11

Theories of Seepage and Design of Weirs and Barrages

11.1. Failure of Hydraulic Structures Founded on Pervious Foundations

Hydraulic structures such as dams, weirs, barrages, head regulators, cross-drainage works, etc. may either be founded on an impervious solid rock foundation or on a pervious foundation. Whenever, such a structure is founded on a pervious foundation, it is subjected to seepage of water beneath the structure, in addition to all other forces to which it will be subjected when founded on an impervious rock foundation. In India, most of these hydraulic structures are required to be founded on alluvial soil foundations, which do allow seepage beneath them. The water seeping below the body of the hydraulic structure, endangers the stability of the structure and may cause its failure, either by :

(*i*) Piping ; or

(*ii*) by Direct uplift.

(*i*) **Failure by Piping or Undermining.** When the seepage water retains sufficient residual force at the emerging downstream end of the work, it may lift up the soil particles. This leads to increased porosity of the soil by progressive removal of soil from beneath the foundation. The structure may ultimately subside into the hollow so formed, resulting in the failure of the structure.

(*ii*) **Failure by Direct Uplift.** The water seeping below the structure, exerts an uplift pressure on the floor of the structure. If this pressure is not counterbalanced by the weight of the concrete or masonry floor, the structure will fail by a rupture of a part of the floor.

The above concepts of the failure of hydraulic structures due to sub-surface flow were introduced by Bligh, on the basis of experiments and the research work conducted after the failure of Khanki weir, which was designed on experience and intuition without any rational theory.

11.2. Bligh's Creep Theory for Seepage Flow

According to Bligh's Theory, the percolating water follows the outline of the base of the foundation of the hydraulic structure. In other words, water creeps along the bottom contour of the structure. The length of the path thus traversed by water is called the *length of the creep*. Further, it is assumed in this theory, that the loss of head is proportional to the length of the creep. If H_L is the total head loss between the upstream and the downstream, and L is the length of creep, then the loss of head per unit of creep length (*i.e.* H_L/L) is called the *hydraulic gradient*. Further, Bligh makes no distinction between horizontal and vertical creep.

Consider a section as shown in Fig. 11.1. Let H_L be the difference of water levels between upstream and downstream ends. (No water is shown on d/s side in Fig. 11.1). Water will seep along the bottom contour as shown by arrows. It starts percolating at A and emerges at B. The total length of creep is given by

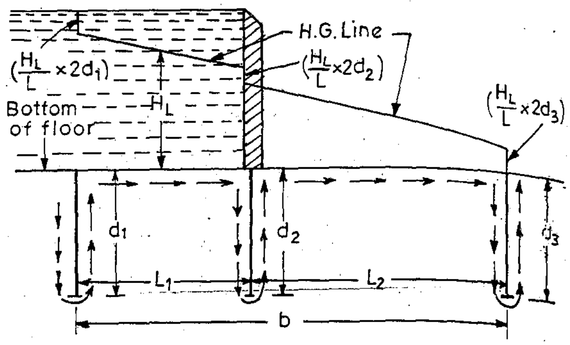


Fig. 11.1. Bligh's Creep.

$$\begin{aligned} L &= d_1 + d_1 + L_1 + d_2 + d_2 + L_2 + d_3 + d_3 \\ &= 2d_1 + (L_1 + L_2) + 2d_2 + 2d_3 \\ &= (L_1 + L_2) + 2\,[d_1 + d_2 + d_3] \\ &= b + 2\,(d_1 + d_2 + d_3) \end{aligned}$$

Head loss per unit length or hydraulic gradient

$$= \left[\frac{H_L}{b + 2.\,(d_1 + d_2 + d_3)}\right] = \frac{H_L}{L}$$

Head losses equal to $\left(\frac{H_L}{L} \times 2d_1\right), \left(\frac{H_L}{L} \times 2d_2\right) \left(\frac{H_L}{L} \times 2d_3\right)$; will occur respectively, in the planes of three vertical cut offs. The hydraulic gradient line (H.G. Line) can then be drawn as shown in Fig. 11.1.

(*i*) **Safety Against Piping or Undermining**. According to Bligh, the safety against piping can be ensured by providing sufficient creep length, given by $L = C.H_L$, where C is Bligh's coefficient for the soil. Different values of C for different types of soils are tabulated in Table 11.1.

Table 11.1. Values of Bligh's Safe Hydraulic Gradient for different types of Soils

S.No.	*Type of soil*	*Value of C*	*Safe Hydraulic gradient should be less than*
1.	Fine micaceous sand (as in North Indian Rivers)	15	1/15
2.	Coarse grained sand (as in Central and South Indian Rivers)	12	1/12
3.	Sand mixed with boulder and gravel, and for loam soil	5 to 9	1/5 to 1/9
4.	Light sand and mud	8	1/8

Note : The hydraulic gradient, *i.e.* H_L/L is then equal to $1/C$. Hence, it may be stated that the hydraulic gradient must be kept under a safe limit in order to ensure safety against piping.

(*ii*) **Safety against uplift pressure.** The ordinates of the H.G. line above the bottom of the floor represent the residual uplift water head at each point. Say for example, if at any point, the ordinate of H.G. line above the bottom of the floor is 1 m, then 1 m head of water will act as uplift at that point. If h' metres is this ordinate, then water pressure equal to h' metres will act at this point, and has to be counterbalanced by the weight of the floor of thickness say t.

$\therefore$ Uplift pressure $= \gamma_w \, h'$

where γ_w is the unit wt. of water

Downward pressure $= (\gamma_w . G.)\, t$

where G is the specific gravity of the floor material.

For equilibrium

$$\gamma_w \cdot h' = \gamma_w . G . t$$

$$h' = Gt$$

Subtracting t on both sides, we get

$$(h' - t) = (G.t - t) = t\,(G - 1)$$

or

$$\boxed{t = \left(\frac{h' - t}{G - 1}\right) = \left(\frac{h}{G - 1}\right)} \qquad \text{...(11.1)}$$

where $(h' - t) = h$ is the ordinate of the H.G. line above the top of the floor. $(G - 1)$ is the submerged specific gravity of the floor material. For concrete, G may be taken equal to 2.4. Hence, the thickness of the floor can be easily determined by using the equation (11.1).This is generally increased by 33%, so as to allow a suitable factor of safety.

It may be mentioned that the floor thickness has to be designed according to equation (11.1) only for the downstream floor and for the worst conditions *i.e.* when maximum ordinates of H.G. line occur. The water standing on the upstream floor, more than counterbalances the uplift caused by the same water, and hence, *only a nominal floor thickness is required on the upstream side*, so as to resist wear, impact of flowing water, etc.

Hence, while designing aprons of hydraulic structures on Bligh's theory for sub-surface flow, the floor thickness, is designed in accordance with the above rules, and sufficient length of pucca floor given by $L = C.H_L$ is provided, so as to ensure a safe value of hydraulic gradient. This will be discussed in details in article 11.5.

11.3. Lane's Weighted Creep Theory

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

Thus in Fig. 11.1, the total Lane's creep length (L_l) is given by

$$L_l = (d_1 + d_1) + \frac{1}{3} L_1 + (d_2 + d_2) + \frac{1}{3} L_2 + (d_3 + d_3)$$

$$= \frac{1}{3} \cdot (L_1' + L_2) + 2\,(d_1 + d_2 + d_3) = \frac{1}{3} \cdot b + 2\,(d_2 + d_2 + d_3)$$

To ensure safety against piping, according to this theory, the creep length L_l must not be less than $C_1 H_L$, where H_L is the head causing flow, and C_1 is Lane's creep coefficient given in table 11.2.

Table 11.2. Values of Lane's Safe Hydraulic Gradient for different types of Soils

S.No.	*Type of soil*	*Value of Lane's Coefficient, C_1*	*Safe Lane's Hydraulic gradient should be less than*
1.	Very find sand or silt	8.5	1/8.5
2.	Fine sand	7.0	1/7
3.	Coarse sand	5.0	1/5
4.	Gravel and sand	3.5 to 3.0	1/3.5 to 1/3
5.	Boulders, gravels and sand	2.5 to 3.0	1/2.5 to 1/3
6.	Clayey soils	3.0 to 1.6	1/3 to 1/1.6

Lane's theory was an improvement over Bligh's theory, but however, was purely empirical without any rational basis, and hence, is generally not adopted in any designs. Bligh's theory, though is still used (even after the invention of modern Khosla's theory), but Lane's theory is practically nowhere used, and is having only a theoretical importance.

11.4. Khosla's Theory and Concept of Flow Nets

Many of the important hydraulic structures, such as weirs and barrages, were designed on the basis of Bligh's theory between the period 1910 to 1925. In 1926-27, the upper Chenab canal syphons, designed on Bligh's theory, started posing undermining troubles. Investigations started, which ultimately lead to Khosla's theory.

A detailed description of this theory is available in C.B.I. publication No. 12, which is available at Publication's Division at Civil Lines, Delhi. The main principles of this theory are summarised below :

(1) The seeping water does not creep along the bottom contour of pucca floor as stated by Bligh, but on the other hand, this water moves along a set of stream-lines as shown in Fig. 11.3. This steady seepage in a vertical plane for a homogeneous soil can be expressed by Laplacian equation.

$$\frac{d^2\phi}{dx^2} + \frac{d^2\phi}{dz^2} = 0$$

where ϕ = Flow potential = Kh where K is 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 (Fig. 11.2). One set of lines is called *Streamlines*, and the other set is called *Equipotential lines*. The resultant flow diagram showing both the sets of curves is called a *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. The remaining streamlines follow smooth curves transiting slowly from the outline of the foundation to a semi-ellipse, as shown in Fig. 11.2.

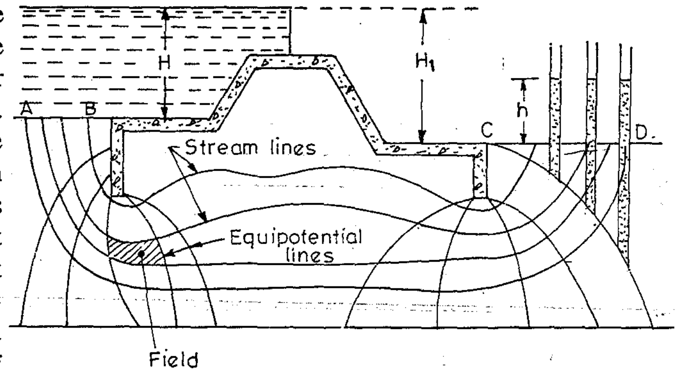


Fig. 11.2. Khosla's Flow Net.

Equipotential Lines. (1) 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 down-stream end into the atmosphere, its head is zero. Thus, the head h_1 is entirely lost during the passage of water along the streamline.

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.

Since an equipotential line represents 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 as shown in Fig. 11.2.

(2) The seepage water exerts a force at each point in the direction of flow and tangential to the streamlines as shown in Fig. 11.3. This force (F) has an upward

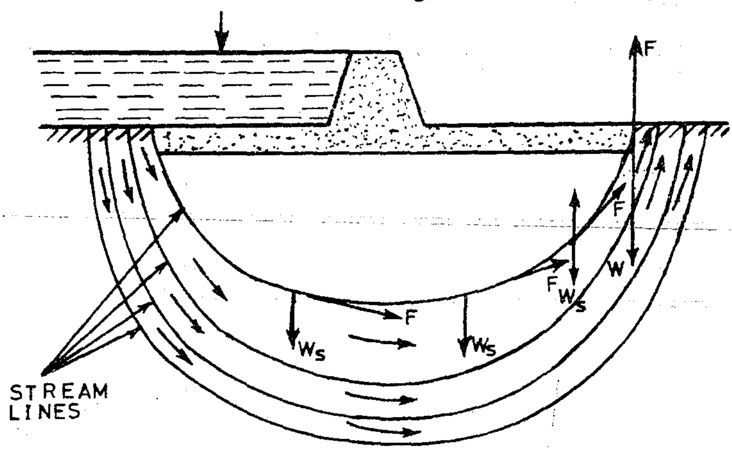


Fig. 11.3

component from the point where the streamline 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 (*i.e. dp/dt*) : 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 or 5 is used, the exit gradient can then be taken as safe. In other words, an exit gradient equal to $\frac{1}{4}$ to $\frac{1}{5}$ of the critical exit gradient is ensured, so as to keep the structure safe against piping.

The submerged weight (W_s) 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 = sp. gravity of soil particles

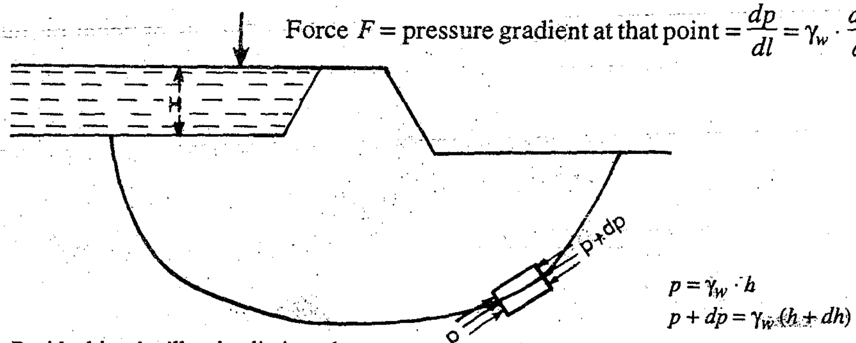
n = porosity of the soil material.

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 $= \frac{dp}{dl} = \gamma_w \cdot \frac{dh}{dl}$



where, h = Residual head still to be dissipated, called hydrostatic excess head.

Fig. 11.4

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

or $$\frac{dh}{dl} = (1 - n)(S_s - 1) \qquad \text{...(11.2)}$$

where $\frac{dh}{dl}$ represents the rate of loss of head or the gradient at the exit end.

Under critical conditions, the critical exit gradient is equal to $(1-n)(S_s-1)$. For most of the river sands, $S_s \cong 2.65$ and $n \cong 0.4$, then the value of critical exit gradient $= (1-0.4)(2.65-1)$

$$= 0.6 \times 1.65 = 0.99 \cong \mathbf{1.0}$$

Hence, an exit gradient equal to $\frac{1}{4}$ to $\frac{1}{5}$ of the critical gradient means that an exit gradient equal to $\frac{1}{4}$ to $\frac{1}{5}$ has to be provided for keeping the structure safe against piping.

Values of safe exit gradient for some of the subsoils are given in Table 11.3.

Table 11.3. Values of Khosla's Safe Exit Gradient for different types of Soils

Type of soil	*Khosla's Safe Exit Gradient*
Shingle	0.25 to 0.20
Course sand	0.20 to 0.17
Fine sand	0.17 to 0.14

Khosla's theory of flow nets made it very clear that the loss of head does not take place uniformly, in direct proportion to the creep length, as stated by Bligh. In fact, it depends upon the whole geometry of the figure, *i.e.* the shape of foundation, depth of impervious boundary and levels of u/s and d/s beds. When the *equipotential lines are traced to be closer, the rate of loss of head will definitely be quicker and vice versa.*

It can, hence be concluded that the safety against piping can not be obtained by providing sufficient floor length, as stated by Bligh, but can be obtained by keeping the exit gradient well below the critical value. The exit gradient may not be safe even if the average hydraulic gradient of Bligh $\left(i.e.\ \frac{l}{C}\right)$ is safe.

(3) Undermining of the floor starts from the downstream end of the d/s pucca floor, and if not checked, it travels upstream towards the weir wall. The undermining starts only when the exit gradient is unsafe for the subsoil on which the weir is founded. *It is, therefore, absolutely necessary to have a reasonably deep vertical cut-off at the downstream end of the d/s pucca floor to prevent undermining.* The depth of this d/s vertical cut off is governed by two considerations *i.e.*

(*i*) maximum depth of scour ; (*ii*) safe exit gradient.

While designing a weir, downstream cutoff from the maximum scoured depth considerations is, first of all, provided, and checked for exit gradient. If a safe value of exit gradient is not obtained, then the depth of cutoff is increased. The depth of cutoff is also governed and limited by practical considerations, as the execution of very deep cutoff may be difficult or unpracticable at site.

A weir or a barrage may fail not only due to seepage (*i.e.* sub-surface flow) as stated by Bligh, but may also fail due to the surface flow. The surface flow (*i.e.* when flood water flows over the weir crest) may cause scour, dynamic action ; and in addition, will cause uplift pressures in the jump trough* (if the hydraulic jump forms on the downstream). These uplift pressures must be investigated for various flow conditions. The maximum uplift due to this dynamic action (*i.e.* for surface flow) should then be compared with the maximum uplift under steady seepage (*i.e.* for sub-surface flow) ; and the maximum of the two chosen for designing the aprons and the floors of the weirs. All these modern aspects and other details about designing weirs on permeable foundations, as per the Khosla's theory, have been discussed in article 11.6.

Khosla's theory differs from Bligh's theory in all the above respects, but owing to the simplicity, Bligh's theory is still used for design of small works. A minimum practical thickness for the floor and a deep vertical cutoff at the downstream end is, however, always provided, in addition to the requirements of Bligh's theory. However, on major works, Bligh's theory should never be used, as it would lead to expensive and unsafe erroneous designs.

11.4.1. Khosla's method of independent variables for determination of pressures and exit gradient for seepage below a weir or a barrage. 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 flownet. 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 w.r.t. the boundary conditions. These are complicated methods and are time consuming. Therefore, for designing hydraulic structures such as weirs or barrages 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 flownets for these simple standard profiles have been presented in the form of equations given in Fig. 11.5, and curves given in Plate 11.1, which can be used for determining the percentage pressures at the various key points. The simple profiles which are most useful are :

(*i*) A straight horizontal floor of negligible thickness with a sheet pile line on the u/s end and d/s end [Fig. 11.5 (*a*) and (*b*)].

(*ii*) A straight horizontal floor depressed below the bed but without any vertical cut-offs [Fig. 11.5 (*c*)].

(*iii*) A straight horizontal floor of negligible thickness with a sheet pile line at some intermediate point [Fig. 11.5 (*d*)].

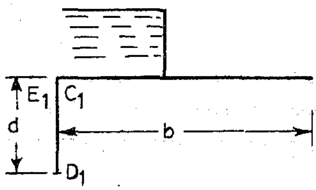
The key points are the junctions of the floor and the pile lines on either side, and the bottom point of the pile line, and the bottom corners in the case of a depressed floor. The percentage pressures at these key points for the simple forms into which the complex profile has been broken is valid for the complex profile itself, if corrected for

(*a*) correction for the mutual interference of piles ;

(*b*) correction for thickness of floor ;

(*c*) correction for the slope of the floor.

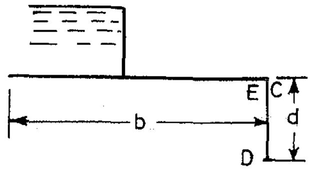
* For detailed description, please refer to Article 11.6.11.



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

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

(*a*)



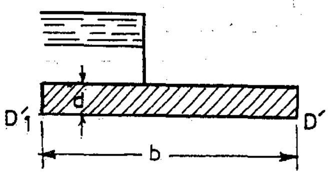
$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$$

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

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

$\alpha = \dfrac{b}{d}$ (respective)

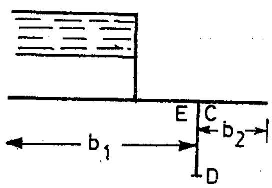
(*b*)



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

$$\phi'_{D_1} = 100 - \phi_{D'}$$

(*c*)



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

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

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

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

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

$\alpha_1 = b_1/d$

$\alpha_2 = b_2/d$

(*d*)

Fig. 11.5. Khosla's simple profiles for a weir of complex profile.

These corrections are described below :

(*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

$$C = 19\sqrt{\frac{D}{b'}}\left[\frac{d + D}{b}\right] \qquad \text{...(11.3)}$$

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 neighbouring 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.

This correction *is positive for the points in the rear or back water; and substractive for the 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.*

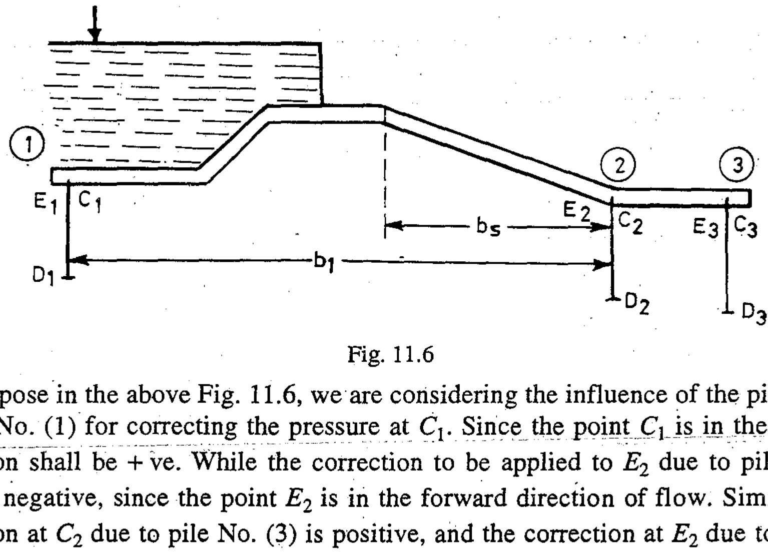


Fig. 11.6

Suppose in the above Fig. 11.6, we are considering the influence of the pile No. (2) on pile No. (1) for correcting the pressure at C_1. Since the point C_1 is in the rear, this correction shall be + ve. While the correction to be applied to E_2 due to pile No. (1) shall be negative, since the point E_2 is in the forward direction of flow. Similarly, the correction at C_2 due to pile No. (3) is positive, and the correction at E_2 due to pile No. (2) is negative.

(*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 pressure at the actual points are calculated by assuming a straight line pressure variation. Since the corrected pressure at E_1 should be less than the calculated pressure at E_1', the correction to be applied for the point E_1 shall be – ve. Similarly, the pressure calculated C_1' is less than the corrected pressure at C_1, and hence, the correction to be applied at point C_1 is + ve.

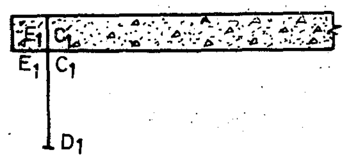


Fig. 11.7

(*c*) **Correction for the Slope of the Floor.** A correction is applied for a sloping floor, and is taken as + ve for the down, and – ve for the *up* slopes following the direction of flow. Values of correction of standard slopes such as 1 : 1, 2 : 1, 3 : 1, etc. are tabulated in Table 11.4.

Table 11.4

Slope Horizontal : Vertical	*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.

Thus, in Fig. 11.6, this correction is applicable only to point E_2. Since the slope is down at point E_2 in the direction of flow, hence, the correction shall be + ve and will be equal to the correction factor for this slope (Table 11.4) multiplied by b_s/b_1, where b_s and b_1 are shown in Fig. 11.6.

Exit Gradient (G_E). 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

$$G_E = \frac{H}{d} \cdot \frac{1}{\pi\sqrt{\lambda}} \qquad \text{...(11.4)}$$

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

$$\text{and} \quad \alpha = \frac{b}{d}.$$

From the curve of Plate 11.2 ; for any value of α, *i.e.* $\frac{b}{d}$, the corresponding value of $\frac{1}{\pi\sqrt{\lambda}}$ can be read. Knowing H and d, the value of G_E can be easily calculated. The exit gradient so calculated must lie within safe limits as given in Table 11.5.

Table 11.5

Type of soil	*Safe exit gradient*
Shingle	$\frac{1}{4}$ to $\frac{1}{5}$ (0.25 to 0.20)
Coarse Sand	$\frac{1}{5}$ to $\frac{1}{6}$ (0.20 to 0.17)
Fine Sand	$\frac{1}{6}$ to $\frac{1}{7}$ (0.17 to 0.14)

The uplift pressures must be kept as low as possible consistent with the safety at the exit, so as to keep the floor thickness to the minimum.

It is obvious from equation (11.4), that if $d = 0$; G_E is infinite. *Hence, it becomes essential that a vertical cutoff at the downstream end must be provided.*

Example 11.1. *Determine the percentage pressures at various key points in Fig. 11.8. Also determine the exit gradient and plot the hydraulic gradient line for pond level on u/s and no flow on d/s.*

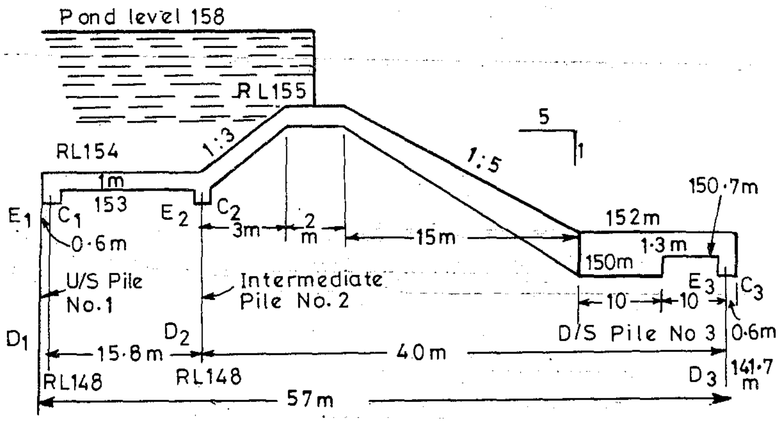


Fig. 11.8

Solution.

(1) **For Upstream Pile Line No. (1)**

Total length of the floor $= b = 57.0$ m.

Depth of u/s pile line $= d = 154.00 - 148.00 = 6.0$ m

$$\alpha = \frac{b}{d} = \frac{57.0}{6.0} = 9.5$$

$$\frac{1}{\alpha} = \frac{1}{9.5} = 0.105$$

From curve Plate 11.1 (*a*)

$$\phi_{C_1} = 100 - 29 = 71\%$$

$$\phi_{D_1} = 100 - 20 = 80\%$$

These values of ϕ_{C_1} and ϕ_{D_1} must be corrected for three corrections as below :

Corrections for ϕ_{C_1}

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

$$\text{Correction} = 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right) \qquad \text{...(11.3)}$$

where D = Depth of pile No. 2.
$= 153.00 - 148.00 = 5.0$ m
d = Depth of pile No. 1 $= 153.0 - 148.00 = 5.0$
b' = Distance between two piles $= 15.8$ m.
b = Total floor length $= 57.0$ m.

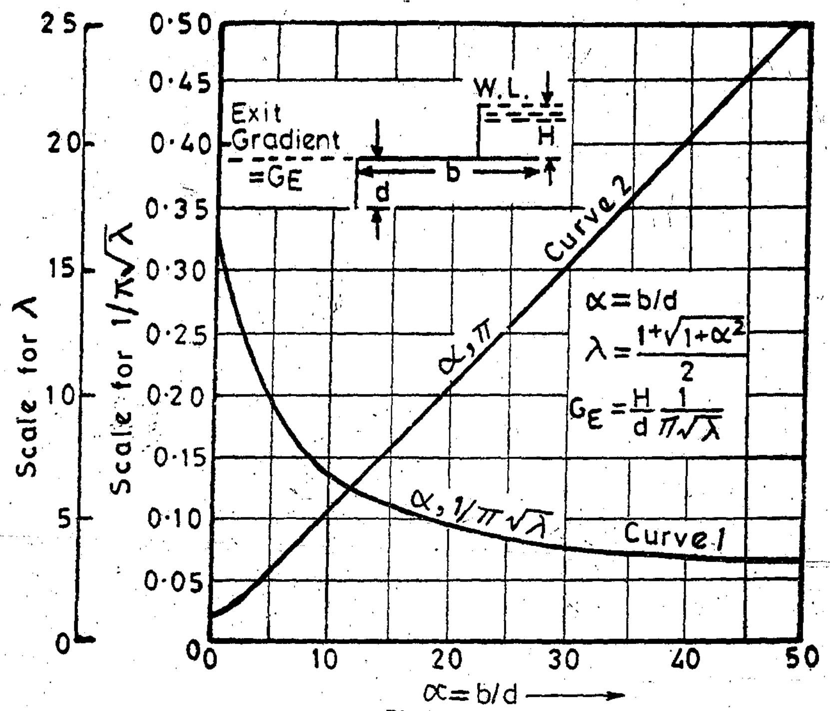

Scale for λ
Scale for 1/π√λ
Exit Gradient =GE
W.L.
H
b
d
Curve 2
α, π
α=b/d
λ = (1+√(1+α²))/2
GE = H/d · 1/(π√λ)
α, 1/π√λ
Curve 1
α= b/d

Plate 11.2

$$\text{Correction} = 19\sqrt{\frac{5}{15.7}}\left[\frac{5+5}{57}\right] = \mathbf{1.88\%}$$

Since the point C_1 is in the rear in the direction of flow, the correction is + ve.

