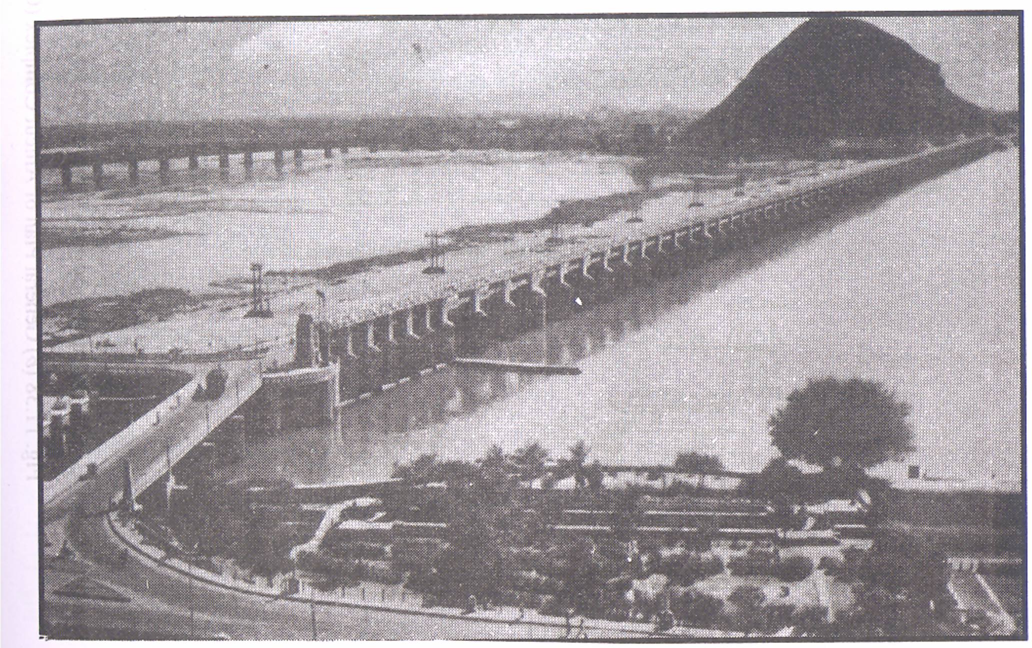
Diversion Head Works

INTRODUCTION

Any hydraulic structure which supplies water to the off-taking canal is called a *headwork.* Headwork may be divided into two

1. Storage headwork.
2. Diversion headwork.

A Storage headwork comprises the construction of a dam on the river. It stores water during the period of excess supplies and releases it when demand overtakes available supplies. A *diversion headwork* serves to divert the required supply to canal from the river. A diversion head works (or a weir) is a structure constructed across a river for the purpose of raising water level in the river so that it can be diverted into the offtaking canals. Diversion headworks are generally constructed on the perennial rivers which have adequate flow throughout the year and, therefore, there is no necessity of creating a storage reservoir. A diversion head works must be differentiated from a storage work or a dam. A dam is constructed on the river for the purpose of creating a large storage reservoir. The storage works are required for the storage of water on a non-perennial river or on a river with inadequate flow throughout the year. On the other hand, in a diversion head works, there is very little storage, if any.



If the storage on the upstream of a diversion head works is significant, it is called a *storage weir*. If a diversion headworks is constructed on the downstream of a dam for the purpose of diverting water released from the u/s dam into the offtaking canals, it is called a *pickup weir.* Generally, the dam is constructed in the rocky or the mountainous reach of the river where the conditions are suitable for a dam, and a pickup weir is constructed near the commanded area in the alluvial reach of the river.

A diversion head works serves the following functions:

1) It raises the water level on its upstream side.
2) It regulates the supply of water into canals.
3) It controls the entry of silt into canals
4) It creates a small pond (not reservoir) on its upstream and provides some pondage.
5) It helps in controlling the vagaries of the river.

