point D is at the same level of DS floor level).

The thickness of floor, T_D, at point D should be h_D/(G-1) where G, is the specific gravity of the concrete floor, let Gs= 2.24, then

$$T_D = h_D/(G-1) = 2.9/(2.24-1) = 2.34 \text{ m}$$

Khosla's Theory

The seepage water does not creep along the outlines of hydraulic structure as started by Bligh, but on the other hand, this water moves along a set of stream-lines. This steady seepage in a vertical plane for a homogeneous soil can be expressed by Laplacian equation:

$$\frac{d^2\phi}{dx^2} + \frac{d^2\phi}{dz^2}$$

Where, φ = Flow potential = Kh; K = the co-efficient of permeability of soil as defined by Darcy's law, and h is the residual head at any point within the soil.

The above equation represents two sets of curves intersecting each other orthogonally. The resultant flow diagram showing both of the curves is called a Flow Net.

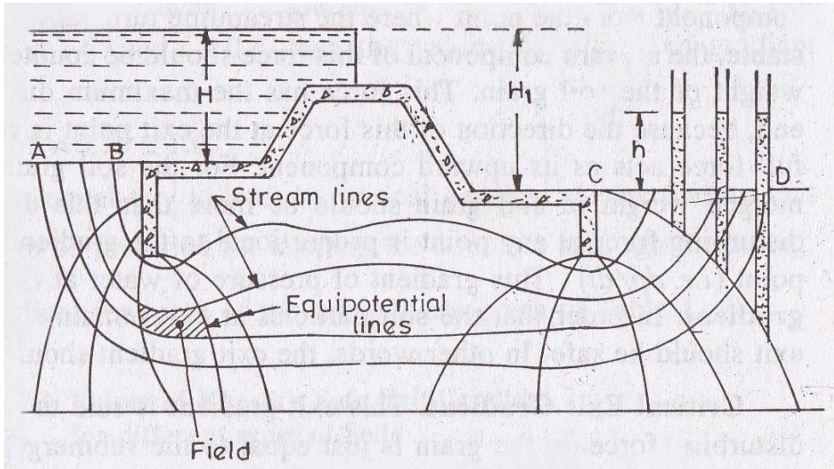
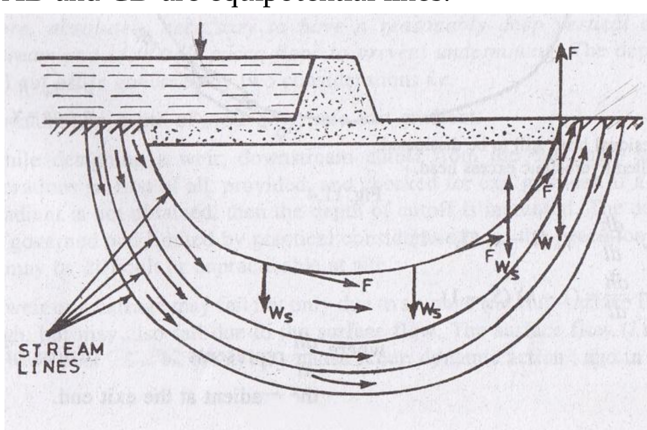


Fig: Khosla's Flow Net

Stream Lines: The streamlines represent the paths along which the water flows through the sub-soil. Every particle entering the soil at a given point upstream of the work will trace out its own path and will represent a streamline. The first streamline follows the bottom contour of the works and is the same as Bligh's path of creep.

An equipotential line represent the joining of points of equal residual head, hence if piezometers were installed on an equipotential line, the water will rise in all of them up to the same level

Every water particle on line AB is having a residual head h = h1, and on CD is having a residual head h = 0, and hence, AB and CD are equipotential lines.



Exit Gradient

The seepage water exerts a force at each point in the direction of flow and tangential to the streamlines as shown in figure above. This force (F) has an upward component from the point where the streamlines turns upward.

This force has the maximum disturbing tendency at the exit end, because the direction of this force at the exit point is vertically upward, and hence full force acts as its upward component. For the soil grain to remain stable, the submerged weight of soil grain should be more than this upward disturbing force. The disturbing force at any point is proportional to the gradient of pressure of water at that point (i.e. dp/dl). This gradient of pressure of water at the exit end is called the exit gradient. In order that the soil particles at exit remain stable, the upward pressure at exit should be safe. In other words, the exit gradient should be safe.