$\therefore$ Correction due to pile interference on C_1

$$= 1.88\% \ (+\text{ve}) \qquad ...(i)$$

(*b*) *Correction at C_1 due to thickness of floor.*

Pressure calculated from curve is at C_1', (Fig. 11.9) 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

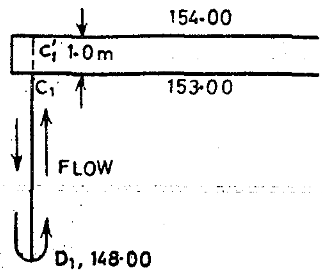


Fig. 11.9

$$= \left[\frac{80\% - 71\%}{154.0 - 148.0}\right] \times (154.00 - 153.00) = \frac{9}{6} \times 1 = 1.5\% \ (+\text{ve}) \qquad ...(ii)$$

(*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 $\phi_{C_1} = 71\% + 1.88\% + 1.5\% = 74.38\%$

Hence **corrected** $\phi_{C_1} = \mathbf{74.38\%}$ **Ans.**

and $\phi_{D_1} = 80\%$

(2) **For Intermediate Pile Line No. (2)**

$$d = 154.00 - 148.00 = 6.0 \text{ m}$$

$$b = 57.0 \text{ m}$$

$$\alpha = \frac{b}{d} = \frac{57.0}{6.0} = 9.5$$

Using curves of Plate 11.1 (*b*), we have b_1 in this case

$$= 0.6 + 15.8 = 16.4 \text{ m}$$

and $b = 57.0$ m

$$\therefore \quad \frac{b_1}{b} = \frac{16.4}{57.0} = 0.298.$$

$$1 - \frac{b_1}{b} = 1 - 0.298 = 0.702.$$

$$\phi_{E_2} = 100 - 30\% = 70\%$$

(where 30% is ϕ_C for a base ratio of 0.702 and $\alpha = 9.5$)

$$\phi_{C_2} = 56\% \text{ (for a base ratio } 0.298 \text{ and } \alpha = 9.5)$$

$$\phi_{D_2} = 100 - 37 = 63\%$$

(where 37% is ϕ_D for a base ratio of 0.702 and $\alpha = 9.5$)

Corrections for ϕ_{E_2}

(*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 – ve. The amount of this correction is given as :

$$19\sqrt{\frac{D}{b'}}\cdot\left[\frac{d+D}{b}\right]$$

where D = Depth of pile No. 1, the effect of which is considered = 153.0 – 148.0 = 5.0 m

d = depth of pile No. 2, the effect on which is considered = 153.0 – 148.0 = 5.0 m.

b' = Distance between the two piles = 15.8 m.

b = Total floor length = 57.0 m.

$$\text{Correction} = 19\sqrt{\frac{5}{15.7}}\left(\frac{5+5}{57}\right) = \mathbf{1.88\%}\ (-\text{ ve})$$

(*b*) *Correction at E_2 due to floor thickness*

$$= \frac{\text{Obs. }\phi_{E_2} - \text{Obs. }\phi_{D_2}}{\text{Distance between } E_2D_2} \times \text{Thickness of floor}$$

$$= \left(\frac{70\% - 63\%}{154.0 - 148.0}\right) \times 1.0 = \frac{7}{6} \times 1.0 = 1.17\%.$$

Since the pressure observed is at E_2' and not at E_2, (Fig. 11.10) and by looking at the direction of flow, it can be stated easily that the 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

$= \mathbf{1.17\%}$ (– 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.

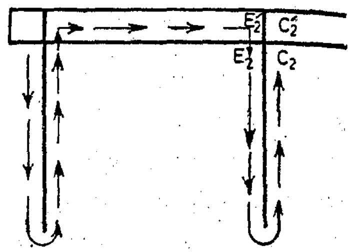


Fig. 11.10

Hence, corrected percentage pressure at E_2

$$= \textbf{Corrected } \phi_{E_2} = 70\% - 1.88\% - 1.17\% = \mathbf{66.95\%\ Ans.}$$

Corrections for ϕ_{C_2}.

(*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 + ve. The amount of this correction is given as :

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

where D = Depth of pile No. (3), the effect of which is considered below the level at which interference is desired

= 153.0 – 141.7 = 11.3 m

d = Depth of pile No. 2, the effect on which is considered

= 153.0 – 148.0 = 5.0 m.

b' = Distance between the pile 2 and pile 3 = 40.0 m

b = Total floor length = 57.0 m.

$$\text{Correction} = 19\sqrt{\frac{11.0}{40.0}}\left(\frac{11.3+5.0}{57.0}\right) = \mathbf{2.89\%}\ (+\text{ ve})$$

(*b*) *Correction at C_2 due to floor thickness.* From Fig. 11.10, it can be easily stated that the pressure at C_2 shall be more than that at C_2', and since the observed pressure is at C_2', this correction shall be + ve 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 11.4 = 4.5
Horizontal length of the slope = 3 m.

Distance between two pile lines between which the sloping floor is located = 40.0 m.

$$\therefore \text{ Actual correction} = 4.5 \times \left(\frac{3}{40}\right) = 0.34\%\ (-\text{ ve})$$

Hence, **corrected** ϕ_{C_2}

$$= 56\% + 2.89\% + 1.17\% - 0.34\% = \mathbf{59.72\%}$$

(3) **Downstream Pile Line**

$$d = 152.0 - 141.7 = 9.3 \text{ m}$$
$$b = 57.0 \text{ m.}$$
$$\frac{1}{\alpha} = \frac{d}{b} = \frac{10.3}{57.0} = 0.181.$$

From curves of Plate 11.1 (*a*), we get

$$\phi_{D_3} = \mathbf{32\%}$$
$$\phi_{E_3} = 38\%$$

Corrections for ϕ_{E_3}

(*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

$$19\sqrt{\frac{D,}{b}} \cdot \left(\frac{d+D}{b}\right)$$

where D = Depth of pile No. 2, the effect of which is considered
= 150.7 − 148.0 = 2.7 m.

d = Depth of pile No. 3, the effect on which is considered
= 150.7 − 141.7 = 9.0 m.

b' = Distance between piles = 40.0 m.

b = Total floor length = 57.0 m.

The correction $= 19\sqrt{\frac{2.7}{40}} \times \left(\frac{9+2.7}{57}\right) = 1.02\%$ (– ve)

(*b*) *Correction due to floor thickness*

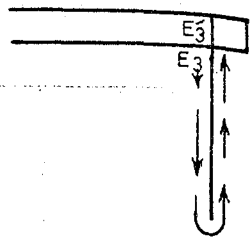


Fig. 11.11

From Fig. 11.11, it can be stated easily that the pressure at E_3 shall be less than at E_3', and since the pressure observed from curves is at E_3'; this correction shall be – ve and its amount

$$= \frac{38\% - 32\%}{152.0 - 141.7} \times 1.3 = \frac{16}{10.3} \times 1.3 = 0.76\% \text{ (– ve)}$$

(*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_{E_3}

$$= 38\% - 1.02 - 0.76\% = \mathbf{36.22\%} \quad \textbf{Ans.}$$

The corrected pressures at various key points are tabulated below in Table 11.6.

Table 11.6

Upstream Pile No. 1	*Intermediate Pile No. 2*	*Downstream Pile No. 3*
$\phi_{E_1} = 100\%$	$\phi_{E_2} = 66.95\%$	$\phi_{E_3} = 36.22\%$
$\phi_{D_1} = 80.0\%$	$\phi_{D_2} = 63.0\%$	$\phi_{D_3} = 32.0\%$
$\phi_{C_1} = 74.38\%$	$\phi_{C_2} = 59.72\%$	$\phi_{C_3} = 0\%$

Exit Gradient

Let the water be headed up to pond level, *i.e.* on *RL* 158.0 m on the upstream side with no flow downstream.

The maximum seepage head $= H = 158.0 - 152.0 = 6.0$ m

The depth of d/s cut-off $= d = 152.0 - 141.7 = 10.3$ m

Total floor length $= b = 57.0$ m.

$$\alpha = \frac{b}{d} = \frac{57.0}{10.3} = 5.53$$

For a value of $\alpha = 5.53$, $\frac{1}{\pi\sqrt{\lambda}}$ from curves of Plate 11.2 is equal to 0.18.

Hence, $G_E = \frac{H}{d} \cdot \frac{1}{\pi\sqrt{\lambda}} = \frac{6.0}{10.3} \times 0.18 = \frac{1}{9.53} = 0.105$

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

Plotting the Hydraulic Gradient Line

The percentage pressures, computed and tabulated in Table 11.6, can be used to work out the elevation of H.G. line above the datum, as given in Table 11.7.

Table 11.7

Flow condition	Upstream water level in metres	Downstream water level in metres	Head in metres	*Height/Elevation of Sub-soil H.G. Line above Datum*								
				Upstream Pile Line			*Intermediate Pile Line*			*Downstream Pile Line*		
				ϕ_{E_1} *100%*	ϕ_{D_1} *80%*	ϕ_{C_1} *74.38%*	ϕ_{E_2} *66.95%*	ϕ_{D_2} *63.0%*	ϕ_{C_2} *59.72%*	ϕ_{E_3} *36.22%*	ϕ_{D_3} *32.0%*	ϕ_{C_3} *0%*
Pond level u/s with no flow d/s	158.00	152.00	6.0	6.0	4.8	4.46	4.02	3.78	3.58	2.68	1.92	0.0
				158.0	156.8	156.46	156.02	155.78	155.58	154.17	153.92	152.0

The subsoil H.G. Line is then plotted in Fig. 11.12.

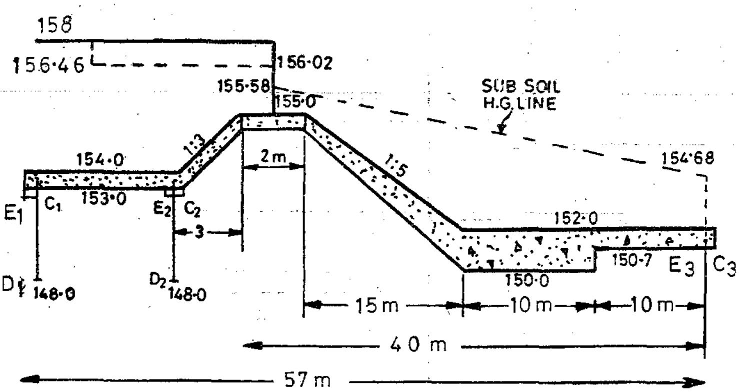


Fig. 11.12

Example 11.2. *In the previous example 11.1, the uncorrected percent residual pressures at* C_1, D_1, E_2, D_2, C_2, E_3 *and* D_3, *were all computed with the use of Khosla's charts. It is now desired to compare these pressures analytically by using the respective formulas, if the charts are not available.*

Solution. (1) **For Upstream Pile Line No. 1.**

$$b = 57 \text{ m}$$

$$d = 154.00 - 148.00 = 6.0 \text{ m}$$

$$\alpha = \frac{b}{d} = \frac{57}{6} = 9.5.$$

Now, ϕ_E for such a case [Refer Fig. 11.5 (*a*)] is given by

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

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

$$\therefore \quad \lambda = \frac{1+\sqrt{9.5^2}}{2} = 5.28$$

$$\therefore \quad \phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{5.28-2}{5.28}\right) = \frac{1}{\pi}\times 51.65^\circ \times \frac{\pi}{180^\circ} = 0.287\text{; i.e. } 28.7\%.$$

$$\therefore \quad \phi_{C_1} = 100 - \phi_E = 100 - 28.7 = 71.3\% \textbf{ Ans.}$$

(as against 71% read out earlier)

Similarly, from Fig. 11.5 (*a*)

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

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

$$= \frac{1}{\pi}\cos^{-1}\left(\frac{5.28-1}{5.28}\right) = 0.199\text{; say } 19.9\%$$

$$\therefore \quad \phi_{D_1} = 100 - 19.9 = 80.1\% \textbf{ Ans.}$$

(as against 80% readout earlier)

(2) **Intermediate Pile Line No. 2**. As given in Fig. 11.5 (*d*),

$$\phi_{E_2} = \phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1-1}{\lambda}\right)$$

$$\phi_{C_2} = \phi_C = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1+1}{\lambda}\right)$$

$$\phi_{D_2} = \phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$$

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

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

$$\alpha_1 = \frac{b_1}{d}\text{; } \alpha_2 = \frac{b_2}{d}.$$

From Fig. 11.8 of the given question,

d = Depth of intermediate pile
= 154.0 − 148.0 = 6 m

α_1 = Floor length *U/S* of intermediate pile
= 16.4 m.

α_2 = Floor length *D/S* of intermediate pile
= 40.6 m.

$$\therefore \quad \alpha_1 = \frac{16.4}{6} = 2.73$$

$$\alpha_2 = \frac{40.6}{6} = 6.77$$

$$\therefore \quad \lambda = \frac{\sqrt{1+2.73^2}+\sqrt{1+6.77^2}}{2} = \frac{2.907+6.843}{2} = 4.875$$

$$\lambda_1 = \frac{2.907-6.843}{2} = -1.968$$

Using, $\phi_{E_2} = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right)$

$$\phi_{E_2} = \frac{1}{\pi}\cos^{-1}\left(\frac{-1.968-1}{4.875}\right) = 0.708\ ;\ i.e.\ 70.8\%.\ \textbf{Ans.}$$

(as against 70% read out earlier).

Now, using

$$\phi_{D_2} = \phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$$

$$= \frac{1}{\pi}\cos^{-1}\left(\frac{-1.968}{4.875}\right) = 0.632 = 63.2\%\ \textbf{Ans.}$$

(as against 63% read out earlier)

Now, using

$$\phi_{C_2} = \phi_C = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda}\right)$$

$$= \frac{1}{\pi}\cos^{-1}\left(\frac{-1.968+1}{4.875}\right) = 0.564 = 56.4\%.$$

(as against 56% read out earlier)

(3) **For Downstream Pile line No. 3.** W.R. to Fig. 11.5 (*b*)

$$b = 57.0\text{ m}$$
$$d = 152.0 - 141.7 = 10.3\text{ m}$$
$$\alpha = \frac{b}{d} = \frac{57}{10.3} = 5.53$$
$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = \frac{1+\sqrt{1+5.53^2}}{2} = 3.31$$

$$\therefore \quad \phi_{E_2} = \phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right)$$

$$= \frac{1}{\pi}\cos^{-1}\left(\frac{3.31-2}{3.31}\right) = \frac{1}{\pi}\,66.65° \times \frac{\pi}{180°} = 0.37\ ;\ i.e.\ 37\%.\ \textbf{Ans.}$$

(as against 38% read out earlier)

$$\phi_{D_2} = \phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right)$$

$$= \frac{1}{\pi}\cos^{-1}\left(\frac{3.31-1}{3.31}\right) = 0.254\ ;\ i.e.,\ 25.4\%.\ \textbf{Ans.}$$

(as against 26% read out earlier)

Example 11.3. *An impervious floor of a weir on permeable soil is 16 m long and has sheet piles at both the ends. The upstream pile is 4 m deep and the downstream pile is 5 m deep. The weir creates a net head of 2.5 m. Neglecting the thickness of the weir* *floor, calculate the uplift pressures at the junction of the inner faces of the pile with the weir floor, by using Khosla's theory.* (*U.P.S.C., Civil Services, 1990*)

Solution. In the given question, since Khosla's curves are not supplied, *it becomes necessary to remember and use* the formulas on which those Khosla's curves are based. These formulas are already mentioned in Fig. 11.5.

For the given question, pressures are required at inner junctions of both piles. Refer Fig. 11.5 (*a*) and (*b*). Pressures are thus required at C_1 for U/S pile, and at E at D/S pile line.

(*a*) **U/S Pile Line** ($\phi_{C_1} = ?$)

$$d = 4 \text{ m}$$
$$b = 16 \text{ m}$$
$$\alpha = \frac{b}{d} = \frac{16}{4} = 4$$
$$\phi_{C_1} = 100 - \phi_E$$
$$\phi_E = \frac{1}{\pi} \cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$$

where $\lambda = \dfrac{1 + \sqrt{1 + \alpha^2}}{2} = \dfrac{1 + \sqrt{1 + 4^2}}{2} = 2.5616$

$$\therefore \quad \phi_E = \frac{1}{\pi} \cos^{-1}\left(\frac{2.5616 - 2}{2.5616}\right) = \frac{1}{\pi} \times (77.34°)$$
$$= \frac{1}{\pi} \times (77.34°) \times \frac{\pi}{180°} = 0.4296 \text{; say } 0.43 \text{ } i.e. \text{ } 43\%.$$
$$\therefore \quad \phi_{C_1} = 100 - 43 = 57\%.$$

Correction due to D/S pile

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

where D = Depth of D/S influencing pile = 5 m
d = Depth of U/S pile being influenced = 4 m
$b = b' = 16$ m

$$C = +19 \cdot \left(\frac{5}{16}\right)\left(\frac{4 + 5}{16}\right) = +6\%.$$

$$\therefore \quad \phi_{C1 \text{ (corrected)}} = 57\% + 6\% = 63\%$$

$$\therefore \quad P_{C_1} = 63\% \times 2.5 \text{ m} = \mathbf{1.575 \text{ m}}. \text{ } \textbf{Ans.}$$

(*b*) **For D/S Pile Line** ($\phi_E = ?$)

$$d = 5 \text{ m}$$
$$b = 16 \text{ m}$$
$$\alpha = \frac{b}{d} = \frac{16}{5} = 3.2$$
$$\lambda = \frac{1 + \sqrt{1 + 3.2}}{2} = 2.176$$

$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{2.176-2}{2.176}\right)$$

$$= \frac{1}{\pi} \times 85.35° \times \frac{\pi}{180°} = 0.474 = 47.4\%.$$

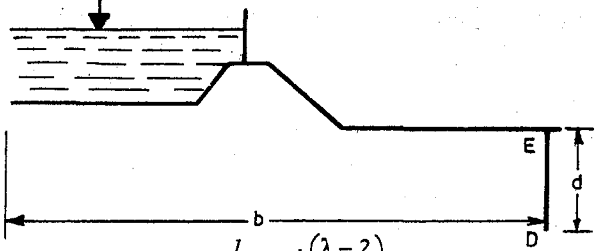
Correction due to U/S pile.

$$C = (-)\ 19 \cdot \sqrt{\frac{4}{16}} \times \left(\frac{5+4}{16}\right) = -5.3\%$$

$$\phi_{E\,(corrected)} = 47.4 - 5.3 = 42.1\%$$

$$P_E = 42.1\% \times 2.5\text{ m} = 1.05\text{ m}. \quad \textbf{Ans.}$$

Example 11.4. *The concrete floor of a head regulator is level with the channel bed (except for the short crest hump) and is 13 m long. The floor is provided with cut off walls at its upstream and downstream ends. The depth of upstream cutoff is 1.5 m (below the floor level) and that of the downstream wall is 2.0 m. Using Khosla's theory (see Fig. 11.13 for definition, sketch and formula), determine the thickness of the floor at its*



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

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

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

Fig. 11.13. Definition sketch for Khosla's theory for uplift pressures.

mid length and also at its junction with the upstream and downstream cutoff walls. The floor thickness may not be less than 30 cm anywhere. The upstream FSL is 1.5 m above the floor level. If the permissible exit gradient is 0.18, is the floor safe against failure by piping ? (*U.P.S.C. Civil Services, 1982*)

Solution. (*i*) For U/S cut off wall.

W.r. to Fig. 11.5 (*a*) and (*b*), we have

$b = 13$ m

d = depth of pile line from top of floor level = 1.5 m.

$$\alpha = \frac{b}{d} = \frac{13}{1.5} = 8.67$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = \frac{1+\sqrt{1+8.67^2}}{2} = 4.86$$

Now, $\phi_{C_1} = 100 - \phi_E$

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

$$= \frac{1}{\pi} \cos^{-1}\left(\frac{4.86-2}{4.86}\right) = \frac{1}{\pi} \times 53.9° \times \frac{\pi}{180°} = 0.30 \text{ ; } i.e., 30\%$$

$$\therefore \quad \boxed{\phi_{C_1} = 100 - 30 = 70\%.}$$

(*i*) *Correction for D/S pile line.*

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

where D = Depth of influencing D/S pile = 2 m

d = Depth of U/S pile getting influenced = 1.5 m

b' = Distance between two piles = 13

$$= (+)\ 19 \cdot \sqrt{\frac{2}{13}}\left(\frac{1.5+2}{13}\right) = 2.0$$

(Since point C_1 is in the rear in the direction of flow, the correction is positive)

(*ii*) *Correction for floor thickness*. Strictly speaking, this **positive** correction needs to be worked out by assuming certain floor thickness of say 1 m at U/S end ; but for that, Khosla's formula for ϕ_{D_1} or $\phi_{D_1} = 100 - \phi_D$; where $\phi_D = \frac{1}{\pi}\cos^{-1}\frac{\lambda-1}{\lambda}$ is required, which is not given in the question, although the other one for ϕ_E is given, which shows that the examiner wants us to ignore this correction. If time permits, this correction can also be worked out.

Hence, **corrected** ϕ_{C_1}

$= 70 + 2 = 72\%$ (Ignoring + ve correction due to floor thickness)

(*a*) P_{C_1} = Residual pressure causing uplift at start point (inner edge of U/S cutoff)

$= \text{Corrected } \phi_{C_1} \times 1.5 = 72\% \times 1.5 = 1.08 \text{ m}$

$\therefore$ **Depth of floor required at start point C_1**

$$= \frac{1.08}{G-1} = \frac{1.08}{1.24} = 0.87 \text{ m ; say 1 m}$$

Use 1.0 m depth (to be conservative for not accounting + ve correction for floor thickness). **Ans.**

(2) **For D/S cutoff wall**

$b = 13$ m

$d = 2$ m

$$\alpha = \frac{13}{2} = 6.5$$

$$\lambda = \frac{1+\sqrt{1+6.5^2}}{2} = 3.79$$

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

$$= \frac{1}{\pi} \cos^{-1} \left(\frac{3.79 - 2}{3.79} \right) = 0.3434 \text{ ; say } 34.34\%.$$

Correction for the effect of U/S pile line. It is – ve, as the point is forward in the direction of flow

$$= (-)\ 19 \cdot \sqrt{\frac{D}{b'}} \cdot \left(\frac{d + D}{b} \right) = (-)\ 19 \cdot \sqrt{\frac{1.5}{13}} \left(\frac{2 + 1.5}{13} \right) = (-)\ 1.74\%.$$

∴ **Corrected** $\phi_E = 34.34 - 1.74 = 32.6\%$.

(*b*) P_E = Residual pressure causing uplift at end point (inner edge of D/S cutoff)

$= \text{Corrected } \phi_E \times 1.5 = 32.6\% \times 1.5 = 0.489 \text{ m}$

Thickness of floor required at end point (E)

$= \dfrac{0.489}{1.24} = 0.394$, say 0.4 m. **Ans.** ; which eventually is more than the min. specified of 0.3 m

(*c*) **Thickness of floor at mid length** can be the average of the two thicknesses, because uplift is varying from start to end, from 72% to 32.6% ; and its value at mid length is just average of the two. Hence, use floor thickness at mid point as

$$= \frac{1.0 + 0.4}{2} = \mathbf{0.7\ m.\ Ans.}$$

Hence, floor thickness at start = 1.0 m. **Ans.**
floor thickness at end = 0.4 m. **Ans.**
floor thickness at mid length = 0.7 m. **Ans.**

(3) **Exit Gradient.** Exit gradient is given by Eqn. 11.4 as

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

where H = Total head = 1.5 m (given)

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

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

d = Depth of D/S cutoff = 2 m

$$\alpha = \frac{b}{d} = \frac{13}{2} = 6.5$$

$$\therefore \quad \lambda = \frac{1 + \sqrt{1 + 6.5^2}}{2} = 3.79,$$

$$\therefore \quad G_E = \frac{1.5}{2} \cdot \frac{1}{\pi \cdot \sqrt{3.79}} = \mathbf{0.123 < 0.18} \text{ (Permissible)}$$

Hence, the floor is safe against piping. **Ans.**

11.5. Design of a Vertical Drop Weir on Bligh's Theory

Many of the vertical drop weirs, such as shown in Fig. 9.6, have been designed on Bligh's theory ; and even though this theory has now been replaced by modern Khosla's theory, yet it is still used at certain places and especially for minor works, owing to its simplicity. The design of a vertical drop weir, on the basis of this theory is explained below :

Design of Pucca-floor and Aprons. As discussed earlier the total length of the pucca floor of the weir (including twice the length of cut-off, if provided) is designed in accordance with the equation $L = CH_L$, and the thickness of the floor is designed by using the equation $t = 1.33\left(\frac{h}{G-1}\right)$. Bligh has further given certain empirical formulas for determining the length of the downstream pucca floor (L_2). The cut-offs may be provided as per the provision of Khosla's theory (explained in article 11.4). The balance floor length [*i.e.* (the total length) – (the downstream length + twice the cut-off length)]

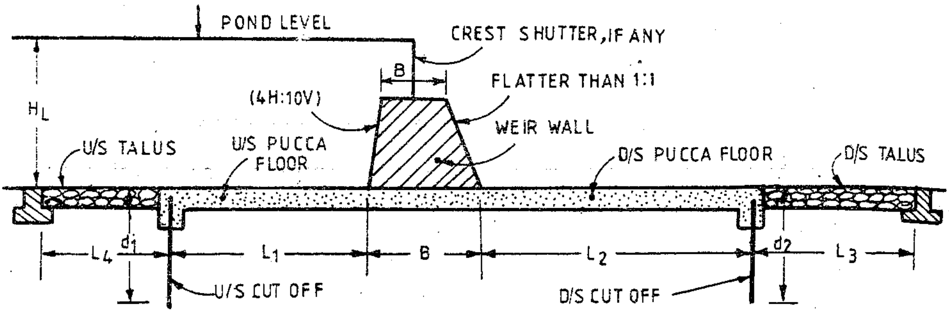


Fig. 11.14. Dimensions of Vertical Drop Weir based on Bligh's Theory.

is then provided under the crest and on the upstream side as shown in Fig. 11.14. The lengths of the upstream and downstream loose stone talus* (or aprons) which are provided in order to prevent the scour from reaching the pucca floor are also worked out by the empirical formulas put forward by Bligh.

The various empirical formulas put forward by Bligh (w.r.t. Fig. 11.14) are given below :

(*a*) $L_2 = 2.21\, C \cdot \sqrt{\frac{H_L}{13}}$ for weirs having crest shutters. ..(11.5)

(*b*) $L_2 = 2.21\, C \cdot \sqrt{\frac{H_L}{10}}$ for weirs having no crest shutters . ..(11.6)

where H_L = the total head loss.

L_2 = the length of d/s pucca floor.

(*c*) $L_2 + L_3 = 18\, C \cdot \sqrt{\frac{H_L}{13} \cdot \frac{q}{75}}$ for weirs having crest shutters ...(11.7)

(*d*) $L_2 + L_3 = 18\, C \cdot \sqrt{\frac{H_L}{10} \cdot \frac{q}{75}}$ for weirs having no crest shutters. ...(11.8)

* In continuation to the upstream pucca floor, a length L_4 (as shown in Fig. 11.14) of loose stone talus is provided to keep the erosion and scour away from the upstream pucca floor. Similarly, a length L_3 of loose sstone talus is provided on the downstream of pucca floor, so as to dissipate the residual energy and to keep the scour away from the pucca floor. As soon as the scour occurs, the talus falls into the scour, and thus preventing the scour to travel and reach upto the pucca floor. The rational design of the talus has been discussed a little later, under "Design of Protection Works", in article 11.6.1.

where q = the discharge intensity in cumecs/metre.
L_3 = the length of d/s loose stone talus.

The length of upstream talus (L_4) may be kept equal to half the length of d/s talus. Thus

$$L_4 = \frac{L_3}{2} \qquad ...(11.9)$$

The above formulas are applicable for designing the proper weir portion ; whereas for designing the 'undersluice' portion of the weir, the following modified formulas are used.

(*i*) $L_2 = 3.87\ C \cdot \sqrt{\frac{H_L}{13}}$ for 'undersluices' having crest shutters . ..(11.10)

(*ii*) $L_2 = 3.87\ C \cdot \sqrt{\frac{H_L}{10}}$ for 'undersluices' having no crest shutters . ..(11.11)

(*iii*) $L_2 + L_3 = 27\ C \cdot \sqrt{\frac{H_L}{13} \cdot \frac{q}{75}}$ for 'undersluices' having crest shutters ...(11.12)

(*iv*) $L_2 + L_3 = 27\ C \cdot \sqrt{\frac{H_L}{10} \cdot \frac{q}{75}}$ for undersluices having no crest shutters ...(11.13)

Design of Weir Wall. Bligh has further given certain empirical formulas for the design of weir wall. According to him, the top width of weir wall (B') is given as :

$$B' = \frac{H}{\sqrt{G-1}} \qquad ...(11.14)$$

where B' = Top width of weir wall and is generally 1.5 to 1.8 m.
H = Head of water over the weir wall at the time of max. flood.
G = Specific gravity of floor material.

Further, the crest width should also be greater by 0.6 m than the height of the crest shutters ; if any.

The bottom width (B) of the weir wall may be obtained by providing suitable side slopes. The u/s batter may by kept as $4H : 10V$ and the d/s batter should not be flatter than 1 : 1. The bottom width (B) of the weir wall should not be less than

$$B = \frac{H + \text{Height of weir}}{\sqrt{G-1}} \qquad ...(11.15)$$

The crest level of the weir wall and the height of solid masonry weir is determined from the considerations of afflux. The afflux produced should not exceed the allowable value, generally kept less than 1.5 m or so. If the crest level, works out to be practically equal to the pond level, then a solid masonry weir can be provided ; and if it is much less than the pond level, then the balance may be provided by crest shutters.

Example 11.5. *A weir with a vertical drop has the following particulars :*

Nature of bed : Coarse sand with the value of Bligh's C = 12
Flood discharge = 300 cumecs

Length of weir = 40 m
Height of weir above low water = 2 m
Height of falling shutter = 0.6 m
Top width of weir = 2.0 m
Bottom width of weir = 3.5 m

Design the length and thickness of aprons and draw the cross-section of the weir.

(*Madras University, 1975*)

Solution. Total max. head loss $= H_L = 2 + 0.6 = 2.6$ m

Total length of creep required including creep along cut-off

$$= L = C.H_L = 12 \times 2.6 = 31.2 \text{ m}.$$

The length of downstream floor is given by Eq. (11.5) as

$$L_2 = 2.21\, C \cdot \sqrt{\frac{H_L}{13}}$$

or $$L_2 = 2.21 \times 12 \cdot \sqrt{\frac{2.6}{13}} = 2.21 \times 12 \times 0.447 = 11.8 \text{ m; say } 12 \text{ m}$$

The bottom width of weir $= B = 3.5$ m.

Provision of cut-offs

Let us first calculate as to what will be the head over the weir when high flood discharge is passing.