LOCATION OF DIVERSION HEADWORKS

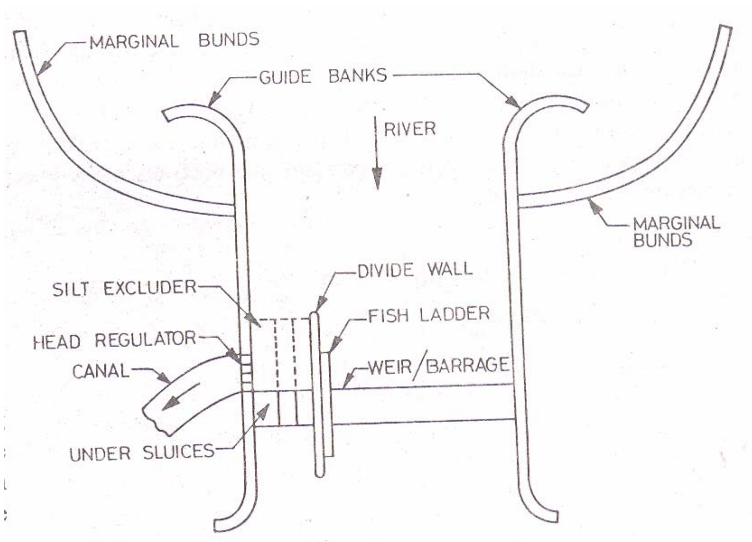
The diversion headworks are generally located in the boulder stage or trough stage of the river at a site which is close to the commanded area of the offtaking canals. If there are a number of sites which are suitable, the final selection is done on the basis of cost. The site which gives the most economical arrangement for the diversion head works and the distribution works (canals) is usually selected.

1) The river section at the site should be narrow and well-defined.
2) The river should have high, well-defined, inerodible and non-submersible banks so that the cost of river training works is minimum.
3) The canals taking off from the diversion head works should be quite economical and should have a large commanded area.
4) There should be suitable arrangement for the diversion of river during construction.
5) The site should be such that the weir (or barrage) can be aligned at right angles to the direction of flow in the river.
6) There should be suitable locations for the undersluices, head regulator and other components of the diversion headworks.
7) The diversion headworks should not submerge costly land and property on its upstream.
8) Good foundation should be available at the site.
9) The required materials of construction should be available near the site.
10) The site should be easily accessible by road or rail.
11) The overall cost of the project should be a minimum.

COMPONENT PARTS OF A DIVERRSION HEADWORK

A diversion headwork consist of the following component parts

1. Weir or barrage
2. Undersluices
3. Divide wall
4. Fish ladder
5. Canal head regulator
6. pocket or approach channel
7. Silt excluders/ Silt prevention devices/
8. River training works (Marginal bunds and guide banks)

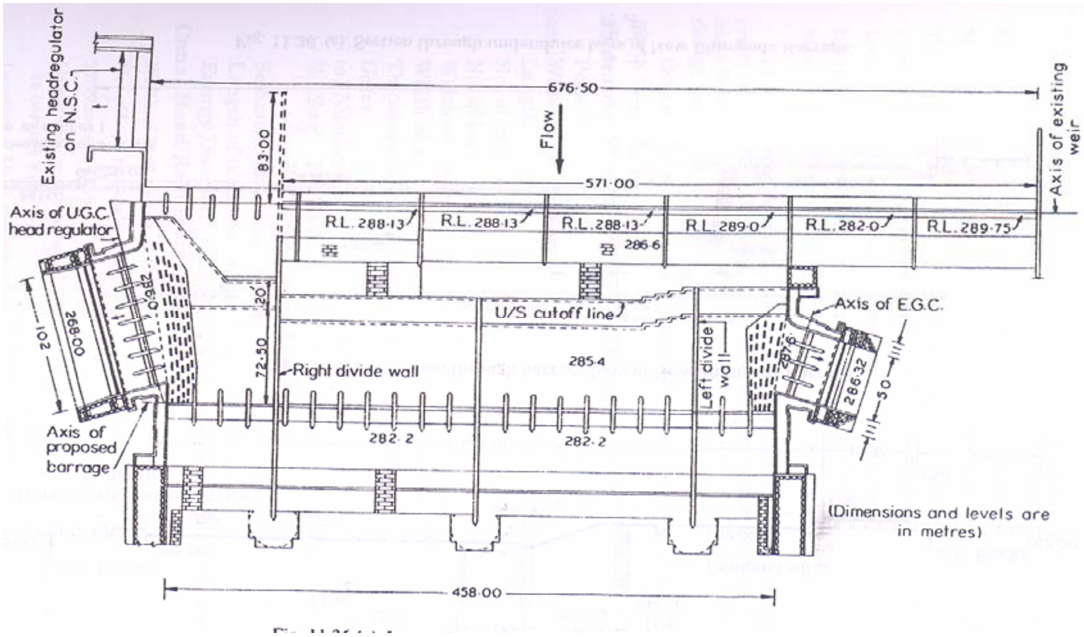


Undersluices

Undersluice sections are provided adjacent to the canal head regulators. The undersluices should be able to pass fair weather flow for which the crest shutters on the weir proper need not be dropped. The crest level of the undersluices is generally kept at the average bed level of the river.

