UNIT-IV

DIVERSION HEAD WORKS

- Types of diversion head works
- Diversion and storage head works
- weirs and barrages
- Layout of diversion head works, components
- Bligh’s creep theory, Khosla’s theory, determination of uplift pressure
- Impervious floors using Bligh’s and Khosla’s theory
- Exit gradient

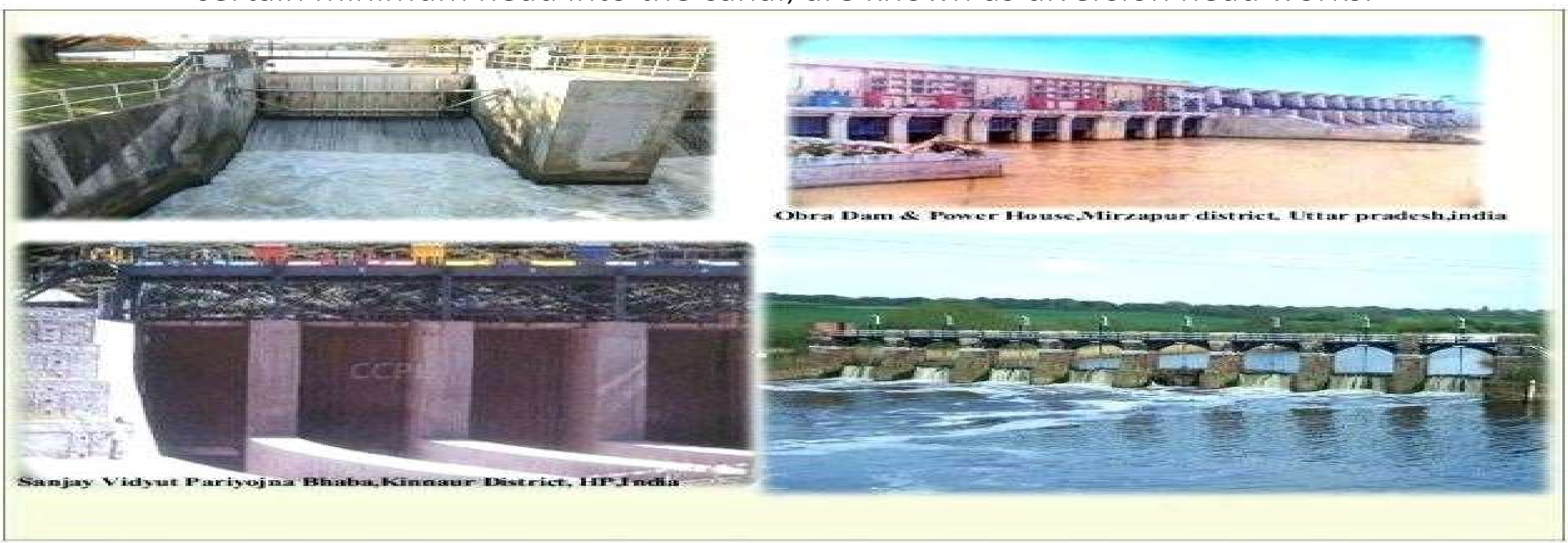
Diversion Headworks

Introduction:

Any hydraulic structure which supplies water to the off-taking canal is called a headwork.

(Or)

The works which are constructed at the head of the canal, in order 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, are known as diversion head works.



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TYPES OF HEADWORKS

Fig1: DIVERSION HEADWORK

Fig 2: STORAGE HEADWORK

Types of diversion head works

Headworks may be divided into two classes:

1. Diversion headwork
2. Storage headwork

Diversion headwork:

To divert required supply to canal from the river. They are two types.

a) Temporary spurs or bunds: which are temporary and constructed every year after floods
b) Permanent weirs and barrages

Storage headwork:

A storage headwork comprises the construction of a dam across the river. It stores water during the period of excess supplies in the river and releases it when demand overtakes available supplies.

Weir:

The weir is a solid obstruction put across the river to raise its water level and divert the water into the canal. If a weir also stores water for tiding over small periods of short supplies, it is called a storage weir.

The main difference between a storage weir and a dam is only in height and the duration for which the supply is stored.

Barrage:

The function of a barrage is similar to that of weir, but the heading up of water is effected by the gates alone. No solid obstruction is put across the river.

The crest level in the barrage is kept at a low level.

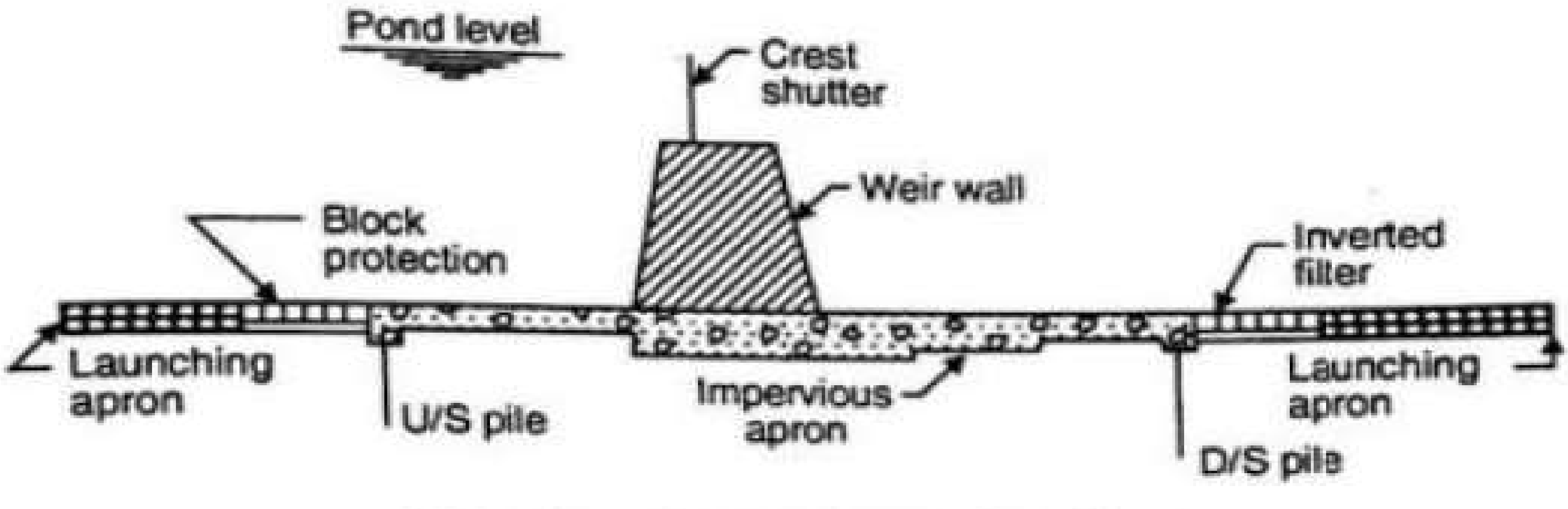


FIG. 12.1. VERTICAL DROP WEIR.

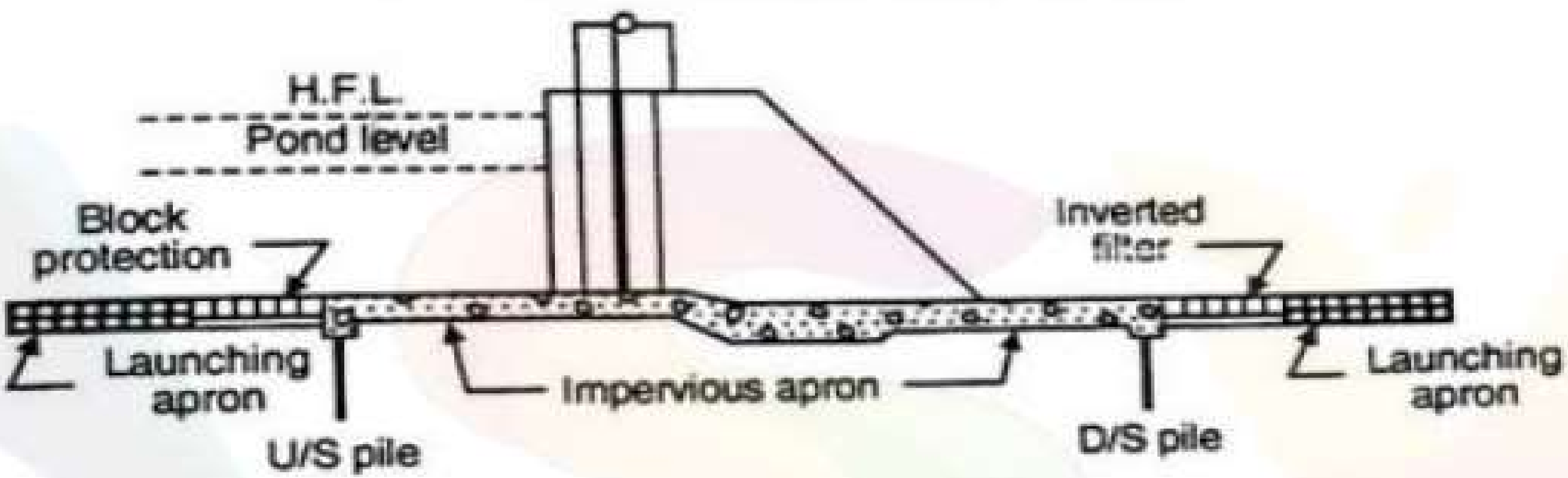
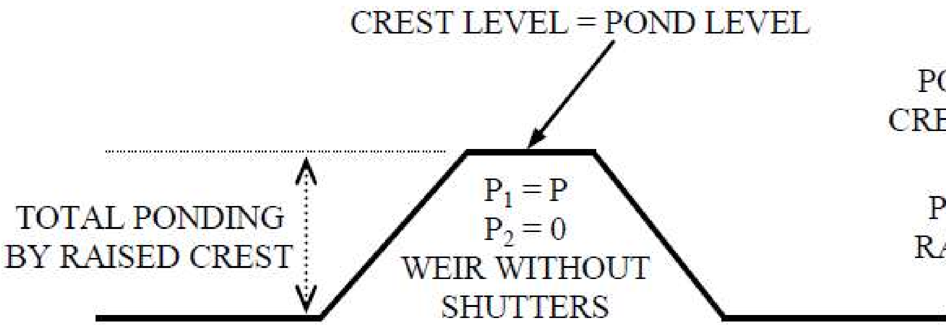
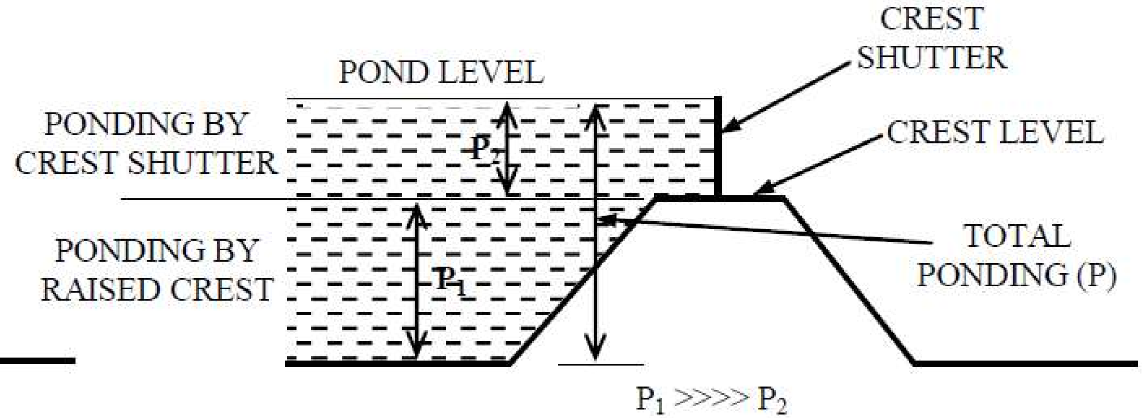