Critical Exit Gradient

This exit gradient is said to be critical, when the upward disturbing force on the grain is just equal to the submerged weight of the grain at the exit. When a factor of safety equal to 4 to 5 is used, the exit gradient can then be taken as safe. In other words, an exit gradient equal to ¼ to 1/5 of the critical exit gradient is ensured, so as to keep the structure safe against piping.

The submerged weight (Ws) of a unit volume of soil is given as:

$$\gamma_w (1 - n) (S_s - 1)$$

Where, γ_w = unit weight of water.

S_s = Specific gravity of soil particles

n = Porosity of the soil material

For critical conditions to occur at the exit point

$$F = W_s$$

Where F is the upward disturbing force on the grain

Force F = pressure gradient at that point = $dp/dl = \gamma_w \times dh/dl$

$$\therefore \quad \gamma_w \cdot \frac{dh}{dl} = \gamma_w (1 - n) (S_s - 1)$$

$$\frac{dh}{dl} = (1 - n) (S_s - 1)$$

2

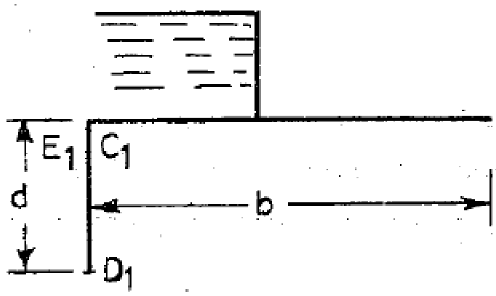
In order to know as to how the seepage below the foundation of a hydraulic structure is taking place, it is necessary to plot the flow net. In other words, we must solve the Laplacian equations.

This can be accomplished either by mathematical solution of the Laplacian equations, or by Electrical analogy method, or by graphical sketching by adjusting the streamlines and equipotential lines with respect to the boundary conditions. These are complicated methods and are time consuming.

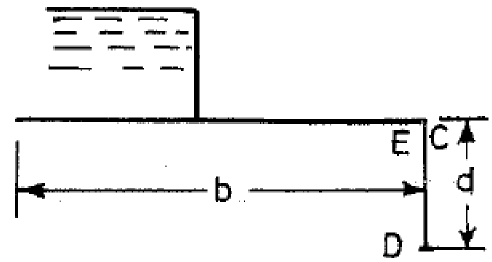
Therefore, for designing hydraulic structures such as weirs or barrage or pervious foundations, Khosla has evolved a simple, quick and an accurate approach, called Method of Independent Variables. In this method, a complex profile like that of a weir is broken into a number of simple profiles; each of which can be solved mathematically.

The simple profiles, which are most useful for analysis, are:

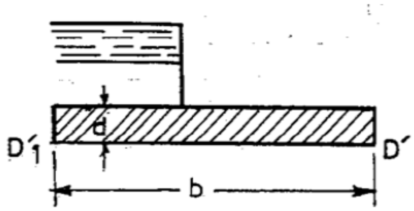
- A straight horizontal floor of negligible thickness with a sheet pile line on the u/s end and d/s end.
- A straight horizontal floor depressed below the bed but without any vertical cut-offs.
- A straight horizontal floor of negligible thickness with a sheet pile line at some intermediate point.



$$\phi_{C_1} = 100 - \phi_E$$
$$\phi_{D_1} = 100 - \phi_D$$

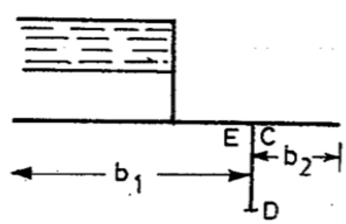


$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$$
$$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right)$$
$$\text{where} \quad \lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2}$$
$$\alpha = \frac{b}{d} \text{ (respective)}$$



$$\phi_{D'} = \phi_D - \frac{2}{3}(\phi_E - \phi_D) + \frac{3}{\alpha^2}$$
$$\phi'_{D_1} = 100 - \phi_{D'}$$

(c)