Divide Wall

A divide wall is a wall constructed parallel to the direction of flow of river to separate the weir section and the undersluices section to avoid cross flows. If there are undersluices at both the sides, there are two divide walls.

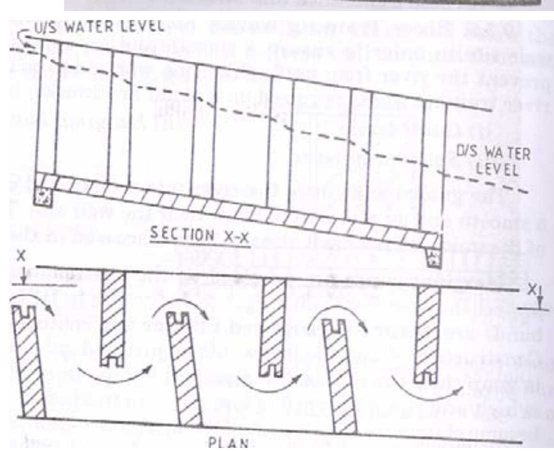


Fish Ladder

A fish ladder is a passage provided adjacent to the divide wall on the weir side for the fish to travel from the upstream to the downstream and vice versa. Fish migrate upstream or downstream of the river in search of food or to reach their sprawling places. In a fish ladder the head is gradually dissipated so as to provide smooth flow at sufficiently low velocity. Suitable baffles are provided in the fish passage to reduce the flow velocity.

Canal Head Regulator

A canal head regulator is provided at the head of the canal offtaking from the diversion headworks. It regulates the supply of water into the canal, controls the entry silt into the canal, and prevents the entry of river floods into canal.



Silt Excluder

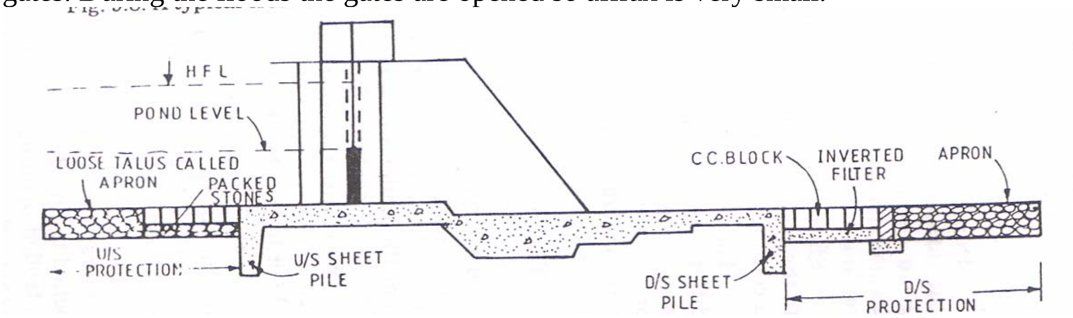
A silt excluder is a structure in the undersluices pocket to pass the silt laden water to the downstream so that only clear water enters into the canal through head regulator. The bottom layer of water which are highly charged with silt pass down the silt excluder an escape through the undersluices.

Guide Banks and Marginal Bunds

Guide banks are provided on either side of the diversion headworks for a smooth approach and to prevent the river from outflanking. Marginal bunds are provided on either side of the river upstream of diversion headworks to protect the land and property which is likely to be submerged during ponding of water in floods.

Weir or Barrage

A diversion head works is a structure constructed across a river for the purpose of raising water level in the river so that it can be diverted into the offtaking canals. A weir is a raised concrete crest wall constructed across the river. It may be provided with small shutters (gates) on its top. In the case of weir, most of the raising of water level or ponding is done by the solid weir wall and little with by the shutters. A barrage has a low crest wall with high gates. As the height of the crest above the river bed is low most of the ponding is done by gates. During the floods the gates are opened so afflux is very small.



