VI. RIVER DIVERSION HEAD WORKS

6.1 Introduction

River diversion headwork is constructed at the head of the canal to divert the river water towards the canal, so as to ensure a regulated continuous supply of silt-free water with a certain minimum head into the canal. It usually provides a small storage capacity.

6.2 Purposes of diversion headwork

(i) It raises the water level in the river so that the commanded area is increased

(ii) It regulates the supply of water into the canal

(iii) It provides storage of water for a short period

(iv) It controls the entry of silt into the canal

(v) It reduces the fluctuations in the level of supply in the river.

6.3 Selection of actual site for canal head works

The selection may be made in accordance with the following considerations.

i) As far as possible a narrow, straight, well defined channel confined b/n banks not submerged by the highest flood;
ii) It should be possible to align the offtaking canal in such a way that the command of its area is obtained without excessive digging.
iii) The material of construction such as stone, sand, etc. should be available in the vicinity of the site.
iv) The site should be accessible by road. And there should be (enough) workers available in the vicinity of project site.

6.4. Components Of Diversion Head Works

The components of diversion head works consists of:

(1) Weir or barrage
(2) Divide wall
(3) Fish ladder
(4) Pocket or approach channel
(5) Undersluices or scouring sluices
(6) Silt excluder
(7) Canal head regulator
(8) River training works, such as marginal bunds, guide banks

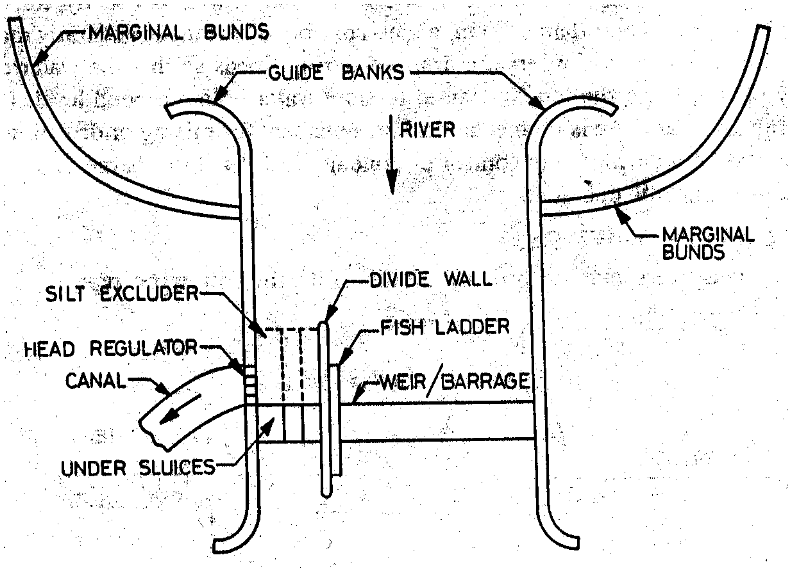


Figure 6.1. Typical layout of diversion headworks

6.4.1. Weirs and Barrages

Weirs and barrages are permanent river diversion works and are relatively low dams constructed across a river to raise the river level sufficiently to divert the flow in full, or in part, into a supply canal or conduit for the purpose of irrigation, power generation, domestic and industrial uses, etc.

Weirs are with or without gates, whereas barrages are always gate controlled.

6.4.1.1. Weirs

Weirs may be classified according to the material of construction and certain design features as

1) Masonry weirs with vertical drop or vertical drop weirs
2) Rockfill weirs with sloping aprons
3) Concrete weirs with a downstream glacis

1) Masonry Weir (Vertical Drop Weir): Consists of:

- An impervious horizontal floor or apron
- A masonry weir wall (with both upstream and downstream faces vertical; or both faces inclined; or upstream face vertical and downstream face inclined)
- Block protection at upstream end of floor, and a graded inverted filter at the downstream end of floor
- Launching aprons or pervious aprons (or floors) after block protection and inverted filters.

This type of weir is suitable for any type of foundation.

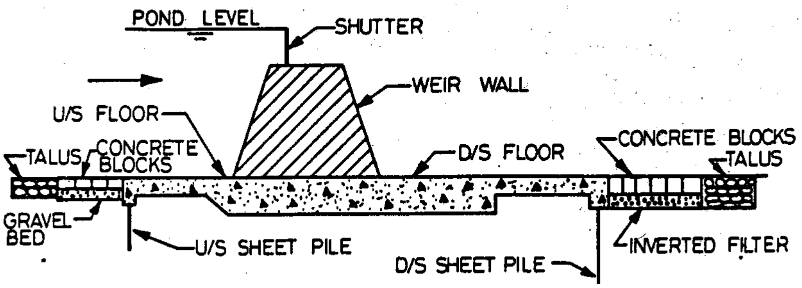


Figure 6.2 Vertical drop weir

2) **Rockfill Weir With Sloping Aprons**: It is the simplest type of construction and Consists of:

- Masonry weir wall
- Dry packed boulders laid in the form of glacis or sloping aprons in the upstream and downstream sides of the weir wall

The downstream slope is generally made very flat. It requires a very large quantity of stone. It also has few intervening core walls.

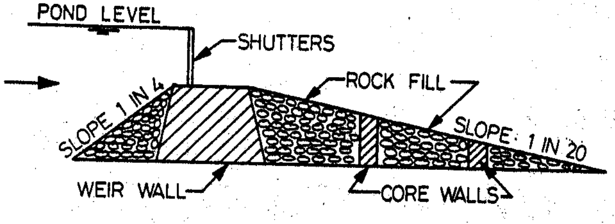


Figure 6.3 Rockfill weir

3) **Concrete weir with downstream glacis**: It is of recent origin and its design is based on sub-surface flow concept. Hydraulic jump is developed on the glacis due to which considerable energy is dissipated. Protection works such as inverted filter; block protection and launching apron are provided. May be constructed on pervious foundation. Sheet piles of sufficient depths are provided both at upstream and downstream ends of the floor.