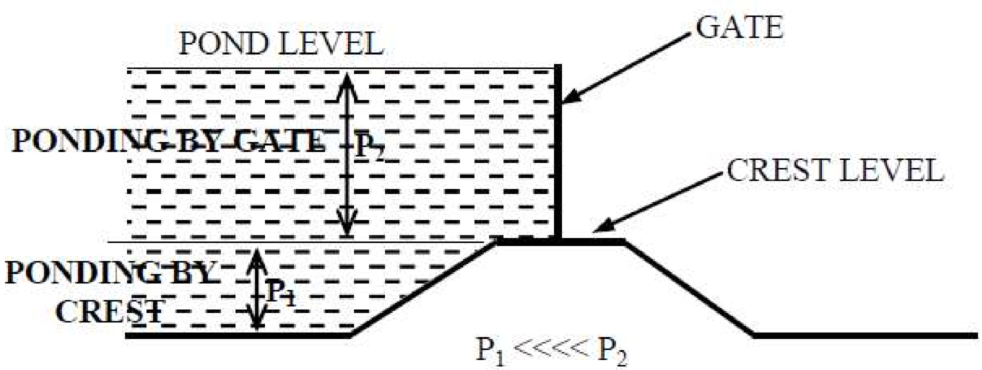
FIG. 12.2. BARRAGE.



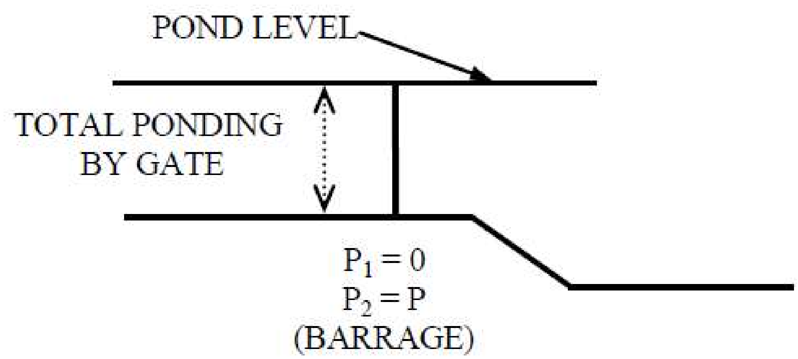
(a) Fig: Weir without shutters



(b) Fig: Weir with shutters



(c) Fig: Barrage with a small raised crest



(d) Fig: Barrage without any raised crest

4. RIVER DIVERSION HEAD WORKS

4.1 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.

4.2 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.

4.3 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.

4.4 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

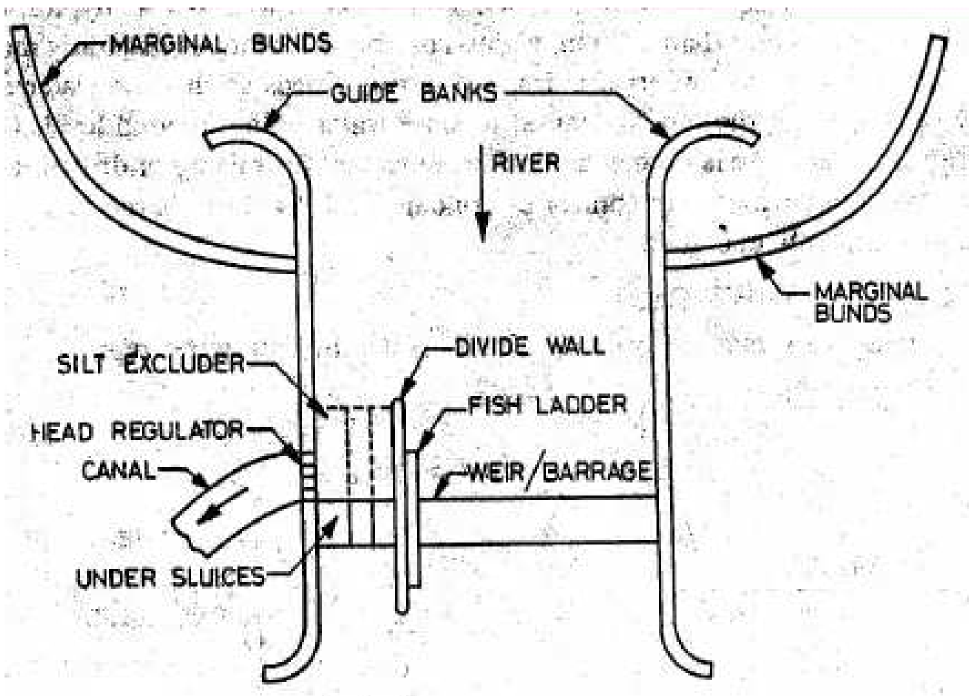


Figure 4.1. Typical layout of diversion headworks

4.4.1 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.

4.4.1.1 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.

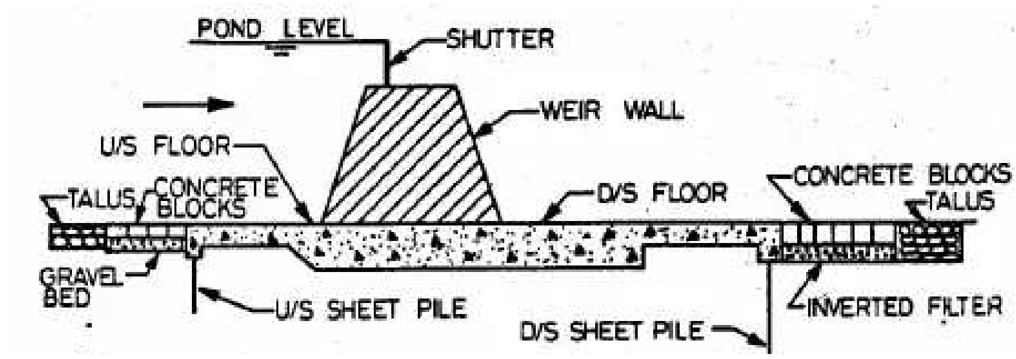


Figure 4.2 Vertical drop weir

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.

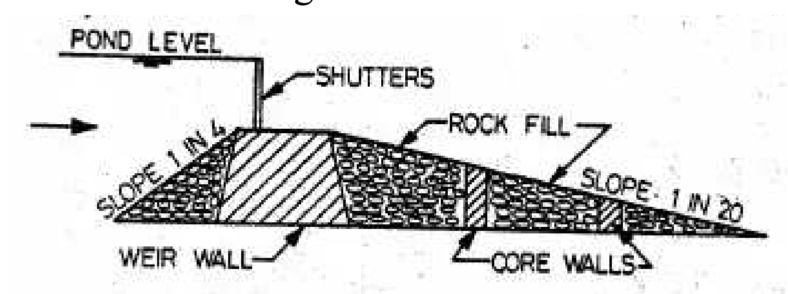


Figure 4.3 Rockfill weir

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.

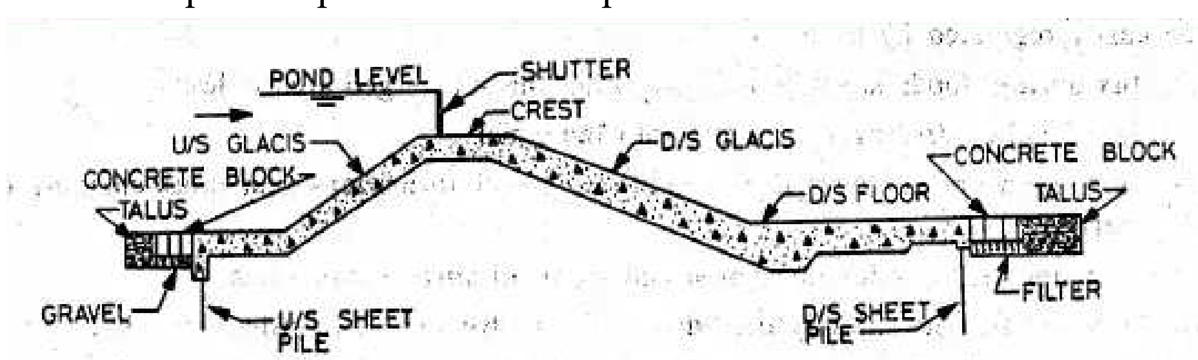


Figure 4.4 Typical cross section of concrete weir with downstream glacis on permeable foundation

4.4.1.2 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.

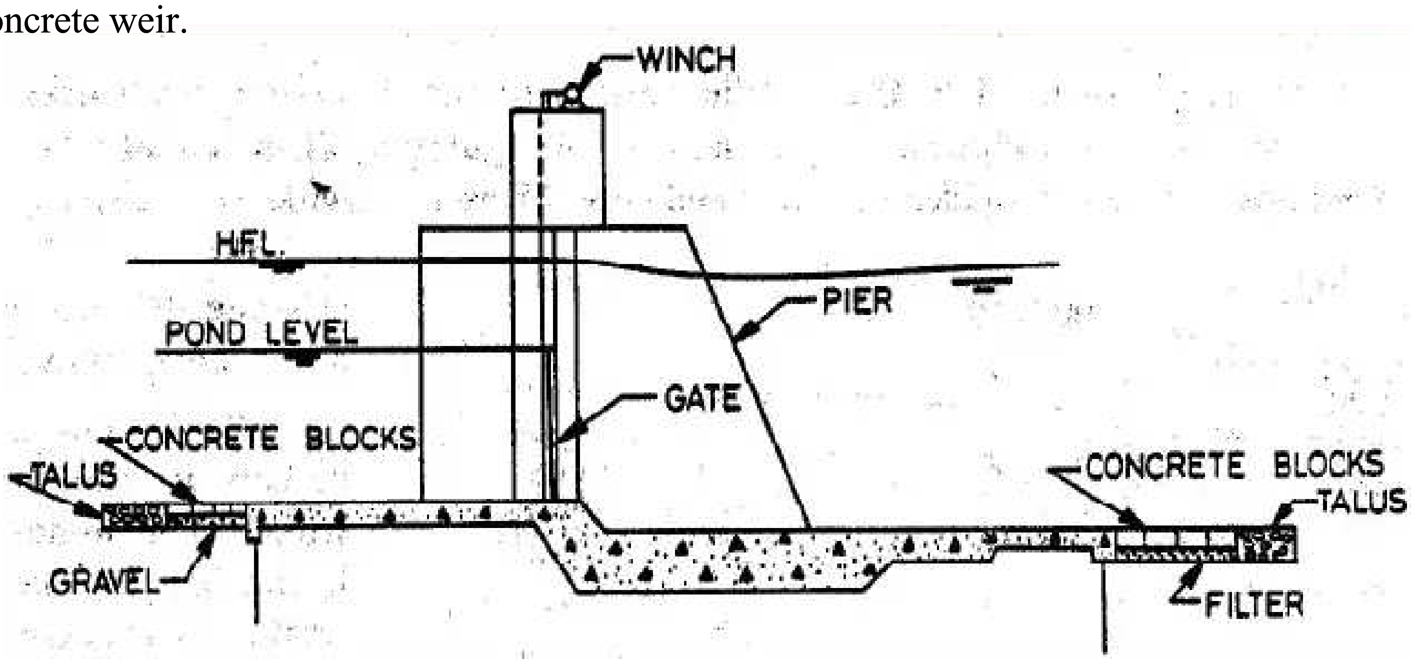


Figure 4.5 Typical cross section of a barrage on pervious foundation

4.4.2 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.