Use $$q = 1.7\, H^{3/2}$$

where $$q = \frac{Q}{L} = \frac{300}{40} = 7.5$$

$$\therefore \quad 7.5 = 1.7\, \text{H}^{3/2}$$

$$\therefore \quad H^{3/2} = \frac{7.5}{1.7} = 4.41$$

or $$H = (4.41)^{2/3} = 2.68 \text{ m}$$

Head over the weir crest = **2.68 m.**

$\therefore$ U/s HFL (assuming bed level as 100.0 m) and crest level as 102 m)

$$= 102.0 + 2.68 = \mathbf{104.68\ m}$$

Now R = Lacey's regime scoured depth $= 1.35 \left(\frac{q^2}{f}\right)^{1/3}$

assuming $f = 1$

$$R = 1.35 \left(\frac{7.5^2}{1}\right)^{1/3} = 1.35 \times 3.84 = \mathbf{5.18\ m}$$

Depth of u/s sheet pile from below u/s HFL

$$= 1.5\, R = 1.5 \times 5.18 = 7.8 \text{ m}$$

$\therefore$ Level of bottom of U/s sheet pile

$$= 104.68 - 7.8 = 96.88 \text{ m}$$

Provide a depth of $100 - 96.88 = 3.02$, say 3 m for u/s cut-off.

Since afflux is not known, the d/s HFL is not known; and the exact calculation for the depth of d/s sheet pile cannot be done. Use a similar d/s cut off of 3 m depth below the weir floor.

Now, total creep length provided except u/s floor

$$= 2 \times 3\text{ m} + 3.5\text{ m} + 12\text{ m} + 2 \times 3\text{ m}$$
$$= 27.5\text{ m}$$

The balance length, *i.e.* 31.2 – 27.5 = 3.7 m; say 5 m; is therefore, provided as u/s floor as shown in Fig. 11.15. Hence, total creep length provided = 27.5 + 5 = 32.2 m.

Now using Eq. (11.7), we have

$$L_2 + L_3 = 18\,C \cdot \sqrt{\frac{H_L}{13} \cdot \frac{q}{75}}$$

where L_3 is the length of downstream loose talus

$$q = \text{Discharge intensity} = \frac{Q}{L} = \frac{300}{40} = 7.5$$

$$L_2 + L_3 = 18 \times 12 \sqrt{\frac{2.6}{13} \cdot \frac{7.5}{75}}$$
$$= 18 \times 12 \times 0.141 = 31.6\text{ m}$$

But $\quad L_2 = 11.8$ m

$\therefore \quad L_3 = 31.6 - 11.8 =$ **19.8 m** ; say 20 m.

Hence provide (say 1 m thick) d/s loose talus of 20 m in length.

(**Note** : This length of 20 m can be partly provided as blocks over inverted filter and partly as launching apron. The length of u/s talus L_4 may be kept as equal to $\frac{1}{2}$ the length of d/s talus, *i.e.* 10 m.

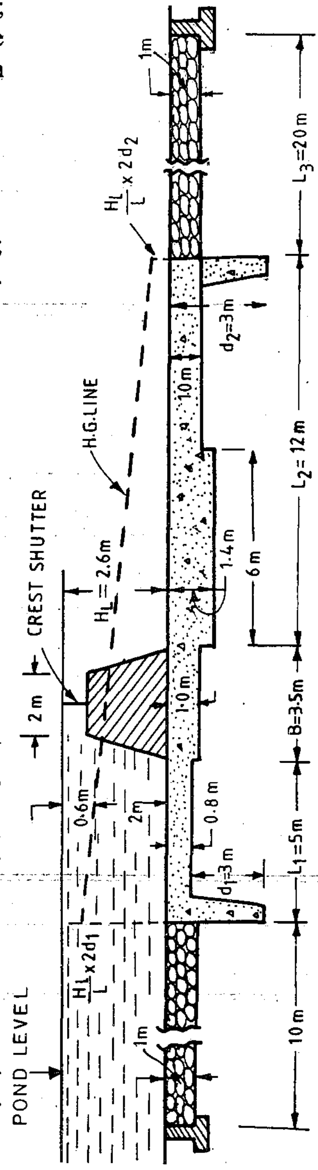


Fig. 11.15

D/s Floor Thickness

The H.G. line is now plotted as shown in Fig. 11.15. The max. ordinate of the H.G. line above the bottom of the floor for the downstream portion at the junction of weir wall

$$= h = \frac{2.6}{32.3} \times 18 = 1.45\text{ m}$$

The thickness of D/s floor at this point is then obtained by Eq.

$$t = 1.33\left(\frac{h}{G-1}\right) = 1.33 \cdot \left(\frac{1.45}{2.4-1}\right)$$
$$= \frac{1.33 \times 1.45}{1.4} = 1.35;\ \text{say } \mathbf{1.4\ m}.$$

Hence, provide 1.4 m thickness for D/s floor from just near its junction with weir wall. The thickness can be curtailed as below.

The thickness required at half way of D/s floor length

$$= \frac{1.33\,h}{1.4}$$

$$\text{where } h = \frac{2.6}{32.3} \times 12 = 0.97 \text{ m}$$

$$= \frac{1.33 \times 0.97}{1.4} = 0.92 \text{ m; say 1 m.}$$

Further, provide a nominal thickness of 0.8 m below the u/s floor and 1 m below the weir wall. The complete details are shown in Fig. 11.15.

11.6. Design of Modern Weirs and Barrages Founded on Permeable Foundations on the Basis of Khosla's Theory

The complete design of a modern glacis-weir or a barrage can be divided into two main aspects, *i.e.*

(1) Hydraulic Design ; and

(2) Structural Design.

11.6.1. Hydraulic Design for the Weir. The 'hydraulic design' involves determining the section of the weir and the details of its u/s cutoff, crest, glacis, floor, d/s cutoff, protection works u/s and d/s, etc. The hydraulic design of weirs on permeable foundations may be classified into :

(*i*) *Design for Sub-surface Flow;* and

(*ii*) *Design for Surface Flow.*

The effects of sub-surface flow or seepage flow on the stability of a hydraulic structure founded on permeable foundations have been thoroughly described earlier and the same hold good for weirs or barrages. *Khosla's method of independent variables is invariably used for determining the uplift pressures exerted by the seeping water on the floor of the weir. The safety of the structure against piping has to be checked by keeping the exit gradient within safe limits.*

The maximum uplift pressure shall occur when the pond is full and there is no water flowing down the weir. But when flood water passes over the weir, entirely new conditions are superimposed. The formation of hydraulic jump causes uplift or unbalanced head in the jump trough, which may be larger than that under steady seepage, as explained below.

Uplift pressure in the jump trough. The maximum difference of head and hence the maximum uplift pressures are normally imposed on the structure when water is ponded upto the highest level on the upstream side without any discharge passing down the weir. The hydraulic gradient line under such a situation shall be as shown in Fig. 11.16 (*a*).

The pressure distribution along the length of the weir section is also shown in Fig. 11.16 (*a*).

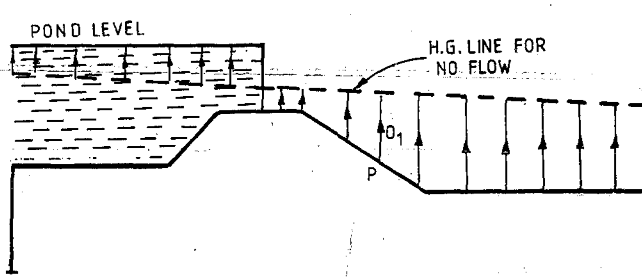


Fig. 11.16 (*a*). Uplift pressures in jump trough for no flow.

When a certain discharge is passing over the weir and hydraulic jump is forming, the seepage head is the difference of the water level of upstream and downstream, which is generally much smaller than the seepage head in case there is no flow. The hydraulic gradient line for this case has also been plotted along with water surface profile, as shown in Fig. 11.16 (*b*).

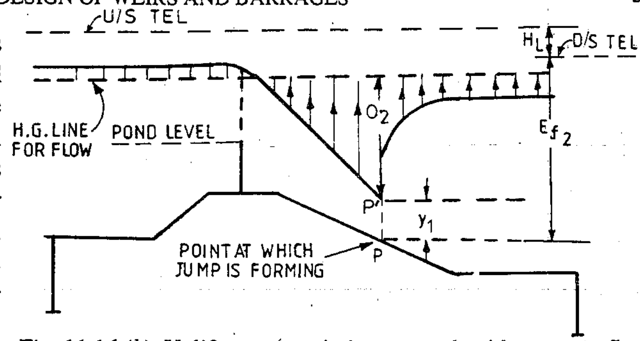


Fig. 11.16 (*b*). Uplift pressures in jump trough with a certain flow.

The ordinate of the uplift pressure is to be measured from the H.G. line to the water surface, as the rest of the uplift is exactly counterbalanced by the weight of the water standing on the floor. The pressure distribution is also shown in Fig. 11.15 (*b*). It is quite evident from these figures that the uplift pressure on the upstream side is more than counterbalanced by the water standing on the floor. Thus, it becomes very clear that there is absolutely no uplift acting on u/s floor, and hence, only a nominal floor thickness is to be provided on the upstream side, and no design is required for u/s floor thickness.

Further, if Fig. 11.16 (*a*) and (*b*) are imposed upon each other, it is found that the uplift ordinate O_2 in the second case is much larger than the uplift ordinate O_1 in the first case at the corresponding point P; where the jump is forming and also at some other points in its vicinity. Since the point of jump formation P is likely to shift with the variation in discharge passing over the weir, the entire glacis may have to be designed for the second condition; while most of the remaining floor may have to be designed for the first condition. In other words, the requirement of floor thickness should be obtained by taking the larger of the two ordinates, and dividing it by the submerged density of the floor material, *i.e.* $(G-1) = (2.24-1) = 1.24$ for concrete.

The water surface profile after the jump and before the jump can be plotted as explained in Chapter 10, and then, H.G. line can be plotted and the net ordinate determined easily. The floor thickness shall have to be designed to withstand the larger pressure.

The uplift pressure due to dynamic action (*i.e.* hydraulic jump) are further reduced due to the following reasons :

(*i*) The backward rolling flow of water in the trough

(*ii*) The uplift pressure due to jump is maximum at the point of the jump but reduces rapidly on either side. As the floor has beam action, it may be designed for average uplift ordinate rather than for maximum uplift ordinate.

(*iii*) The vertical component of the momentum remains unaffected in the jump, which exerts a downward pressure in the vertical direction.

Due to these reasons, the uplift pressure due to dynamic action is reduced to $\frac{2}{3}$rd of the theoretical value, and the design is done for maximum of either the $\frac{2}{3}$rd of the largest ordinate due to dynamic action or the largest ordinate due to steady seepage.

Uplift pressures at the point of jump formation may also approximately be taken as $= [50\% \ (y_2 - y_1) + \phi \cdot H_L]$, *where* ϕ *is the percentage of pressure at the jump location. (Generally used in the design of falls, etc.)*

The other aspects of surface flow are scour considerations. The sheet piles at the ends must go below the deepest anticipated scour level.

For a discharge intensity q, the normal depth of scour (R) is given by Lacey's equation (4.20) *i.e.*,

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

The value of q will be different for the weir and for the undersluice section, and should, therefore, be taken separately for each. For the design of sheet piles, it is just enough to take them down to the level obtained by measuring the normal depth of scour R, below the H.F.L. Though sometimes, even 1.5 R on the upstream side and 2.0 R on the d/s side is taken in conservative designs. A value of 1.25 R on u/s side and 1.5 R on d/s is widely accepted as a *via media*.

Length of Pucca Concrete Floor. The total length of the pucca floor is mainly governed by the exit gradient considerations. For a safe exit gradient and a depth of downstream cut-off suitable from scour considerations, the length of the horizontal floor 'b' can be worked out as $b = \alpha \cdot d$ (α is known when G_E is fixed).

The main turbulence of the hydraulic jump is generally confined to a length equal to five times the jump height. Hence, a pucca floor equal to or more than 5 $(y_2 - y_1)$ in length is provided after the lowest point of jump formation, *i.e.* the end point of glacis. *The glacis should be sloped down at a slope of 3 : 1 to 5 : 1 for maximum dissipation of energy coupled with economy and stability of the jump.* The top width of the crest is generally kept as 2.0 to 3.0 metres from practical considerations, and the upstream slope to the crest is kept as 1 : 1 to 3 : 1. The length of the upstream floor may be adjusted so as to provide the necessary total floor length b, calculated above, as $b = \alpha.d$.

By providing a deeper d/s cut-off, it is possible to reduce the floor length 'b' and *vice-versa*, and hence b and d can be mutually adjusted to provide the most economical and suitable combination and to keep a safe exit gradient.

Design of Protection Works. Protection works are required on the upstream as well as on the downstream in order to obviate the possibility of scour hole travelling close to the pucca floor of the weir and to relieve any residual uplift pressure through the filter. The arrangement consists of (*i*) Inverted filter, and (*ii*) Launching apron, as explained below.

(*A*) *Downstream Loose Protections*

(*i*) **Inverted filter.** Just after the end of the concrete floor, an '*Inverted filter*', 1.5 to 2 D long is generally provided, where D is the depth of scour below the original river bed. The total scour below HFL is taken as xR, where R is Lacey's normal scour depth and values of x for different classes of scour are tabulated below in Table 11.8. *Value of x is generally taken as 1.5 for design of d/s protection works and 1.25 for design of u/s protection works.*

Table 11.8

Class of scour	*Reach*	*Mean value of x*	*$D = xR$ – Water depth above bed*
A	Straight	1.25	$1.25\,R - y$
B	Moderate Bend	1.50	$1.50\,R - y$
C	Severe Bend	1.75	$1.75\,R - y$
D	Right-angled Bend	2.00	$2\,R - y$

The depth of inverted filter is kept equal to the depth of d/s *launching apron.* It generally consists of 1.0 to 1.2 m deep concrete blocks with open joints laid over 0.6 m thick graded filter material. The openings between the blocks are filled with *clean bajri.*

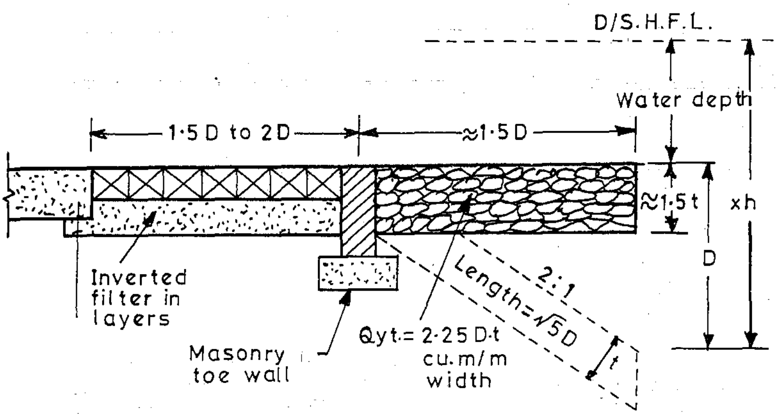


Fig. 11.17. Details of d/s loose projections.

An 'inverted filter' invariably reduces the possibility of piping, as it allows free flow of seepage water through itself without allowing the foundation soils to be lifted upward. *The filter, therefore, consists of layers of materials of increasing permeability from bottom to top.* The gradation should be such that while it allows free flow of seepage water, the foundation material does not penetrate to clog the filter. The design criteria to satisfy these conditions are discussed in chapter 20 on "Earth Dams". To prevent filter from dislocation under surface flow, concrete or masonry blocks are laid over the filter material.

(*ii*) **Launching apron**. After the inverted filter, the loose apron called 'launching apron' is provided for a length, generally equal to 1.5 D, where D has the same meaning as given above in inverted filter calculations.

The apron generally launches to a slope of 2 : 1, and if t is the thickness of the apron in the launched position, length being $\sqrt{5}\,D$; the volume of stone per metre width will then be

$$= \sqrt{5} \cdot D \cdot t = 2.24\,D.t.; \text{ or say } \mathbf{2.25\,D.t.}$$

Hence, Volume of stone per metre is given by 2.25 $D.t.$cu. m/metre.

Since the volume of stone should be the same in launched and unlaunched apron, and if the unlaunched apron is laid in a length equal to 1.5 D, the thickness of the unlaunched apron is given as :

$$= \frac{2.25\,D.t}{1.5\,D} = 1.5\,t$$

If t is taken as 1 m ; then an apron of 1.5 m thickness and 1.5 D metres long has to be laid as d/s launching apron. Different values of t have been recommended by different investigators.

Blench has recommended t equal to 1.5 to 2 times the size of the stone (d_s) required by equation (11.15) as :

$$V_{av} = 4.915\, d_s^{1/2} \quad \ldots(11.15)$$

where V_{av} = average velocity of flow in m/sec.

d_s = mean diameter of stone in metres.

Spring has recommended t as about 0.9 m. Gales recommended t varying from 1.35 to 1.9 m, for discharge varying from 7,000 to 70,000 cumecs. Blench's recommendations are quite adoptive.

(*B*) *Upstream Loose Protections*

Just upstream of the concrete floor of the weir, block protection is provided. It generally consists of concrete blocks laid over packed stone, for a length equal to D. ($D = xR - y$, where x = 1.0 to 1.5, generally taken as 1.25).

Upstream of the blocks, a launching apron is provided in the same way as described for the downstream portion, except that the proper value of x should be chosen.

Toe walls are always generally constructed in between the 'filter' and the 'apron' as shown in Fig. 11.17.

11.6.2. Structural Design for the Weir Floor. The concrete floor is usually designed for the uplift pressures as a pure gravity section at each point. Sometimes, when H_L is enormous, the uplift pressures are too high, and as such, the thickness of the required concrete gravity section becomes enormous and, therefore, becomes very costly. In such cases, the floor may be designed as a reinforced concrete raft structure held down by the weight of the raft and piers. A raft, in such cases, may be cheaper and more desirable as the thin section of raft reduces deep excavations and dewatering problems. Recently, Farraka Barrage, Narora Barrage, and Durgapur Barrage have been designed and constructed as reinforced raft structures. However, the treatment of this type of design is a simple structural problem. The total downward weight of raft and piers is counterbalanced by a uniform uplift, and design is done by working out bending moments, etc.

11.6.3. Effects Produced by Weir on River Regime and Retrogression of Downstream Levels. Before we start with the actual design of a weir, let us first review the effects that are produced by the weir construction, on the regime of the river. The first effect produced by the construction of a weir across a river, is that : *the downstream bed of the river goes on eroding, consequently causing progressive lowering of the downstream levels. This progressive lowering of the downstream levels is known as Retrogression of downstream levels or* **retrogression.**

The basic cause for retrogression is the variation in the silt carrying capacity of the channel. As soon as a weir is constructed, the water starts ponding on its upstream side, causing the water surface slope to flatten for some distance behind the weir. This reduces the silt carrying capacity of the river in this reach, and consequently the silt deposition starts, *i.e.* the river starts dropping sediment, and this leads to formation of shoals and islands on the upstream side. This clearer water passes over the weir and picks up sediment from its downstream bed, so as to fulfil the increased demand of the silt carrying capacity of the channel in the downstream, *i.e.* the sediment deficit caused on the upstream is made up by eroding extra sediment from the downstream. This causes the progressive lowering of the downstream bed levels. The process continues for a number of years till the river starts to regain its original slope in the upstream portion by extending the afflux more and more upstream. The stage is gradually reached, when the upstream pond absorbs no more silt. As the off-taking channel takes comparatively silt free water, the sediment will go downstream, while the discharge going down will be below normal, and hence, the sediment taken shall be more than the 'carrying capacity of the river' ; consequently resulting in sediment deposition on the down stream river bed and long range recovery of d/s bed levels.

A provision must, therefore, be made for retrogression of d/s bed, while designing a weir, as it shall lower the d/s TEL and increase H_L. Hence, if retrogression is not taken into account, it may lead to undermining of floor. Observations on various weirs in Punjab have exhibited a retrogression of 1.2 to 2.2 metres. The retrogression of such a high magnitude has been observed at low water levels, but at high flood levels, the maximum retrogression is between 0.3 to 0.5 metres.

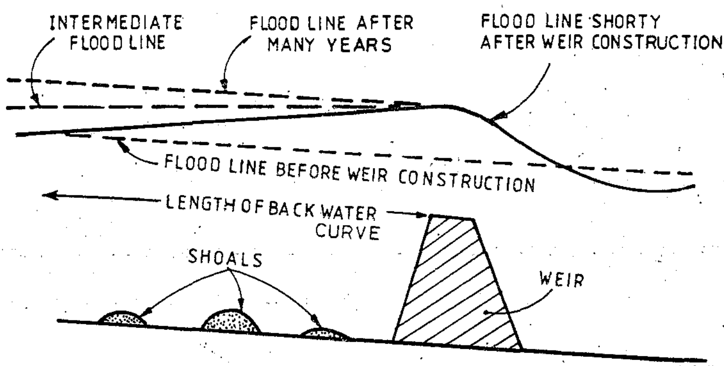


Fig. 11.18. Figure showing the effects of weir construction on flood levels.

Hence, a retrogression of 0.5 m at high flood stage, and a higher retrogression varying linearly upto 2.0 m at lower discharges, is generally considered in the design of weirs or barrages.

The recovery of downstream bed levels, sometimes continues even beyond the original bed levels. This may lead to reduced control on silt regulation. Hence, sufficient margin must be provided between the canal full supply level and the pond level, so as to allow raising of the crest of the canal Head Regulator, if found necessary, in future.

Since the afflux extends on the upstream of the weir with the passage of time, it increases the high flood levels on the upstream even beyond the original backwater curve. Hence, the marginal bunds will have to be extended upstream as soon as the above effects come into picture. Since it happens after many years, it is economical to construct marginal afflux bunds only for the backwater length in the beginning, and to extend them upstream afterwards.

11.6.4. Factors Governing the Design of a Weir or a Barrage The following data must be collected before a weir or a barrage can be designed :

(*i*) High flood levels for the river at the weir site.

(*ii*) High flood or maximum flood discharge for the river at the weir site.

(*iii*) River cross-section at the weir site.

(*iv*) The stage discharge curve for the river at the weir site.

All these informations can be obtained from Topographical maps of the area and by consulting the Hydrological and Meteorological Departments of the area.

Factors to be Decided. Besides Retrogression, certain other factors which have to be decided while designing a weir or a barrage are :

(*i*) Crest Levels.

(*ii*) Afflux.

(*iii*) Waterway and the discharge per metre.

(*iv*) Pond Level.

They are described below :

(*i*) **Crest Levels.** It has been stated earlier that the weir consists of two parts :

(*a*) The main weir section, called Weir Bay Section.

(*b*) Undersluice Section.

The undersluice section is kept at a lower level, so as to provide deeper silent river pocket near the canal head regulator. The undersluice crest is, therefore, kept slightly lower than the barrage bay crest so as to attract a deep current infront of the canal head regulator, so that dry weather flow may remain rear the regulator.

The undersluice crest is generally kept as near the bed level in the existing deepest channel, as is practically possible.

The crest level of other barrage bays is generally kept 1.0 to 1.5 metres higher than the crest level of the undersluices. It is also guided by the general bed level of the river in the barrage bay portion.

It can be easily seen that the afflux and discharge per metre are dependent upon the crest levels. If crest level is low, afflux shall be less, and since the depth of water over the crest will be more, it shall lead to higher discharge per metre. A low set barrage, with increased depth of water over the crest may, therefore, result in the increase in height of gates, thickness of floor, and cost of superstructure above floor level.

(*ii*) **Afflux**. It was defined earlier that the rise in the maximum flood level of the river upstream of the weir after construction is known as afflux. This afflux is confined only to a short reach (equal to the length of the back water curve, in the beginning, but extends gradually very far up, as explained earlier.

The amount of afflux will determine the top levels of guide banks and marginal bunds. The lengths and sections of marginal bunds are also dependent upon it. It will govern the dynamic action downstream of the work as well as the depth and location of the hydraulic jump. *By providing a higher afflux, the waterway and, therefore, the length of the weir can be reduced, but it will increase the cost of training works and the risk of failure by outflanking.* At the same time, the discharge intensity and the consequent scour shall go up, and hence, the *sections of loose protections upstream and downstream as well as the depths of pile lines at either ends shall have to be increased,* thereby making it costly. *It is, therefore, always desirable to limit the afflux to a safe value of*

1.0 to 1.2 metres, more commonly 1.0 metre. However, in steep reaches with rocky bed, a higher value of afflux may be permitted.

(*iii*) **Waterway and discharge per metre.** The waterway and afflux are correlated. If afflux is increased, waterway is reduced and *vice-versa*. Hence, a limit placed on maximum afflux shall limit the minimum waterway. It shall be seen that the cost of works as a whole is minimum for a certain waterway and afflux. Attempts should, therefore, be made to attain the most economical combination of these two factors. This can be made by trial and error, generally limiting the maximum value of afflux.

A likely figure for the waterway is obtained by Lacey's wetted perimeter formula, given by eq. (4.18) as

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

A waterway equal to 1.2 to 1.4 P is generally assumed in rivers in plains. Some engineers preferred to keep a shorter waterway inspite of costlier works, as it was thought that a shorter waterway reduces shoaling, but as a matter of fact, it is not so.

(*iv*) **Pond levels.** The pond level is the minimum water level required in the undersluice pocket upstream of the canal head regulator, so as to feed the canal with its full supply. *The pond level is generally obtained by adding 1.0 to 1.2 metres to the canal FSL.*

Water in the undersluice pocket has to be maintained at Pond Level, even during dry weather flow. This can be accomplished either by a raised crest or by shutters or by a combination of both. A permanently raised crest will lead to higher afflux during floods and is likely to result in loss of control on the river.

In modern design of a barrage, the entire ponding is done by gates which are opened during floods and the crest level of the undersluices is generally taken as the available river bed level in the deepest channel. *No raised crest is thus generally provided for the undersluices*. A raised crest is provided where possible, as it improves the coefficient of discharge.

The **discharge formulas** to be used in the design of a weir or a barrage are

(*a*) **For a broad crested weir :**

$$Q = 1.7\,(L - Kn.H)\,H^{3/2} \qquad ...(11.16)$$

where Q = Discharge in cumecs

H = Total head in metres including velocity head

n = No. of end contractions (Twice the number of gated bays)

L = Clear waterway length in metres.

K = coefficient of end contraction varying from 0.1 for thick blunt peir noses to 0.04 for thin pointed noses : generally taken as 0.1 in ordinary calculations.

(*b*) **For a sharp crested weir :**

$$Q = 1.84 \cdot (L - KnH)\,H^{3/2}.$$

Q, L, n, K and H have the same meanings as given above.

If the head over the weir crest is more than 1.5 times the width of the weir, the weir behaves as a *sharp crested weir.*

Concentration factor. While calculating the cistern levels and the depths of cutoffs, the possibility of non-uniform flow is taken into account by providing a suitable concentration factor. This factor is chosen arbitrarily. Generally, a 20% concentration is taken at any particular section. Hence, the calculated maximum discharge intensity is increased by 20% in designs. No concentration of flow is taken while designing protection works.

Example on the design of a Barrage

Example 11.6. *A barrage is to be constructed on a river having a high flood discharge of about 8,100 cumecs, with the given data as follows :*

Average bed level of the river	= *257.0 m*
High Flood Level (before construction of barrage)	= *262.2 m*
Permissible afflux	= *1.0 m*
Pond Level	= *260.6 m.*

Stage Discharge curve for the river at the barrage site is given in Fig. 11.19. Prepare a complete hydraulic design for the undersluice section as well as for other barrage bay section, on the basis of Hydraulic jump theory and Khosla's theory. A safe exit gradient of 1/6 may be assumed. 0.5 metres retrogression and 20% discharge concentration may be assumed where non-uniform flow is likely to occur. Assume any other data if not given.

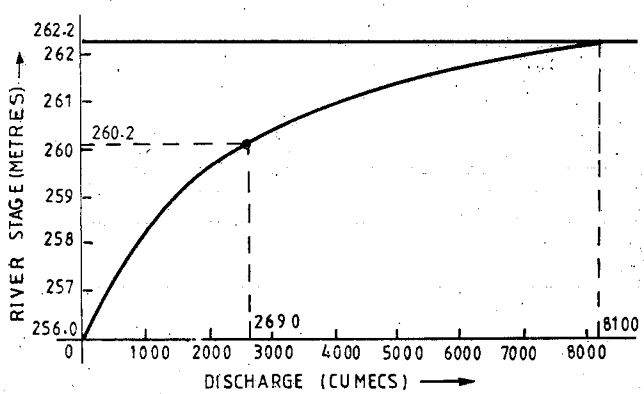


Fig. 11.19

Solution. Fixing the Crest Levels and Waterway

Crest Levels. The average bed level of the river is given to be 257.0 m, which may be taken as the crest level of the undersluices. The upstream floor of the undersluices may be kept the same, and thus there will be no raised crest in the undersluices. The crest level of other barrage bays may be kept 1.0 to 1.5 m higher than the crest level of undersluices. Let us keep it 1.3 m higher, *i.e.* say $257.0 + 1.3 = 258.3$ m.

Waterway : The waterway, as per Lacey's wetted perimeter equation, is given by

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

$$= 4.75\sqrt{8100} = 4.75 \times 90 = \mathbf{427.5\ m.}$$

Now let us provide a waterway approximately equal to 1.2 P by trial, in such a way that approximately 15 to 20% of the discharge is passed through the undersluices and the total provided waterway should be able to pass the entire discharge.

Assume the waterway as below

(*a*) **Undersluice portion :**

5 bays of 15 m each	75 m
4 piers of 2.5 m each	10 m
Overall waterway	85 m

(*b*) **Other barrage bays portion**

27 bays of 12 m each	324.00 m
25 piers of 2 m each	50.00 m
Overall waterway	374.00 m

Assume a divide wall of 3.0 m thickness. Hence, total overall waterway provided between abutments $= 85 + 374 + 3 = \mathbf{462\ m.}$

Now, let us check whether the maximum flood can pass through this waterway with the maximum permissible afflux of 1.0 metre or not.