A weir maintains a constant pond level on its upstream side so that the water can flow into the canals with the full supply level (F.S.L.). If the difference between the pond level and the crest level is less than 1·5 m or so, a weir is usually constructed. On the other hand, if this difference is greater than 1·50 m, a gate-controlled barrage is generally more suitable than a weir. In the case of a weir, the crest shutters are dropped during floods so that the water can pass over the crest. During the dry period, these shutters are raised to store water upto the pond level. Generally, the shutters are operated manually, and there is no mechanical arrangement for raising or dropping the shutters. On the 'other hand, in the case of a barrage, the control of pondage and flood discharge is achieved with the help of gates which are mechanically operated

ADVANTAGES AND DISADVANTAGES OF WEIRS AND BARRAGES

1. Weirs

Advantages: The initial cost of weirs is usually low.

Disadvantages:

(i) There is a large afflux during floods which causes large submergence.
(ii) Because the crest is at high level, there is great silting problem
(iii) The raising and lowering of shutters on the crest is not convenient. Moreover, it requires considerable time and labour.
(iv) The weir lacks an effective control on the river during floods.

2. Barrages

Advantages

(i) The barrage has a good control on the river during floods. The outflow can be easily regulated by gates.
(ii) The afflux during floods is small and, therefore, the submerged area is less.
(iii) There is a good control over silt entry into the canal.
(iv) There is a good control over flow conditions, shoal formations and cross-currents on the upstream of the barrage.
(v) There are better facilities for inspection and repair of various structures.

(vi) A roadway can be conveniently provided over the structure at a little additional cost.

Disadvantages: The initial cost of the barrage is quite high.

Conclusion: A barrage is generally better than a weir. Most of the diversion headworks these days usually consist of barrages.

TYPES OF WEIRS

The weirs may be broadly divided into the following types

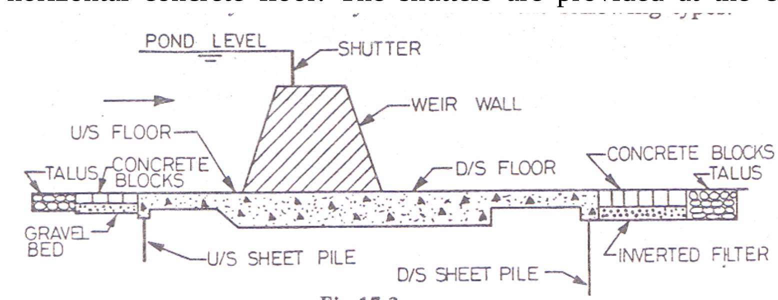
(i) Vertical drop weirs.

(ii) Rockfill weirs.

(iii) Concrete glacis or sloping weirs.

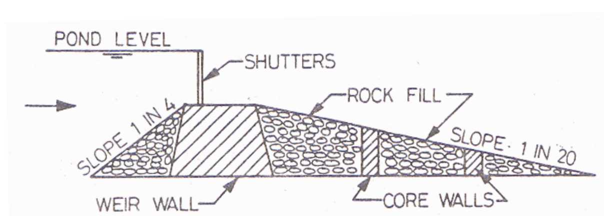
1. Vertical drop weirs

A vertical drop weir consists of a masonry wall with a vertical (or nearly vertical) downstream face and a horizontal concrete floor. The shutters are provided at the crest, which are dropped during floods so as to reduce afflux. The water is ponded upto the top of the shutters during the rest of the period. Vertical drop weirs were quite common in early diversion headworks, but these are now becoming more or less obsolete. The vertical drop weir is suitable for hard clay foundation as well as consolidated gravel foundations, and where the drop is small. The upstream and downstream cutofIwalls (or piles) are provided upto the scour depth. The weir floor is designed as a gravity section.



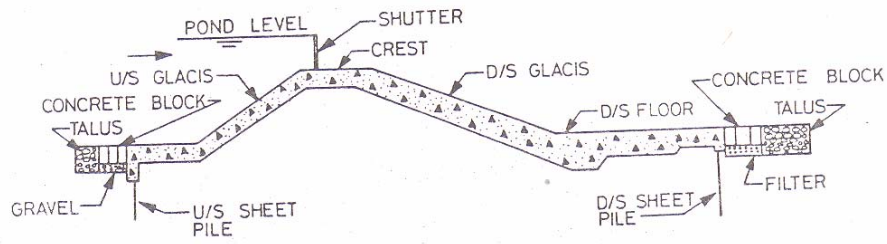
2. Rockfill weirs

In a rockfill type weir, in addition to the main weir wall, there are a number of core walls. The space between the core walls is filled with the fragments of rock (called rockfill). A rockfill weir requires a lot of rock fragments and is economical only when a huge quantity of rockflll is easily available near the weir site. It is suitable for fine sand foundation. The old Okhla Weir across the Yamuna river is a rockfill weir. Such weirs are also more or less obsolete these days.