4.4.3 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.

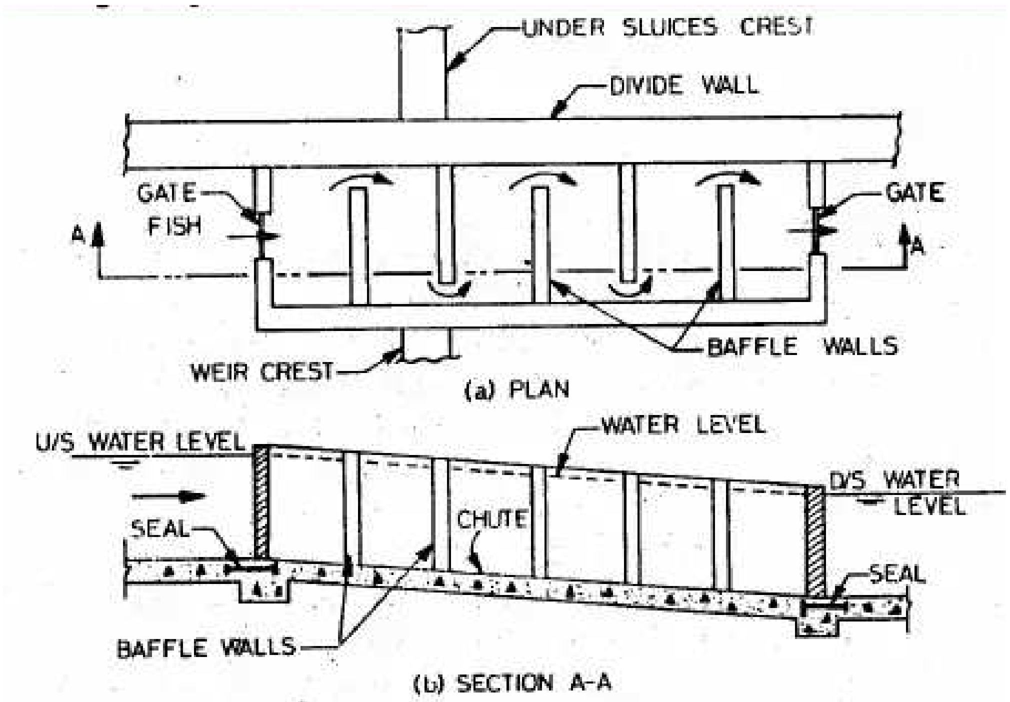


Figure 4.6 Fish ladder

4.4.4. Undersluices or Scouring Sluices

They are the openings provided in the weir wall with their crest at low level. The openings are fully controlled by gates. They are located on the same side of the off-taking canal.

Functions of Undersluices:

(i) They preserve a clear and well defined river channel towards the canal head regulator;
(ii) They scour the silt deposited on the river bed in the pocket upstream of the canal head regulator;
(iii) They pass low floods without the necessity of dropping the weir crest shutters;
(iv) They help to lower the high flood level by supplementing the discharge over the weir during high floods.

Capacity of Undersluices:

The discharging capacity is fixed from the following considerations:

(i) To ensure proper scouring, its capacity should be at least two times the maximum discharge of the off-taking canal;
(ii) It should have sufficient capacity to discharge maximum winter flood – without the necessity of dropping the weir shutter;
(iii) 10 to 20% of the maximum flood discharge – to supplement the discharge over the weir during high floods.

4.4.5. Canal head regulator

The structure controlling diversion into a supply canal is called 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 generally aligned at right angle to the weir, but slightly larger angles (between 90^0 and 110^0) are now considered preferable for providing smooth entry of water into the regulator. The regulation is done by means of gates.

The design principles are the same as those used in the design of barrages, except that the regulators are a smaller version of barrages. An important consideration in designing the regulator is silt exclusion from canals. Silt-excluder tunnels are often provided in the barrage bays adjacent to the regulator, so that the heavier silt-laden bottom layers of water bypass through the tunnels (Figure 4.7).

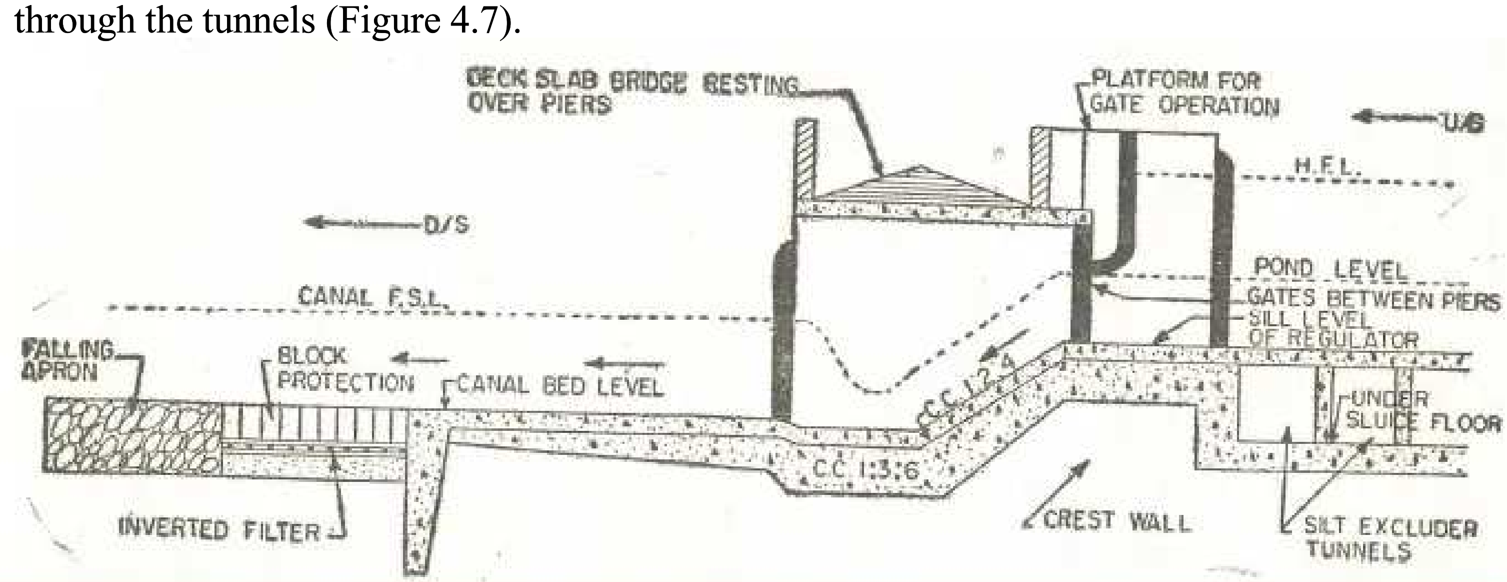


Figure 4.7 Head regulator

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.

4.4.6. Guide banks (river training works)

Guide banks direct the main river flow as centrally as possible to the diversion structure. They also safeguard the barrage from erosion and may be designed so that a desirable curvature is induced to the flow for silt exclusion from the canals. The side slopes of the guide banks must be protected by stone pitching, with a sufficient 'self-launching' stone apron at the lowest feasible level. The top levels of the guide banks will depend on the increase in the maximum flood level upstream of the barrage.

4.4.7. 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.

4.5. Designs of Weirs and Barrages

4.5.1 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

4.5.1.1. Failures due to seepage or subsurface 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 into the resulting depression and more soil is removed which progressively result in subsidence of the floor in the hollows so formed.

- **To prevent these 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.

4.5.1.2. Failures due to surface flow

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.

4.5.2. Criteria for the Design of Weirs and Barrages

Design of weirs and barrages consists of;

i) Hydraulic design
ii) Structural design

The hydraulic design deals with the evaluation of the hydraulic forces acting on the structure and the determination of the configurations of the structure which will be most economical and will have the best functional efficiency.

The structural design consists of dimensioning the various parts of the structure to enable it to resist safely all the forces acting on it.

The hydraulic design is treated in respect of both subsurface and surface flows. The various aspects of design in respect of subsurface flow involves determination of;

- Uplift pressure,
- Exit gradient,
- Length of impervious floor,
- Depth of sheet piles or cutoffs at upstream and downstream ends of the impervious floor;

- Protection works.

Note: *These aspects shall be discussed in detail in section 4.5.3.*

On the other hand, the design in respect of surface flow involves determination of;

- *Pond level;*
- *Afflux;*
- *Levels of upstream floor and crest of weir or barrage;*
- *Shape of weir crest;*
- *Waterway;*
- *Effect of retrogression.*

Pond level: Pond level, in the undersluice pocket, u/s of the canal head regulator may be obtained by adding the working head to the designed full supply level in the canal. The working head should include the head required for passing the design discharge into the canal and the head loss in the regulator.

Afflux: It is the rise in water level on the u/s of a weir or barrage as a result of its construction. The value of afflux corresponding to the design flood is important for the design of the length of the weir, crest levels, river training works, etc.

Levels of u/s floor and crest of weir or barrage

The u/s floor level of a weir or a barrage bays (other than undersluice bays) is fixed at the general river bed level, at or below the level of the crest of the weir or barrage. Usually the floor level is kept at 0.5 to 1.0 m higher than the u/s floor level of the undersluice bays.

The crest levels of weirs or barrages are fixed as follows:

(i) For weirs without shutters, the crest level should be at the required pond level;

(ii) For weirs with shutters, the crest level should not be lower than 2 m below the pond level as the maximum height of the falling shutters is limited to 2 m.

(iii) For barrages, the crest level is determined by the depth required to pass the design flood at the desired afflux. The level of crest in this case should be fixed by adjustment of the waterway. It should in any case be kept higher than the undersluice crest level.

Shape of the weir crest

- A vertical drop weir is usually trapezoidal in cross section and its dimensions may be obtained on the basis of stability considerations;

- A glacis type weir is provided with a top width of about 2.0 m, and u/s slope of 2:1 to 3:1 depending on site conditions and d/s slope as required for the glacis of stilling basin.