HFL before barrage construction = 262.2 m.

Permissible afflux = 1.0 m.

Now u/s HFL = d/s HFL + Afflux

d/s HFL = HFL before weir construction = 262.2 m

∴ u/s HFL = 262.2 + 1.0 = 263.2 m.

$$\text{Average discharge intensity} = q = \frac{8{,}100}{462} = 17.6$$

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

Assume Lacey's silt factor $f = 1.0$

$$R = 1.35\left[\frac{(17.6)^2}{1}\right]^{1/3} = 1.35\,(17.6)^{0.667} = 9.20\text{ m.}$$

$$\text{Velocity of approach} = V = \frac{q}{R} = \frac{17.6}{9.20} = 1.90\text{ m/sec.}$$

$$\text{Velocity Head} = \frac{V^2}{2g} = \frac{(1.90)^2}{2 \times 9.81} = 0.19\text{ m.}$$

u/s TEL = u/s HFL + Velocity Head = 263.2 + 0.19 = 263.39 m.

Head (*i/c* velocity head) over the undersluice crest

= u/s TEL − Undersluice crest level = 263.39 − 257.0 = 6.39 m.

Head (*i/c* velocity head) over the crest of other barrage bays

= u/s TEL − Crest level of barrage bays = 263.39 − 258.3 = **5.09 m.**

Discharge passing through undersluices is given by the discharge formula for a broad crested weir; since the crest and u/s floor are at the same level, and the width of the crest is sufficient, it will behave as a broad crested weir.

$$\therefore \quad Q_1 = 1.7\,(L - K\,n\,H)\,H^{3/2}$$

where L is the clear waterway

Q_1 = Discharge through undersluices

$= 1.7\,[75 - 0.1 \times 10 \times 6.39]\,(6.39)^{3/2}$

$= 1.7\,[75 - 6.39]\,(6.39)^{3/2} = 1.7 \times 68.61 \times 16.1 =$ **1,880 cumecs.**

Let us keep the width of the crest of other barrage bays portion as 2.0 m. Since the head over the other barrage bays crest is 5.09 m., which is more than 1.5 times the width of the crest, it shall behave like a sharp crested weir. The discharge is then given by

$Q = 1.84\,(L - 0.1\,nH)\,H^{3/2}$

$\therefore$ Q_2 = Discharge through other barrage bays

$= 1.84\,(312.0 - 0.1 \times 52 \times 5.09)\,(5.09)^{3/2}$

$= 1.84\,(324.0 - 26.4)\,(5.09)^{3/2}$

$= 1.84 \times 297.6 \times 11.4 =$ **6, 260 cumecs**

Total discharge that can pass through the barrage

$= Q_1 + Q_2 = 1{,}880 + 6{,}260 = 8{,}140$ cumecs

$= 8{,}140$ cumecs $> 8{,}100$ cumecs.

Hence, the assumed waterway and crest levels are in order.

Actual overall waterway provided = 462 m against Lacey's wetted perimeter of 427.5 m.

$\therefore$ Looseness factor $= \dfrac{462}{427.5} =$ **1.08**

The design of undersluice section and that of other barrage bays section shall be carried out separately.

Design of Undersluice Portion

There are two major flow conditions : (*i*) When high flood is passing ; and (*ii*) When flow is at Pond Level (with all gates open). Let us calculate q and H_L for these two conditions.

(1) High flood condition

(*a*) *Assuming no concentration and retrogression*

u/s TEL = d/s HFL + Afflux + Velocity head = 262.2 + 1.0 + 0.19 = 263.39 m.

d/s TEL = d/s HFL + Velocity head = 262.2 + 0.19 = 262.39 m.

Head Loss $= H_L = 263.39 - 262.39 =$ **1.0 m**

Discharge intensity between piers = q

$= CH^{3/2} = 1.7\,(6.39)^{3/2} \simeq 1.7 \times 16.1 = 27.4$ cumecs/metre.

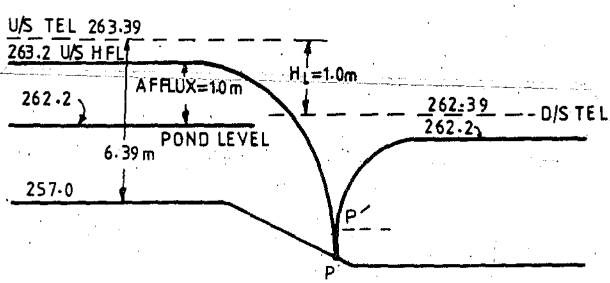


Fig. 11.20 (*a*) High Flood condition with no retrogression.

(*b*) *High flood condition with 20% concentration and 0.5 m retrogression*

The discharge intensity is increased by 20%. Therefore, new discharge intensity

$$= 1.2 \times 27.4 = 32.9 \text{ cumecs/metre}$$

New head (*i/c* velocity head) required for this discharge intensity to pass

$$= \left(\frac{32.9}{1.7}\right)^{2/3} = (19.5)^{0.667} = \mathbf{7.2\ m.}$$

The conditions are shown in Fig. 11.20 (*b*).

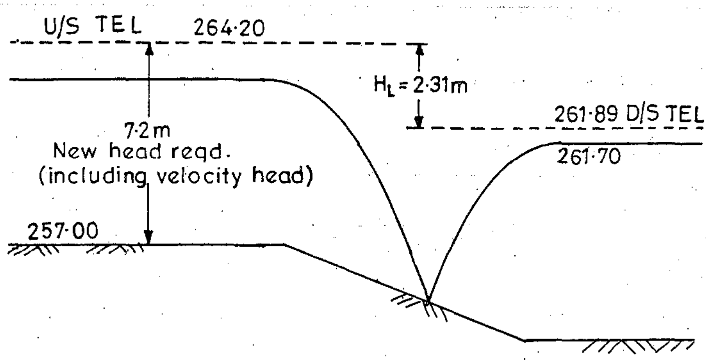


Fig. 11.20 (*b*) High flood condition with 20% concentration and 0.5 m retrogression.

$$\text{u/s TEL} = 257.0 + 7.2 = 264.2 \text{ m.}$$

The d/s HFL is depressed by 0.5 m due to retrogression, *i.e.* it becomes 261.7 m

$\therefore$ d/s TEL = 261.7 + 0.19 = 261.89 m

$\therefore$ $H_L = 264.20 - 261.89 = \mathbf{2.31\ m.}$

(2) Pond Level Flow Condition

(*a*) *With no concentration and retrogression*

Pond Level (given) = 260.6 m.

Head over the crest of undersluices under this condition

$$= 260.6 - 257.0 = 3.6 \text{ m}$$

Head over the crest of other barrage bays

$$= 260.6 - 258.3 = 2.3 \text{ m.}$$

Neglecting velocity of approach for this flow condition, the total discharge passing down the barrage

$$Q = Q_1 + Q_2$$
$$= 1.7\,[75 - 0.1 \times 10 \times 3.6]\,(3.6)^{3/2} + 1.84\,[324 - 0.1 \times 52 \times 2.3]\,(2.3)^{3/2}$$
$$= 1.70 \times 71.4 \times 6.81 + 1.84 \times 312 \times 3.26$$
$$= 825 + 1865 = \mathbf{2690\ cumecs.}$$

$$\text{Average discharge intensity} = \frac{2690}{462} = 5.82 \text{ cumecs/metre}$$

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

$$= 1.35 \times \left[\frac{(5.82)^2}{1}\right]^{1/3} \doteq 1.35 \times (5.82)^{0.667} = 4.45 \text{ m}$$

Velocity of approach

$$= V = \frac{q}{R} = \frac{5.82}{4.45} = 1.31 \text{ m/sec.}$$

$$\text{Velocity head} = \frac{V^2}{2g} = \frac{(1.31)^2}{2 \times 9.81} = 0.088 = \textbf{say 0.09 m.}$$

These conditions are shown in Fig. 11.21 (*a*).

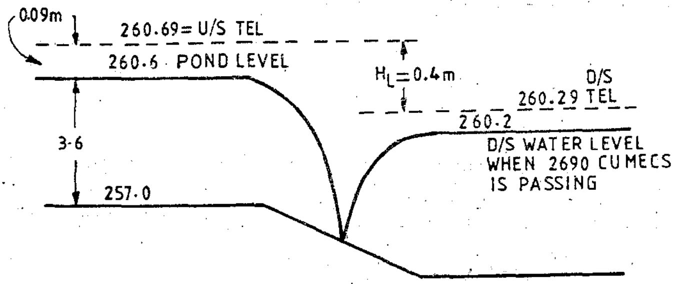


Fig. 11.21 (*a*) Pond level with no concentration and retrogression.

$\therefore$ d/s TEL $= 260.6 + 0.09 = 260.69$ m.

The d/s water level when a discharge of 2,690 cumecs is passing can be found from the Stage-Discharge curve of the river, given in Fig. 11.19. It is found to be 260.2 m.

$\therefore$ d/s TEL $= 260.2 + 0.09 = 260.29$ m.

$H_L = 260.69 - 260.29 = \mathbf{0.40\ m.}$

Discharge intensity between piers $= 1.7 \times (3.6)^{3/2}$

$= 1.7 \times 6.81 = 11.54$ cumecs/metre.

(*b*) *Pond Level flow with 20% concentration and 0.5 m retrogression.*

New discharge intensity $= 1.2 \times 11.54 =$ **13.86** cumecs/metre

New head reqd. (*i/c* velocity head) over the crest $= \left(\frac{13.86}{1.7}\right)^{2/3} = 4.06$ m.

U/s TEL $= 257.0 + 4.06 = 261.06$ m.

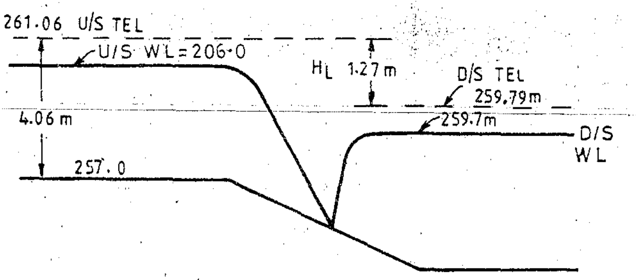


Fig. 11.21 (*b*) Pond level flow with 20% concentration and 0.5 m retrogression.

d/s HFL which was 260.2 m is depressed by 0.5 m. The new d/s HFL shall be 259.7 m.

$\therefore$ d/s TEL = 259.7 + 0.09 = 259.79 m.

$\therefore$ H_L = 261.06 − 259.79 = **1.27 m.**

The values of q, H_L, the water levels and energy levels for all these four cases are tabulated in Table 11.9.

Table 11.9. Undersluice Portion of Barrage

S. No.	Item	*High Flood Flow*		*Pond Level Flow*	
		condition 1(a) without concentration and retrogression	*condition 1(b) with concentration and retrogression*	*condition 2(a) without concentration and retrogression*	*condition 2(b) with concentration and retrogression*
(1)	(2)	(3)	(4)	(5)	(6)
1.	Discharge intensity (q) in cumecs/metre	27.4	32.7	11.54	13.86
2.	Upstream water level	263.2 m	263.2 m	260.60 m	260.6 m
3.	Downstream water level	262.2 m	261.7 m	260.20 m	259.7 m
4.	U/s TEL	263.39 m	264.20 m	260.69 m	261.06 m
5.	D/s TEL	262.39 m	261.89 m	260.29 m	259.79 m
6.	Head loss H_L	1.0 m	2.31 m	0.40 m	1.27 m
7.	E_{f2} (from Plate No. 10.1)	7.40 m	9.00 m	4.00 m	5.00 m
8.	Level at which jump will form, *i.e.* (d/s TEL − E_{f2})	254.99 m	252.89 m	256.29 m	254.79 m
9.	$E_{f1} = E_{f2} + H_L$	8.40 m	11.31 m	4.40 m	6.27 m
10.	y_1 corresponding to E_{f1} (Plate No. 10.2)	2.4 m	2.5 m	1.65 m	1.30 m
11.	y_2 corresponding to E_{f2} (Plate No. 10.2)	6.40 m	8.0	3.30 m	4.5 m
12.	Length of concrete floor required = 5 $(y_2 - y_1)$	20.0 m	27.5 m	8.25 m	16.00 m
13.	Froude No. $F_1 = \frac{q}{\sqrt{g y_1^3}}$	2.36 m	2.66 m	1.74 m	2.97 m

All these four cases of jump formation are separately analysed. Using Blench curves, the values of $E_{f2} \cdot E_{f1}$, position of jump point P, y_2, y_1, etc. are all calculated as explained earlier and shown in Table 11.9. The table is self-explanatory. It can be seen from this Table 11.9, that the maximum value of 5 $(y_2 - y_1)$ is 27.5 metres for the worst Case 1 (*b*), *i.e.* high flood flow with concentration and retrogression. Hence, we provide a *slightly conservative value of 29 m as the length of the downstream floor.* The lowest level at which jump will form, from this table, is 252.89 m [Case I (*a*)] and hence, we provide the downstream floor at a level of say *252.7 metres* (slightly lower than the calculated value of 252.89).

Hence, the d/s floor is provided at R.L. 252.7 m and equal to 29 metres in length, as shown in Fig. 11.22. The glacis is provided in 3 : 1 slope with a horizontal length of 12.9 metres, as shown in Fig. 11.22. Let us now calculate the total floor length required

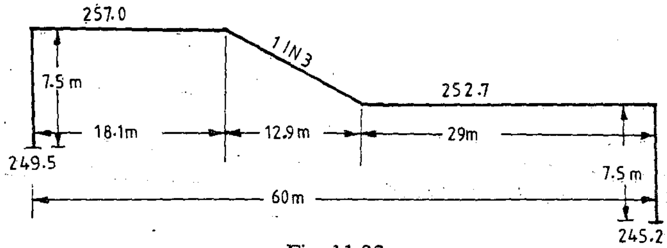


Fig. 11.22

from Khosla's Safe Exit Gradient Theory, so that the balance length of floor can be provided on the u/s side. Before this is calculated, we shall have to decide the depth of d/s cut off provided from scour considerations.

Depth of Sheet Pile Lines from Scour Considerations

Total discharge passing through undersluices = 1,880 cumecs (calculated earlier)

Overall waterway of undersliuces = 85 m.

Average discharge intensity $= \dfrac{1880}{85} = 22.3$ cumecs/metre.

Depth of scour

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

$$= 1.35\left[\left(\frac{22.3}{1}\right)^2\right]^{1/3} = 1.35(22.3)^{2/3} = 1.35 \times 8 = 10.8 \text{ m, say } 11 \text{ m.}$$

Let us provide a downstream cut-off at 1.5 R below the d/s water level (which is 261.7 m with retrogression). Hence, the R.L. of bottom of d/s cut-off

$$= 261.7 - 1.5 \times 11.0 = 261.7 - 16.5 = 245.2 \text{ m.}$$

Let us provide the d/s cut-off upto a bottom level of 245.2 m, i.e. a depth of 7.5 metres, as shown in Fig. 11.22.

U/s cut-off. Let us provide u/s cut-off at depth of 1.25 R (*i.e.* $1.25 \times 11.0 = 13.75$ m) from top of u/s water level.

Level of bottom of u/s cut-off $= 263.20 - 13.75 = 249.45$ m.

Let us, therefore, provide the u/s cut-off up to a bottom level of 249.5 m, *i.e.* 7.5 m deep, as shown in Fig. 11.22.

Total Floor length and Exit Gradient

Safe-exit gradient $(G_E) = \dfrac{1}{6}$

Maximum Static Head $(H) = 260.6 - 252.7 = \mathbf{7.9\ m}$

Depth of d/s cut-off $(d) = 252.7 - 245.2 = \mathbf{7.5\ m}$

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

$$\therefore \quad \frac{1}{6} = \frac{7.9}{7.5} \cdot \frac{1}{\pi\sqrt{\lambda}}$$

$$\frac{1}{\pi\sqrt{\lambda}} = \frac{7.5}{7.9} \times \frac{1}{6} = 0.158$$

From Plate 11.2, value of α for a value of $\frac{1}{\pi\sqrt{\lambda}}$ as 0.158, comes out to be approximately 8.0.

$$\therefore \quad b = \alpha \cdot d = 8 \times 7.5 = \mathbf{60\ m.}$$

Hence a balance of 18.1 m length can be provided as u/s floor length (shown in Fig. 11.22) so as to make the total floor length equal to 60.0 metres

Uplift Pressures

To calculate uplift pressures from Khosla's theory, let us first assume floor thickness at u/s cut-off as 1.0 m and at d/s cut-off as 1.5 m. Fig. 11.22 then becomes as shown in Fig. 11.23. No intermediate cut-off is necessary and hence not provided.

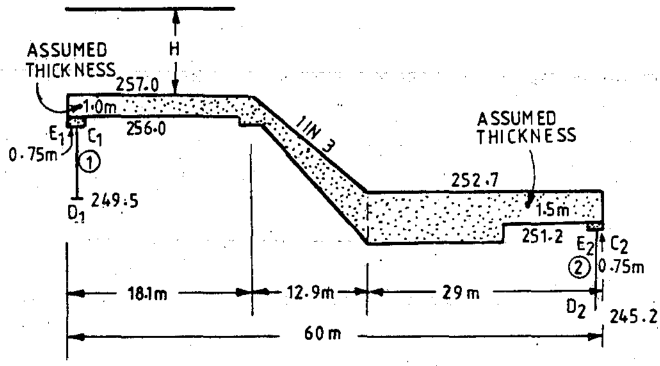


Fig. 11.23

Upstream Pile No. (1)

$$b = 60\ \text{m}$$

$$d = 7.5\ \text{m}$$

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{7.5}{60} = 0.125$$

From Plate 11.1 (a)

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

$$\phi_{C_1} = 100 - \phi_E = 100 - 32 = 68\%$$

$$\phi_{D_1} = 100 - \phi_D = 100 - 22 = 78\%$$

D/s Pile Line

$$d = 7.5\ \text{m}$$

$$b = 60\ \text{m}$$

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{7.5}{60} = 0.125$$

From Plate 11.1 (a)

$$\phi_{C_2} = 0\%$$

$$\phi_{E_2} = \phi_E = 32\%$$

$$\phi_{D_2} = \phi_D = 22\%$$

Let us correct these pressures

$$\phi_{C_1} = 68\%$$

$$\phi_{E_2} = 32\%$$

Corrections to ϕ_{C_1} :

(*i*) *Effect of sheet pile No. (2) on pile No. (1) of depth d*

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

where $D =$ depth of pile (2) below the point C_1, *i.e.* the point at which interference is desired

$= 256.0 - 245.2 = \mathbf{10.8\ m}$

$d = 256.0 - 249.5 = 6.5$ m

$b' = 58.5$ m ; $b = 60$ m

$$\text{Correction} = 19 \times \sqrt{\frac{10.8}{58.5}}\left[\frac{6.5+10.8}{60}\right] = 19 \times \frac{1}{2.33} \times \frac{17.3}{60} = 2.35\%\ (+\text{ ve})$$

(*ii*) *Correction for depth ;*

$$\text{Correction} = \frac{78\% - 68\%}{257.0 - 249.5} \times 1.0 = \frac{10}{7.5} \times 1.0 = 1.33\%\ (+\text{ ve})$$

ϕ_{C_1} (corrected) $= 68\% + 2.35\% + 1.33\% = \mathbf{71.68\%}$.

Corrections to ϕ_{E_2}

(*i*) *Effect of sheet pile No. (1) on pile No. (2) of depth d*

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

where $D = 251.2 - 249.5 = 1.7$ m

$d = 251.2 - 245.2 = 6.0$ m

$b' = 58.5$ m

$b = 60$ m

$$\text{Correction} = 19 \times \sqrt{\frac{1.7}{58.5}}\left(\frac{6.0+1.7}{60.0}\right) = 19 \times \frac{1}{5.86} \times \frac{7.7}{60} = 0.42\%\ (-\text{ ve})$$

Table 11.10

Condition of flow	*u/s water level in metres*	*d/s water level in metres*	*Head in metres (H)*	*Height/Elevation of Subsoil H.G. Line above Datum*					
				Upstream Pile Line No. (1)			*Downstream Pile Line No. 2*		
				ϕ_{E_1} *100%*	ϕ_{D_1} *78%*	ϕ_{C_1} *= 71.68%*	ϕ_{E_2} *= 29.58%*	ϕ_{D_2} *22%*	ϕ_{C_2} *0%*
				7.9	6.17	5.66	2.33	1.74	0
No flow, maximum static Head	260.6	252.7 (No water d/s)	7.9	260.6	258.87	258.36	255.03	254.44	252.70
				1.5	1.17	1.07	0.44	0.33	0
(High Flood with concentration and retrogression)	263.2	261.7	1.5	263.2	262.87	262.77	262.14	262.03	261.7
				0.9	0.70	0.64	0.27	0.20	0
Flow at pond level (with concentration and retrogression)	260.6	259.7	0.9	260.6	260.40	260.34	259.97	259.9	259.7

(*ii*) Correction due to thickness of floor :

$$\text{Correction} = \left(\frac{32\% - 22\%}{252.7 - 245.2}\right)1.5$$

$$= \frac{10}{7.5} \times 1.5 = 2.0\% \ (-\text{ve})$$

$$\phi_{E_2} \ (\text{Corrected}) = 32\% - 0.42\% - 2.0\% = \mathbf{29.58\%}$$

The levels of H.G. line at key points for various flow condition are tabulated in Table 11.10 as explained earlier.

We shall now determine the *hydraulic jump profiles* for the two flow conditions, *i.e.* High flood flow with concentration and retrogression, and pond level flow with concentration and retrogression.

(*a*) **Pre-jump profile** : It is worked out in Table 11.11 as explained earlier.

Table 11.11. Pre-jump Profile Calculations

Distance from the d/s end of the crest, i.e. the start of glacis, in metres	*Glacis level in metres*	*High Flood Flow q = 32.9 cumecs/metre*		*Pond Level Flow q = 13.86 cumecs/metre*	
		E_{f_1} *u/s TEL – Glacis Level, i.e.* 264.20 – *col.* (2)	y_1 *from Plate 10.2*	E_{f_1} *u/s TEL – Glacis Level, i.e.* 261.06 – *Col.* (2)	y_1 *from Plate 10.2*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
0	257.0	7.2	—	4.06	—
3	256.0	8.2	3.5	5.06	1.8
6	255.0	9.2	3.0	6.06	1.4
6.63	254.79 Point at which jump is formed at Pond Level	9.41	2.9	6.27	1.3
9.00	254.00	10.20	2.7		
12.33	252.89 Point at which jump is formed at high flood	11.31	2.5		

(*b*) **Post jump Profile**

From Table 11.9

Froude No. for high flood condition, F $= 2.66$, or $F^2 = 7.1$

Depth y_1 " " " $= 2.5$ m.

Froude No. for pond level flow condition $= 2.97$, or $F^2 = 8.8$

Depth y_1 " " " $= 1.3$ m.

From Plate 10.3 (*a*), the Table 11.12 is worked out :

Table 11.12. Post Jump Profile Calculations.

$\frac{x}{y_1}$ on Plate 10.3 (a)	High Flood Flow $F^2 = 7.1$, $y_1 = 2.5$ m			Pond Level Flow $F^2 = 8.8$, $y_1 = 1.3$ m		
	$\frac{y}{y_1}$ from Plate 10.3 (a)	y	$x =$ Col. (1) × 2.5	$\frac{y}{y_1}$ from Plate 10.3 (a)	y	$x =$ Col. (1) × 1.3
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1.2	1.3	2.85	2.5	1.3	1.69	1.3
2.5	1.9	4.75	6.25	1.9	2.47	3.25
5.0	2.5	6.25	12.5	2.6	3.38	6.5
10.0	3.0	7.5	25.0	3.2	4.16	13.0

The Hydraulic jump profiles for these two flow conditions are plotted along with the H.G. lines for their respective conditions, as shown in Figs. 11.24 (*a*) and (*b*). From these

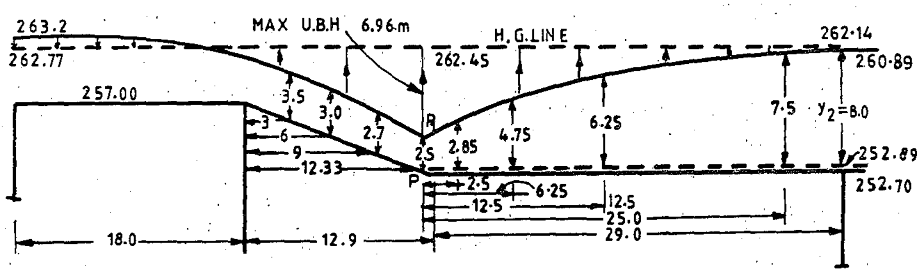


Fig. 11.24. (*a*). Unbalanced head in jump trough at high flood flow.

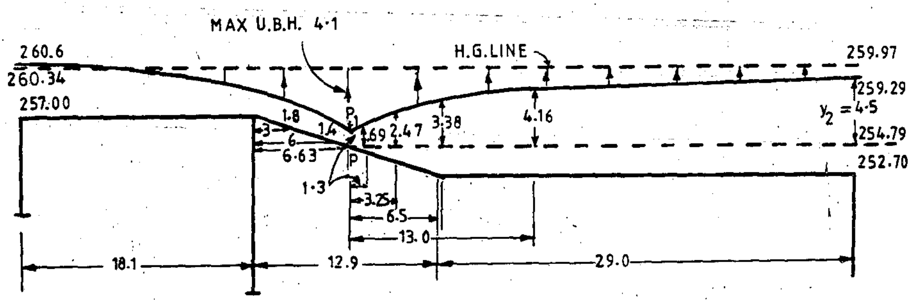


Fig. 11.24. (*b*). Unbalanced head in jump trough at pond level flow.

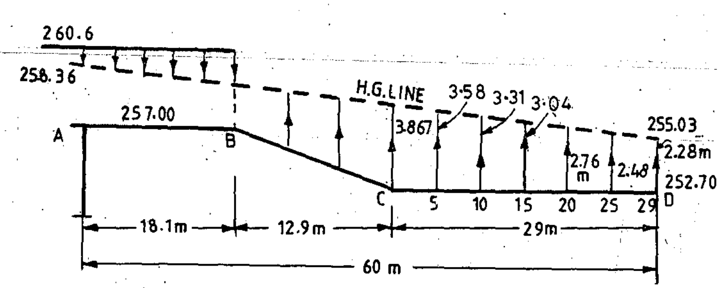


Fig. 11.24. (*c*). Unbalanced head in jump trough at static head at pond level.

figures, it is found that the maximum unbalanced head in the jump trough is equal to 6.96 m. The thickness for glacis shall, therefore, be designed for 2/3rd of this head, *i.e.* equal to 4.64 *metres*, or for the static condition head, whichever is greater. The H.G. line for the static head at pond level is also plotted in Fig. 11.24 (*c*).

It can be seen from the study of all these three figures, *i.e.* Fig. 11.24 (*a*), (*b*) and (*c*), that the dynamic action (*i.e.* flow) governs the floor thickness upto a length of about 5 m beyond the toe of the glacis, and the static head governs the rest of the downstream floor thickness.

Thickness required at point of jump

$$= \frac{4.64}{1.24} = 3.74 \text{ m}$$

Thickness required at 5 m beyond toe

$$= \frac{3.58}{1.24} = 2.88 \text{ m}$$

Thickness required at 15 m beyond toe

$$= \frac{3.04}{1.24} = 2.45 \text{ m}$$

Thickness required at 20 m beyond toe

$$= \frac{2.76}{1.24} = 2.23 \text{ m}$$

Thickness required at 25 m beyond toe

$$= \frac{2.48}{1.24} = 2.0 \text{ m}$$

Thickness required at 29 m beyond toe

$$= \frac{2.28}{1.24} = 1.84 \text{ m}$$

Provide floor thicknesses as shown in Fig. 11.25.

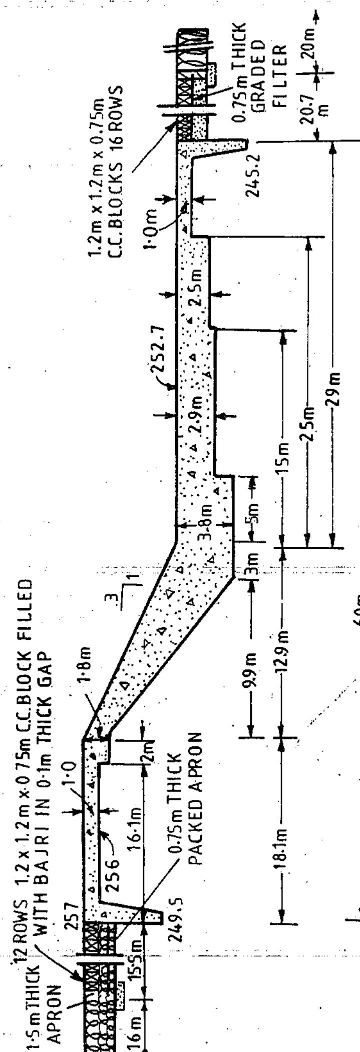


Fig. 11.25. Section of undersluice portion of Barrage.