3. Concrete sloping weir

Concrete sloping weirs (or glacis weirs) are of relatively recent origin. The crest has glacis (sloping floors) on upstream as well as downstream. There are sheet piles (or cut off walls) driven upto the maximum scour depth at the upstream and downstream ends of the concrete floor. Sometimes an intermediate pile is also driven at the beginning of the upstream glacis or at the end of downstream glacis. The main advantage of a sloping weir over the vertical drop weir is that a hydraulic jump is formed on the d/s glacis for the dissipation of energy. Therefore, the sloping weir is quite suitable for large drops.



Modes of Failure

Irrigation structures (or hydraulic structures) for the diversion and distribution works are weirs, barrages, head regulators, distributary head regulators, cross regulators, cross-drainage works, etc. These structures are generally founded on alluvial soils which are highly pervious. Moreover, these soils are easily scoured when the high velocity water passes over the structures. The failures of weirs constructed on the permeable foundation may occur due to various causes, which may be broadly classified into the following two categories:

1. Failure due to- subsurface flow 2. Failure due to surface flow

1. Failure due to subsurface flow

The failure due to subsurface flow may occur by piping or by rupture of floor due to uplift.

(a) Failure by piping Piping (or undermining) occurs below the weir if the water percolating through the foundation has a large seepage force when it emerges at the downstream end of the impervious floor. When the seepage force exceeds a certain value, the soil particles are lifted up at the exit point of the seepage. With the removal of the surface soil particles, there is further concentration of flow in the remaining portion and more soil particles are removed. This process of backward erosion progressively extends towards the upstream side, and a pipe-like hollow formation occurs beneath the floor. The floor ultimately subsides in the hollows so formed and fails. This type of failure is known as *piping failure.*

(b) Failure by rupture of floor The water percolating through the foundation exerts an upward pressure on the impervious floor, called the uplift pressure. If the weight of the floor is not adequate to counterbalance the uplift pressure, it may fail by rupture.

2. Failure due to surface f1ow

The failure due to surface flow may occur by suction pressure due to hydraulic jump or by scouring of the bed.

(a) Failure by suction pressure In the glacis type of weirs, a hydraulic jump is formed on the d/s glacis. In this case, the water surface profile in the hydraulic jump trough is much lower than the subsoil H.G.L. Therefore uplift pressure occurs on the glacis. This uplift pressure is known as the *suction pressure.* If the thickness of floor is not adequate, the rupture of floor may occur.

(b) Failure by scour During floods, scouring occurs in the river bed. The bed of the river may be scoured to a considerable depth. If no suitable measures are adopted, the scour may cause damage to the structure and may lead to the failure.

Design aspects

The basic principles for the design of all irrigation structures on pervious foundations are as follows:

(a) Subsurface flow

1. The structure should be designed such that the piping failure does not occur due to subsurface flow.
2. The downstream pile must be provided to reduce the exit gradient and to prevent piping.
3. An impervious floor of adequate length is provided to increase the path of percolation and to reduce the hydraulic gradient and the seepage force.
4. The seepage path is increased by providing piles and impervious floor to reduce the uplift pressure.
5. The thickness of the floor should be sufficient to resist the uplift pressure due to subsurface flow. The critical section is d/s of the weir/crest wall.
6. A suitably graded inverted filter should be provided at the downstream end of the impervious floor to check the migration of soil particles along with water. The filter layer is loaded with concrete blocks. Concrete blocks are also provided at the upstream end.