Waterway: The length of waterway which is equal to the length of the weir or barrage is fixed to pass safely the maximum flood discharge. The length of the waterway should be equal to the stable river width for the maximum flood discharge so that shoaling upstream is mostly eliminated and a nearly straight and stable approach to the weir or barrage is obtained.

The clear waterway to be provided between guide banks or abutments, excluding thickness of piers, is usually taken equal to the Lacey's regime perimeter given by

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

Where

P = Lacey's regime perimeter in m,
Q = design flood discharge (m^3/s)

To account for the silt load carried by the alluvial rivers, the clear waterway of 1.1 to 1.25 times Lacey's regime perimeter is provided.

Effect of Retrogression: As a result of construction of a weir or barrage progressive retrogression or degradation of the downstream river bed is caused which results in lowering of the downstream river stages and the same has to be suitably provided for in the design of downstream cisterns.

4.5.3. Design of Impervious Floor for Subsurface Flow

4.5.3.1. 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.

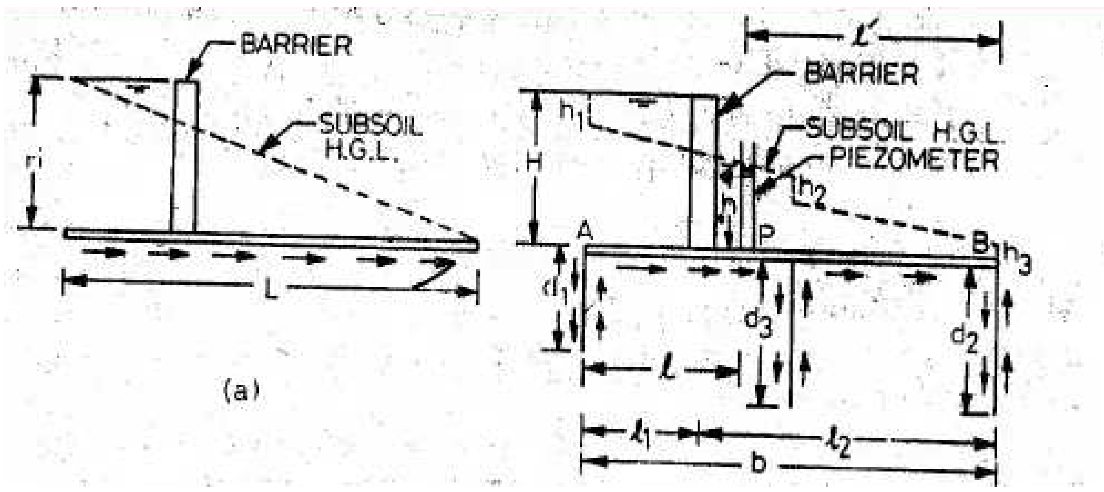


Figure 4.8 Bligh's creep theory – definition sketch

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

The total creep length, L, is given by

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

$$\mathbf{L = b + 2(d^1 + d^2 + d^3)}$$

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

$$\frac{H}{L} = \frac{H}{b + 2d_1 + 2d_2 + 2d_3} = \frac{H}{[b + 2(d_1 + d_2 + d_3)]}$$

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 vertical cutoffs will be:

$[(H/L)\ 2d_1]$, $[(H/L)\ 2d_2]$ and $[(H/L)\ 2d_3]$

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

(a) Safety against piping and undermining

According to Bligh, the safety against piping can be ensured by providing sufficient creep length, given by L = C.H, where C is the Bligh's Coefficient for the soil.

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

Table 4.1 Recommended values of Bligh coefficient of creep C and safe hydraulic gradient

Type of soil	Value of C	Safe Hydraulic Gradient
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
Light sand & mud	8	1/8

(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.

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, $\rho_m = 2400\ kg/m^3$). 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 $h' = \frac{\rho_m}{\rho} t = S_m t$

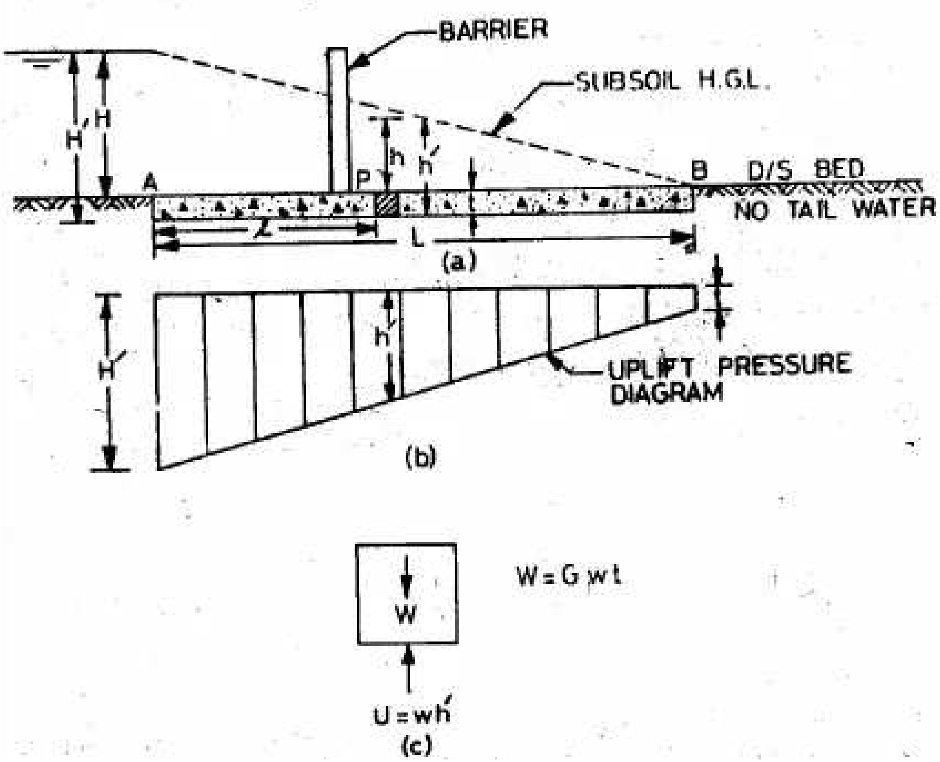


Figure 4.9 Determination of floor thickness

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 ≈ 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 tl

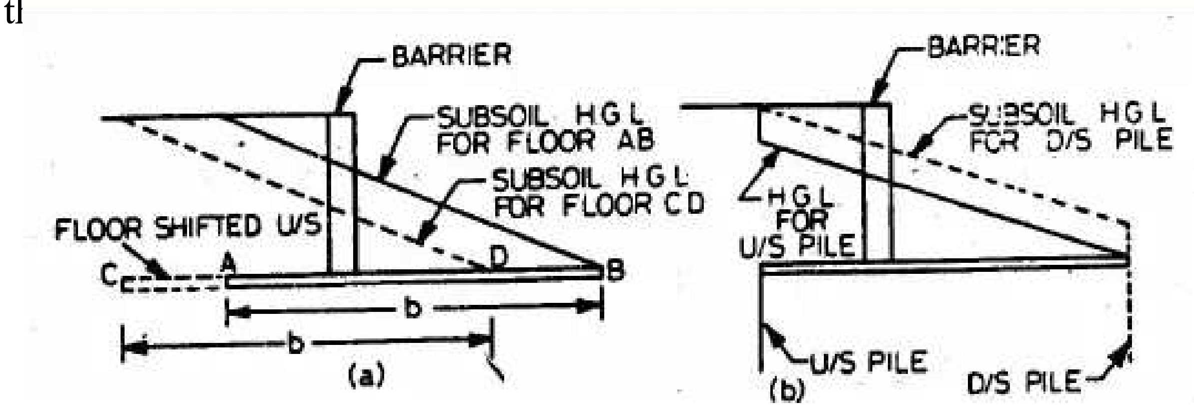


Figure 4.10

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 4.10a). 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 4.10b) 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

(i) Bligh made no distinction between horizontal and vertical creep.
(ii) The theory holds good as long as horizontal distance between cut-offs or pile lines is greater than twice their depth.
(iii) 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.
(iv) 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).
(v) 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.
(vi) 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.

4.5.3.2. 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 $\left(\frac{H}{L_w}\right) \leq \left(\frac{1}{C_1}\right)$ safety against piping can be ensured.

Table 4.2. 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 fine sand or silt	8.5	1/8.5
Fine sand	7.0	1/7
Coarse sand	5.0	1/5
Gravel & Sand	3.5 to 3.0	1/3.5 to 1/3
Boulders, with some cobble & gravel	2.5	1/2.5
Boulders, gravel and sand	2.5 to 3.0	1/2.5 to 1/3
Clayey Soils	3.0 to 1.6	1/3 to 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.

4.5.3.3. Khosla's Theory and Concept of Flow Nets

The main principles of this theory are summarized below:

a) The seepage water does not creep along the bottom contour of pucca-floor as stated by Bligh, but moves along a set of streamlines. 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 coefficient 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.*

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. The remaining streamlines follows smooth curves transiting slowly from the outline of the foundation to a semi-ellipse, as shown in Figure 4.11.

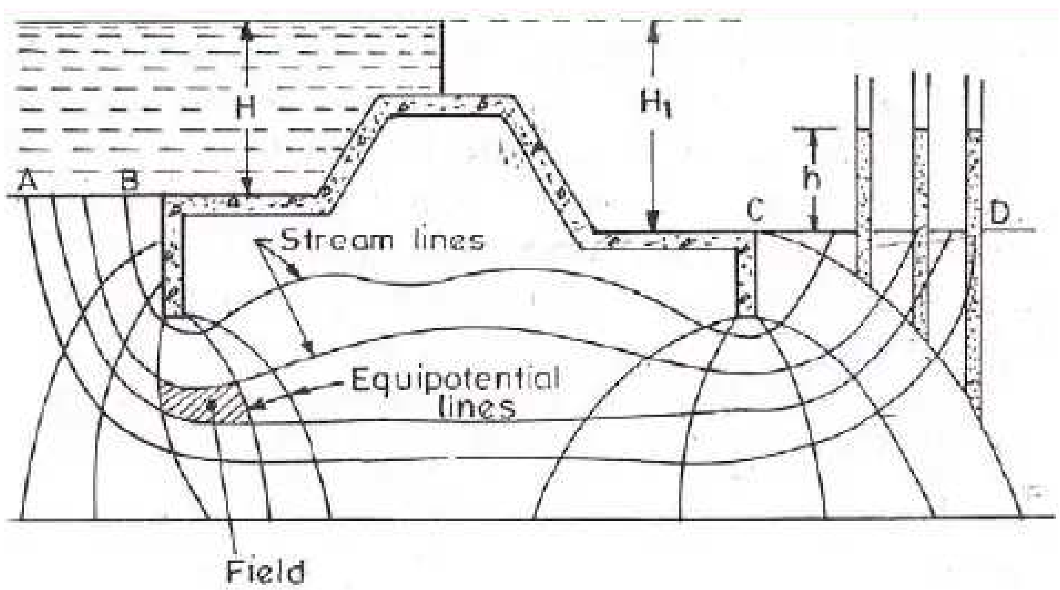


Figure 4.11 Khosla's flow net

Treating the downstream bed as datum and assuming no water on the downstream side, it can be easily stated that every streamline possesses a head equal to h_1 while entering the soil; and when it emerges at the downstream end into the atmosphere, its head is zero. Thus, the head h_1 is entirely lost during the passage of water along the streamlines.