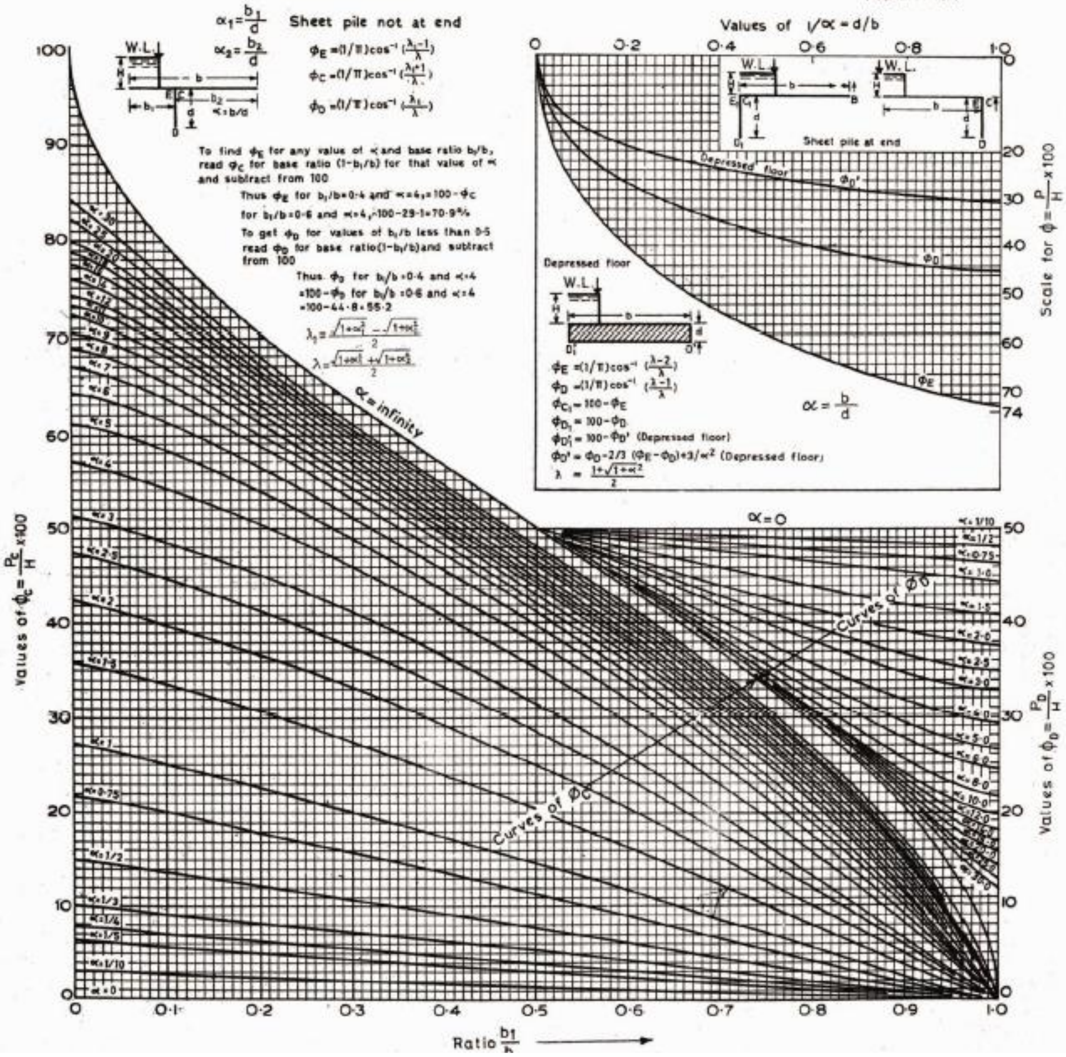


$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right)$$
$$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$$
$$\phi_C = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda}\right)$$
$$\text{where} \quad \lambda = \frac{\sqrt{1 + \alpha_1^2} + \sqrt{1 + \alpha_2^2}}{2}$$
$$\lambda_1 = \frac{\sqrt{1 + \alpha_1^2} - \sqrt{1 + \alpha_2^2}}{2}$$
$$\alpha_1 = b_1/d$$
$$\alpha_2 = b_2/d$$

(d)

Khosla's Pressure Curves

Plate 11·1(a)