(b) Surface flow

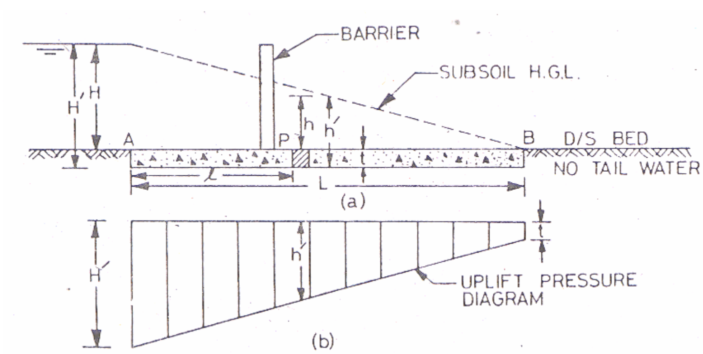
1. The piles (or cutoff walls) at the upstream and downstream ends of the impervious floor should be provided upto the maximum scour level to protect the main structure against scour.
2. The launching aprons should be provided at the upstream and downstream ends to provide a cover to the main structure against scour.
3. A device is required at the downstream to dissipate energy. For large drops, hydraulic jump is used to dissipate the energy.
4. Additional thickness of the impervious floor is provided at the point where the hydraulic jump is formed to counterbalance the suction pressure.
5. The floor is constructed as a monolithic structure to develop bending resistance (or beam action) to resist the suction pressure.

Floor Thickness

The floor should have appropriate thickness to counteract the uplift pressure acting on it. At selected point let the residual head is h which is the subsoil H.G.L. measured from the top surface of the floor. If h' is the head measured above the bottom surface of the floor, then

$$h' = h + t$$

where t is the thickness of floor. Fig. shows the uplift pressure diagram on the bottom surface. It is more convenient to measure the intercept h than the intercept h'. The intercept h' above the bottom surface of the floor can be determined only after the thickness t has been determined or has been assumed. For the determination of the floor thickness t. let us consider the force acting on the unit area of the floor (shown hatched) so as



$$\gamma_w h' = \gamma_c t \qquad \Rightarrow \qquad h + t = S_c t \qquad \Rightarrow \qquad t = h/(S_c - 1)$$

where S_c = specific gravity of the floor material. For plain concrete floor, the value of S_c usually varies from 2.0 to 2.3 depending upon the type of aggregates used. A value of 2.24 is usually adopted. Generally, a factor of safety of 4/3 is adopted. Thus

$$t = \frac{4}{3}\frac{h}{S_c - 1} = \frac{4}{3}\frac{P_{uH}}{S_c - 1}$$

where $P_{uH} = h$ is uplift pressure head at that point above the top surface of the floor.

Bligh's Theory

In 1910, W.G. Bligh gave creep theory. According to this theory, the percolating water creeps along the contact surface of the base profile of the structure with the subsoil. The length of the path thus traversed by the percolating water is called the length of creep or the creep length. As the water creeps from the upstream end to the downstream end, the head loss occurs. The head loss is proportional to the creep distance travelled. Bligh made no distinction between the creep in the horizontal direction below the floor and the creep in the vertical direction along the faces of the piles. Bligh's theory is quite simple and convenient. A large number of early irrigation structures were designed using this theory. Some of these structures are existing even today, but unfortunately a few of them failed. The theory is now rarely used for the design of large, important irrigation structures. However, sometimes it is used for the design of small structures or for the preliminary design of large structures. Limitations are: (1) The Bligh theory does not differentiate between the vertical creep and the horizontal creep and gives the same weightage to both, actually, the vertical creep is more effective than the horizontal creep. (2) The theory assumes that the head loss variation is linear, while the actual head loss variation is non-linear. (3) No distinction is made between the head loss on the outer faces and that on the inner faces of the sheet piles. Actually, the outer faces are more effective than the inner faces. (4) The theory does not emphasise the importance of the downstream pile without which piping failure occurs. It considers the downstream pile only as a component of the total creep length and not as a controlling factor for the exit gradient and the piping. (5) The theory does not give any theoretical or practical method for the determination of the safe gradient. (6) Bligh did not consider the effect of the length of the intermediate pile. Later investigations by Khosla indicated that the intermediate pile is ineffective if its length is shorter than that of the outer piles. However, there is some local redistribution of uplift pressure.

Further, according to Bligh, the subsoil hydraulic gradient, which is the loss of head per unit length of creep, is constant throughout the seepage path. Thus if the seepage head (which is the difference of water levels on the upstream and downstream of weir) is H_s (the total loss of head) and L_T is the total creep length, the loss of head per unit length is equal to $H_s\ /L_T$ which is inverse of Bligh’s Creep coefficient C (usually varies from 10 to 18 depending upon the bed material). Therefore for known seepage head and creep coefficient the required creep length or seepage path $L_T = C\ H_s$. The uplift pressure at any point can be determined by Bligh theory and then required thickness to counteract it.