Further, at every intermediate point in its path, there is certain residual head, h, still to be dissipated in the remaining length to be traversed to the downstream end. This fact is applicable to every streamline, and hence, there will be points on different streamlines having the same value of residual head h. If such points are joined together, the curve obtained is called an ***equipotential line***.

Every water particle on line AB is having a residual head $h = h_1$, and on CD is having a residual head $h = 0$, and hence, AB and CD are equipotential lines.

b) The seepage water exerts a force at each point in the direction of flow and tangential to the streamlines. This force (F) has an upward component from the point where the streamlines turns upward. For soil grains to remain stable, the upward component of this force should be counterbalanced by the submerged weight of the soil grain. 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. 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.

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.

4.5.3.4. Khosla's Theory of Independent Variables

In order to know 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 graphically sketching and 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 on 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. Mathematical solutions of flow nets for these simple standard profiles have been presented in the form of equations and curves (given in Figure 4.18 (a), (b) and (c)), which can be used for determining the percentage pressures at the various key points. The simple standard profiles used are:

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

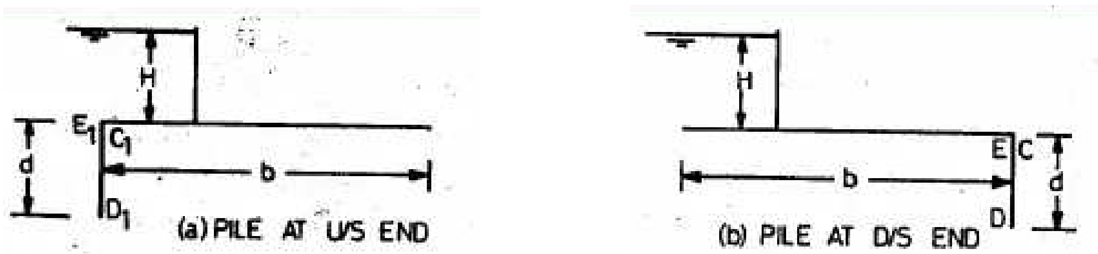


Figure 4.12 (a) Pile at upstream end and (b) Pile at the downstream end

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

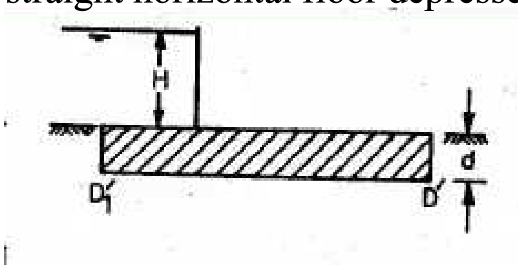


Figure 4.13 Depressed floor

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

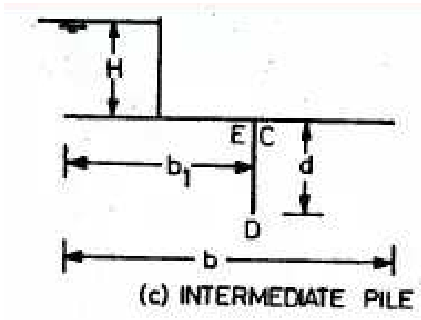


Figure 4.14 Intermediate pile

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 of piles;
(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 between 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.

The 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.

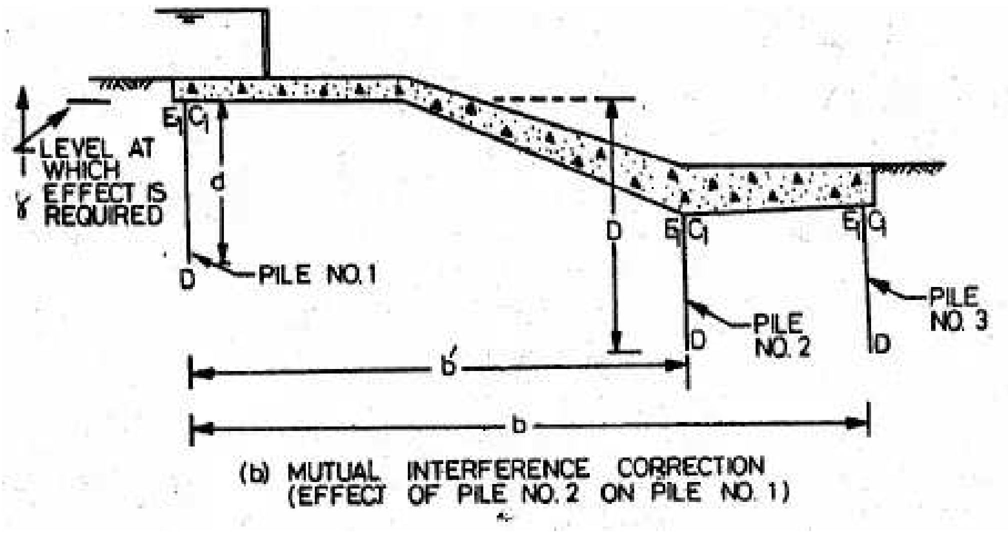


Fig.4. 15 Mutual interference of piles

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.

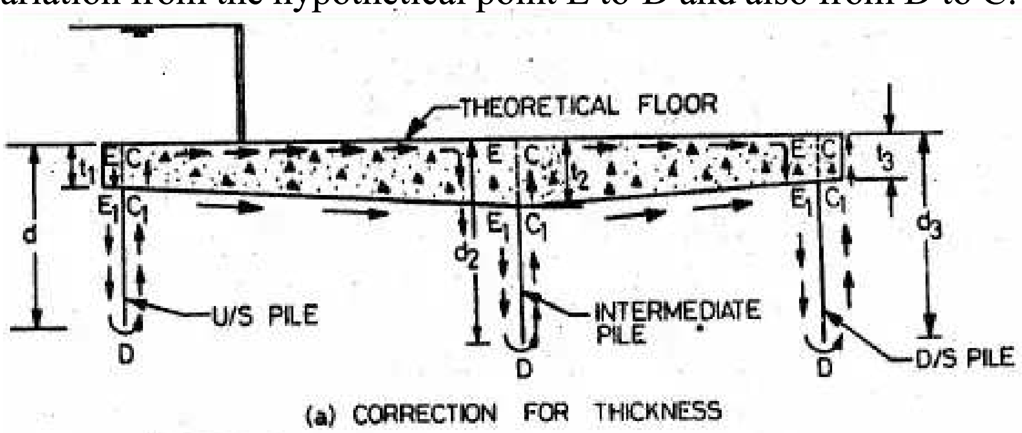


Fig. 4.16 Correction for floor thickness

For pile no. 1, since the corrected pressure at E_1 should be less than the calculated pressure at E, the correction to be applied for the joint E_1 shall be negative. Similarly, the pressure calculated at C is less than the corrected pressure at C_1, and hence, the correction to be applied at point C_1 is positive.

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 (Table 4.3).

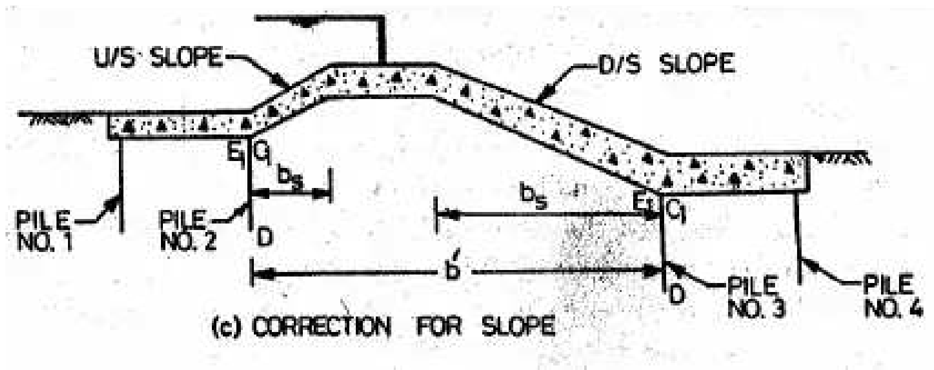


Fig. 4.17 Correction for slope of the floor

Table 4.3: 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 form 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
$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}, \ and$$