Protection Works

Downstream Protection

Normal scour depth $= R = 11.0$ m

$$D = 2R - y = 2 \times 11.0 - (261.7 - 252.7)$$
$$= 22 - 9 = 13 \text{ m.}$$

Provide a launching apron equal $1.5D$, *i.e.* say 20 m in length and of such a thickness (t) that its quantity is approximately $2.25\ D$ cu. m/m.

$$\therefore \quad t = \frac{2.25 \times 13.0}{20} = 1.46 \text{ m} \text{ ; } \textbf{Say 1.5 m thick.}$$

Let us provide c.c. blocks of size $1.2 \text{ m} \times 1.2 \text{ m} \times 0.75$ m over an inverted filter of 0.75 m thickness for a length equal to say $1.5\ D$, *i.e.* approximately 20 m. 16 rows of c.c. blocks of size $1.2 \text{ m} \times 1.2 \text{ m} \times 0.75$ m having 10 cm gaps filled with bajri, shall be provided in length equal to 20.7 m, as shown in Fig. 11.25.

Upstream Protection

Normal scour depth $= R = 11.0$

Anticipated scour = 1.5 R = 16.5 m

$$D = 1.5\,R - y$$
$$= 1.5 \times 11.0 - (263.2 - 257.0) = 16.5 - 6.2 = 10.3 \text{ m}$$

Provide a launching apron of 1.5 m thickness in a length

$$= \frac{2.25 \times 10.3}{1.5} = 15.5 \text{ ; } \textbf{say 16 m in length.}$$

Let us provide c.c. blocks of size 1.2 m × 1.2 m × 0.75 m over packed apron of 0.75 m thickness for a length equal to say 1.5 D, *i.e.* 15.5 m in length. 12 rows of c.c. blocks of size 1.2 m × 1.2 m × 0.75 m having 10 cm gaps filled with bajri shall be provided in length equal to 15.5 m, as shown in Fig. 11.25.

Design of Other Barrage Bays Portion

Here the crest level is 258.3 m. The four cases are analysed as given below :

Condition 1 (a) High flood flow with no concentration and retrogression.

u/s water level = 263.2 m
d/s water level = 262.2 m
u/s TEL = 263.39 m
d/s TEL = 262.39 m

$$H_L = 263.39 - 262.39 = 1.0 \text{ m.}$$

Head, including velocity head, over the crest

$$= 263.39 = 258.3 = 5.09 \text{ m}$$

Discharge intensity

$$q = 1.84\,(5.09)^{3/2} = 1.84 \times 11.4 = 21 \text{ cumecs/metre.}$$

Condition 1 (b) High flood flow with 20% concentration and 0.5 m retrogression

New discharge intensity

$$= 1.2 \times 21 = 25.2 \text{ cumecs/metre}$$

New head required, including velocity head, for this discharge to pass.

$$= \left[\frac{25.2}{1.84}\right]^{2/3} = (13.7)^{0.667} = 5.73 \text{ m.}$$

u/s TEL = 258.3 + 5.73 = 264.03 m
u/s TEL = 261.89 m

$$H_L = 264.03 - 261.82 = 2.14 \text{ metres.}$$

Condition 2 (a) : Pond level flow with no concentration and retrogression.

u/s W.L. = Pond Level = 260.6 m
d/s W.L. = 260.20 m
u/s TEL = 260.69 m
d/s TEL = 260.29 m

$$H_L = 260.69 - 260.29 = 0.4 \text{ m}$$

Head, including velocity head, over the crest = 260.69 − 258.3 = 2.39 m

Discharge intensity = 1.84 $(2.39)^{3/2}$ = 1.84 × 3.69 = 6.78 cumecs/metre.

Condition 2 (b) : Pond level flow with 20% concentration and 0.5 m retrogression.

New discharge intensity = 1.2 × 6.78 = 8.13 cumecs/metre.

New head required, including velocity head, for this discharge to pass

$$= \left[\frac{8.13}{1.84}\right]^{3/2} = (4.42)^{0.667} = 2.69 \text{ m}$$

$$\text{u/s TEL} = 258.3 + 2.69 = 260.99 \text{ m}$$

$$\text{d/s TEL} = 260.29 - 0.5 = 259.79 \text{ m}$$

$$H_L = 260.99 - 259.79 = \mathbf{1.2\ m.}$$

These values are tabulated in Table 11.13 and the table is completed as shown. The lowest level at which jump will form is found from this table as 254.39 m. Hence,

Table 11.13. Other Barrage Bays Portion

S. No.	Item	*High Flood Flow*		*Pond level Flow*	
		Condition 1 (a) without concentration and retrogression	*Condition 1 (b) with 20% concentration and 0.5 m retrogression*	*Condition 2 (a) without concentration and retrogression*	*Condition 2 (b) with 20% concentration and 0.5 m retrogression*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
1.	Discharge intensity q in cumec/metre	21.0	25.2	6.78	8.13
2.	Upstream water level	263.2 m	263.2 m	260.6 m	260.6 m
3.	Downstream water level	262.2 m	262.2 m	260.2 m	259.7 m
4.	u/s TEL	263.39 m	264.03 m	260.69 m	260.99 m
5.	d/s TEL	262.39 m	261.89 m	260.29 m	259.79 m
6.	Head loss H_L	1.0 m	2.14 m	0.4 m	1.2 m
7.	E_{f1} from plate 10.1	6.3 m	7.5 m	3.0 m	3.7 m
8.	Level at which jump will form, *i.e.* d/s TEL $- E_{f2}$	256.09 m	**254.39 m***	257.29 m	256.09 m
9.	$E_{f1} = E_{f2} + H_L$	7.3 m	9.64 m	3.4 m	4.9 m
10.	y_1 corresponding to E_{f1} (plate 10.2)	2.1 m	1.9 m	1.1 m	0.9 m
11.	y_2 corresponding to E_{f2} (plate 10.2)	5.7 m	6.8 m	2.7 m	3.3 m
12.	Length of concrete floor required, *i.e.* $5\,(y_2 - y_1)$	5 (3.6) = 18.0 m	5 (4.9) = 24.5 m.	5 (1.6) = 8.0 m.	5 (2.4) = 12.0 m
13.	Froude No. $F_1 = \dfrac{q}{\sqrt{g y_1^3}}$	2.52	3.09	1.91	3.04 m

*Lowest point at which jump will form.

provide the downstream floor at level, say 254.2 m. (slightly lower than the calculated value of 254.39 m), and equal to *26 metres* in length. The glacis is provided in 3 : 1 slope with a horizontal length of $3\,(258.3 - 254.2) = 3 \times 4.1 = 12.3$ *metres*. Crest width provided is 2 metres and upstream glacis in a slope of 1 : 1 with a horizontal length equal to $258.3 - 257.0 = 1.3$ metres, as shown in Fig. 11.26.

Depth of Sheet Piles from Scour

Discharge passing = 6,260 cumecs

Overall waterway = 374 metres

$$\text{Average discharge intensity} = \frac{6{,}260}{374} = 16.8 \text{ cumecs/metre}$$

$$R = 1.35\left[\frac{(16.8)^2}{1}\right]^{1/3} = 1.35 \times 6.6 = 9.07 \text{ m.}$$

Let us provide a downstream cutoff upto depth 1.5 R below the d/s water level which is 261.7 m with retrogression. Hence, the R.L. of bottom of d/s cut-off

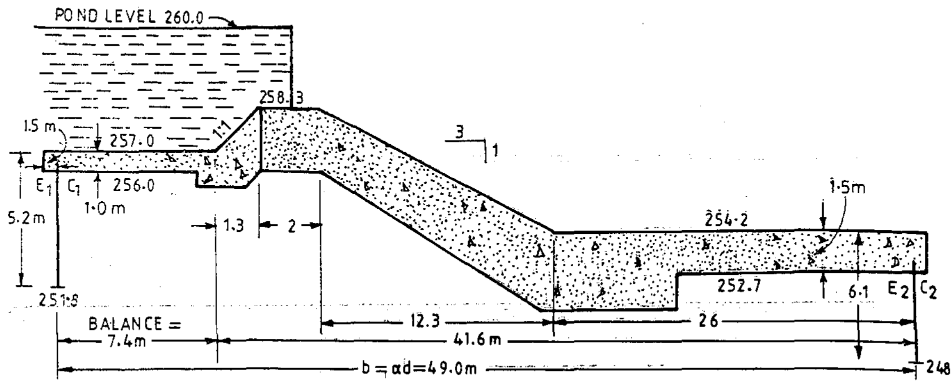


Fig. 11.26

$$= 261.7 - 1.5 \times 9.07$$
$$= 261.7 - 13.6 = 248.1 \text{ m}.$$

Let us provide d/s cut-off upto a bottom level of 248.1 m, *i.e.* depth of 6.1 m, as shown in Fig. 11.26.

U/s Cutoff. Let us provide 1.25 R, *i.e.*

$$1.25 \times 9.07 = 11.4 \text{ m for u/s cutoff.}$$

∴ The level of bottom of u/s cutoff = 263.2 − 11.4 = 251.8 m.

Let us provide u/s cutoff upto a bottom level of 251.8 m, *i.e.* for a depth of 5.2 m, as shown in Fig. 11.26.

Total Floor Length and Exit Gradient

$$G_E = \frac{1}{6}.$$

Maximum Static Head = H = 260.6 = 254.2 = 6.4 m

Depth of d/s cutoff = d = 254.2 − 248.1 = 6.1 m

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

∴ $$\frac{1}{6} = \frac{6.4}{6.1} \cdot \frac{1}{\pi \cdot \sqrt{\lambda}}$$

or $$\frac{1}{\pi \cdot \sqrt{\lambda}} = \frac{1}{6} \times \frac{6.1}{6.4} = 0.159.$$

From Plate 11.2, value of α for a value of $\frac{1}{\pi \sqrt{\lambda}}$ as 0.159 comes out to be approximately 8.0.

∴ $$b = \alpha d = 8 \times 6.1 = 48.8 \text{ m} \text{ ; say provide } b = 49 \text{ m}.$$

∴ Balance length of 49 − 1.3 − 2.0 − 12.3 − 26.0 = 49 − 41.6 = 7.4 m is provided as upstream floor, as shown in Fig. 11.26.

Uplift Pressures

Let us assume floor thickness as 1.0 m at u/s cutoff end and 1.5 m at d/s cutoff end, as shown in Fig. 11.26.

Upstream Pile No. (1)

$$b = 49 \text{ m}$$

$$d = 5.2 \text{ m}$$

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{5.2}{49.0} = 0.106.$$

From Plate 11.1 (*a*)

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

$$\phi_{C_1} = 100 - \phi_E = 100 - 29 = 71\%$$

$$\phi_{D_1} = 100 - \phi_D = 100 - 20 = 80\%$$

Downstream Pile No. (2)

$$d = 6.1 \text{ m}$$

$$b = 49.0 \text{ m}$$

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{6.1}{49.0} = 0.125$$

From Plate 11.1 (*a*)

$$\phi_{C_2} = 0\%$$

$$\phi_{E_2} = \phi_E = 32\%$$

$$\phi_{D_2} = \phi_D = 22\%$$

Let us correct these pressures

$$\phi_{C_1} = 71\%$$

$$\phi_{E_2} = 32\%$$

Corrections to ϕ_{C_1}

(*i*) *Effect of sheet pile No. (2) on pile No. (1) of depth d*

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

where $d = 256.0 - 251.8 = 4.2$ m
$D = 256.0 - 248.1 = 7.9$ m
$b' = 47.5$ m
$b = 49.0$ m

$$\text{Correction} = 19\sqrt{\frac{7.9}{47.5}}\left(\frac{4.2+7.9}{49}\right)$$

$$= 19 \times \frac{1}{2.45} \times \frac{12.1}{49} = 1.49\% \text{ (+ ve).}$$

(*ii*) Correction for depth of floor

$$\text{Correction} = \frac{80\% - 71\%}{257.0 - 251.8} \times 1.0 = \frac{9}{5.2} \times 1.0 = 1.73\% \text{ (+ ve)}$$

ϕ_{C_1} (*corrected*) $= 71\% + 1.49 + 1.73\% = 74.22\%$

Corrections to ϕ_{E_2}

(i) Effect of sheet pile No. (1) on sheet pile No. (2) of depth d

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

where $d = 252.7 - 248.1 = 4.6$ m

$D = 252.7 - 251.8 = 0.9$ m

$b' = 47.5$ m

$b = 49.0$ m

$$\text{Correction} = 19 \times \sqrt{\frac{0.9}{47.5}}\left(\frac{4.6+0.9}{49}\right) = 0.29\% \;(-\text{ve})$$

(ii) Correction due to thickness of floor

$$\text{Correction} = \frac{32\% - 22\%}{254.2 - 248.1} \times 1.5 = 2.46\% \;(-\text{ve})$$

ϕ_{E_2} *(corrected)* $= 32 - 0.29 - 2.46 = 29.25\%$

The levels of H.G. line at key points for various flow conditions are tabulated in Table 11.14.

Table 11.14

Condition of flow	*U/s water level in metres*	*D/s water level in metres*	*Head in metres H*	*Height/Elevation of sub soil H.G. line above datum*					
				Upstream pile line No. (1)			*Downstream pile line No. (2)*		
				ϕ_{E_1} *100%*	ϕ_{D_1} *80%*	ϕ_{C_1} *74.22%*	ϕ_{E_2} *29.25%*	ϕ_{D_2} *22%*	ϕ_{C_2} *0%*
				6.4	5.12	4.75	1.87	1.41	0
No flow, maximum static head	260.6	254.2 (No water)	6.4	260.6	259.32	258.95	256.07	255.61	254.2
				1.5	1.2	1.11	0.44	0.33	0
High flood flow with concentration and retrogression	263.2	261.7	1.5	263.2	262.90	262.81	262.14	262.03	261.7
				0.9	0.72	0.67	0.26	0.20	0
Flow at pond level with concentration and retrogression	260.6	259.7	0.9	260.60	260.42	260.37	259.96	259.90	259.7

We shall now determine the hydraulic jump profiles. For the two flow conditions. **Pre-jump Profile** is worked out in Table 11.15.

Table 11.15. Pre-jump Profile Calculations

Distance from the start of 3 : 1 glacis	*Glacis level in metres*	*High Flood Flow with concentration and retrogression $q = 25.2$ cumecs/metre*		*Pond Level Flow with concentration and restogression $q = 8.13$ cumecs/metre*	
		E_{f_1} = u/s TEL – glacis level i.e. 264.03 —col. (2)	y_1 From Plate 10.2	E_{f_1} = u/s TEL – glacis level i.e. 260.99 – col. (2)	y_1 From Plate 10.2
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
0	258.3	5.73	—	2.69	—
3	257.3	6.73	3.0	3.69	1.15
6	256.3	7.73	2.25	4.69	1.00
6.63	256.09	7.94	2.2	4.90	0.90
	Point at which jump forms for Pond Level flow				
9.0	255.3	8.73	2.1	—	—
11.73	254.39	9.64	1.9	—	—
	Point at which jump forms for High Flood flow				

Post Jump Profile. If (x, y) are the ordinates of any point taking the point of jump P' as the origin, the profile can be worked out using plate 10.3 (b) as shown in Table 11.16. This profile can also be worked out by using plate 10.3 (a) as was done in the design of undersluice portion (Table 11.12). We have used both these plates just to explain their respective use.

Table 11.16. Post Jump Profile Calculations

Values of x where x is the horizontal distance from the point of jump	*High Flood Flow $y_2 - y_1 = 4.9$, $F_1 = 3.09$*			*Pond Level Flow $y_2 - y_1 = 2.4$, $F_1 = 3.04$*		
	$\frac{x}{y_2 - y_1}$	$\frac{y}{y_2 - y_1}$ *From Plate 10.3 (b)*	y	$\frac{x}{y_2 - y_1}$	$\frac{y}{y_2 - y_1}$ *From Plate 10.3 (b)*	y
2 m	0.41	0.16	0.79	0.83	0.33	0.79
4 m	0.81	0.32	1.57	1.67	0.51	1.22
6 m	1.20	0.38	1.86	2.50	0.68	1.63
8 m	1.61	0.50	2.45	3.33	0.77	1.85
10 m	2.01	0.58	2.84	4.17	0.88	2.11
15 m	3.06	0.76	3.73	6.25	0.98	2.35
20 m	4.08	0.86	4.21	8.25	1.00	2.40
25 m	5.10	0.95	4.67	10.4	—	—

Hydraulic jump profile for two flow conditions, their H.G. lines and the uplift pressure diagrams are plotted in Fig. 11.27 (a) and (b). The H.G. line and the uplift pressure diagram for maximum static head is also plotted in Fig. 11.27 (c). By studying

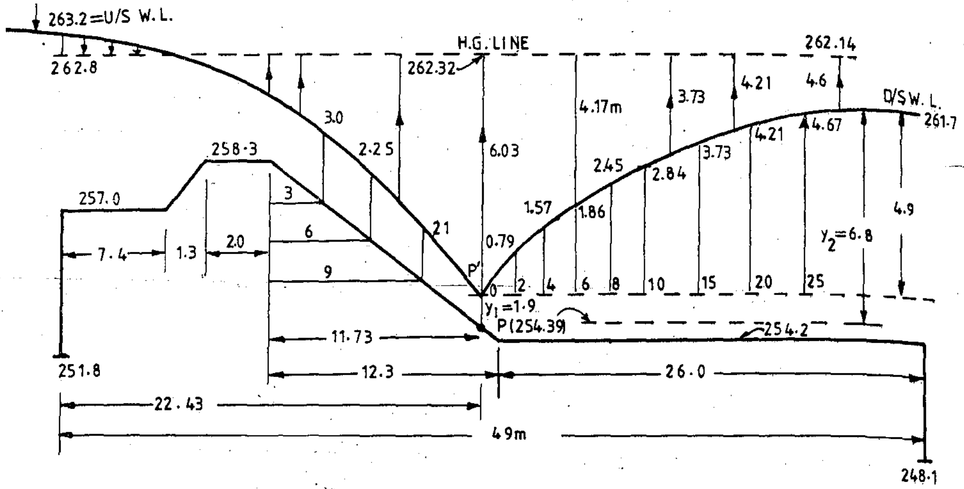


Fig. 11.27. (*a*). Uplift pressures in the jump trough for high flood flow.

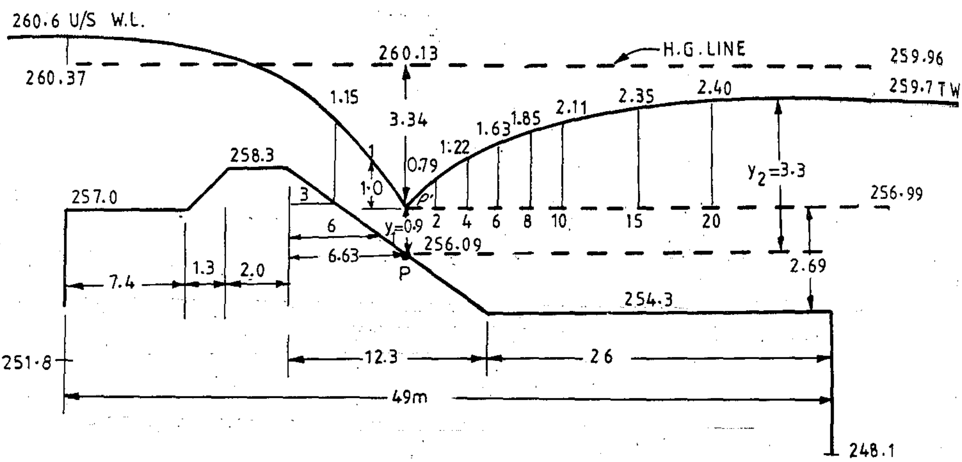


Fig. 11.27. (*b*). Uplift pressures in the jump trough for pond level flow.

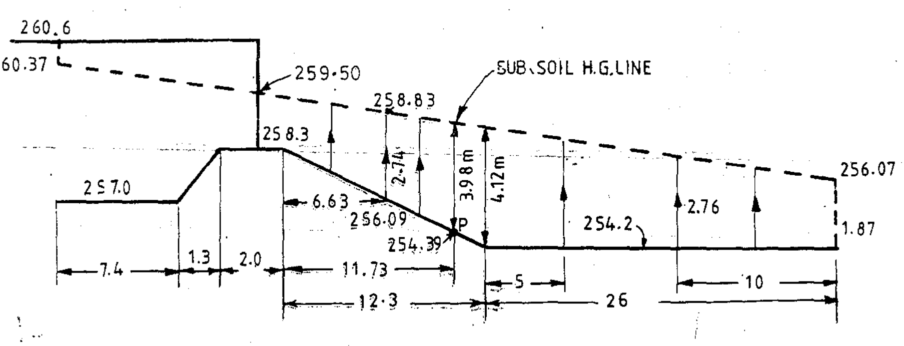


Fig. 11.27. (*c*). Uplift pressures in the jump trough for max. static head at pond level.

these diagrams, it is found that the maximum designed head for dynamic action is $\frac{2}{3} \times 6.03 = 4.02$ m [Fig. 11.27 (*a*)]. The maximum ordinate for uplift in static head condition = 4.12 m [Fig. 11.27 (*c*)]. Hence, static head condition becomes the governing factor. So minimum thickness required.

$$= \frac{4.12}{1.24} = 3.32 \text{ m.}$$

So we *provide 3.5 m.*

For the rest of the downstream floor beyond 5 m of the toe of the glacis, the thickness is decided by dividing the uplift ordinates of Fig. 11.27 (*c*) by 1.24; and by making suitable adjustments, the depths are provided as shown in Fig. 11.28.

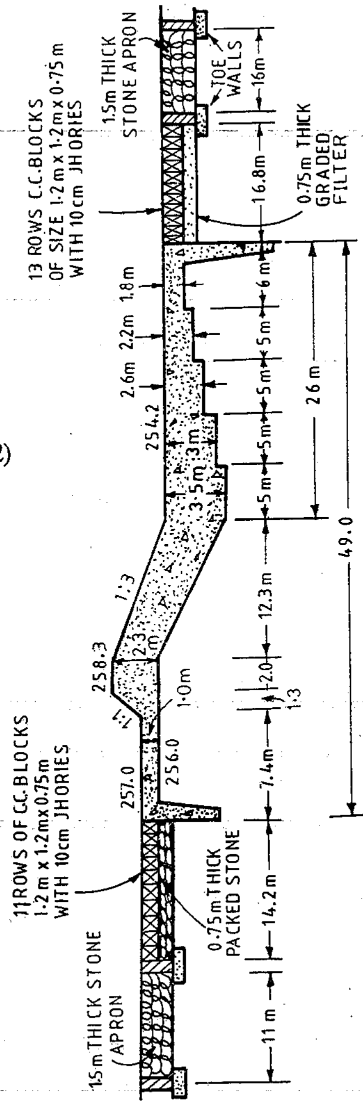


Fig. 11.28. Section of other Barrage Bays

Protection Works

(*i*) *Downstream Protection*

Normal scour depth

$$R = 9.07 \text{ m}$$

$$D = 2R - y = 2 \times 9.07 - (261.7 - 254.2)$$
$$= 18.14 - 7.5 = 10.64 \text{ m}$$

Provide a launching apron equal to 1.5 D, *i.e.* say 16 m in length and of thickness say 1.5 m.

Let us provide C.C. blocks of size 1.2 m × 1.2 m × 0.75 m over a graded filter of 0.75 m thickness for a length equal to 1.5 D, *i.e.* approximately 16 m. 13 rows of C.C. blocks of size 1.2 m × 1.2 m × 0.75 m having 10 cm gaps filled with bajri shall hence, be provided in length equal to 16.8 metres.

(*ii*) *Upstream Protection*

Normal scour depth

$$= R = 9.07 \text{ m}$$

$$D = 1.5\,R - y$$
$$= 1.5 \times 9.07 - (263.2 - 257.0)$$
$$= 13.6 - 6.2 = 7.4 \text{ m.}$$

Provide a launching apron of thickness 1.5 m in a length

$$= \frac{2.25 \times 7.4}{1.5} = 11.1 \text{ m.} = \textbf{Say 11 m.}$$

Let us provide C.C. blocks of size 1.2 m × 1.2 m × 0.75 m over packed stone of 0.75 m thickness for a length equal to say 1.5 D *i.e.* 13.6 m. Hence, provide 11 rows of C.C. blocks of size 1.2 m × 1.2 m × 0.75 m having 10 cm jhories (*i.e.* gaps filled with bajri), in a length equal to 14.2 metres, as shown in Fig. 11.28.

Canal Head Regulator Design

Example 11.7. *Design a suitable head regulator for the barrage designed in example 11.6. The following data for the off-taking canal are also given :*

Full supply discharge of canal =*180 cumecs*
Anticipated maximum full supply level of canal = *260.2 m*
Bed level of canal = *257.2 m*
Safe exit gradient for canal bed material = *1/5.*

Solution. Students are advised to go through the theory for Canal Head Regulator and its design, etc. given in Chapter 9, before solving this problem.

The crest level of the canal head regulator is kept 1.2 to 1.5 m higher than the crest level of the undersluices. The crest level of undersluices (from previous example) = 257.0 m

Pond Level (from previous example, = 260.6 m
u/s H.F.L. (from previous example) = 263.2 m

Let us keep the **crest level** of regulator (*i.e.* sill level) = 257.0 + 1.5 = **258.5 m.**

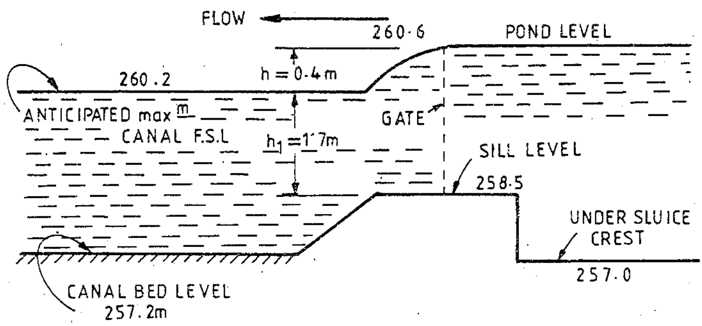


Fig. 11.29.

Let u/s now fix the waterway for the regulator, such that full supply discharge of 180 cumecs can pass through it.

Discharge Q through the regulator is given by

$$\frac{2}{3} C_{d_1} \cdot \sqrt{2g} \cdot B \cdot (h + h_v)^{3/2} + C_{d_2} B \cdot h_1 \sqrt{2g\,(h + h_v)}$$

Taking $C_{d_1} = 0.577$

and $C_{d_2} = 0.80$

and neglecting velocity head h_v, we get

$$Q = \frac{2}{3} \times 0.577 \cdot \sqrt{2g} \cdot B \,.\, h^{3/2} + 0.80 \,.\, B \,.\, h_1 \sqrt{2gh.}$$

Here in this example, we have

$h = 0.4$ m
$h_1 = 1.7$ m
$Q = 180$ cumecs

$$\therefore \quad 180 = \frac{2}{3} 0.577 \sqrt{19.62.} \quad B\,(0.4)^{3/2} + 0.80\,B \,.\, (1.7) \sqrt{19.62 \times 0.4}$$

$$= B\,(1.69 \times 0.253 + 1.36 \times 2.8) = 4.237\,\mathbf{B}$$

or $$B = \frac{180}{4.237} = \mathbf{42.5\ m}$$

Provide 6 bays of 7.5 m each giving a clear waterway of 45 m. Provide 5 piers of 1.5 m each ; Overall waterway of the regulator $= 45 + 5 \times 1.5 = 52.5$ m.

Hydraulic Calculations for Various Flow Conditions

(*i*) *Full supply discharge passing down the regulator during high flood.* When u/s water level is 263.2 m (*i.e.* High flood), water shall pass over the regulator and the gated opening provided between the sill level and pond level shall have to be adjusted by partially opening this gate, as shown in Fig. 11.30.

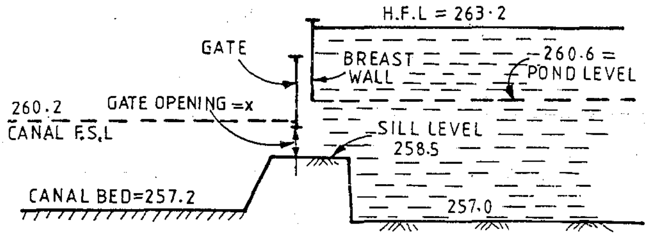


Fig. 11.30

Let the gate opening be x metres. The discharge can then be calculated by submerged orifice formula. *i.e.*

$$Q = C_d \cdot A \cdot \sqrt{2gh}$$

where $C_d = 0.62$

$A = B \cdot x = 45 \cdot x\ \text{m}^2$

$Q = 180$ cumecs.

$h =$ head causing flow $= 263.2 - 260.2 = 3.0$ m.

$\because \quad 180 = 0.62\,(45x)\,\sqrt{2 \times 9.81 \times 3}$

or $\quad 180 = 0.62 \times 45 \times 7.67\,x$

or $$x = \frac{180}{0.62 \times 45 \times 7.67} = \mathbf{0.84\ m.}$$

Velocity of flow through opening

$$= \frac{180}{45 \times 0.84} = 4.76\ \text{m/sec}$$

Loss of head at entry

$$= 0.5\frac{V^2}{2g} = \frac{0.5 \times (4.76)^2}{2 \times 9.81} = 0.58\ \text{metres}$$

TEL just u/s of gate

$$= 263.2 + 0.19 = 263.39\ \text{m}$$

[$\because$ Vel. head is calculated in previous example = 0.19 m]

TEL just downstream of gate $= 263.39 - 0.58 = 262.81$ m.

Downstream water level = 260.2 m.

Head loss $= H_L = 262.81 - 260.2 = \mathbf{2.61\ m.}$

Discharge intensity $q = \frac{180}{45} = 4.4$ cumecs/metre.

(ii) Full supply discharge passing down the regulator at pond level.

$$H_L = 260.6 - 260.2 = 0.4 \text{ m}$$

$$q = 4.0 \text{ cumecs/metre.}$$

Hydraulic jump calculations for the two flow conditions are tabulated in Table 11.17.