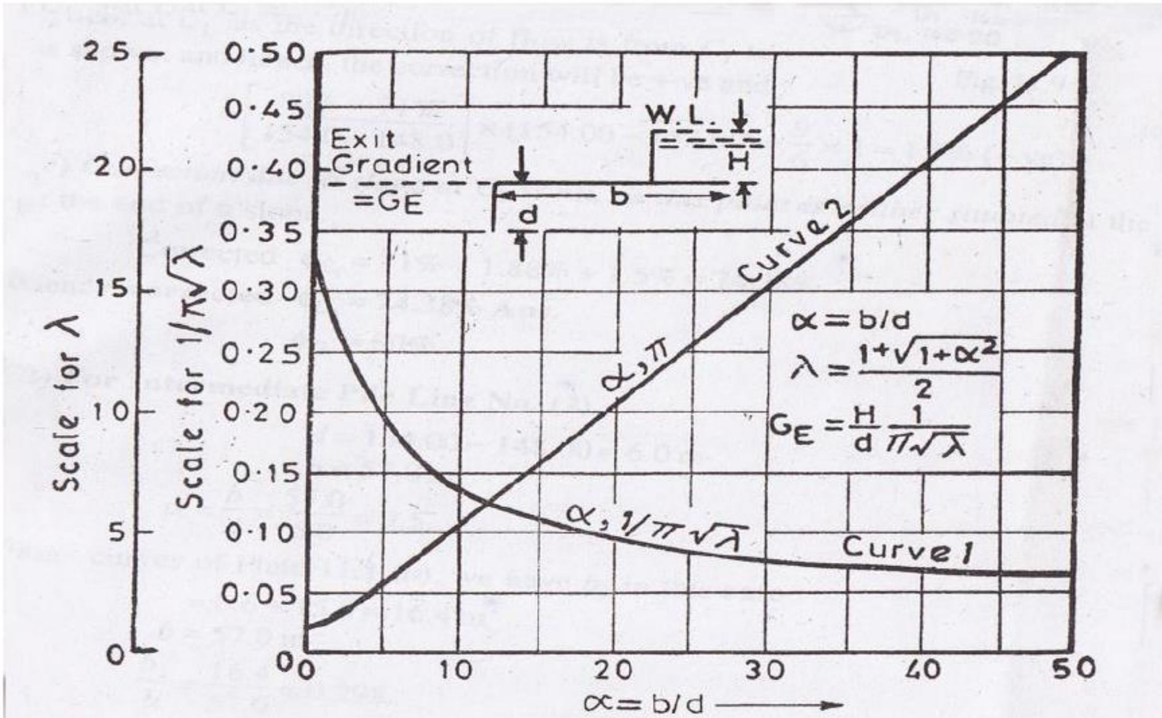


Plate 11.2

The percentage pressures at these key points for the simple forms into which the complex profile has been broken is valid for the complex profile itself, if corrected for

(a) Correction for the mutual interference of piles

(b) Correction for the thickness of floor

(c) Correction for the slope of the floor

(a) Correction for the Mutual interference of Piles

The correction, C, to be applied as percentage of head due to this effect, is given by

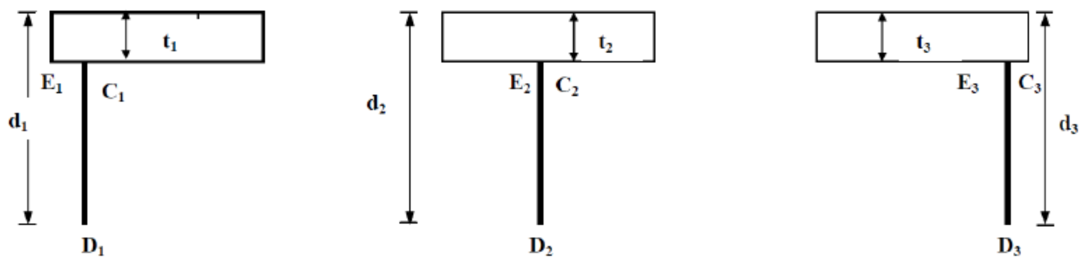
$$C = 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right)$$

Where,
b' = The distance between two pile lines.
D = The depth of the pile line, the influence of which has to be determined on the neighboring pile of depth, d. D is to be measured below the level at which interference is desired.
d = The depth of the pile on which the effect is considered
b = Total floor length

The correction is positive for the points in the rear of back water, and subtractive for the points forward in the direction of flow.

(b) Correction for the thickness of floor

In the standard form profiles, the floor is assumed to have negligible thickness. Hence, the percentage pressures calculated by Khosla's equations or graphs shall pertain to the top levels of the floor. While the actual junction points E and C are at the bottom of the floor. Hence, the pressures at the actual points are calculated by assuming a straight line pressure variation.