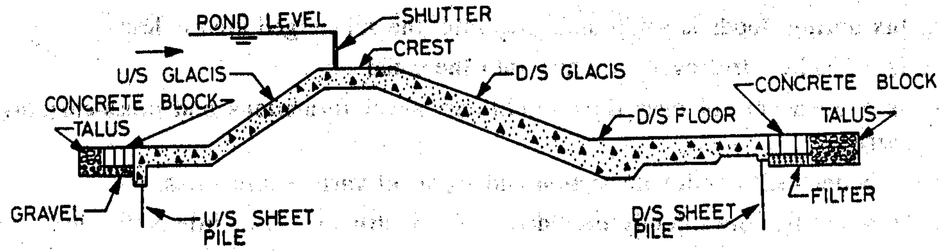


Figure 6.4 Typical cross section of concrete weir with downstream glacis on permeable foundation

6.4.1.2. Barrages

The crest level is kept at a low level and the raising up of water level (or pording) is accomplished mainly by means of gates. During floods these gates can be raised clear off the high flood level and thus enable the high flood to pass with minimum of afflux (or heading up of water on the upstream side). A barrage provides better control on the water level in the river but it is comparatively more costly. The design of a barrage involves the same procedure as a concrete weir.

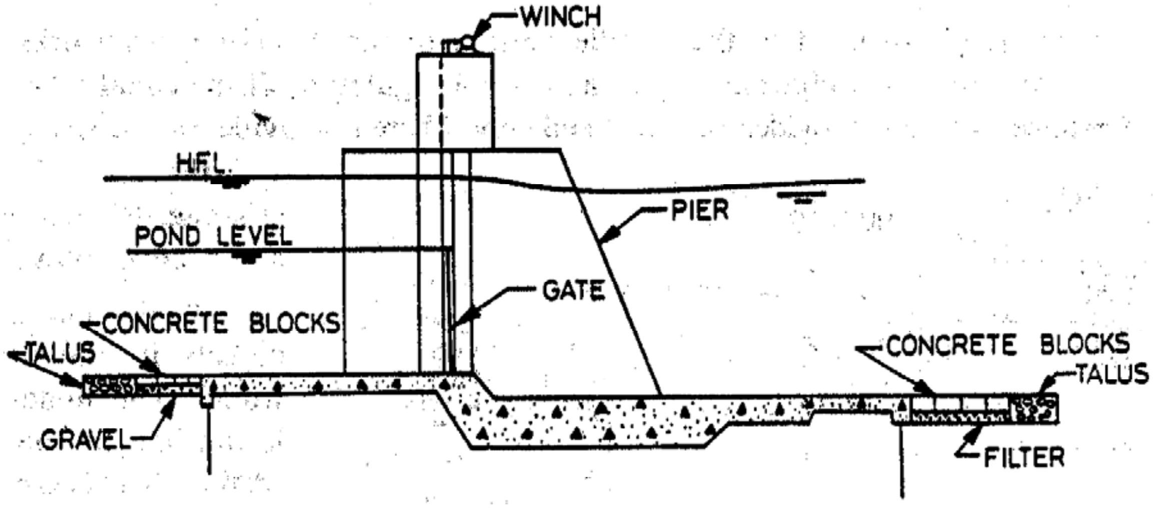


Figure 6.5 Typical cross section of a barrage on pervious foundation

6.4.2. Divide wall

It is masonry or concrete wall with top width of 1.5 to 3m constructed at right angles to the axis of the weir and separates the 'weir proper' from under sluices. The divide wall extends on the upstream side beyond the beginning of the canal head regulator and on the downstream side, it extends up to the end of downstream protection of the under sluices.

The main functions of a divide wall are:

a) To separate the floor of the under sluices which is at lower level from the weir proper;
b) To help in providing a comparatively less turbulent pocket near the canal head regulator resulting in deposition of silt in this pocket and, thus, to help entry of silt-free water into the canal;
c) To isolate the pocket upstream of the canal head regulator and facilitate scouring operation;
d) To prevent formations of cross-currents to avoid their damaging effects on the weir.

6.4.3. Fish Ladder

This structure enables the fish to pass upstream. It is device by which the flow energy can be dissipated in such a manner as to provide smooth flow at sufficiently low velocity, not exceeding 3 to 3.5m/s. This object is generally accomplished by providing a narrow opening adjacent to the divide wall and provide suitable baffles or staggering devices in it, so as to control the flow velocity.

The various types of fish ladder are (i) pool type, (ii) steep channel type, (iii) fish lock type and (iv) fish lift or elevator type. Types (iii) and (iv) are suitable for high dams only. Types (i) and (ii) are generally provided for barrages.

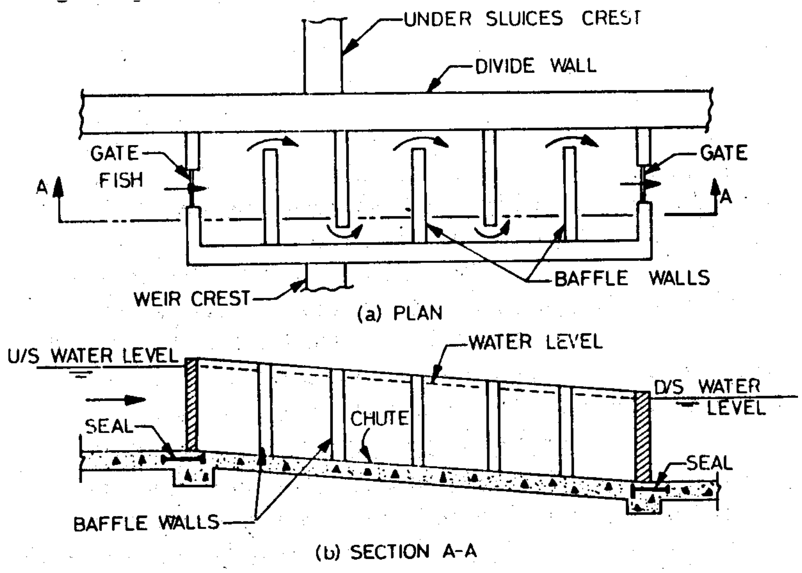


Figure Fish Ladder

6.4.4. Canal head regulator

It is provided at the head of the offtaking canal and serves the following functions:

(a) It regulates the supply of water entering in the canal;
(b) It controls the entry of silt in the canal;
(c) It prevents the river floods from entering the canal.

The head regulator is normally aligned between 90^0 and 120^0 to the axis of the weir. The regulation is done by means of gates.

The maximum height of gated opening is determined by the differences in crest level of regulator (sill level) and the pond level. During high floods, the water level in the river will be much higher than the pond level. To avoid spilling of this water over the gates, a R.C. wall, called Breast wall, is provided from pond level up-to river HFL. This wall rests over the piers of the regulator bays.

The entry of silt into the canal is controlled by keeping the crest of the head regulator by about 1 to 1.5m higher than the crest of the under sluices.

Head regulators are generally provided with a very wide and shallow waterway and drowned weir formula given below is used to calculate the discharge (see Fig)

$$Q = \frac{2}{3} C_{d1} \sqrt{2g} B \left[(h + h_a)^{3/2} - h_a^{3/2} \right] + C_{d2} B h_1 \sqrt{2g(h + h_a)}$$

Where $Cd_1 = 0.577$ and $Cd_2 = 0.80$
h = difference between upstream and downstream water levels, i.e.
(Pond level – Maximum FSL of canal)
h_a = head due to velocity of approach
B = clear width of water way
h_1 = depth of downstream water level above the crest.

When all other variables are fixed and known, value of B can be calculated.

The width of the waterway (B) calculated above; generally works out to be more than the normal width of the canal downstream. In such a case, the sill level may be lowered, so as to increase the head and to decrease the waterway to make it equal to the width of the canal. But the sill level is also governed by silt exclusion considerations, and therefore, many times it may not be possible to lower the sill level. In such a case, the calculated value of waterway is provided and the normal required width of the canal is obtained by contracting the wings. The length and thickness of horizontal floor, glacis protection aprons, etc. is designed on the same principles as are applicable to weir design.

6.4.5. Protection Works

The concrete floor of a weir or barrage is protected on the upstream as well as downstream by loose apron. In the immediate vicinity of the floor, a certain portion of the loose apron is made non-launching. The non-launching apron prevents the scour hole travel close to the floor or sheet pile line; whereas launching apron is designed to launch along the slope of the scour hole to prevent further scooping out of the underlying river bed material.

6.5. Designs Of Weirs And Barrages

6.5.1 Causes Of Failures of Weirs on Permeable Foundation

Causes of failures of weirs on permeable foundations may be classified into two broad categories.

(1) Due to seepage or subsurface flow
(2) Due to surface flow

The seepage may cause the failure of a weir in two ways.

i) **By piping or undermining:** If the water percolating through the foundation has sufficient force when it emerges at the downstream end of the impervious floor it may lift up the soil particles at the end of the floor. With the removal of the surface soil there is further concentration of flow in to the resulting depression and more soil is removed which progressively result in subsidence of the floor in the hollows so formed.

- **To prevent three kinds of failures:**
    - (a) Provide sufficient length of the impervious floor (so that the path of percolation is increased) and reduce exit gradient.
    - (b) Provide piles at upstream and downstream ends of the impervious floor

(ii) **By uplift pressure:** If the uplift pressure is not counterbalanced by the weight of the floor, it may fail by rupture.

- **To prevent failure by uplift:**

    - (a) Provide sufficient thickness of the impervious floor
    - (b) Provide pile at the upstream end of the impervious floor so that uplift pressure is reduced on the downstream side.

The Surface Flow may cause the failure of a weir in the following two ways:

(i) By suction due to standing wave or hydraulic jump: The standing wave or hydraulic jump developed on the downstream side of the weir causes suction or negative pressure which also acts in the direction of uplift pressure. If the floor thickness is insufficient it may fail by rupture in suction.

The following measures may be taken to prevent such kind of failure:

- (a) Providing additional thickness of the impervious floor to counterbalance the suction pressure due to standing wave.
- (b) Constructing floor as monolithic concrete mass instead of in different layers of masonry.

(ii) By scour on the upstream and downstream of the weir: Upstream and downstream ends of the impervious floor and bed of the river may be scoured during floods. If not prevented, lead to damage to the floor and an ultimately failure.

Preventive measures which should be taken against failure due to scour are:

- (a) Providing deep piles both at upstream and downstream ends of the impervious floor. The piles should be driven much below the calculated scour depth.
- (b) Providing launching aprons of suitable length and thickness at upstream and downstream ends of the impervious floor.

6.5.2. Design Of Impervious Floor For Subsurface Flow

6.5.2.1. Bligh's Creep Theory

Bligh assumed that the percolating water creeps along the base profile of the structure, which is in contact with the subsoil. The length of the path thus traversed by the percolating water is called the creep length. Bligh also assumed that the head loss per unit length of creep (called hydraulic gradient) is proportional to the distance of the point from the upstream of the foundation (constant).