Table 11.17

S.No.	*Item*	*High flood flow condition*	*Pond level flow condition*
1	Discharge intensity q in cumecs/m	4.0	4.0
2	Upstream water level	263.2 m	260.6 m
3	Downstream water level	260.2 m	260.2 m
4	u/s TEL	262.81 m	260.6 m
5	d/s TEL	260.2 m	260.2 m
6	Head loss H_L	2.61 m	0.4 m
7	E_{f2} (from Plate 10.1)	2.75 m	2.05 m
8	Level at which jump will form *i.e.* d/s TEL $- E_{f2}$	257.45 m	258.15 m
9	$E_{f1} = E_{f2} + H_L$	5.36 m	2.54 m
10	y_1 corresponding to E_{f1} (Plate 10.2)	0.45 m	0.68 m
11	y_2 corresponding to E_{f2} (Plate 10.2)	2.67 m	1.90 m
12	Length of concrete floor required = 5 $(y_2 - y_1)$	$5 \times 2.22 = 11.1$	$5 \times 1.22 = 6.1$ m
13	Froude No. $F_1 = \frac{q}{\sqrt{g y_1^3}}$		

Provide downstream floor at R.L. 257.2 metres with a horizontal length of 13 metres.

Depth of Sheet Piles from Scour Considerations

Discharge intensity $q = 4.0$ cumecs/metre

D/s Sheet Pile. Depth of scour 'R' $= 1.35 \cdot \left(\frac{q^2}{f}\right)^{1/3}$

$$= 1.35 \cdot \left(\frac{4^2}{1}\right)^{1/3} = 1.35 \times 2.52 = 3.4$$

Let us provide a d/s cutoff upto 1.5 R *i.e.* 5.1 m below the d/s water level which is 260.2 m.

$\therefore$ R.L. of bottom of d/s cutoff $= 260.2 - 5.1 = 255.1$ m.

This value gives only $257.2 - 255.1 = 2.1$ m deep cutoff, which is small. *So let us provide d/s cutoff up to a bottom level of 253.0 m i.e. 4.2 m deep below the d/s floor level.*

U/s Sheet Pile. Provide upstream sheet pile line down to the elevation 249.5 m (*i.e.* the same as that of the undersluices).

Total Floor Length and Exit Gradient

The worst condition of flow occurs when the maximum flood is passing in the river (u/s water level 263.2 m) and there is no water in the canal (*i.e.* the canal is completely closed). The bed level of d/s floor = 257.2 m.

Maximum static head under this condition

$$= H = 263.2 - 257.2 = 6.0 \text{ m}$$

Depth of d/s cutoff = (d) = 4.2 m.

$$G_E = \frac{1}{5} \text{ (Given)}$$

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

$$\therefore \quad \frac{1}{5} = \frac{6.0}{4.2} \cdot \frac{1}{\pi\sqrt{\lambda}}$$

or

$$\frac{1}{\pi\sqrt{\lambda}} = \frac{1}{5} \times \frac{4.2}{6.0} = \mathbf{0.140}$$

From Plate 11.2

$$\alpha = 9$$

$$\therefore \quad b = \alpha \cdot d = 9 \times 4.2 = 37.8 \text{ m}$$

Adopt total floor length = 38 metres.

The floor length shall be provided as below and as shown in Fig. 11.31.

D/s horizontal length	= 13.0 m
D/s glacis length with 3 : 1 slope	= 4.2 m.
Crest width	= 2.0 m
Balance provided as upstream floor	= 18.8 m
	Total = 38.0 m

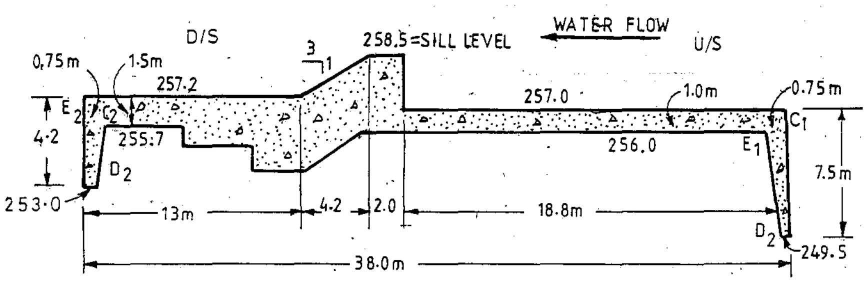


Fig. 11.31

Uplift Pressures

Let us assume 1.0 m floor thickness on u/s and 1.5 m on downstream, as shown in Fig. 11.31.

Upstream pile No. 1.

$$b = 38 \text{ m}$$

$$d = 7.5 \text{ m}$$

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{7.5}{38} = 0.197.$$

From Plate 11.1,

$$\phi_{E_1} = 100\%$$
$$\phi_{D_1} = 100 - \phi_D = 100 - 27\% = 73\%$$
$$\phi_{C_1} = 100 - \phi_E = 100 - 39\% = 61\%$$

Downstream pile No. (2)

$$b = 38 \text{ m}$$
$$d = 4.2$$
$$\frac{1}{\alpha} = \frac{d}{b} = \frac{42.}{38} = 0.11.$$

From Plate 11.1

$$\phi_{E_2} = \phi_E = 30\%$$
$$\phi_{D_2} = \phi_D = 20\%$$
$$\phi_{C_2} = 0\%$$

These pressures are to be corrected as follows :

Corrections to ϕ_{C_1}

(*i*) *Correction due to the effect of sheet pile No. (2) on pile No. (1) of depth d.*

$$\text{Correction} = 19 \cdot \sqrt{\frac{D}{b'}} \cdot \left[\frac{d+D}{b}\right]$$

where $d = 256.0 - 249.5 = 6.5$ m
$D = 256.0 - 253.0 = 3.0$ m
$b' = 36.5$ m
$b = 38$ m.

$$\text{Correction} = 19 \cdot \sqrt{\frac{3.0}{36.5}} \left(\frac{6.5+30}{38}\right) = 1.36\% \,(+\text{ve})$$

(*ii*) *Correction due to floor thickness*

$$\text{Correction} = \frac{73\% - 61\%}{7.5} \times 1.0 = 1.6\% \,(+\text{ve})$$

ϕ_{C_1} (*corrected*) $= 61\% + 1.36\% + 1.6\% =$ **63.96%**

Corrections to ϕ_{E_2}

$$\phi_{E_2} = 30\%$$

(*i*) *Correction due to the effect of sheet pile No. (1) on sheet pile No. (2) of depth d.*

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

where $d = 255.7 - 253.0 = 2.7$ m
$D = 255.7 - 249.5 = 6.2$ m
$b' = 36.5$ m
$b = 38$ m.

$$\text{Correction} = 19 \cdot \sqrt{\frac{6.2}{36.5}} \left(\frac{2.7+6.2}{38}\right) = 1.84\% \,(-\text{ve}).$$

(*ii*) *Correction for floor thickness*

$$\text{Correction} = \frac{30\% - 20\%}{257.2 - 253.0} \times 1.5 = 3.57\% \; (-\text{ ve})$$

$$\phi_{E_2}\,(corrected) = 30 - 1.84 - 3.57 = 24.59\%$$

The levels of the H.G. lines at key points for different flow conditions are tabulated in Table 11.18.

Table 11.18

Condition of flow	U/s water level in metres	D/s water level in metres	Head in metres (H)	Height/elevation of sub-soil H.G. line above datum: Upstream pile line No. (1) ϕ_{E_1} 100%	ϕ_{D_1} 73%	ϕ_{C_1} 63.96%	Downstream pile line No. (2) ϕ_{E_2} 24.59%	ϕ_{D_2} 20.0%	ϕ_{C_2} 0%
				6.0	4.38	3.84	1.48	1.20	0
No flow, maximum static head	263.2	257.2 No water in canal	6.0	263.2	261.58	261.04	258.68	258.40	257.2
				3.0	2.19	1.92	0.74	0.60	0
High flood on barrage and full supply discharge through canal head regulator	263.2	260.2 (Canal FSL)	3.0	263.2	262.39	262.12	260.94	260.8	260.2
				0.4	0.29	0.26	0.10	0.08	0
Flow at pond level	260.6	260.2 (Canal FSL)	0.4	260.6	260.49	260.46	260.30	260.28	260.2

Floor Thicknesses

The maximum static head on the floor occurs for the worst condition *i.e.* when high flood is passing down the barrage but the canal is empty and regulator gates are closed.

$\therefore$ The maximum static head $= 263.2 - 257.2 = 6.0$ metres

The subsoil H.G. line shall be drawn for the maximum static head only (Fig. 11.32) since the floor thicknesses are governed by this critical condition.

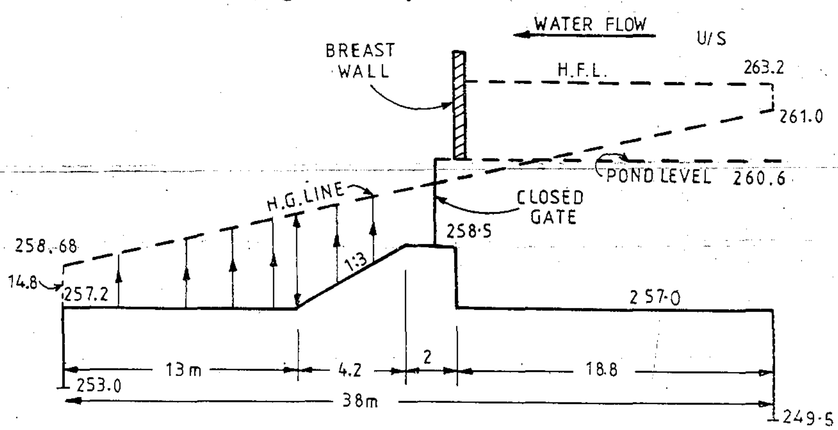


Fig. 11.32. Max. Static Head condition.

Thickness reqd. at d/s end of floor $= \dfrac{1.48}{1.24} = 1.19$ m.

At 4 m from end, d/s unbalanced head $= 1.48 + \dfrac{2.36}{38} \times 4 = 1.73$ m.

Thickness reqd. $= \dfrac{1.73}{1.24} = 1.4$ m.

Provide 1.5 metres thickness in this 4 m reach

At 8 m from d/s end, unbalanced head $= 1.48 + \dfrac{2.36}{38} \times 8$

$= 1.98$ m

Thickness reqd. $= \dfrac{1.98}{1.24} = 1.61$ m.

Provide 1.8 m thickness in this portion

At 13 m from d/s end, unbalanced head

$$= 1.48 + \frac{2.36}{38} \times 13 = 2.29 \text{ m}$$

Thickness required $= \dfrac{2.29}{1.24} = 1.85$ m

Provide 2 m thickness in this portion.

The provided floor thicknesses are shown in Fig. 11.33.

Protection works

(*i*) *U/s Protection.* It shall be equal to what was provided in upstream of under-sluices

(*ii*) *D/s Protection*

$R = 3.4$ m (calculated earlier)

Anticipated scour $= 2R = 2 \times 3.4 = 6.8$ m.

The level of d/s scour hole

$=$ d/s water level $- 2R$

$= 260.2 - 6.8 = 253.4$ m

Scour depth (D) below d/s floor

$= 257.2 - 253.4 = \mathbf{3.8}$ **m**

Let us provide a launching apron in a length equal to 1.5 D (say 5.7 m) and of thickness

$$= \frac{2.25 \times 3.8}{5.7} = 1.5 \text{ m.}$$

Use 6 m length of 1.5 m thick launching apron.

Let us use 1.3 m × 1.3 m × 0.75 m C.C. block in a length equal to app. 1.5 D, *i.e.* 5.7 m. Hence, use 4 rows of 1.3 m × 1.3 m × 0.75 m C.C. blocks with 10 cm jhories in between filled with bajri, in a total length of 5.5 m.

The protection works are shown in Fig. 11.33.

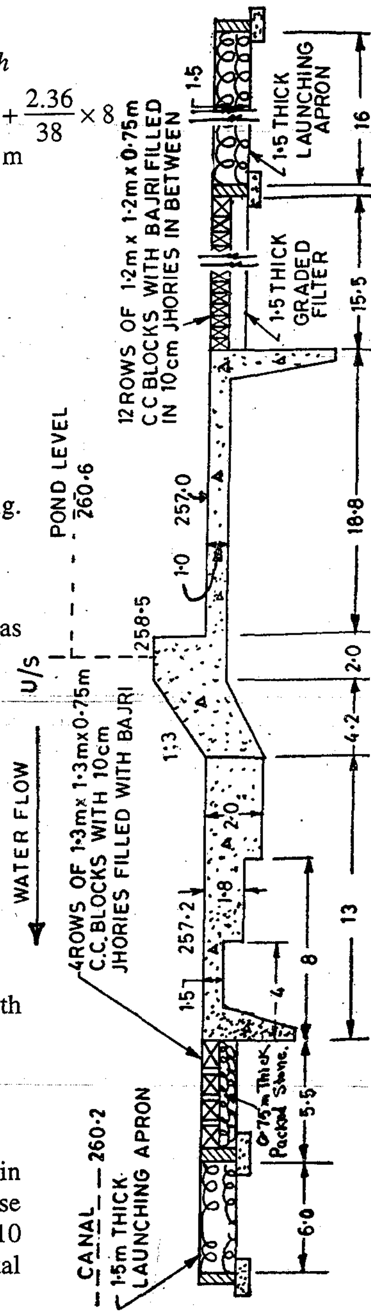


Fig. 11.33. Section of Head Regulator.

CERTAIN IMPORTANT INDIAN BARRAGES

11.7. Data Pertaining to Certain Important Barrages of India

The data pertaining to the location, purpose, flood discharge, and other characteristics of certain important barrages/weirs constructed on Indian rivers, are given below :

11.7.1. Durgapur Barrage. Durgapur barrage (Fig. 11.34) is situated at **Durgapur in Burdwan Distt., West Bengal State** (on Damodar river). It helps in serving *irrigation, industrial, water supply,* and *navigation* needs. The work on this barrage was started in

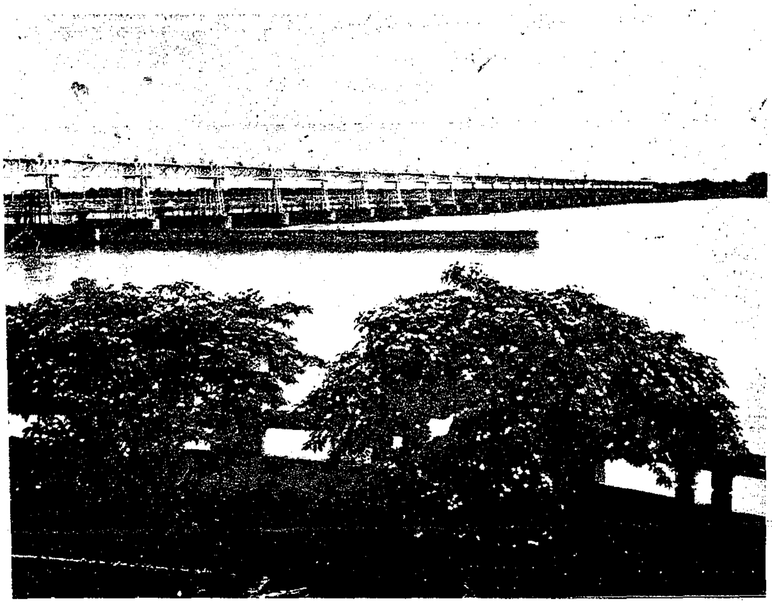
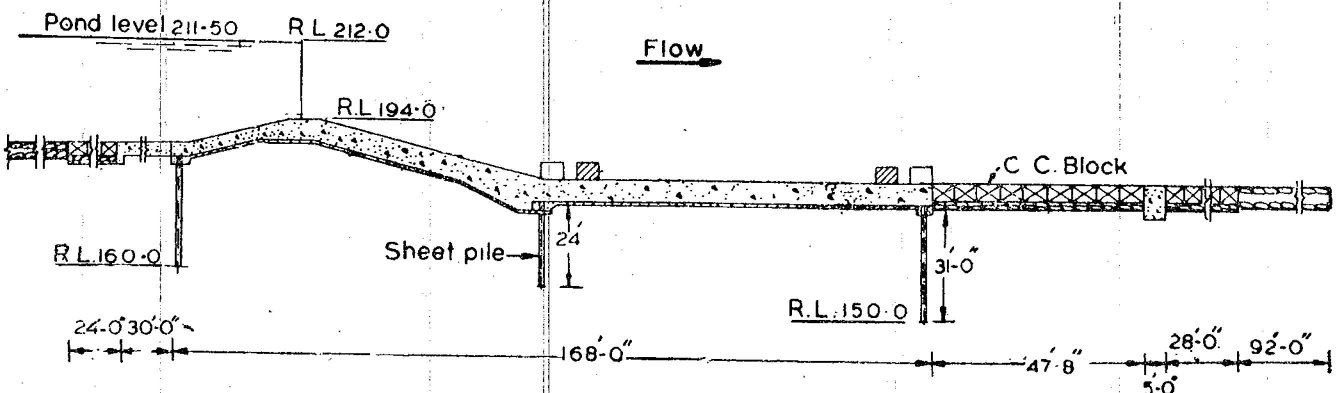


Fig. 11.34. (*a*). Photoview of Durgapur Barrage.

the year 1952, and completed in 1955. The *design flood discharge* for the barrage site is 15,576 cumecs (5.5 lakh cusecs). Other details of the barrage and its canal head regulator are given below.

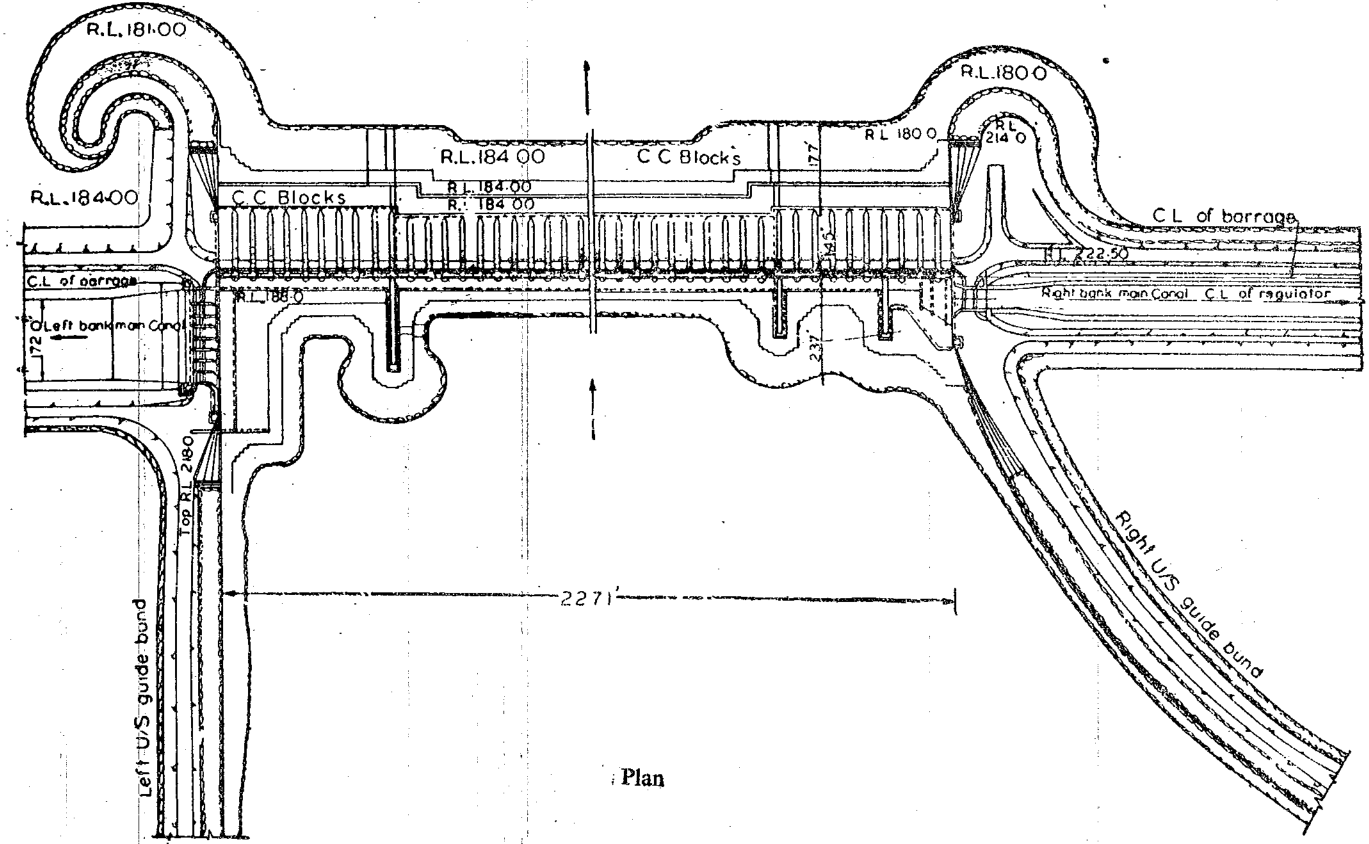
Hydraulic Particulars of Durgapur Barrage

Pond Level	...211.5 m
Width of river	...2,000 m
Length of barrage/anicut	...692.2 m
No. of under-sluice bays	...10
No. of barrage bays	...24
Width of under-sluice bays	...18 m each
Width of barrage-bays	...18 m each
Thickness of intermediate piers	...2.13 m



SECTION THROUGH UNDERSLUICE

Fig. 11.34. (*b*). Section through under-sluices of Durgapur Barrage.



Plan

Fig. 11.34. (c). Plan of Durgapur Barrage.

Gates.	(*a*) *Number*		...24 in weir bays
			...10 in undersluice (barrage) bays.
	(*b*) *Size*	Barrage	...4.87 m× 18 m
		Left undersluice	...5.38 m × 18 m
		Right undersluice	...5.79 m × 18 m.
Sediment excluding devices			...Silt excluder and under-sluices.
Length of upstream divide wall			...80.47 m.

Canal Head Regulator Details

	Left	*Right*
Width of head regulator	59.44 m	13.71 m
No. of bays and their width	8 of 6.09 m each	2 of 6.09 m each
Thickness of piers	2.44 m	1.52 m
Orientation with respect to barrage axis	Perpendicular to the barrage axis	
Location of head regulator from barrage axis	16 m from G.L. of barrage axis	
Energy dissipating devices below head regulator	c.c. blocks and boulder pitching apron	
Max. discharge of canal	260.18 cumecs	64.31 cumecs

11.7.2. Bhimgoda Barrage. Bhimgoda barrage is a new barrage, the construction of which, has recently been completed, across the holy Ganges at Hardwar in U.P. This barrage is a replacement of the old Bhimgoda Weir (Photoview in Fig. 11.35). The old weir is presently being dismantled.

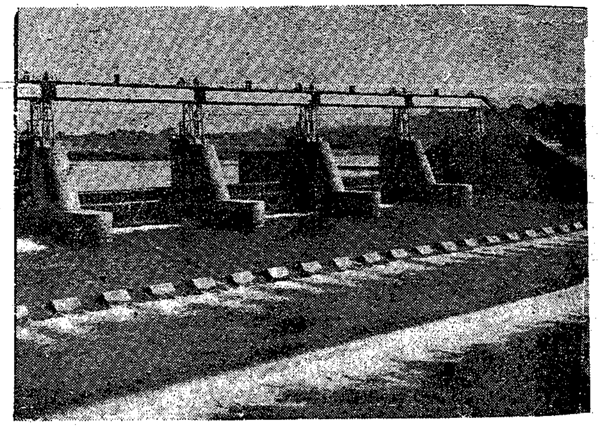


Fig. 11.35. Photoview of the Old Bhimgoda Weir (constructed in the year 1920).

The layout plan of the new barrage w.r. to the old weir is shown in Fig. 11.36 (*a*). Sections of barrage bays and undersluice bays are also shown in Fig. 11.36 (*b*) and 11.36 (*c*); respectively.

This headwork serves the irrigation needs of the adjoining areas, and is designed for a flood discharge of 19300 cumecs (6.8 lakh cusecs).

Other particulars of this headworks are given below.

Hydraulic Particulars of Bhimgoda Barrage

Pond level	...290.2 m (min.) and 293.7 m (max.)

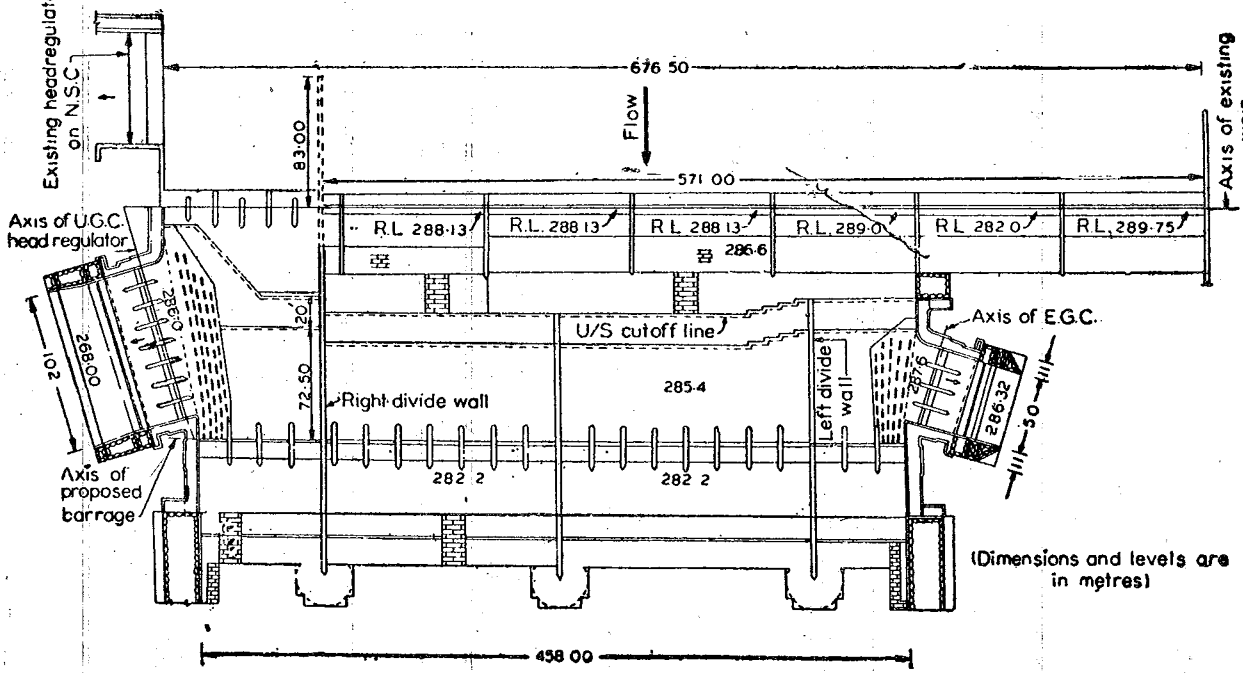


Fig. 11.36. (*a*) Layout Plan of New Bhimgoda Headworks (Barrage) on Ganga river in U.P.

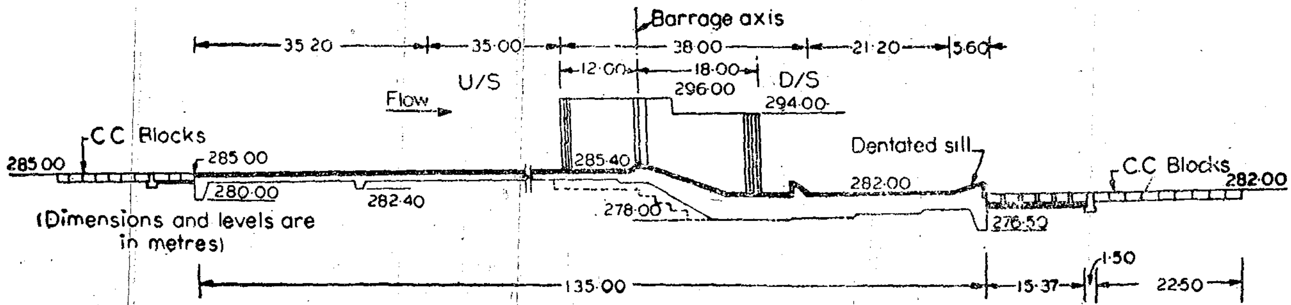


Fig. 11.36. (*b*). Section through barrage bays of New Bhimgoda Barrage

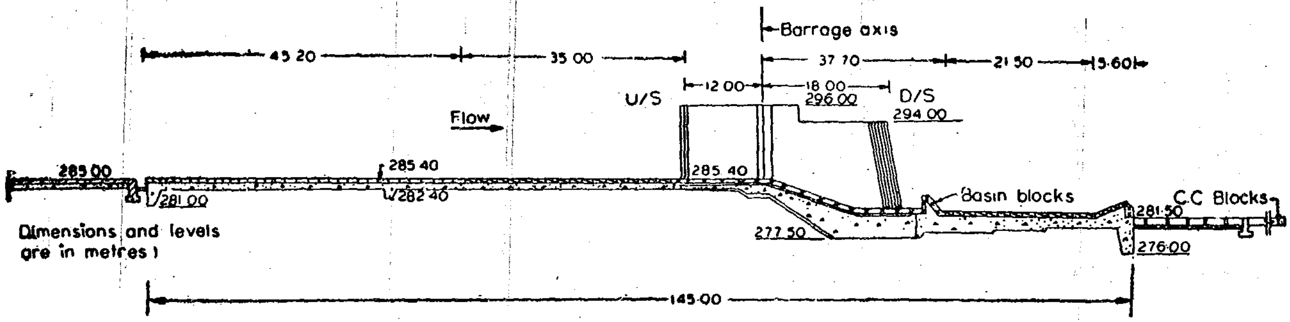


Fig. 11.36. (*c*). Section through undersluice bays of New Bhimgoda Barrage.