US sheet pile   Intermediate sheet pile   DS sheet pile

$\varphi_{E1} = 1$ **don't need any correction**

$$(C_t)_{C1} = \frac{t_1 * (\varphi_{D1} - \varphi_{C1})}{d_1} \quad (+\text{ ve})$$

$$(C_t)_{E2} = \frac{t_2 * (\varphi_{E2} - \varphi_{D2})}{d_2} \quad (-\text{ve})$$

$$(C_t)_{C2} = \frac{t_2 * (\varphi_{D2} - \varphi_{C2})}{d_2} \quad (+\text{ve})$$

Ct represent correction

$$(C_t)_{E3} = \frac{t_3 * (\varphi_{E3} - \varphi_{D3})}{d_3} \quad (-\text{ve})$$

$\varphi_{C3} = 0$ **don't need any correction**

(c) Correction for the slope of the floor

A correction is applied for a slopping floor, and is taken as positive for the downward slopes, and negative for the upward slopes following the direction of flow. Values of correction of standard slopes such as 1 : 1, 2 : 1, 3 : 1, etc.,

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

The correction factor given above is to be multiplied by the horizontal length of the slope and divided by the distance between the two pile lines between which the sloping floor is located. This correction is applicable only to the key points of the pile line fixed at the start or the end of the slope.

It has been determined that for a standard form consisting of a floor length (b) with a vertical cutoff of depth (d), the exit gradient at its downstream end is given by

$$G_E = \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}} \qquad \text{Where, } \lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$$

$\alpha = b/d$

H = Maximum Seepage Head

Type of Soil	Safe exit gradient
Shingle	1/4 to 1/5 (0.25 to 0.20)
Coarse Sand	1/5 to 1/6 (0.20 to 0.17)
Fine Sand	1/6 to 1/7 (0.17 to 0.14)

3.5. DESIGN OF VERTICAL DROP WEIR

- A vertical drop weir consists of a masonry/concrete crest wall with its d/s face vertical (or nearly vertical).
- In this type of weir, the energy is dissipated by the impact of water, as no hydraulic jump is formed.
- On the top of the crest wall, shutters are provided, if necessary.
- Vertical drop weirs are usually provided when the flood discharge is not very large.
- It is suitable for all types of foundations.
- The design is usually done by Bligh's theory.
- Finally, the thickness and length of floor is checked by Khosla's theory.

The following data should be collected for the design of vertical drop weir:

1. Maximum flood discharge (Q)
2. H.F.L. before construction of weir.
3. Average bed level of the river.
4. F.S.L. of off·taking canals.
5. Lacey's silt factor (f)
6. Permissible exit-gradient (GE)
7. Permissible Afflux

I. Lacey's silt factor (f) for the silt at the river site is generally decided by experience. It can also be determined from the average size of the particles as

$$f = 1.76\sqrt{d_s}$$

where ds is the average size (not radius) of particle (mm).

II. Afflux is the rise in water level on u/s of structure after the construction of the weir. The high flood level on the u/s is higher than that at the d/s. The area of submergence and the top levels of the marginal banks and guide banks depend upon the afflux. The location of the hydraulic jump on the d/s glacis also depends upon u/s TEL which is a function of afflux. If the afflux is very large, the length of the weir will be small because of high discharge intensity (q) over the crest. However, the cost of the river training works (guide banks, marginal bunds, etc) will increase. The risk of the failure of the structure due to outflanking will also increase.

Further, the scour depth will be large and it will increase the cost of protection works on the u/s and d/s of the impervious floor. The afflux is usually limited to 1 m. IS: 6933-1973 recommends an afflux of 1 m for the alluvial rivers in the upper and middle reaches of the river and of 0.3 m in the lower reaches.

III. The design flood discharge (Q) will be able to pass over the crest without exceeding the afflux. IS: 6966-1973 recommends the discharge formula:

$$Q = C_d L_e H_e^{3/2}$$

where Cd is the coefficient of discharge which depends upon the type of crest,
Le is the effective length, and
He is the head over the crest, including the head due to velocity of approach.
For sharp crested weir (top width < 2 He/3) Cd = 1.84 and for broad crested weir (top width > 2 He/3) Cd = 1.705.

IV. The effective length Le.is determined as:

$$L_e = L' - 2(NK_p + K_a)H_e$$

where L' = clear length excluding total width of piers,
N is the number of piers,
Kp is the pier contraction coefficient and
Ka is the abutment contraction coefficient.
Coefficients Kp and Ka depend on the shape of piers and abutment respectively.