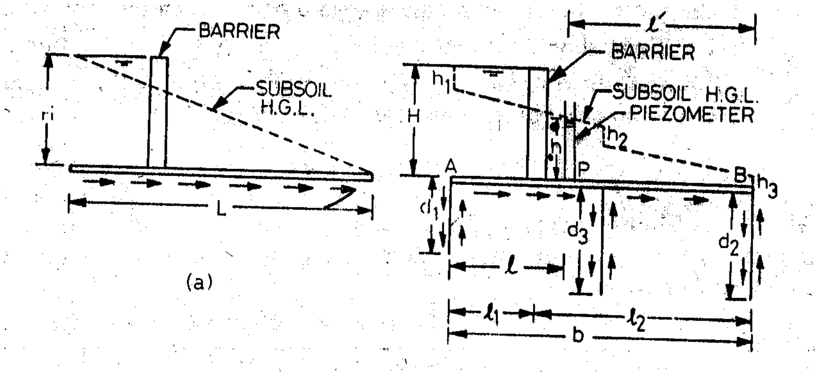


Figure 6.6a Bligh’s creep theory – Definition sketch

Shortcoming of this theory is that it does not discriminate between the horizontal and vertical creeps in estimating the exit hydraulic gradient.

Creep length, L, is given by

$$\mathbf{L = b + 2d_1 + 2d_2 + 2d_3}$$

The hydraulic gradient or the loss of head per unit length of creep is,

$$\frac{H}{L} = \frac{H}{b + 2d_1 + 2d_2 + 2d_3}$$

Therefore, for any point the head loss is proportional to the creep length.

As the hydraulic gradient is constant, if L_1 is the creep length up to any point, then head loss up to this point will be (H/L) L_1 and the residual head at this point will be (H - (H/L) L_1).

The head losses at the three cutoffs will be (e.g.)
(H/L) $2d_1$, (H/L) $2d_2$ and (H/L) $2d_3$

The reciprocal of the hydraulic gradient, i.e., L/H is known as Bligh’s coefficient of creep, C.

According to Bligh

(a) **Safety against piping:** The creep length should be sufficient to provide a safe hydraulic gradient according to the type of soil.

$$L = CH$$

Bligh recommended certain values of C for different soils. According to Bligh if the hydraulic gradient (H/L = 1/c) $\leq \frac{1}{c}$ (for the soil) there is no danger of piping.

Table 6.1 Recommended values of Bligh coefficient of creep C and safe hydraulic gradient

Type of soil	Value of C	Safe Hydraulic Gradient
Light sand &mud (River Nile)	18	1/18
Fine Micaceous sand	15	1/15
Coarse grained sand	12	1/12
Sand mixed with boulder and gravel; and for loam soil	5 to 9	1/9 to1/5

(b) Safety against uplift pressure

The ordinate of the subsoil hydraulic gradient line above the bottoms of the floor at any point represents the residual seepage head or the uplift pressure at that point.

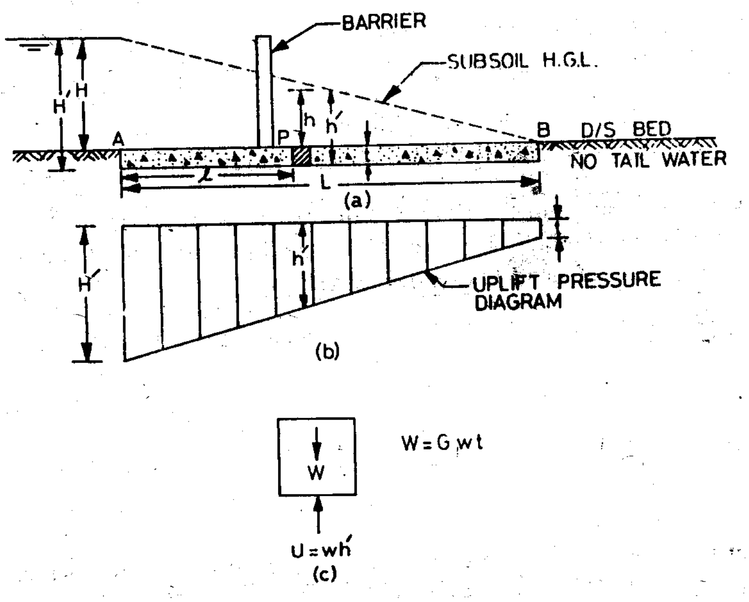


Figure 6.6b

If h´ is the uplift pressure head at a point under the floor, the pressure intensity is,

$$P = \rho g h'$$

This is to be resisted by the weight of the floor, the thickness of which is t and density ρ_m (for concrete, $\ell m = 2400$ kg/m^3). Downward force per unit area due to the weight of the floor is

$$W = \rho_m g\, t$$

Therefore, equating

$$\rho_m\, g\, t = \rho g h'$$

which gives $h' = \frac{\rho_m}{\rho} t = S_m t$

where S_m is the relative density of the floor material. Thus, we can write,

$$h' - t = S_m t - t$$

which gives the thickness of the floor, $t = \frac{h' - t}{S_m - 1} = \frac{h}{S_m - 1}$

where h is the pressure head (ordinate of hydraulic gradient) measured above the top of floor, and (S_m-1) is submerged specific gravity of the floor material.