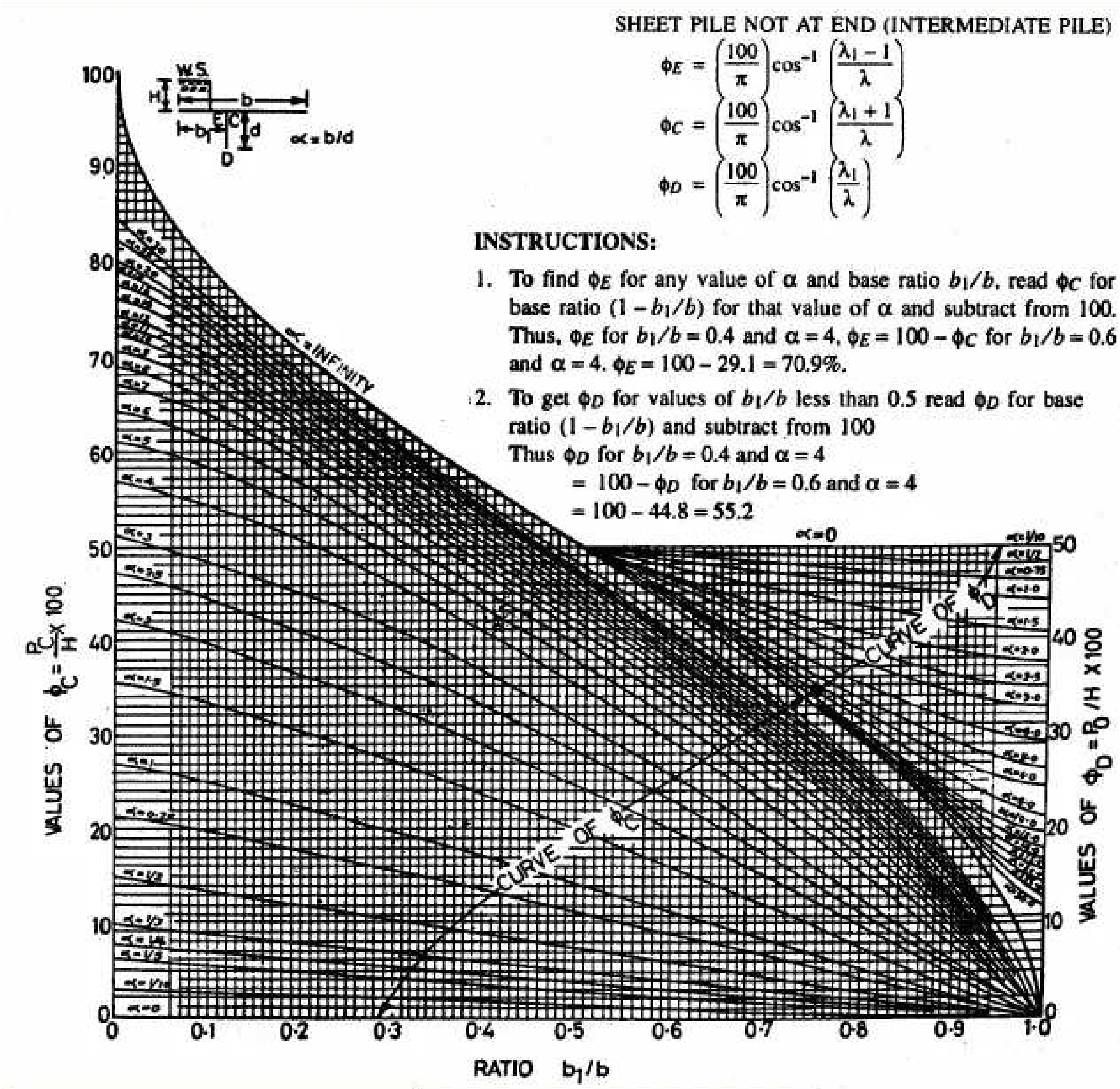
$$\alpha = \frac{b}{d}$$

H = maximum seepage head

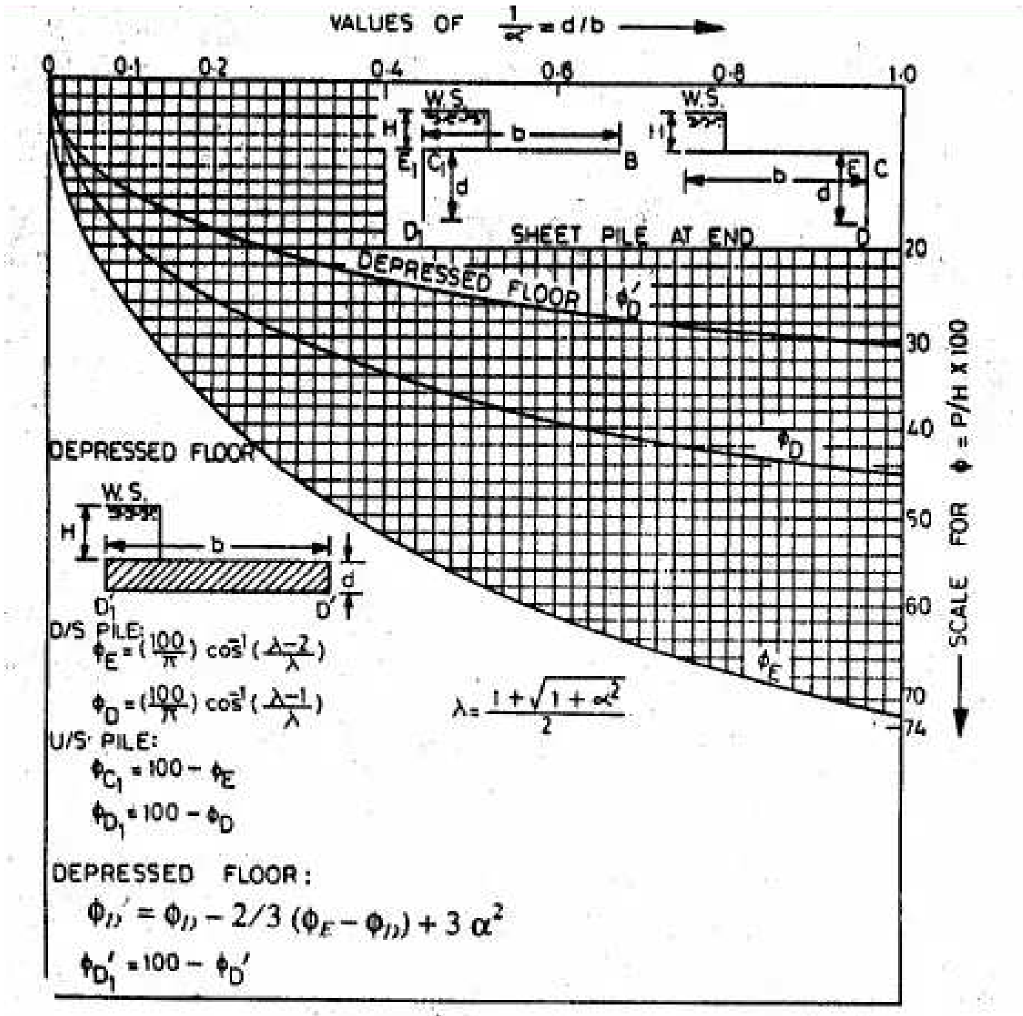
The exit gradient so calculated must lie within safe limits as given in the following table (Table 4.4).

Table 4.4: Safe exit gradient for different types of soils

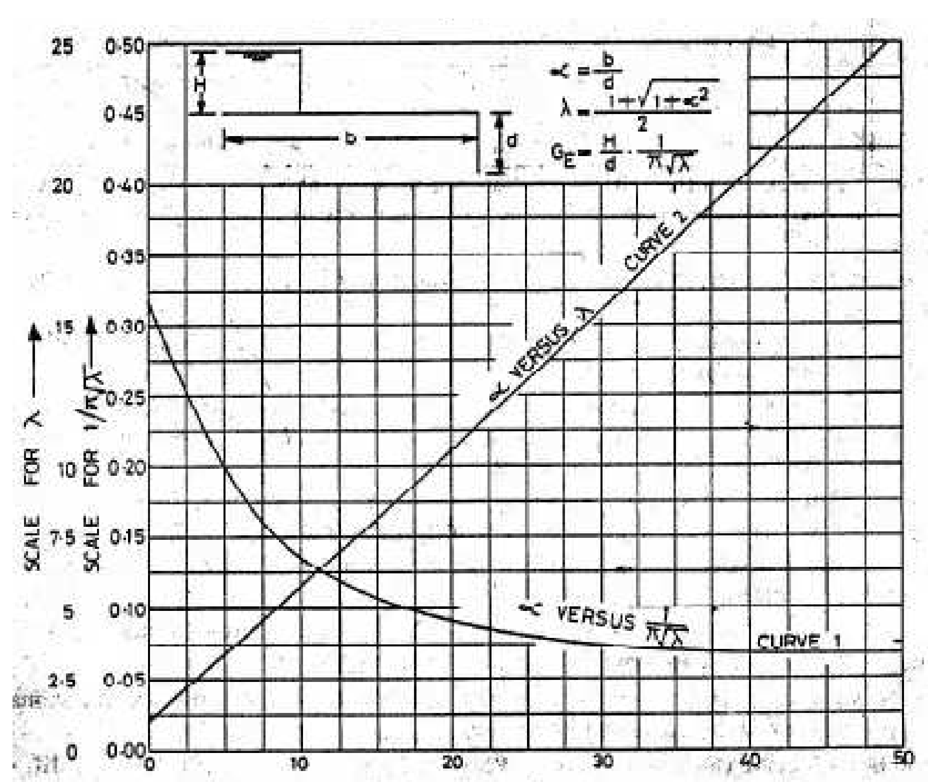
Type of soil	Safe exit gradient
Shingle	¼ to 1/5
Coarse sand	1/5 to 1/6
Fine sand	1/6 to 1/7



(a) Intermediate sheet pile



(b) Sheet pile at the end and depressed floor



(c) Exit gradient

Figure 4.18 Khosla's Pressure Curves (a, b, and c)

Depth of sheet piles on upstream and downstream of impervious floor

The sheet pile must be taken up to the level of possible deepest scour below the bed of the river. According to Lacey the depth of scour in alluvial soils is given by

$$R = 1.35\left(\frac{q^2}{f}\right)^{1/3}$$

Where

R = scour depth measured below the highest flood level (HFL),
q = discharge per unit length,
f = Lacey's silt factor.

In order to ensure further safety, for the design of sheet piles the scour depth is considered as 1.25 to 2 times R given by the above equation.

Design of protection works at the u/s and d/s ends of the impervious floor

In order to further safeguard the impervious floor against failure due to piping certain protection works are provided at both the u/s and d/s ends of the impervious floor. These protection works consist of

(i) Inverted filter,
(ii) Block protection, and
(iii) Launching apron or pervious apron (as shown in 4.19)

Inverted Filter: consists of layers of materials of increasing permeability from bottom to top. The thickness of the inverted filter varies from 0.5 to 1.25 m. To prevent the filter material from dislocation by surface flow they are loaded with large size stones or concrete blocks. The blocks are usually 0.9 to 1.2 m thick and are placed with open joints filled with river sand or filter material.

It is provided immediately at the d/s end of the impervious floor to relieve the uplift pressure. The length depends on the scour depth D below the river bed and it usually varies from 1.5 D to 2 D, where D is given by

$$D = XR - Y$$

Where XR = depth of deepest scour level below high flood level

X = a multiplying factor (varies from 1.25 to 2)
Y = depth of the river bed or impervious floor below high flood level

Y = High flood level – River bed level (or floor level)

Block Protection: It is provided immediately at the u/s end of the impervious floor. It consists of 0.6 to 1.0 m thick stone or concrete blocks laid on 0.4 to 0.6 m thick loosely packed stone. The length of the block protection is usually equal to the depth of scour, D, below the river bed at the u/s end of the impervious floor.

Figure 4.19. Upstream and downstream protection works

Launching apron or pervious apron: It is an apron of loosely packed stones. Its function is to protect the impervious floor and the pile from the scour holes progressing towards the floor and the pile.

The protection is provided by a launching apron by forming a protective covering of stones over a certain slope below the bed of the river at which the apron is originally laid to the bottom of the deepest scour likely to occur. As shown in Figure 4.20, when scour occurs, the new position attained by this apron is called launched position.

The size of the stones (that shall not be washed away during maximum flood) is given by USBR as

$$d = \left(\frac{V_A}{4.915}\right)^2$$

Where V_A = average velocity of flow in m/s and d = mean diameter of stones in m.

It is generally assumed that the stones launch at a slope of 2:1. The quantity of stone in a launching apron should be sufficient to provide about 1.0 m thick cover over a slope of 2:1 in the launched position. Thus if D is the depth of scour, the length of the launched apron would be about $\sqrt{5}D = 2.236D$. Since the thickness of the launched apron is 1 m, the quantity of stone required is 2.236D m^3 per m length of the apron (See Figure 4.20).

Figure 4.20. Launching apron

Example 4.1

The figure below (Figure E-4.1) shows a section of a hydraulic structure on permeable foundation. Calculate the average hydraulic gradient according to

(a) Bligh's creep theory

(b) Lane's weighted creep theory

Also find the uplift pressures at points A and B and the floor thickness required at these points.