There are FOUR broad design steps

1. hydraulic calculation to fix various levels
2. design for weir wall
3. design for impervious floor and piles
4. design for u/s and d/s protection

Step 1: Hydraulic calculation to fix various levels

- Length of waterway (L) = regime perimeter P. From Lacey's regime theory, $L = P = (4.5 \ to \ 6.3)\sqrt{Q}$, where Q is the design discharge.
- Discharge intensity q = Q/L
- Using Lacey's theory Normal Scour depth, $R = 1.35(q^2/f)^{1/3}$ where f is Lacey's silt factor
- Determine the regime velocity of flow V = q/R and then the velocity head $h_a = V^2/2g$.
- d/s TEL = HFL before construction + h_a
- u/s TEL = d/s TEL + afflux
- u/s HFL. = u/s TEL - ha
- Determine the head required over the crest (He) for passing the design intensity q, assuming that the weir acts as a broad or sharp crested weir.

$$q = 1.705(H_e)^{3/2} \; or \qquad q = 1.84(H_e)^{3/2}$$

- Crest level = u/s TEL - He (If the crest level is lower, afflux will be less because the head over the crest is increased and consequently, the discharge intensity is also increased. However, a low crest gives rise to an increased depth of water over the crest upto the pond level. It results in the increased height of gates, thickness of floor and the overall cost of the structure).
- Pond level = FSL of off taking canal + Head loss through head regulator (The head loss through the head regulator is usually taken between 0.5 m to 1.0 m, depending upon the type of regulator). The full supply level (FSL) of the canal depends upon a number of factors, such as the water requirements of crops, the topography, the head loss in the canal system and the cumulative fall in the water surface levels from the head to the tail of the canal.
- Height of shutters (s) = Pond level - Crest level

Step 2 : Design of Weir Wall

- The top level of the weir wall is kept at the required crest level therefore Height ofWeir H = Crest level - Bed level of river bed.
- Top width (a) and Base width (B) of weir can be estimated considering elementary profile of a gravity dam. The top width of the weir wall is fixed as the largest of the three values

$$a = \frac{d}{\sqrt{S_c}}; \qquad \frac{d}{\mu S_c}; \quad s\,(height\ of\ shutters) + 1$$

where d = water depth above crest = u/s HFL level - Crest level

= He - ha ;

Sc = specific gravity of weir wall material;

μ = friction factor.

The top width should be sufficient so that when the shutter is laid over it during floods, it does not project beyond the wall.

- Bottom/Base width (B) should be sufficient so that the maximum compressive stresses are within the allowable limits and the tension does not develop. For preliminary design, the base width may be taken as the largest of the three values

$$B = \frac{H+d}{\sqrt{S_c}}; \qquad \frac{H+d}{\mu S_c}; \qquad a + 0.8H$$

The last criteria is based on the slopes on u/s (1:0.3) and d/s (1:0.5) faces.

- Stability analysis assuming weir a gravity dam should be performed under different conditions (eg No flow, High flood, Normal water at pond level etc) to determine the developed stresses, those must be within permissible values.

Step 3: Design of Impervious Floor and Piles

- Seepage head is the difference between water levels u/s and d/s of weir due to which seepage takes place. During high flood the head difference is equal to Afflux. During lean period gates are raised and all the river water is diverted into canal so water level is at pond level at u/s and no tail water at d/s and hence seepage head is equal to pond level minus river bed level. Normally the worst condition occurs during lean period. Thus the maximum seepage head (Hs) = Max. of (Afflux, Pond level - river bed level) or Hs = Pond level - river bed level = H + s.

- Total length of impervious floor LT = C Hs from Bligh's theory where C = Bligh's creep coefficient.
- Depths of u/s and d/s piles are fixed based upon the maximum scour depth, which is 1.25 R to 1.5 R for u/s pile and 1.5 R to 2 R for d/s pile. Thus d/s Max scour depth = d/s HFL – 2R and u/s Max scour depth = u/s HFL – 1.5R . Therefore d2 = 2R – (d/s HFL - d/s bed level) and d1 = 1.5R – (u/s HFL - u/s bed level). where d1 and d2 are the depths of the u/s and d/s piles below the bed levels, respectively.
- The length of the horizontal floor (b) = $l_u + B + l_d = LT - 2d_1. - 2d_2$ where lu = length of impervious floor u/s of weir and l_d= length of impervious floor d/s of weir.
- Certain minimum length of impervious floor d/s of weir is always required to dissipate energy and avoid scouring, which is from Bligh's consideration.