Considering a safety factor of 4/3 to 3/2

$$t = \frac{4}{3}\frac{h}{S_m - 1} \quad \text{to} \quad \frac{3}{2}\frac{h}{S_m - 1}$$

with S_m= 2.24, $t \approx 1.08\,h$ to $1.2\,h$

The design will be economical if the greater part of the creep length (i.e. of the impervious floor) is provided upstream of the weir where nominal floor thickness would be sufficient. The downstream floor has to be thicker to resist the uplift pressure. However, a minimum floor length is always required to be provided on the downstream side from the consideration of surface flow to resist the action of fast flowing water whenever it is passed to the downstream side of the weir

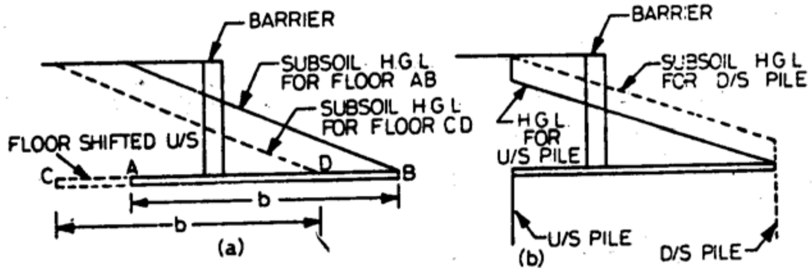


Figure 6.7

Moreover, the provision of maximum creep length on the upstream side of the weir (barrier) also reduces uplift pressures on the portion of the floor provided on the downstream side of the barrier (Fig 6.7a). This is because a large portion of the total creep having taken place up to the barrier; the residual heads on the downstream floor are reduced. Further, (see Fig 6.7b) a vertical cutoff at the upstream end of the floor reduces uplift all over the floor. Thus, according to Bligh's theory a vertical cutoff at the upstream end of the floor is more useful than the one at the downstream end of the floor.

Limitations of Bligh's Theory

1. Bligh made no distinction between horizontal and vertical creep.
2. The theory holds good as long as horizontal distance between cut-offs or pile lines is greater than twice their depth.
3. No distinction is made between the effectiveness of the outer and inner faces of sheet piles and short and long intermediate piles. However, investigations, later, have shown that the outer faces of the end piles are much more effective than the inner ones. Also intermediate piles of shorter length than the outer ones are ineffective except for local redistribution of pressure.
4. No indication on the significance of exit gradient. Average value of hydraulic gradient gives idea about safety against piping. Exit gradient must be less than critical exit gradient (for safety).
5. The assumption, loss of head is proportional to creep length is not true and actual uplift pressure distribution is not linear, but it follows a sine curve.

6. Bligh did not specify the absolute necessity of providing a cutoff at the downstream end of the floor, whereas it is absolutely essential to provide a deep vertical cutoff at the downstream end of the floor to prevent undermining.

6.5.2.2. Lane's Weighted Creep Theory

Lane made distinction between vertical and horizontal creep. He indicated that the horizontal creep is less effective in reducing uplift (or in causing head loss) than the vertical creep. He, therefore, used a weightage factor of (1/3) for the horizontal creep. Thus, the weighted creep length, L_w, is given by

$$L_w = \frac{1}{3}N + V$$

Where N = sum of all the horizontal contacts and all the sloping contacts less than 45^0 to the horizontal.

V = sum of all the vertical contacts and all sloping contacts greater than 45^0 to the horizontal.

To ensure safety against piping $L_w > C_1H$

Where H = Total seepage head (difference in water head between upstream and downstream)

C_1 = Lane's coefficient (empirical) of creep

Further if the hydraulic gradient $\left(\frac{L}{L_w}\right) \leq \left(\frac{1}{C_1}\right)$ safety against piping can be ensured.

Table 6.2. Recommended values of Lane's coefficient of creep C_1 and safe hydraulic Gradient.

Type of Soil (Material)	**Value of C_1**	**Safe Hydraulic Gradient $\left(\frac{1}{C_1}\right)$**
Very fire sand or silt	8.5	$1/8.5$
Fire sand	7.0	$1/7.0$
Medium sand	6.0	$1/6$
Coarse sand	5.0	$1/5$
Fire Gravel	3.5	$1/3.5$
Medium Gravel	3.0	$1/3$
Gravel & Sand	3.5 to 3.0	$1/3.5$ to $1/3$
Coarse gravel including cobbles	3.0	$1/3$
Boulders, with some cobble & gravel	2.5	$1/2.5$

Soft clay	3.0	$\frac{1}{3}$
Medium clay	2.0	$\frac{1}{2}$
Hard clay	1.8	$\frac{1}{1.8}$
Very hard clay or Hard pan	1.6	$\frac{1}{1.6}$
Clayey Soil	3.0 – 1.0	$\frac{1}{3}$ *to* $\frac{1}{1.6}$

Lane's method for determination of the uplift pressure is criticized on the grounds that it is an empirical method and not based on any mathematical approach. However, because of the simplicity of the method it is also widely used.

6.5.2.3. Theory of Seepage Flow: Darcy's low holds good and Laplace equation can be used in the soil and results in flow net. From the flow net one can determine quantity of seepage, hydraulic gradient, and uplift pressure.

6.5.2.4. Khosla's Theory of Independent Variables

The application of the previous three methods is limited to foundation of regular geometry. When the base of the foundation is on different levels, several cut-off walls are provided and / or the floor is too thick to ignore its influence on uplift pressure, the previous methods do not give reliable results or are complicated (e.g. Flow net).

Khosla understood the following points over Bligh's Theory:

- The outer faces of the end sheet piles are more effective than the inner ones and the horizontal length of the floor.
- The intermediate sheet piles if shorter than the outer ones in length are ineffective except for redistribution of pressure.
- Undermining starts at downstream end and if exit hydraulic gradient is more than the critical it results in progressive degradation of the subsoil as a result of piping.
- Reasonably deep vertical cut off at downstream end is necessary to prevent undermining.