Width of river	...675 m
Length of barrage	...455 m
No. of under-sluice bays	...7
No. of barrage bays	...15
Width of under-sluice bays	...18 m each
Width of barrage bays	...18 m each
Thickness of intermediate piers	...2.5 m each
Gates	
(*a*) Number	...22
(*b*) Size	...18 m × 7.8 m for barrage bays
	...18 m × 8.4 m for under-sluices
Sediment Excluding Devices	...Silt excluder and silt ejector
Length of u/s divide wall	...110 m
Energy Dissipation Arrangements	...Basin blocks and dentated sills

Canal Head Regulator Details

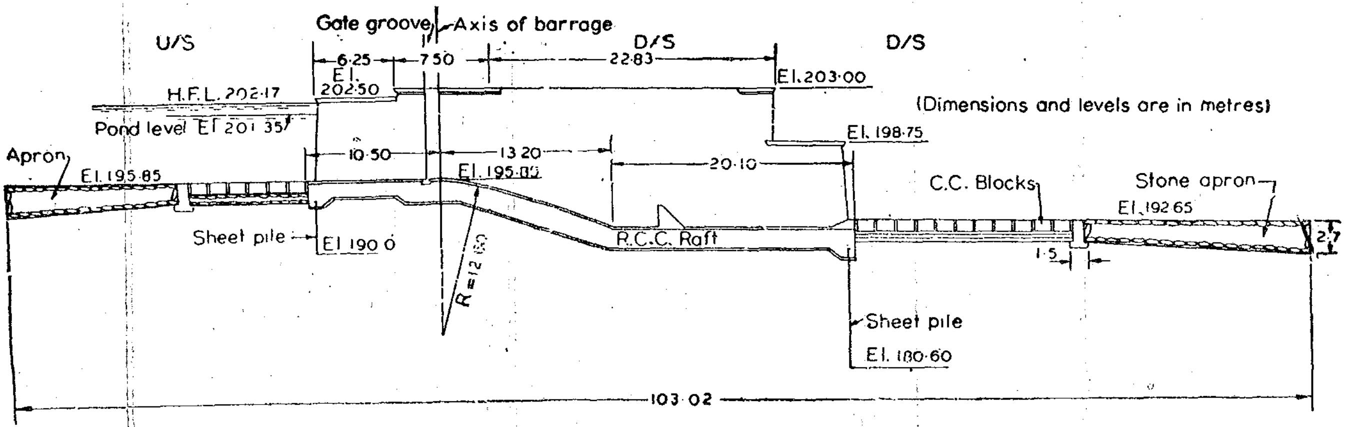
	Right Bank Regulator	*Left Bank Regulator*
Width of head regulator	102 m	50 m
No. of bays and their width	8 of 11 m each	4 of 11 m each
Thickness of piers	2 m	
Orientation with respect to barrage axis	107°	
Location of head regulator from barrage axis	66 m (app.)	40 m (app.)
Energy Dissipation method below regulator	Basin blocks	
Max. discharge of canal	410 cumecs	164 cumecs

11.7.3. New Okhla Barrage. This is a new barrage constructed on Yamuna river, at New Delhi, about 3 km downstream of the existing Old Okhla weir. It aims to serve *irrigation* and *water supply* needs through the New Agra Canal. The design flood discharge is 8495 cumecs. (3 lakh cusecs).

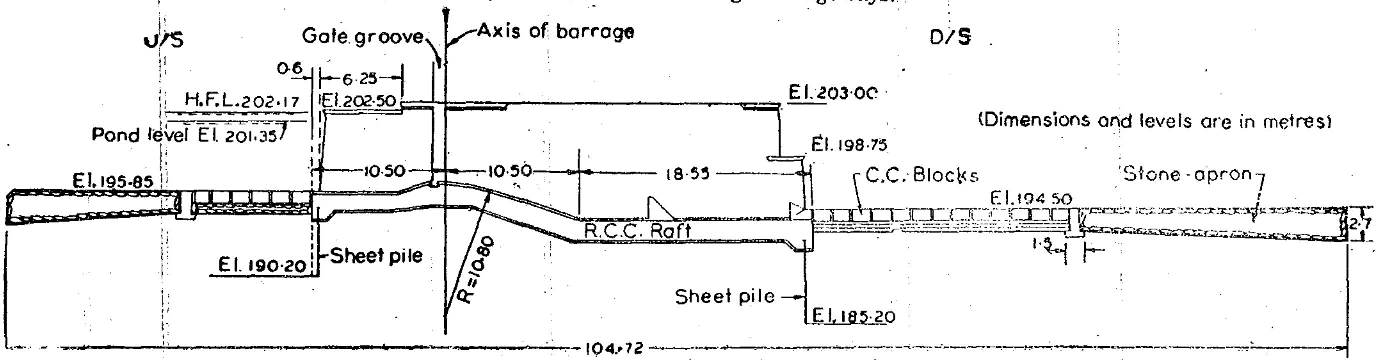
Other particulars of this barrage (Fig. 11.37) and its canal head regulator are given below :

Hydraulic Particulars of New Okhla Barrage

Pond level	...201.35 m



(*a*) Section through barrage bays.



(*b*) Section through undersluice bays.

Fig. 11.37. New Okhla Barrage at New Delhi on Yamuna river.

Width of river	...445.73 m
Length of barrage	...552 m
No. of under-sluice bays	...5
No. of barrage bays	...22
Width of under-sluice bays	...18.3 m each
Width of barrage bays	...18.3 m each
Thickness of intermediate pier	...2.13 m

Gates

(*a*) *Number* ...27

(*b*) *Size* For under-sluices : 5 of 18.3 m × 6 m each

For others : 22 of 18.3 m × 5.1 m each

Sediment Excluding Devices	...Silt excluder
Length of u/s Divide wall	...85.42 m
Energy Dissipation Arrangements	...Baffle blocks and Dentated sills

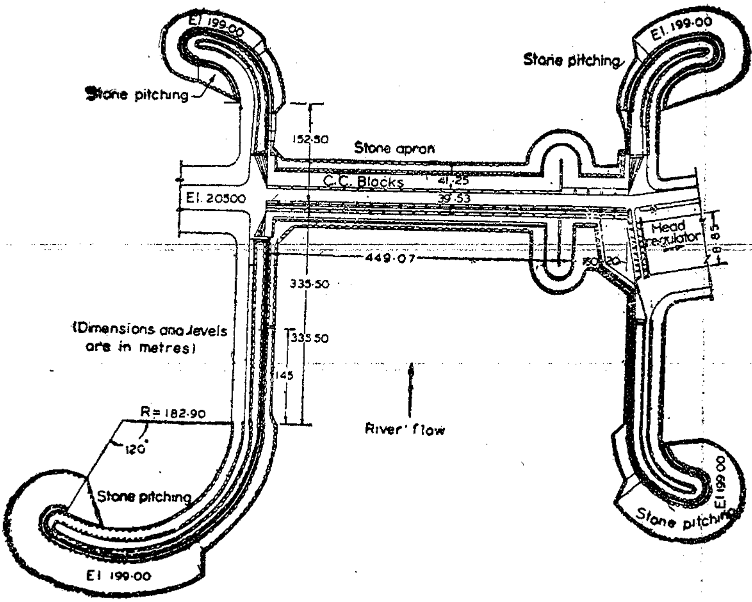


Fig. 11.37 (*c*) Layout Plan of New Okhla Barrage at New Delhi on Yamuna river.

Canal Head Regulator Details

Width of head regulator	...80.85 m
No. of bays and their width	...9 of 7.65 m each
Thickness of piers	...1.5 m each
Orientation with respect to barrage axis	...100°

Location of Head regulator from barrage axis	...22.25 m u/s on right bank,
Energy dissipation arrangements below regulator	...Baffle blocks and end sills
Max. discharge of canal	...242.4 cumecs.

11.7.4. Grand Anicut. The famous old *Grand Anicut* (Fig. 11.38) is located at 16 km from Tiruchirapalli (Tamil Nadu State) on Cauvery river, and is situated 193 km downstream of the famous Mettur dam (Coloured Photo Fig. 11.39) near Salem.

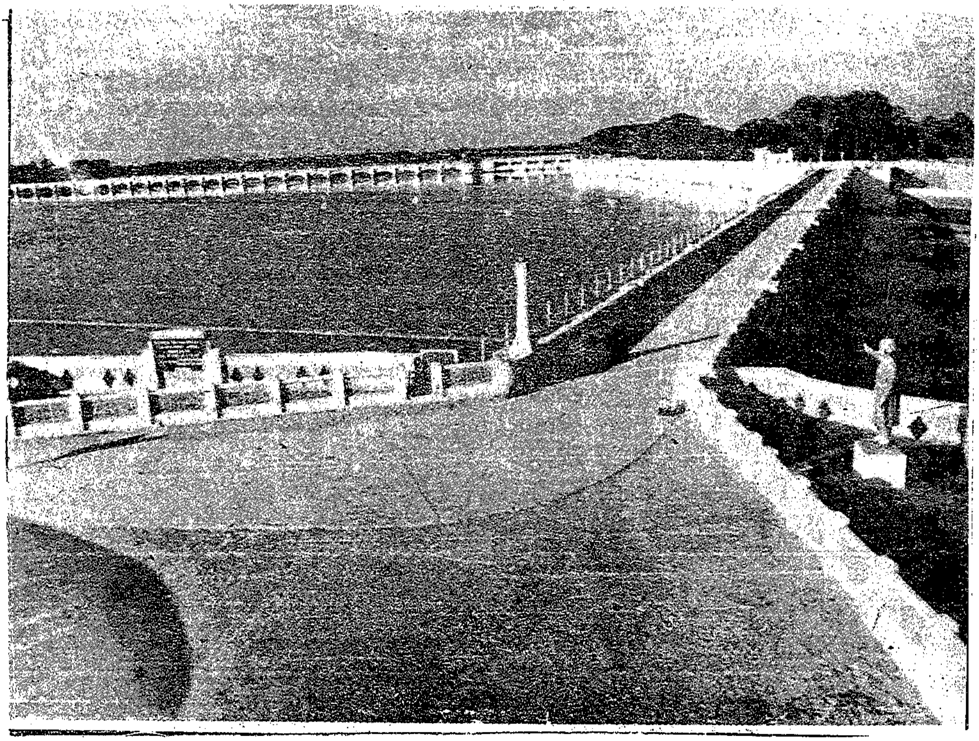


Fig. 11.38 (*a*) Photoview (close) of Grand Anicut.

It serves the *irrigation* needs of the adjoining areas. The design flood discharge is 5094 cumecs (1.8 lakh cusecs). It was constructed in the year 200 A.D. Other particulars of this project are given below :

Hydraulic Particulars of the Grand Anicut

Pond Level	...61.567 m
Width of river	...419 m
Length of anicut	...329 m
No. of Under-sluice bays	...5
No. of Weir bays	...30
Width of Under-sluice bays	...6.1 m each
Width of weir bays	...9.75 m each

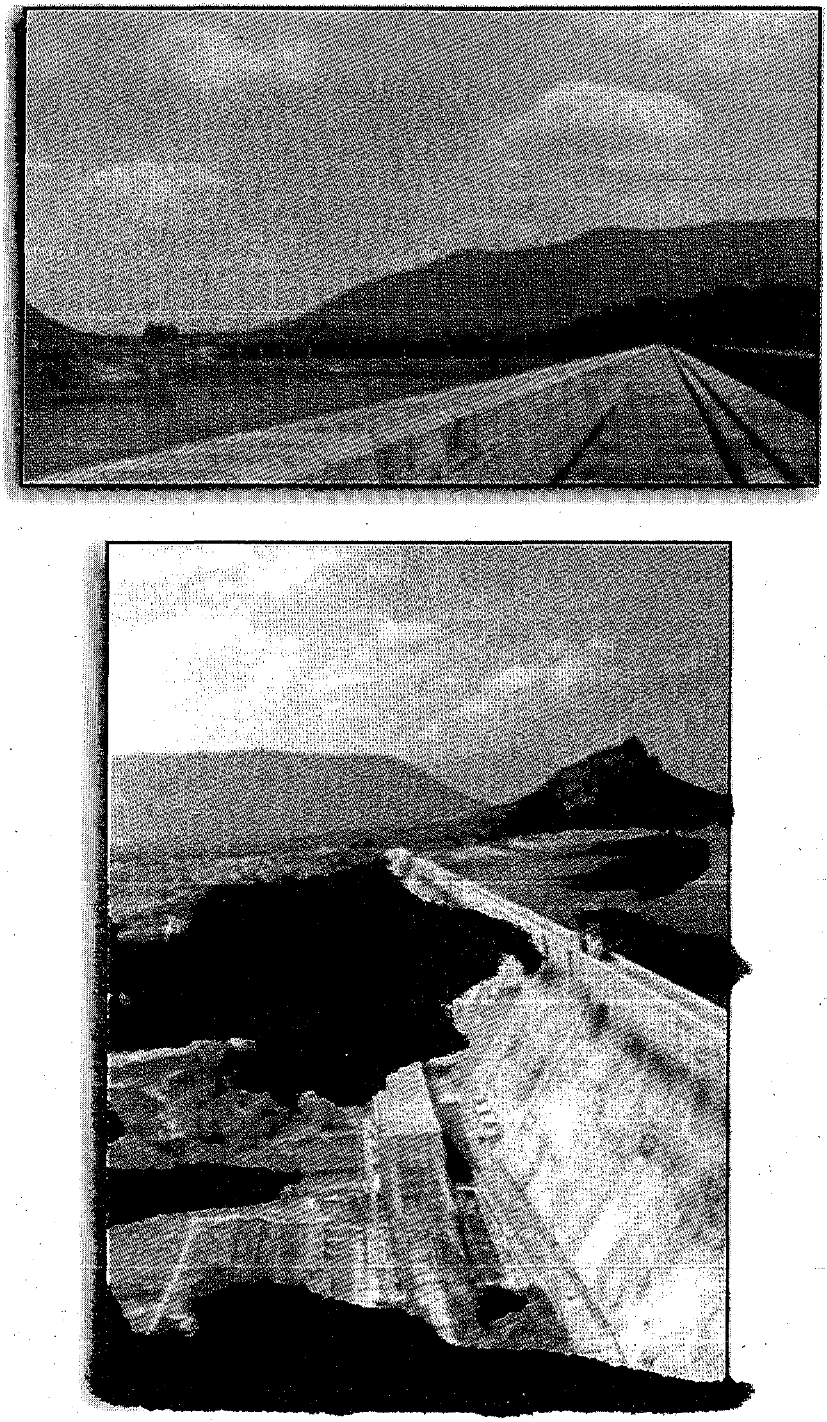


Fig. 11.39. Two views of Mettur dam.

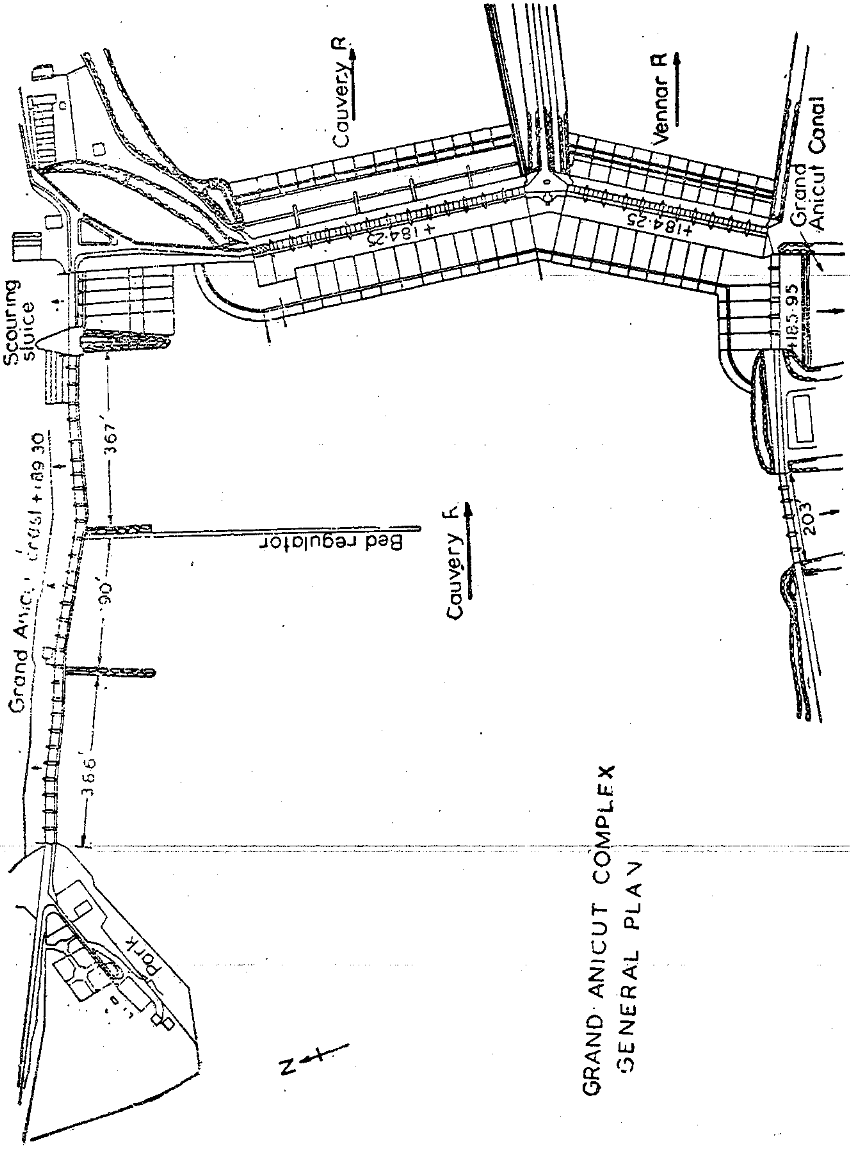


Fig. 11.38 (*b*) General Plan of Anicut Complex (Grand Anicut).

Thickness of intermediate pier	...1.22 m (weir) and 1.83 m (under-sluices)
Gates	
(*a*) Number	...30
(*b*) Size	...9.75 m × 1.52 m
Sediment Excluding Devices	...Under-sluices
Energy Dissipation Arrangements	...Stepped apron

Canal Head Regulator Details

	Cauvery	*Vennar*	*Grand Anicut canal*
Width of head regulator	183 m	135 m	61 m
No. of bays and their width	42 of 3.05×2.74 m each	33 of 3.05×3.45 m each	6 of 9.14×1.6 m each
Thickness of Piers	4.27 m	3.35 m	1.22 m
Orientation with respect to barrage axis	90°	90°	Parallel
Location of head regulator from barrage axis	61 m	61 m	419 m on the right side
Energy Dissipation Devices below regulator.	Baffle blocks		
Max. Discharge of canal	441 cumecs	376 cumecs	116 cumecs

11.7.5. Prakasm Barrage. The famous Prakasm barrage (Fig. 11.40) is located at Vijaywada (in Andhra Pradesh State) on Krishna river. It basically serves the *irrigation*

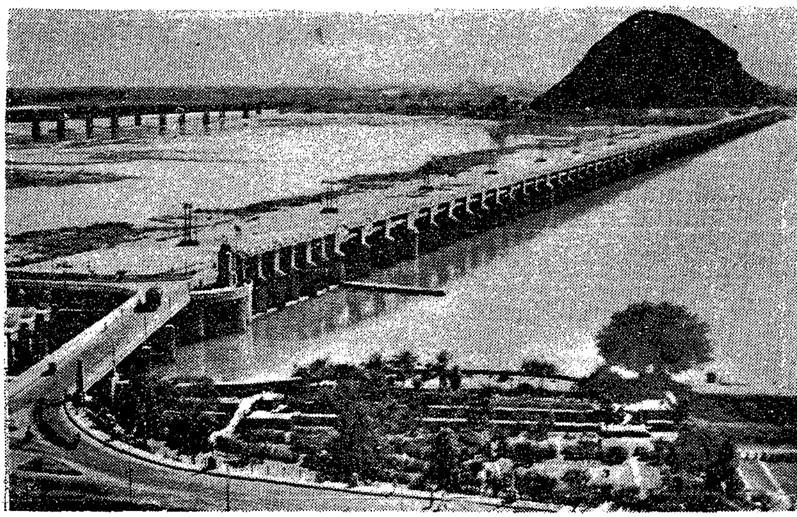


Fig. 11.40. (*a*) Photoview of Prakasm Barrage.

needs of the adjoining areas. The Design flood discharge is as high as 33,984 cumecs (12 lakh cusecs). Its construction started in 1954 and got completed in 1957. Other particulars and details of this barrage are given below :

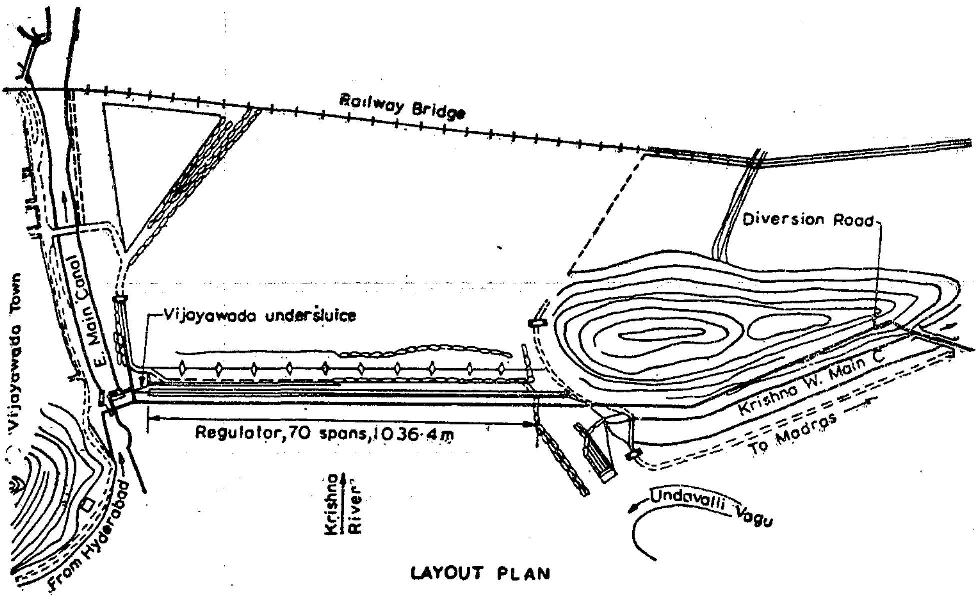


Fig. 11.40. (*b*) Layout plan of Prakasm Barrage.

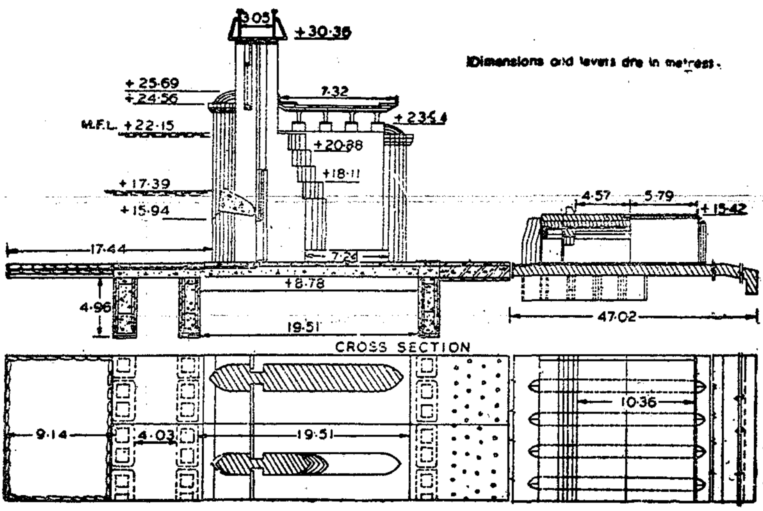


Fig. 11.40. (*c*) Plan (Sitanagram side) of Prakasm Barrage.

Hydraulic Particulars of Prakasm Barrage

Pond Level	...17.38 m
Length of barrage	...1138.73 m
No. of Under-sluice bays	...14
No. of Barrage bays	...70
Width of Under-sluice bays	...5.18 m each
Width of Barrage bays	...12.19 m each
Thickness of intermediate piers	...2.44 m each

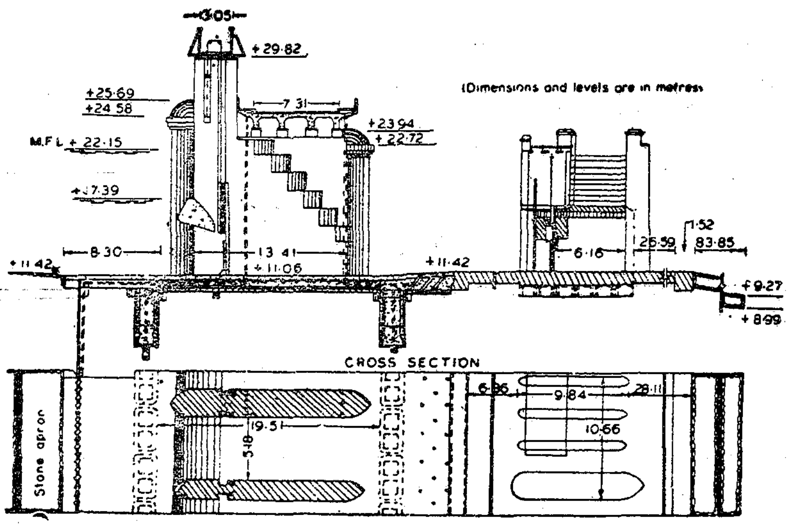


Fig. 11.40. (*d*) Plan (Vijaywada side) of Prakasm Barrage.

Gates

(*a*) *Number*	70
(*b*) *Size*	12.19 m × 3.66 m
Energy Dissipation Arrangements	Cistern

11.7.6. Shah Nehar Barrage. Shar Nehar barrage (Fig. 11.41) is located 5 km downstream of *Beas dam,* in Talwara township (Punjab State) on the Beas river. It caters to the *irrigation* needs of the adjoining areas. The design flood discharge at the barrage site is 11,043 m^3/sec (3.9 lakh cusecs).

Other particulars of this diversion headworks are given below :

Hydraulic Particulars of the Shah Nehar Barrage

Pond level	...330.7 m
Width of the river	...609.6 m
Length of barrage	...561.87 m
No. of Under-sluice bays	...4
No. of Barrage bays	...46
Width of Under-sluice bays	...9.15 m each
Width of Barrage bays	...9.15 m each
Thickness of intermediate pier	...2.13 m (at top)
Gates : (*a*) *Number*	...50
(*b*) *Height*	...6.7 m
Length of Upstream Divide Wall	...41.775 m
Energy Dissipation Arrangements	...Hydraulic jump on cistern

Canal Head Regulator Details

Width of Head regulator	...67.1 m
No. of bays and their width	...8 bays of 7.32 m clear span
Thickness of piers	...1.22 m (at top)
Orientation with respect to barrage axis	at 100°

Energy Dissipation Devices below regulator	...Hydraulic jump in cistern
Max. Discharge of Canal	...325.64 cumecs

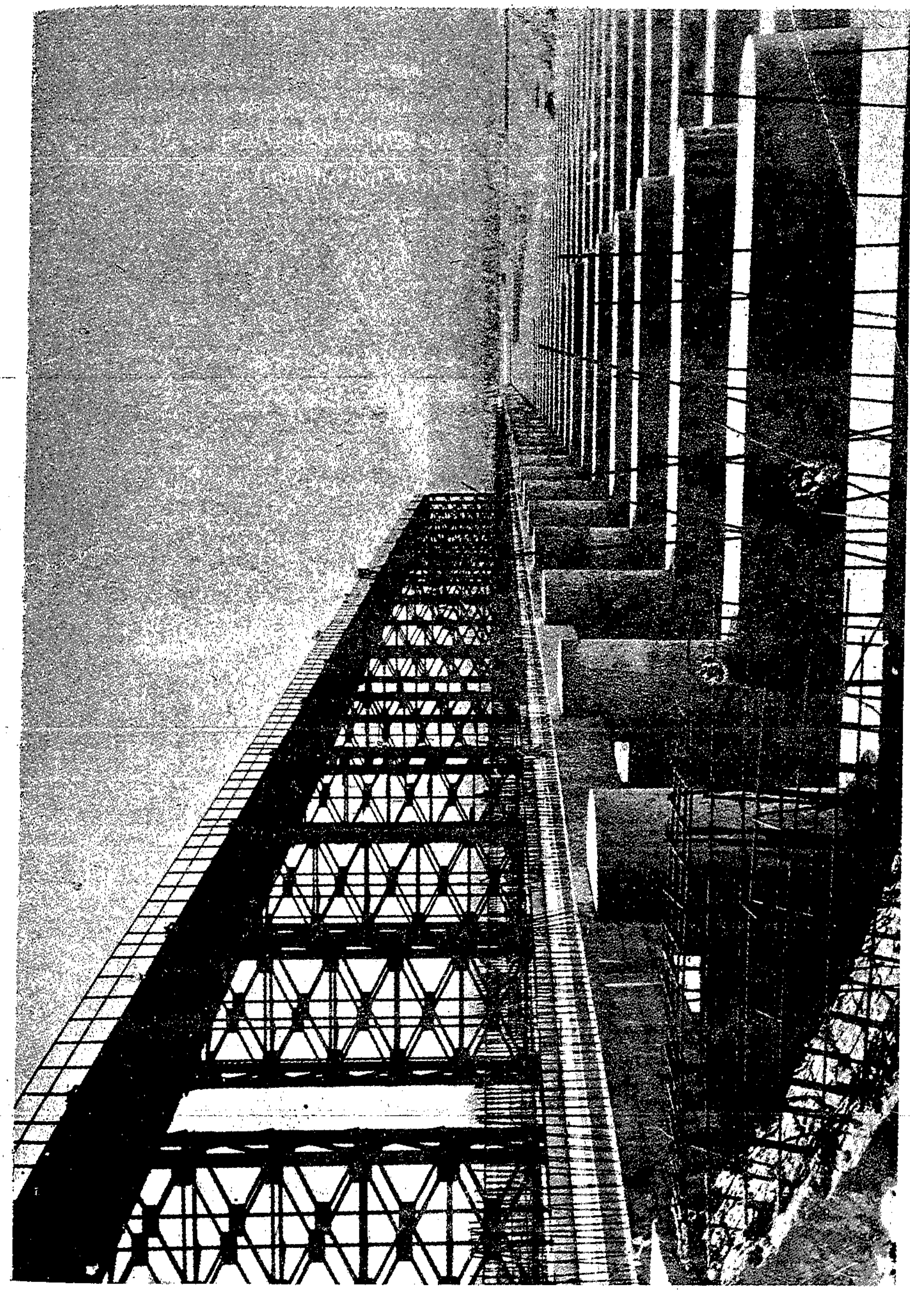


Fig. 11.41. (*a*) Photoview of Shah Nehar Barrage.