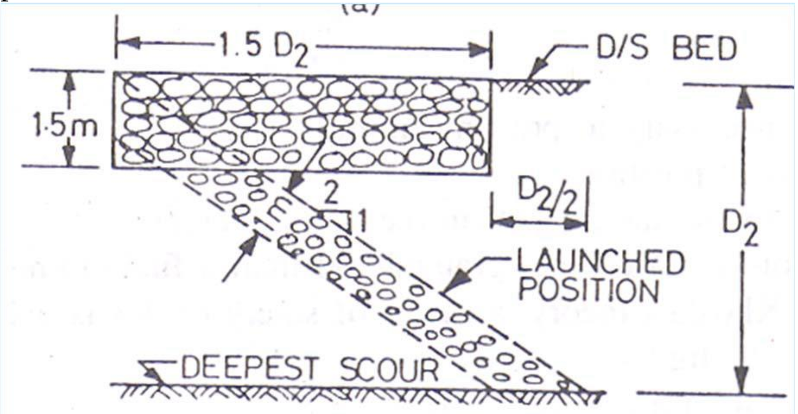
$$l_d = 2.21C\sqrt{H_s/13}$$

- Length of u/s impervious floor lu = $L_T - 2d_1 - 2d_2 - B - l_d$ if it is excessive provide Intermediate pile of depth $d_3 > d_1$ &d_2
- Thickness of u/s floor: The upstream floor is provided with a nominal thickness of about 0.6 m to 1.0 m, as the net uplift force is zero on the u/s floor.
- The thickness of the d/s floor in ld length is determined by computing uplift pressures at selected points. The section just d/s of weir is critical where maximum thickness has to be provided. In the remaining part the thickness may be provided in suitable number of steps. Bligh's theory may be used to determine uplift pressures at selected points.
- Uplift pressure head just d/s of weir $P_{uH1} = (2d_2 + l_d)H_s/L_T$ and just before d/s pile $P_{uH2} = (2d_2)H_s/L_T$ and hence thicknesses at these points are $t_1 = 1.333P_{uH1}/(S_c - 1)$ and $t_2 = 1.333P_{uH2}/(S_c - 1)$ respectively. The thickness at 3 to 5 points is generally found; depending upon the length of d/s floor. The thickness of floor from the weir wall to d/s end is reduced in steps for ease in construction.
- Use Khosla's method to determine uplift pressures and corresponding thicknesses and also exit gradient for tentative dimensions fixed using Bligh's theory. The final dimensions of the impervious floor and piles must not be unsafe as well over safe.

Step 4: Design of d/s and u/s Protection Works

- An inverted filter is provided immediately d/s of d/s impervious floor beyond the d/s pile to relieve the pressure along with filtering out foundation material so that washing out of fine particles does not occur. The filter is properly graded, with the finer layer at the bottom. The total thickness of filter is usually between 50 to 75 cm. The length of the inverted filter is generally kept equal to 1.5 d2 to 2 d2.
- To prevent the damage and dislocation of the inverted filter due to surface flow and counteract uplift, it is generally loaded with concrete blocks or block stones of size 90 to 120 cm cube, generally 1 m × 1 m × 1 m. The joints between the concrete blocks are 10 cm thick filled with sand or bajri.
- On the d/s on the inverted filter, a launching apron of length 1.5 d_2 to 2.5 d_2 is provided. It consists of loosely packed stones. The apron is initially laid horizontal at the river bed level but when scouring occurs, it settles and takes an inclined position. The launching apron protects the impervious floor, d/s pile and inverted filter, as it forms a protective

covering of stones over a certain slope below the river bed. It is generally assumed that the aprons launch at a slope of 2:1 to 3:1. The thickness of the apron in the launched position is usually specified as 0.9 m to 1.0 m. The thickness of the apron in the horizontal position can be found from the volume of stone in the launched position. 'For example. for slope of 3: 1 and the launched thickness of 1 m, the thickness in horizontal position = $\sqrt{10}d_2 / 3d_2$



(b)

- U/s protection works Concrete blocks The concrete blocks of thickness 90 to 120 cm are laid over gravel on the upstream of the u/s impervious floor for a length = d_1 to 1.5 d_1 .
- U/s Launching apron The horizontal length of the u/s launching apron is usually kept = 1.5 d_l to 2d_l the thickness is determined as for d/s launching apron.