To apply the analytical solution to any practical composite profile of a weir or a barrage, Khosla evolved the method of independent variables. In this method the base of the structure is split up into a number of simple standard forms of known analytical solutions. The standard forms proposed are:

(a) A straight horizontal floor of negligible thickness with a sheet pile at either end, i.e. at up

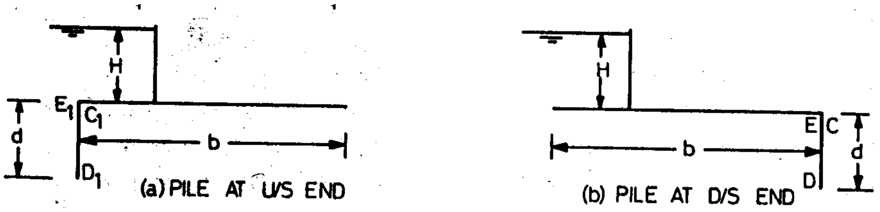


Figure 6.8a Pile at upstream end and Figure 6.8b Pile at the downstream end

(b) A straight horizontal floor depressed below the bed but with no vertical cut-off.

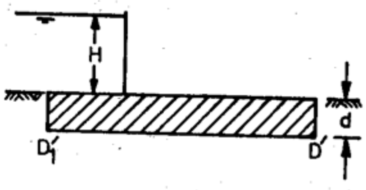


Figure 6.9 Depressed floor

(c) A straight horizontal floor of negligible thickness with a sheet pile line at some intermediate position.

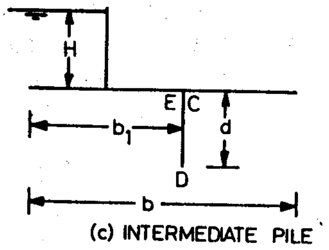


Figure 6.10 Intermediate pile

In general, the usual weir section consists of a combination of all or some of the three forms mentioned above. Each elementary form is treated as independent of the others. The pressures as a percentage of the water head are read from Khosla's curves at the key points. The key points are the junction of the floor and the pile or cut-off walls, the bottom points of the pile or walls, and the bottom corners in the case of depressed floor. The percentage pressure observed from the curves for the simple form into which the profile has been broken up, is valid for the profile as a whole if corrected for:

(i) Mutual interference;
(ii) The floor thickness; and
(iii) The slope of the floor.

i) Correction for Mutual Interference of Piles

The correction C to be applied as a percentage of head is given by

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

Where b= the distance b/n two pile lines

D= the depth of 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 pile on which the effect is to be determined.

b= total floor length.

This correction is positive for points in the rear or backwater and subtractive for points forward in the direction of flow. This equation does not apply to the effect of an outer pile on an intermediate pile, if the intermediate pile is equal to or smaller than the outer pile and is at a distance less than twice the length of the outer pile.

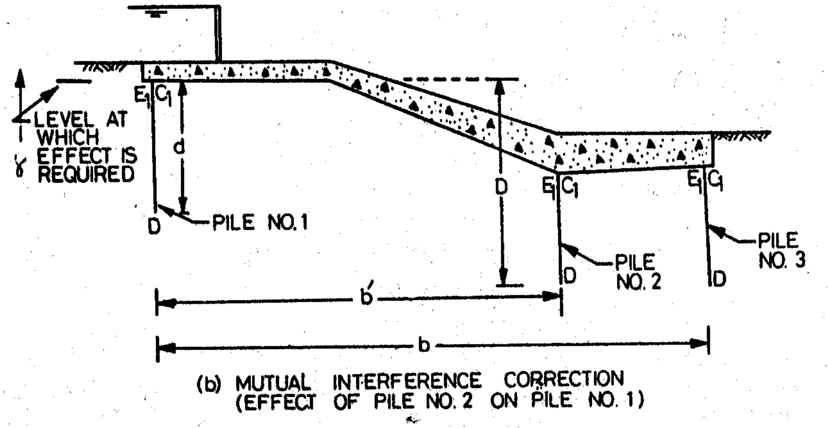


Fig.6. 11 Mutual Interference of Piles

ii) Correction for Floor Thickness

In the standard forms with cutoffs, the thickness of the floor is assumed to be negligible. Thus as observed from Khosla's curves, the percentage pressures at the junction points E and C pertain to the level at the top of the floor whereas the actual junction is with the bottom of the floor.

The percentage pressures at the actual points E and C are interpolated by assuming a straight line pressure variation from the hypothetical point E to D and also from D to C.

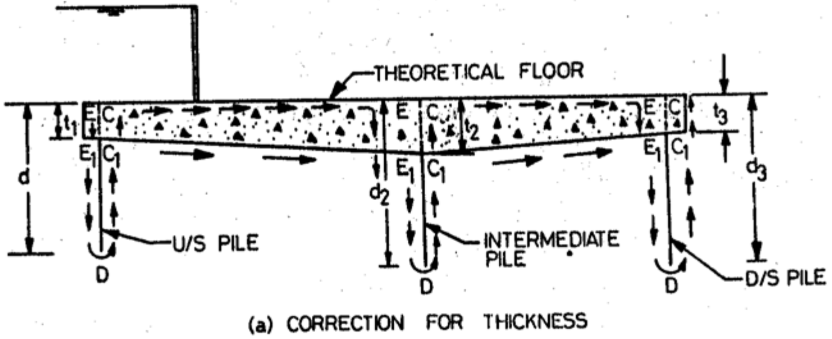


Fig. 6.12 Correction for Floor Thickness

iii) Correction for Slope of the Floor

A correction is applied for a sloping floor, and is taken as positive for the down and negative for the up slopes following the direction of flow. The values of correction for various slopes are tabulated below.