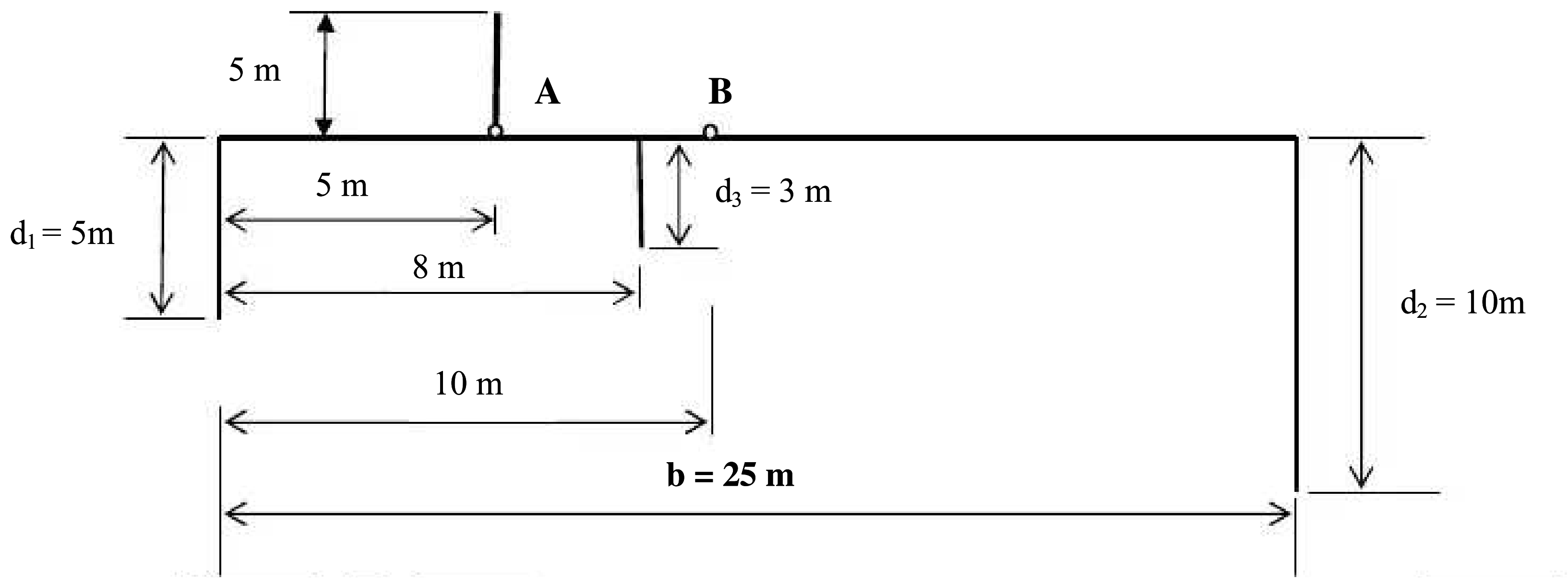


Figure E-4.1

Solution

(a) ***Bligh's creep theory***

- Total creep length, L = b + 2(d1 + d3 + d2) = 25 + 2 (5 + 3 + 10) = 61 m

Hydraulic gradient = H/L = 5/61 = 1/12.2

From Table 4.1, H/L < 1/C = 1/12 (Coarse grained sand)

Therefore, the structure would be safe on coarse grained sand.

- *Uplift pressure at point A*

Length of creep up to point A, $L_A = 2d_1 + 5 = 2 \times 5 + 5 = 15$ m

Residual seepage head at point A, $h_A = [H - (H/L)L_A] = H\,(1 - L_A/L)$

$$= 5\,(1 - 15/61) = 3.77 \text{ m}$$

Uplift pressure at point A, $P_A = \gamma h_A = 9.81 \times 3.77 = 36.98 \text{ kN/m}^2$

- Thickness of the floor at point A

$$t_A = \frac{4}{3}\frac{h_A}{S_m - 1} = \frac{4}{3}\frac{3.77}{2.24 - 1} = 4.05 \text{ m}$$

- *Uplift pressure at point B*

  Length of creep up to point B, $L_B = 10 + 2(5 + 3) = 26$ m

  Residual seepage head at point B, $h_B = H[1 - L_B/L] = 5\ (1 - 26/61) = 2.87$ m

  Uplift pressure at point B, $P_B = \gamma h_B = 9.81 \times 2.87 = 28.15$ kN/m^2

- Required thickness of floor at B,

$$t_B = \frac{4}{3}\frac{h_B}{S_m - 1} = \frac{4}{3} x \frac{2.87}{2.24 - 1} = 3.09 \text{ m}$$

(b) ***Lane's weighted creep theory***

- Total weighted creep length, $L_w = 2\ (5 + 3 + 10) + 1/3\ (25) = 44.33$ m

Hydraulic gradient, $H/L_w = 5/44.33 = 1/8.87 < 1/C_1 = 1/8.5$ (Table 4.2)

Therefore, the structure would be safe on very fine sand or silt.

- *Uplift pressure at point A*

Weighted creep length at A, $L_{wA} = 2 \times 5 + 1/3\ (5) = 11.67$ m

Residual head at A, $h_A = 5(1 - 11.67/44.33) = 3.68$ m

$P_A = 9.81 \times 3.68 = 36.1$ kN/m^2

$$t_A = \frac{4}{3}\frac{h_A}{S_m - 1} = \frac{4}{3} x \frac{3.68}{2.24 - 1} = 3.96 \text{ m}$$

- *Uplift pressure at B,*

$L_{wB} = 2(5 + 3) + 1/3\ (10) = 19.33$ m

$h_B = 5\ (1 - 19.33/44.33) = 2.82$ m

$P_B = 9.81 \times 2.82 = 27.66$ kN/m^2

$$t_B = \frac{4}{3}\frac{h_B}{S_m - 1} = \frac{4}{3} x \frac{2.82}{2.24 - 1} = 3.03 \text{ m}$$

Example 4.2

Determine the percentage pressures at various key points in figure E-4.2. Also determine the exit gradient and plot the hydraulic gradient line for pond level on upstream and no flow on downstream.

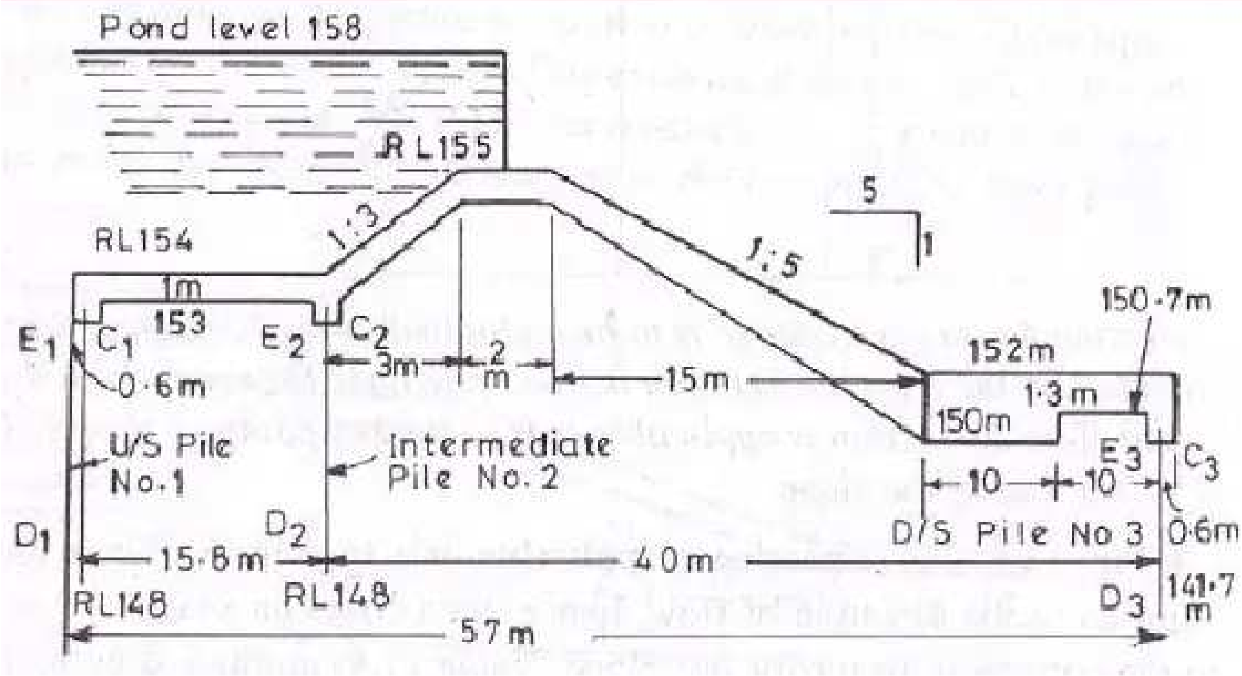


Figure E-4.2

***Solution*:**
(1) For upstream Pile Line No. 1

Total length of the floor, b = 57.0 m

Depth of u/s pile line, d = 154 – 148 = 6 m

$\alpha = b/d = 57/6 = 9.5$

$1/\alpha = 1/9.5 = 0.105$

From curve Figure 4.18 (b)

$\varphi^{C1} = 100 - 29 = 71\ \%$
$\varphi_{D1} = 100 - 20 = 80\ \%$

These values of φ_{C1} must be corrected for three corrections as below:

Corrections for φ_{C1}

(a) Correction at C_1 for Mutual Interference of Piles: (φ_{C1}) is affected by intermediate pile No.2

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

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

$$= 1.88\ \%$$

Where, D = Depth of pile No.2 = 153 – 148 = 5 m

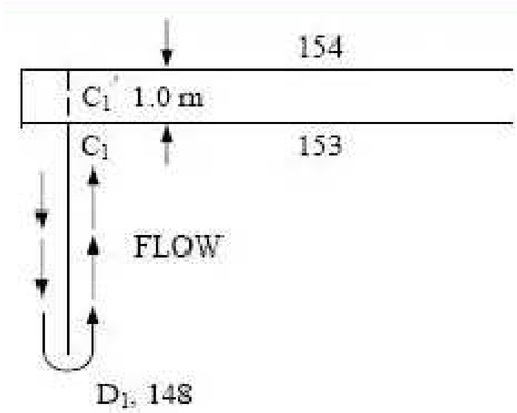
d = Depth of pile No. 1 = 153 – 148 = 5 m

b′ = Distance between two piles = 15.8 m

b = Total floor length = 57 m

Since point C_1 is in the rear in the direction of flow, the correction is positive. Therefore, correction due to pile interference on C_1 = 1.88 % (+ ve)

(b) Correction at C_1 due to thickness of floor:
Pressure calculated from curve is at C_1', (Fig) but we want the pressure at C_1. Pressure at C_1 shall be more than at C_1' as the direction of flow is from C_1 to C_1' as shown; and hence, the correction will be + ve and



$$= \left[\frac{80\% - 71\%}{154 - 148}\right] \times (154 - 153)$$
$$= (9/6) \times 1$$
$$= 1.5\% \ (+ \text{ve})$$

(c) Correction due to slope at C_1 is nil, as this point is neither situated at the start nor at the end of a slope.