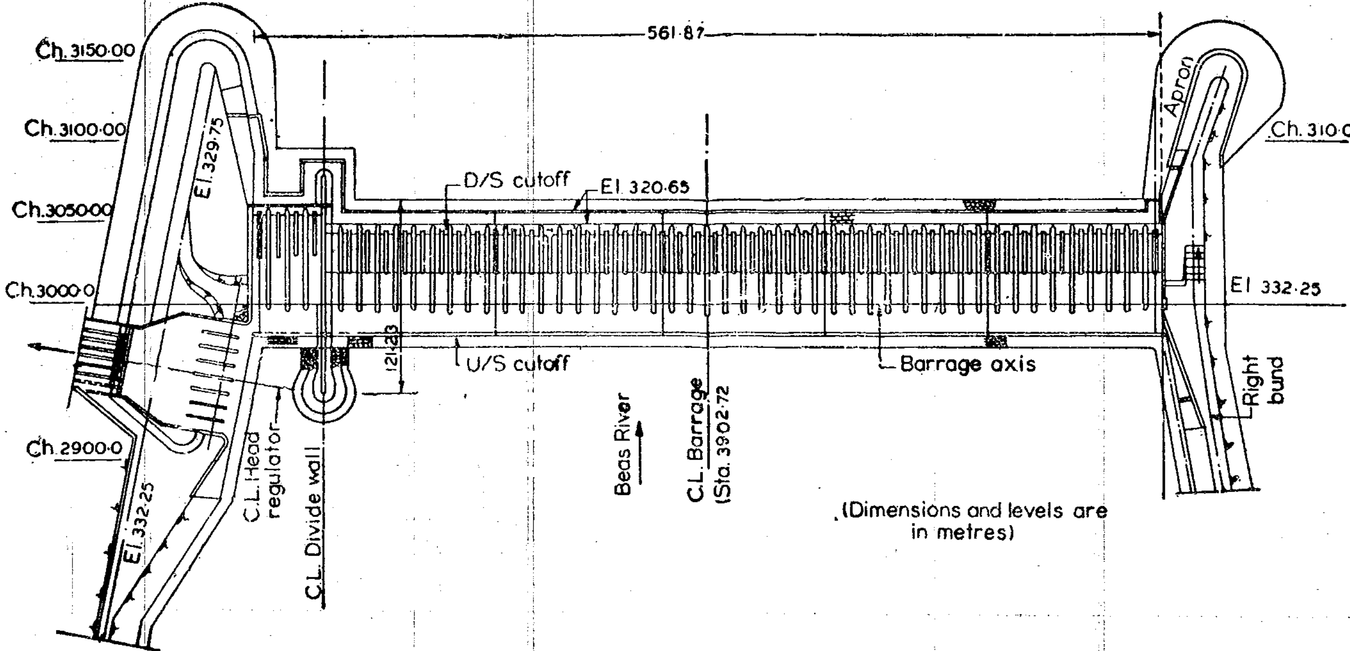


Fig. 11.41 (*b*) Layout plan of Shah Nehar Barrage on Beas river in Punjab.

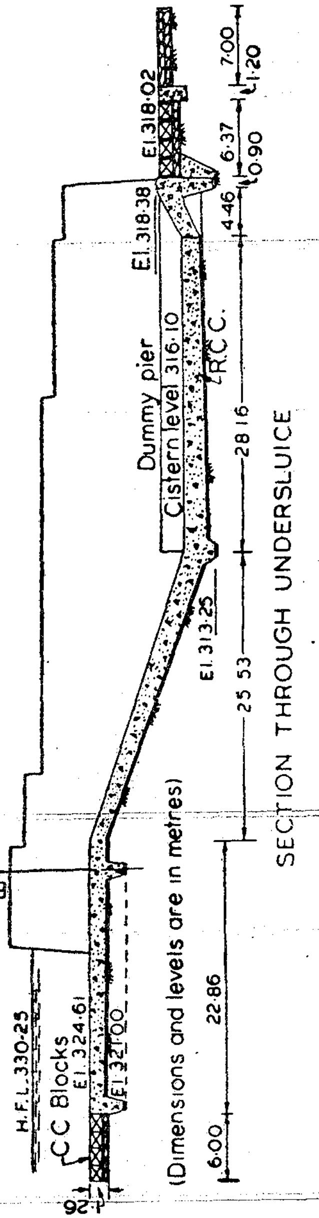


Fig. 11.41. (*c*) Section through Under-sluice bays of Shah Nehar Barrage

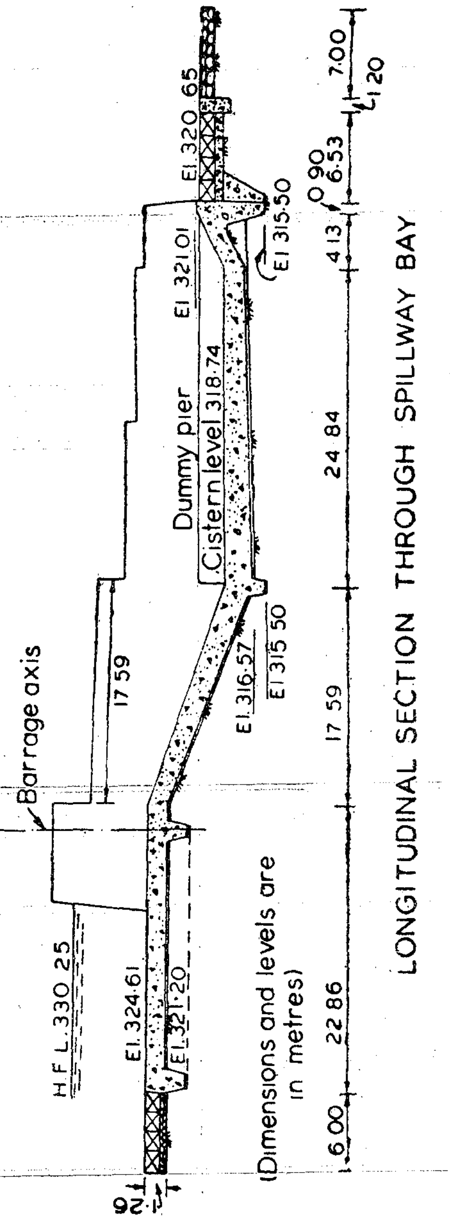


Fig. 11.41. (*d*) Section through Spillway bays of Shah Nehar Barrage.

11.7.7. Tajewala Head Works. Tajewala headworks (Fig. 11.42) was constructed as long back as the year 1873, across Yamuna river. This barrage/anicut is located 37 km from Jagadhri in Haryana, and the famous Western Yamuna Canal (W.J.C.) and the Eastern Yamuna Canal (E.J.C.) take off from this headworks. The design capacity of this barrage is 10,024 cumecs (3.5 lakh cusecs).

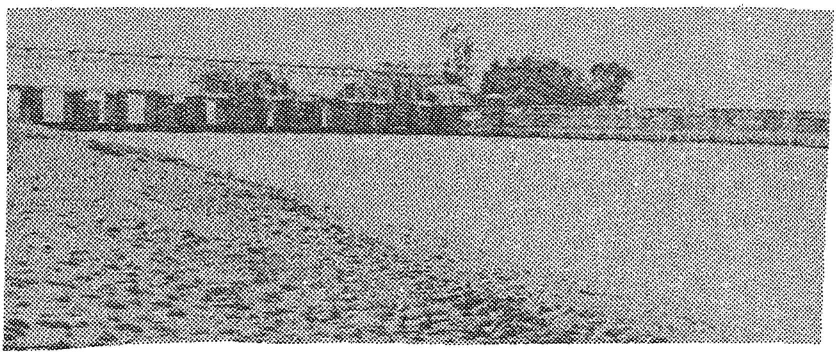


Fig. 11.42. (*a*) Photoview of Tajewala Headworks.

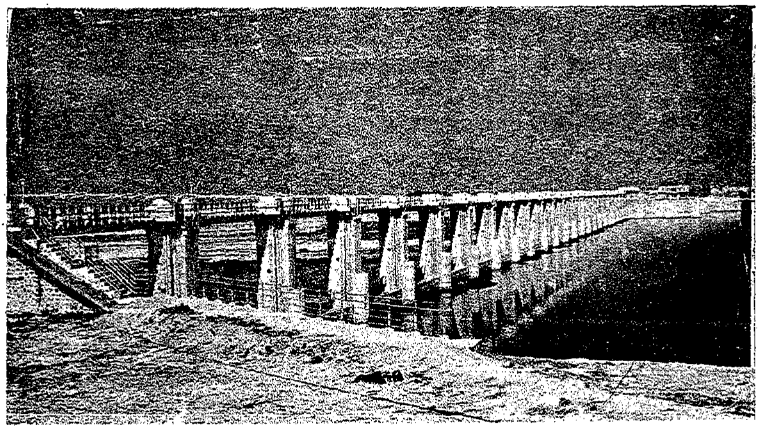


Fig. 11.42. (*b*) Closeview of Tajewala weir bays portion.

Other particulars of this barrage are given below :

Hydraulic Particulars of Tajewala Head works

Length of the barrage/anicut	...753 m
Width of the under sluice bays	...7 spans — 6 m 10 spans—8 m 8 spans—7 m
Width of barrage bays	...564 m
Pond Level	...324 m
Sediment Excluding Devices	...Shingle Excluder
Length of upstream Divide wall	...No. Divide wall
Energy Dissipation devices	...Friction Blocks

Western Yamuna Canal Head Regulator Details

Width of head regulator	...150 m
No. of bays and their width	...13.7 m each and one of 5.4 m
Thickness of piers	...1.37 m

Orientation w.r. to Barrage axis	...90°
Energy Dissipation devices	...Friction Blocks
Max. Discharge of canal	...453 cumecs.

11.7.8. Pick-Up Weir at Ottakkal. This pick-up weir (Fig. 11.43) is located at Ottakkal, Quilon district in Kerala State on Kallada river. The design flood discharge is 2830 cumecs (1 lakh cusecs).

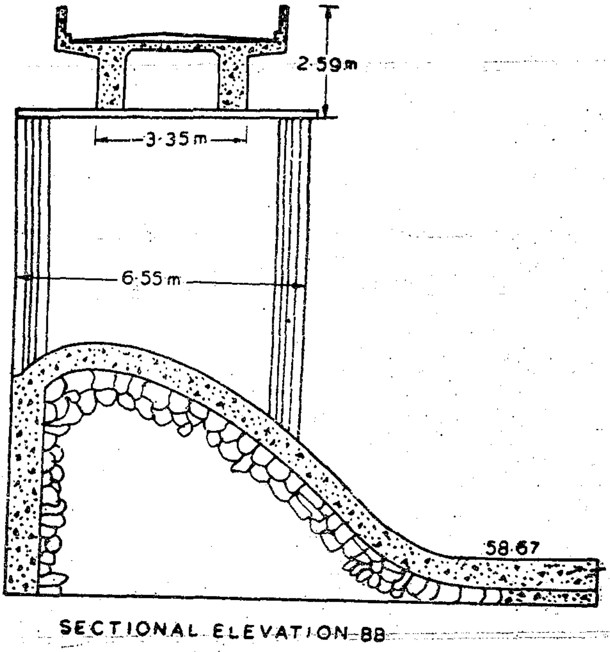


Fig. 11.43. (*a*) Sectional elevation of Ottakkal weir.

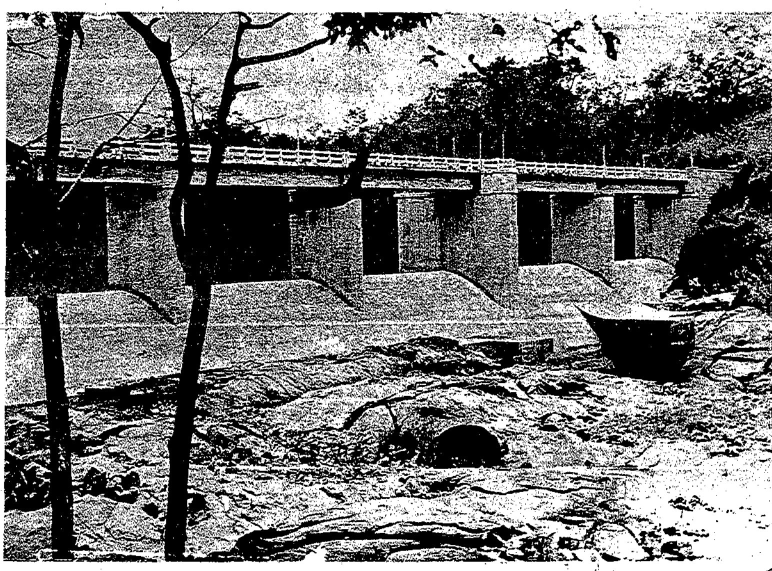


Fig. 11.43. (*b*) Photoview of Ottakkal weir.

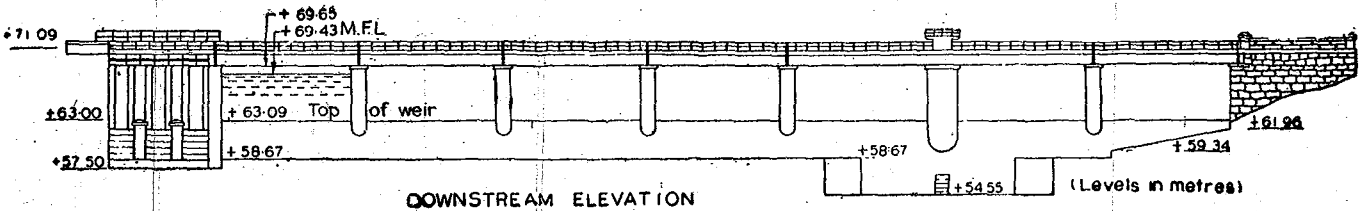


Fig. 11.43. (*c*) Downstream elevation of Ottakal weir.

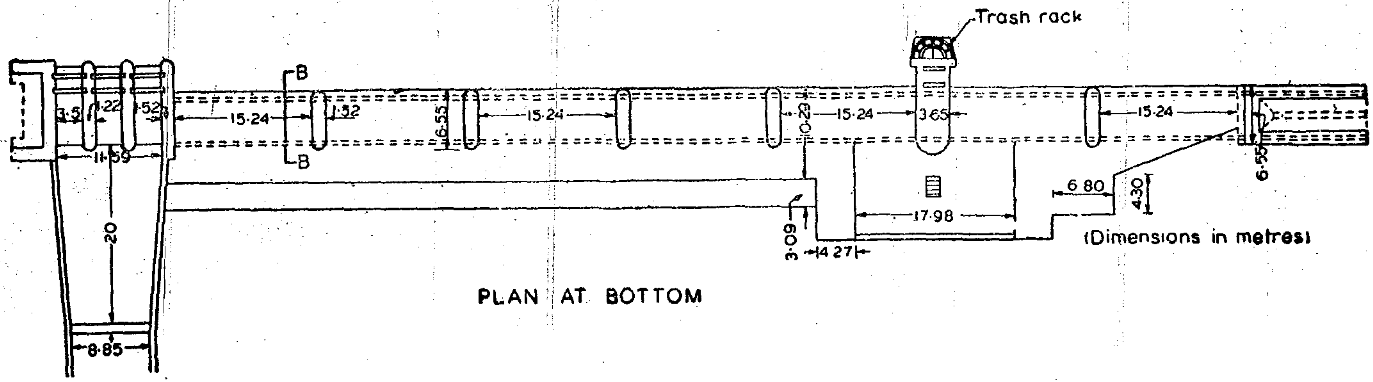


Fig. 11.43. (*d*) Plan at bottom of Ottakkal weir.

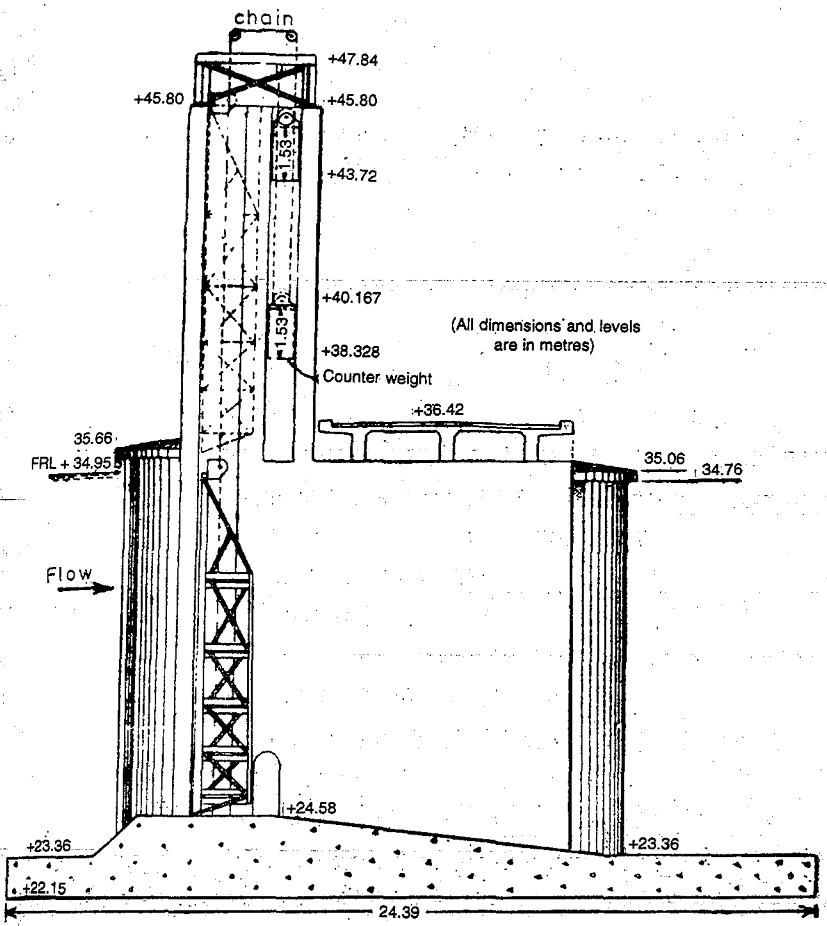


Fig. 11.43. (*e*) Cross-section of Spillway Gate at Ottakkal weir.

Other particulars of this diversion headworks are given below :

Hydraulic Particulars of the Ottakkal weir

Width of the river	...133 m
Length of the weir	...131.04 m
No. of undersluice bays	...1
No. of weir bays	...7, and R.B.C. head regulator bay
Width of undersluice bays	...1.83 m each
Width of weir bays	...15.24 m each
Energy Dissipation Arrangement	...Nil

Canal Head Regulator Details

Width of Head regulator	...11.59 m
No. of bays and their width	...3 of 3.05 m each
Thickness of piers	...1.22 m each
Orientation with respect to weir axis	...90°

Location of Head regulator from weir axis	...R.B.C.* head regulator at the right, and L.B.C.** head regulator at 120 m upstream of the weir axis and 90 m from left edge of the river.
Max. Canal Discharge	...39.08 cumecs (R.B. Canal) 220 cumecs (L.B. Canal)

PROBLEMS

1. (*a*) What are the Main causes of failures of weirs on permeable foundations, and what remedies would you suggest to prevent them ?

(*b*) Explain the surface flow considerations involved in the design of thickness of the sloping glacis and the downstream floor of a weir for different flow considerations.

2. Discuss the main causes of failure of weirs founded on pervious foundation. Also discuss the important theories which have been put forward for designing such weirs to avoid there failure due to the above causes.

3. What are the different types of weirs ? Explain with neat sketches circumstances under which each type is adopted.

What is meant by "piping" in a hydraulic structure ? What are ill-effects of piping ? What are the precautionary methods to avoid the ill effect of piping ? (*Madras University, 1976*)

4. Briefly explain the salient features of Khosla's theory and how it is used in the design of permeable foundations ?

How does Lane's theory differ from Bligh's creep theory ? (*Madras University, 1974*)

5. (*a*) What is meant by "piping" on foundation of a weir. Explain Bligh's method of safe guarding the foundation against the ill effects of piping. (*Madras University, 1975*)

(*b*) Briefly outline Khosla's theory on the design of weirs on permeable foundation. Enumerate the various corrections that are needed in the application of this theory.

6. Explain briefly Khosla's exit gradient concept.

When a weir is constructed across a river ; it disturbs the existing regime. Explain the changes that occur till the river attains regime conditions again. (*Madras University, 1975*)

7. How does Khosla's theory differ from Bligh's theory with regard to the design of weirs on permeable foundations ?

Explain the criteria adopted in designing the various components of a weir built on permeable foundations using Khosla's theory. (*Madras University, 1976*)

8. (*a*) Discuss briefly the causes of failure of hydraulic structures, founded on pervious foundations.

(*b*) State the fundamental difference between Khosla's theory and Bligh's creep theory for seepage below a weir.

(*c*) Following corrected pressure potentials were determined underneath a barrage floor by Khosla's theory :

At junction of upstream sheet pile with floor $\phi_{E_1} = 82\%$

At junction of downstream sheet pile with floor $\phi_{C_2} = 35\%$

Calculate the minimum thickness of the cistern floor at the beginning (*i.e.* at the toe of the glacis), and the end of the cistern (*i.e.* at the junction of the downstream sheet pile with cistern)

The following data are given :

Full reservoir level	= 105 m.
River bed level	= 100 m.
Cistern floor level	= 99 m.

* Right Bank Canal

** Left Bank Canal

Total length of barrage between upstream and downstream sheet piles (*i.e.* between E_1 and C_2) = 40 m; Length of cistern = 15 m (from downstream sheet pile).

Assume tail water depth to be nil on the downstream side. Specific gravity of concrete floor = 2.4.

9. (*a*) Draw a neat sectional view of a weir showing the various parts. What is exit gradient ? How does it affect the design of a weir.

(*b*) Following data refer to a weir :

Total number of vertical gates	= 51
Span of each gate	= 10 m.
Full reservoir level (u/s)	= 110 m.
Crest level	= 106 m.
Coefficient of end contraction for piers	= 0.02
Coefficient of discharge (in Francis formula) C_d	= 1.70 $m^{1/2}$/sec.

Compute the max. flood discharge which can safely pass over the weir without exceeding the full reservoir level. Neglect Velocity of approach.

[**Solution** Use $Q = 170 L_e \cdot H^{3/2}$, where $H = 110 - 106 = 4$ m

$L_e = L - [0.1 \times \text{No. of end contractions for abutments} + 0.02 \times \text{No. of end contractions for piers}]\, H$

$L_e = 51 \times 10 - [0.1 \times 2 + 0.02 \times 50]\, H = 501.2$ m

($\because$ each pier gives 2 contractions, and No. of piers = 50)

$\therefore$ $Q = 1.7 \times 501.2 \times (4)^{3/2}$ cumecs. = **6820 cumecs. Ans.**]

10. Use Khosla's curves to calculate the percentage uplift pressures at the three cut-offs for a barrage foundation profile, shown in Fig. 11.44 applying corrections as applicable.

(Given slope correction for 1 in 4 slope is 3.3%)

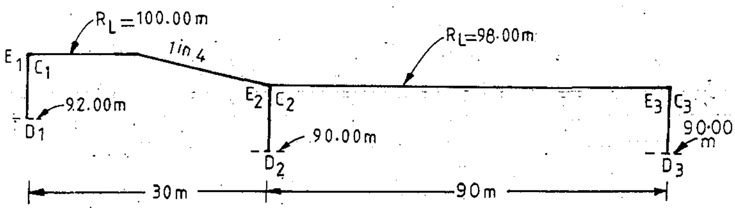


Fig. 11.44

Having determined the percentage uplift pressures, explain how the foundation floor thicknesses are determined corresponding to a known specific gravity of the material of the floor.

11. Write short notes on any three of the following :

(*i*) Stream-lines and Equipotential lines.

(*ii*) Bligh's creep theory for seepage flow.

(*iii*) Khosla's theory and concept of flow net.

(*iv*) Exit gradient and its importance.

(*v*) Khosla's method of independent variables for determining pressures and exit gradient for seepage below a weir.

(*vi*) Design of inverted filters and launching aprons for weirs (u/s as well as d/s).

(*vii*) Retrogression of levels due to weir construction.

(*viii*) Factors governing the design of weirs.

(*ix*) Uplift pressures in jump trough and design of weir floor thickness.

12. Design a barrage across a river at a site which is situated at 25 km downstream of a 120 m high dam, from the following data :

Total length of barrage between upstream and downstream sheet piles (*i.e.* between E_1 and C_2) = 40 m; Length of cistern = 15 m (from downstream sheet pile).

Assume tail water depth to be nil on the downstream side. Specific gravity of concrete floor = 2.4.

9. (*a*) Draw a neat sectional view of a weir showing the various parts. What is exit gradient ? How does it affect the design of a weir.

(*b*) Following data refer to a weir :

Total number of vertical gates	= 51
Span of each gate	= 10 m.
Full reservoir level (u/s)	= 110 m.
Crest level	= 106 m.
Coefficient of end contraction for piers	= 0.02
Coefficient of discharge (in Francis formula) C_d	= 1.70 $m^{1/2}$/sec.

Compute the max. flood discharge which can safely pass over the weir without exceeding the full reservoir level. Neglect Velocity of approach.

[**Solution** Use $Q = 170 L_e \cdot H^{3/2}$, where $H = 110 - 106 = 4$ m

$$L_e = L - [0.1 \times \text{No. of end contractions for abutments} + 0.02 \times \text{No. of end contractions for piers}]\, H$$

$$L_e = 51 \times 10 - [0.1 \times 2 + 0.02 \times 50]\, H = 501.2 \text{ m}$$

($\because$ each pier gives 2 contractions, and No. of piers = 50)

$\therefore$ $Q = 1.7 \times 501.2 \times (4)^{3/2}$ cumecs. = **6820 cumecs. Ans.**]

10. Use Khosla's curves to calculate the percentage uplift pressures at the three cut-offs for a barrage foundation profile, shown in Fig. 11.44 applying corrections as applicable.

(Given slope correction for 1 in 4 slope is 3.3%)

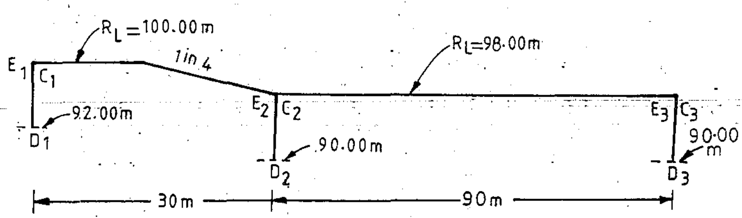


Fig. 11.44

Having determined the percentage uplift pressures, explain how the foundation floor thicknesses are determined corresponding to a known specific gravity of the material of the floor.

11. Write short notes on any three of the following :

(*i*) Stream-lines and Equipotential lines.

(*ii*) Bligh's creep theory for seepage flow.

(*iii*) Khosla's theory and concept of flow net.

(*iv*) Exit gradient and its importance.

(*v*) Khosla's method of independent variables for determining pressures and exit gradient for seepage below a weir.

(*vi*) Design of inverted filters and launching aprons for weirs (u/s as well as d/s).

(*vii*) Retrogression of levels due to weir construction.

(*viii*) Factors governing the design of weirs.

(*ix*) Uplift pressures in jump trough and design of weir floor thickness.

12. Design a barrage across a river at a site which is situated at 25 km downstream of a 120 m high dam, from the following data :

High flood discharge = 6100 cumecs.
High flood level (before the construction of barrage)
= 229.9 m.
River bed level (winter) = 224.3 m.
Pond level = 228.15 m.
Permissible afflux = 1.0 m.
Canal discharge = 151.5 cumecs.
Canal bed width = 50 m
Canal full supply depth = 3 m.
Canal bed level = 224 m.
Angle of off-take = 110°
Silt factor in the river = 1.5

The gauge discharge curve of the river is given below in Fig. 11.45.

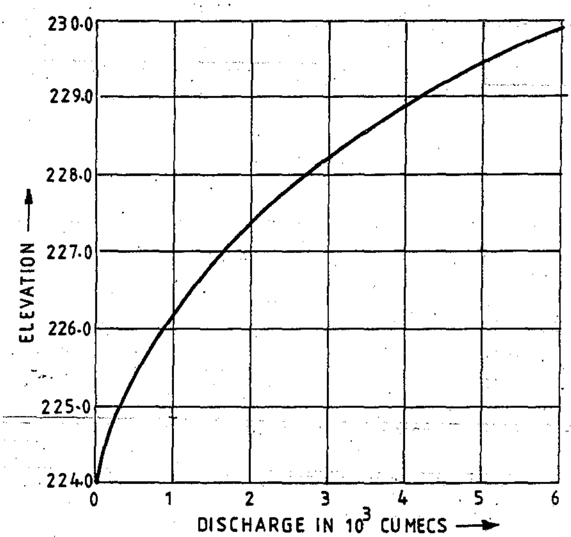


Fig. 11.45