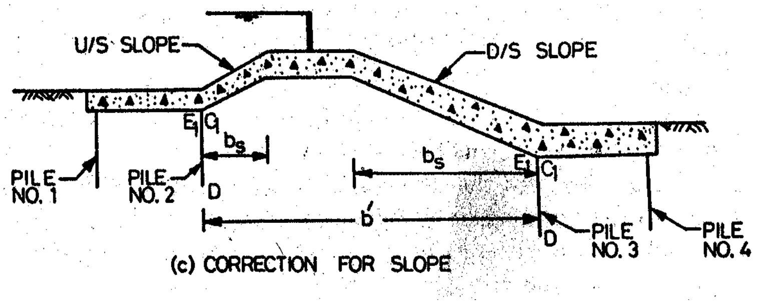


Fig. 6.13 Correction for Slope of the Floor

Table 6.3: Correction for floor slope

Slope (V. H)	**Correction (% of pressure)**
1.1	11.2
1.2	6.5
1.3	4.5
1.4	3.3
1.5	2.8
1.6	2.5
1.7	2.3
1.8	2.0

The correction 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 beginning or the ends of the slope.

iv) Exit Gradient (G_E)

For standard from consisting of a floor length b with a vertical cutoff of depth d, the exit gradient at its d end is given by:

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

Where $$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}, \textit{ and}$$

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

The exit gradient so calculated must lie within safe limits as given in the following table.

Table 6.4: Safe exit gradient for different types of soils

Type of soil	**Safe exit gradient**
Shingle	¼ to 1/5
Coarse sand	1/5 to 1/6
Fine sand	1/6 to 1/7

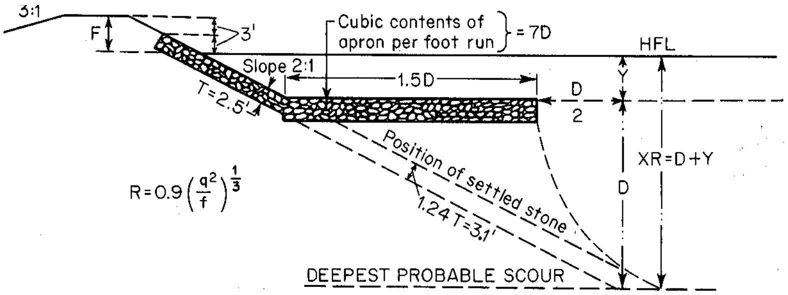


Figure 6.14- Launching apron

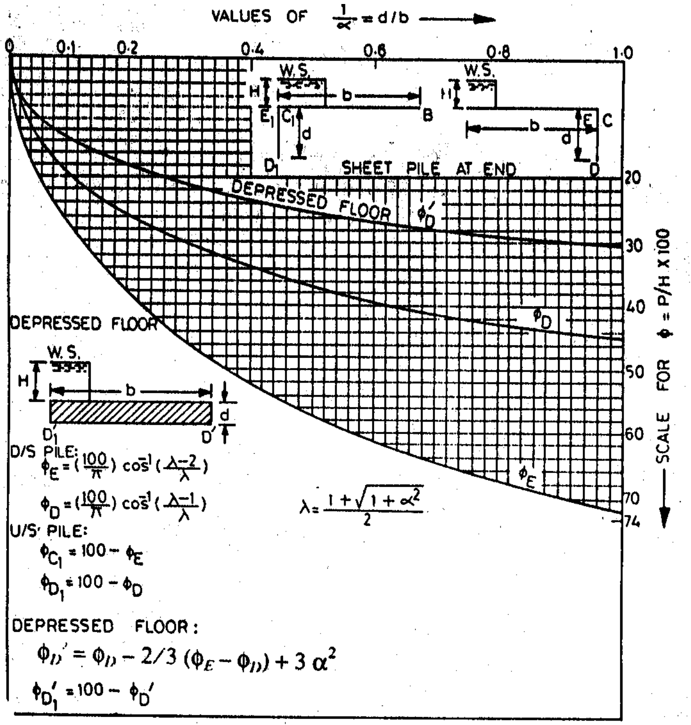
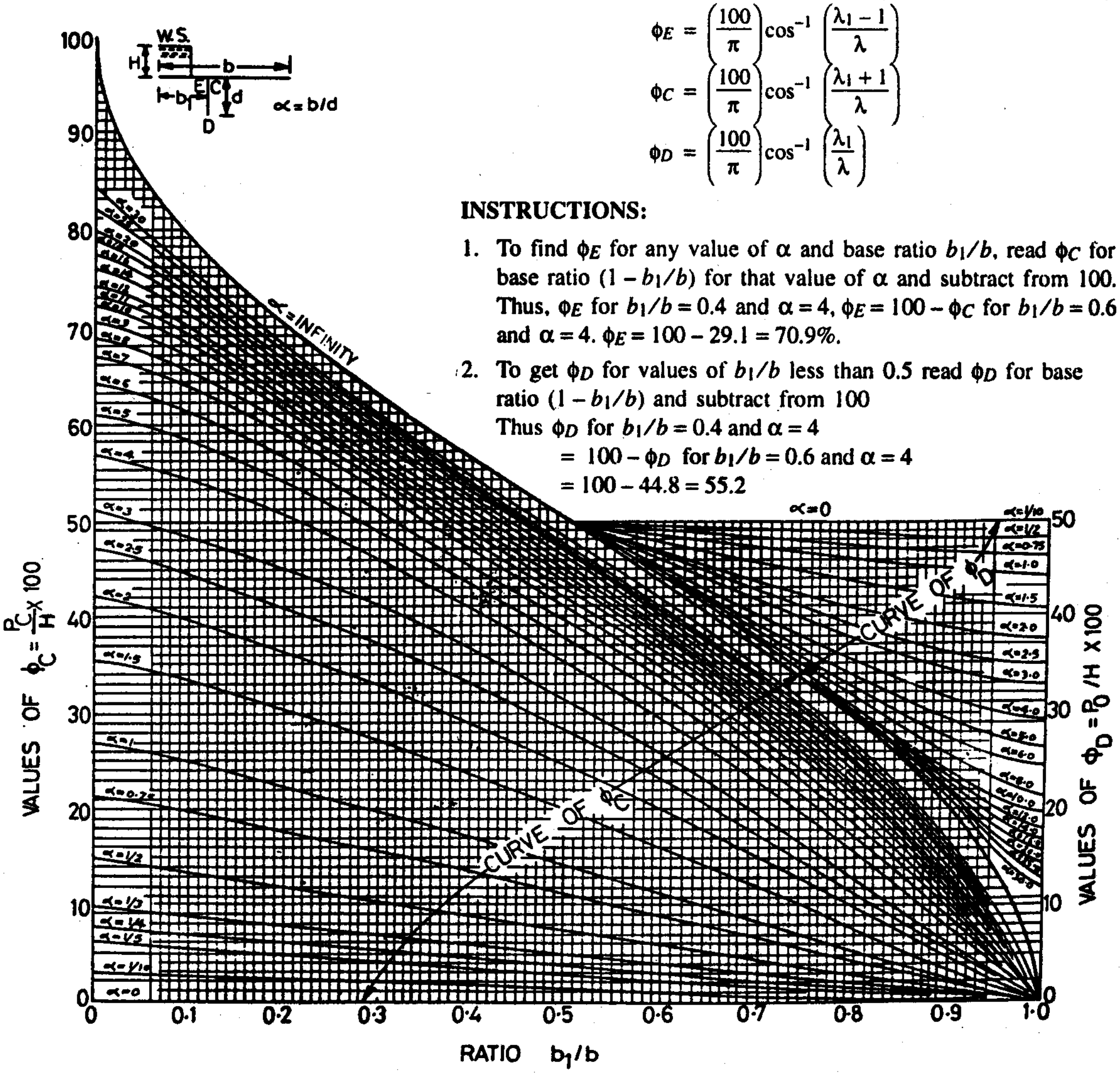