Therefore, corrected (φ_{C1}) = 71 % + 1.88 % + 1.5 %
= 74.38 % (answer)

And (φ_{D1}) = 80 %

(2) For intermediate Pile Line No. 2

$d = 154 - 148 = 6$ m

$b = 57$ m

$\alpha = b/d = 57/6 = 9.5$

Using curves of Fig. 4.18 (a), we have b_1 in this case

$b_1 = 0.6 + 15.8 = 16.4$

$b = 57$ m

Therefore, $b_1/b = 16.4/57 = 0.298$ (for φ_{C2})

$(1 - b_1)/b = 1 - 0.298 = 0.702$

$\varphi_{E2} = 100 - 30 = 70$ % (Where 30 % is φ_C for a base ratio of 0.702 and $\alpha = 9.5$)

$\varphi_{C2} = 56$ % (For a base ratio 0.298 and $\alpha = 9.5$)

$\varphi_{D2} = 100 - 37 = 63$ % (Where 37 % is φ_D for a base ratio of 0.702 and $\alpha = 9.5$)

Corrections for φ_{E2}

(a) Correction at E_2 for sheet pile lines. Pile No. (1) will affect the pressure at E_2 and since E_2 is in the forward direction of flow, this correction shall be negative. The amount of this correction is given as:

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

$$= 19\times\sqrt{\frac{5}{15.7}}\times\left(\frac{5+5}{57}\right)$$

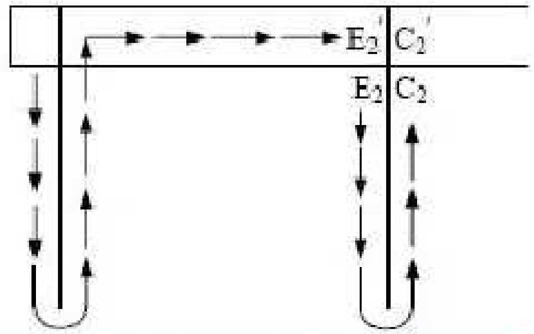
$$= 1.88\ \%\ (-\text{ve})$$

Where, D = Depth of pile No.1, the effect of which is considered = 153 – 148 = 5 m
d = Depth of pile No. 2, the effect on which is considered = 153 – 148 = 5 m
b' = Distance between two piles = 15.8 m
b = Total floor length = 57 m

(b) Correction at E_2 due to floor thickness

$$= \frac{\text{Obs } \varphi_{E2} - \text{Obs } \varphi_{D2}}{\text{Distance between } E_2D_2} \times \text{Thickness of floor}$$

$$= \left[\frac{70\% - 63\%}{154-148}\right] \times 1.0 = (7/6)\times 1.0 = 1.17\ \%$$



Since the pressure observed is at E_2' and not at E_2, (Fig.) and by looking at the direction of flow, it can be stated easily that pressure at E_2 shall be less than that at E_2', hence, this correction is negative,

Therefore, correction at E_2 due to floor thickness = 1.17 % (- ve)

(c) Correction at E_2 due to slope is nil, as the point E_2 is neither situated at the start of a slope nor at the end of a slope.

Hence, corrected percentage pressure at E_2 = Corrected φ_{E2} = (70 – 1.88 – 1.17) % = 66.95 %

Corrections for φ_{C2}

(a) Correction at C_2 due to pile interference. Pressure at C_2 is affected by pile No.(3) and since the point C_2 is in the back water in the direction of flow, this correction is positive. The amount of this correction is given as:

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

$$= 19\times\sqrt{\frac{11}{40}}\times\left(\frac{11+5}{57}\right)$$

$$= 2.89\ \%\ (+\text{ve})$$

Where, D = Depth of pile No.3, the effect of which is considered below the level at which interference is desired = 153 – 141.7 = 11.3 m
d = Depth of pile No. 2, the effect on which is considered = 153 – 148= 5 m
b' = Distance between two piles (2 &3) = 40 m
b = Total floor length = 57 m

(b) Correction at C_2 due to floor thickness. It can be easily stated that the pressure at C_2 shall be more than at C_2', and since the observed pressure is at C_2', this correction shall be positive and its amount is the same as was calculated for the point E_2 = 1.17 %

Hence, correction at C_2 due to floor thickness = 1.17 % (+ ve)

(c) Correction at C_2 due to slope. Since the point C_2 is situated at the start of a slope of 3:1, *i.e.* an up slope in the direction of flow; the correction is negative.

Correction factor for 3:1 slope from Table 4.3 = 4.5

Horizontal length of the slope = 3 m

Distance between two pile lines between which the sloping floor is located = 40 m

Therefore, actual correction = 4.5 × (3/40) = 0.34 % (- ve)

Hence, corrected φ_{C2} = (56 + 2.89 + 1.17 – 0.34) % = 59.72 %

(3) Downstream Pile Line No. 3

d = 152 – 141.7 = 10.3 m

b = 57 m

1/α = 10.3/57 = 0.181

From curves of 4.18 (b), we get

φ_{D3} = 26 %

φ_{E3} = 38 %

Corrections for φ_{E3}

(a) Correction due to piles. The point E^3 is affected by pile No. 2, and since E^3 is in the forward direction of flow from pile No. 3, this correction is negative and its amount is given by

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

$$= 19\times\sqrt{\frac{2.7}{40}}\times\left(\frac{9+2.7}{57}\right)$$

$$= 1.02\ \%\ (-\text{ve})$$

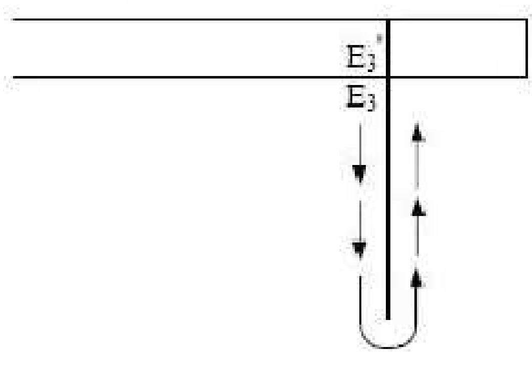
Where, D = Depth of pile No.2, the effect of which is considered = 150.7 – 148 = 2.7 m

d = Depth of pile No. 3, the effect on which is considered = 150 – 141.7 = 9 m

b' = Distance between two piles = 40 m

b = Total floor length = 57 m

(b) Correction due to floor thickness



From the Figure, it can be stated easily that the pressure at E_3 shall be less than at $E^3{}'$, and hence the pressure observed form curves is at E_3'; this correction shall be negative and its amount

$$= \left[\frac{38\% - 32\%}{152 - 141.7}\right]\times 1.3 = (16/10.3)\times 1.3$$

$$= 0.76\ \%\ (-\text{ve})$$

(c) Correction due to slope at E_3 is nil, as the point E_3 is neither situated at the start nor at the end of any slope

Hence, corrected φ_{E3} = (38 – 1.02 – 0.76) % = 36.22 %

The corrected pressures at various key points are tabulated below in Table below

Upstream Pile No. 1	Intermediate Pile No.2	Downstream Pile No. 3
$\varphi_{E1} = 100\ \%$	$\varphi_{E2} = 66.95\ \%$	$\varphi_{E3} = 36.22\ \%$
$\varphi_{D1} = 80\ \%$	$\varphi_{D2} = 63\ \%$	$\varphi_{D3} = 26\ \%$
$\varphi_{C1} = 74.38\ \%$	$\varphi_{C2} = 59.72\ \%$	$\varphi_{C3} = 0\ \%$

Exit gradient

Let the water be headed up to pond level, *i.e.* on *RL* 158 m on the upstream side with no flow downstream

The maximum seepage head, H = 158 – 152 = 6 m

The depth of d/s cur-off, d = 152 – 141.7 = 10.3 m

Total floor length, b = 57 m

$\alpha = b/d = 57/10.3 = 5.53$

For a value of $\alpha = 5.53$, $\frac{1}{\pi\sqrt{\lambda}}$ from curves of Figure 4.18 is equal to 0.18.

Hence, $G_E = \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}} = \frac{6}{10.3} \times 0.18 = 0.105$

Hence, the exit gradient shall be equal to 0.105, *i.e.* 1 in 9.53, which is very much safe.

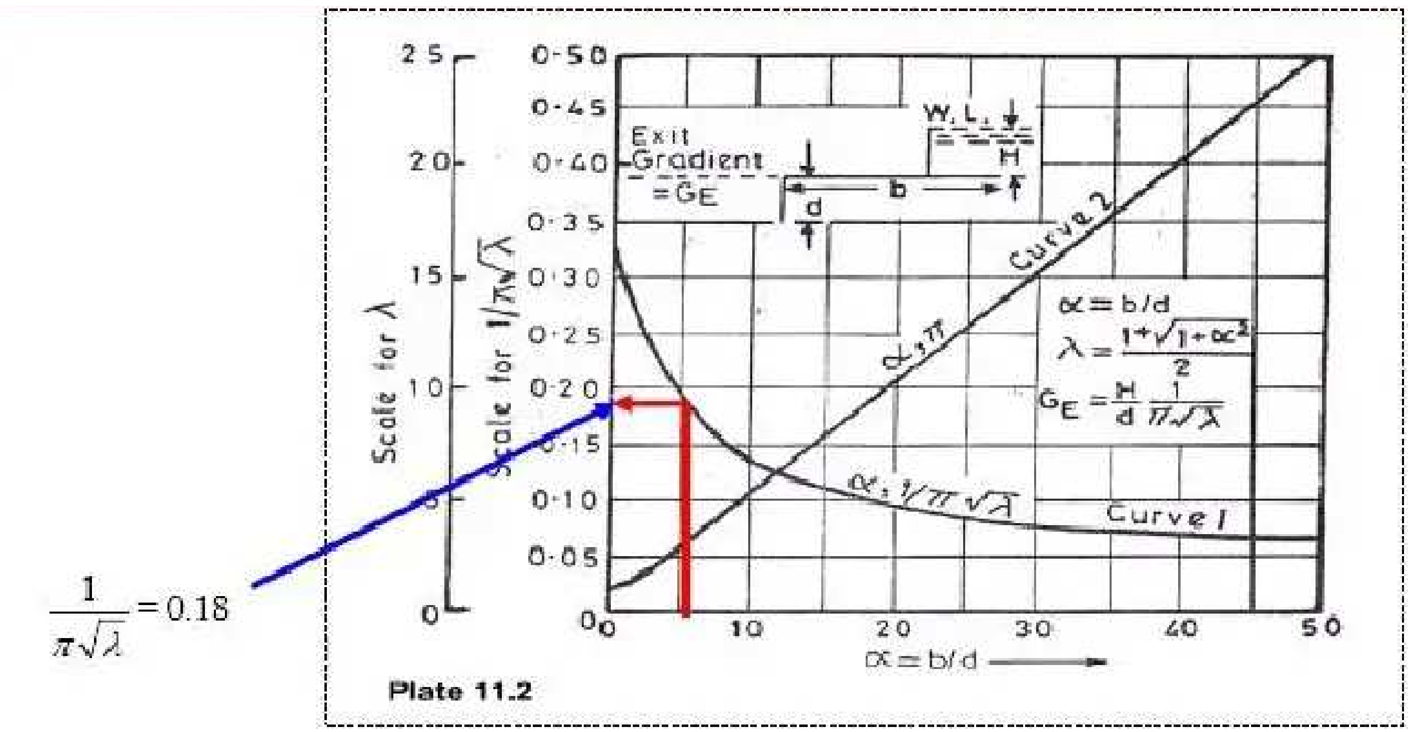


Plate 11.2