Figure A. Khosla Chart for d/s pile, u/s pile and depressed floor



SHEET PILE NOT AT END (INTERMEDIATE PILE)

$$\phi_E = \left(\frac{100}{\pi}\right)\cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right)$$

$$\phi_C = \left(\frac{100}{\pi}\right)\cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda}\right)$$

$$\phi_D = \left(\frac{100}{\pi}\right)\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$$

INSTRUCTIONS:

1. To find ϕ_E for any value of α and base ratio b_1/b, read ϕ_C for base ratio $(1 - b_1/b)$ for that value of α and subtract from 100. Thus, ϕ_E for $b_1/b = 0.4$ and $\alpha = 4$, $\phi_E = 100 - \phi_C$ for $b_1/b = 0.6$ and $\alpha = 4$. $\phi_E = 100 - 29.1 = 70.9\%$.
2. To get ϕ_D for values of b_1/b less than 0.5 read ϕ_D for base ratio $(1 - b_1/b)$ and subtract from 100
   Thus ϕ_D for $b_1/b = 0.4$ and $\alpha = 4$
   $= 100 - \phi_D$ for $b_1/b = 0.6$ and $\alpha = 4$
   $= 100 - 44.8 = 55.2$

Figure B. Khosla Chart for Intermediate Pile

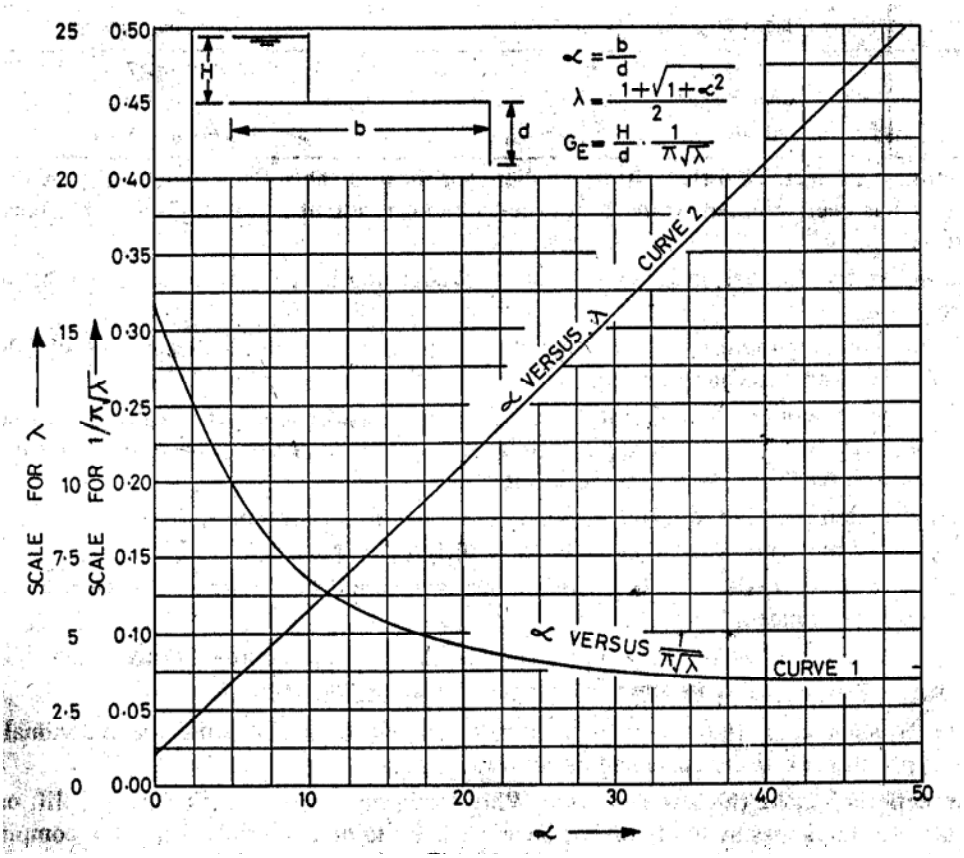


Figure C. Khosla Chart for exit gradient determination