Lane's Theory

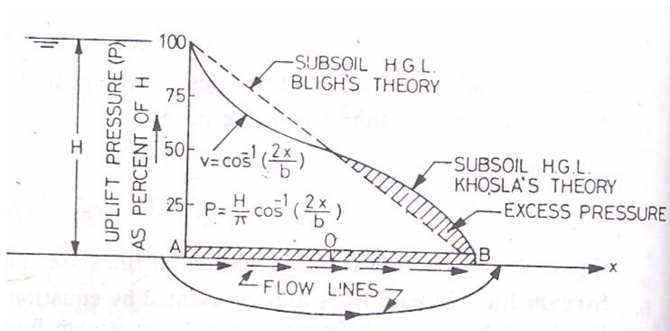
Lane analysed a large number of dams and weirs founded on pervious foundations which failed or did not fail. He brought out deficiencies in Bligh's creep theory and gave a new theory on statistical basis known as Lane's weighted creep theory. This theory gives the vertical creep three times more weightage as compared to the horizontal creep.

Khosla's Theory

In 1926-27, some siphons constructed on the Upper Chenab Canal on the basis of Bligh's creep theory, had undermining problems. Uplift pressure measurements by Dr. A.N. Khosla, Dr. N.K. Bose and Dr. E.M. Taylor indicated that the actual uplift pressures were quite different from those computed on the basis of Bligh's theory. These investigations showed that (1) the outer faces of the end sheet piles are much more effective than the inner faces and the horizontal length of floor, (2) the intermediate piles of smaller in length than the outer piles are ineffective except for some local redistribution of pressure, (3) undermining (piping) of the floors starts from the tail end when the hydraulic gradient at the exit is greater than the critical gradient for that particular soil. The soil particles move with the flow of water, thus causing progressive degradation of the subsoil and resulting in cavities below the floor and ultimate failure. Therefore it is absolutely essential to have a reasonably deep cutoff (or pile) at the downstream end of the floor to prevent undermining (or piping).

Khosla et al provided a complete rational solution of the problem based on potential flow theory and Schwartz-Christoffel transformation. Various cases were analysed and studied by them. The resultant Khosla's theory gives uplift pressure at various points of the structure, depending upon its profile. It also gives the exit gradient. To ensure that the piping failure does not occur, there must be a downstream pile and the exit gradient should be safe. Moreover, the thickness of the floor should be adequate to resist uplift pressure. The uplift pressure head at any point is

$$P_{uH} = \frac{H_s}{\pi}\cos^{-1}\left(\frac{2x}{b}\right)$$



where H_s = seepage head and b = horizontal distance between entry and exit points of impervious floor.

Khosla's expression for the exit gradient is

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

where $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$; $\alpha = \frac{b}{d}$; and d = depth of d/s pile.

Computation of Uplift Pressure by Khosla's Theory

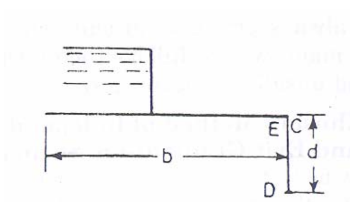
The practical profile of a hydraulic structure rarely conforms to any single elementary form for which the mathematical solution was obtained by Khosla et al. For the determination of uplift pressure at the key points of a composite structure Khosla et al gave the theory of independent variables. According to this theory a composite profile is split into a number of simple elementary standard forms for which the mathematical solution can easily be obtained. Each elementary form is then treated independent of the other and the pressures at its key points are obtained from the solution already available. Then the solutions of these elementary forms are superposed to obtain the pressure distribution at all the key points of the entire structure. Further these pressures are to be corrected as the individual pressures have been obtained based on the assumptions (i) the floor is of negligible thickness, (ii) there is only one pile line, (iii) the floor is horizontal. Therefore for any given profile of a weir/barrage/hydraulic structure on pervious foundation the following three steps may be adopted to compute uplift pressure at any point:

1. Decompose the general profile into elementary profiles
2. Assemble/superposition of uplift pressures at key points
3. Correction and interpolation

1. Uplift Pressures at Key Points in Elementary cases

(i) Pile at downstream end

The uplift pressure head at key points E, D and C as shown in Fig is given by



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

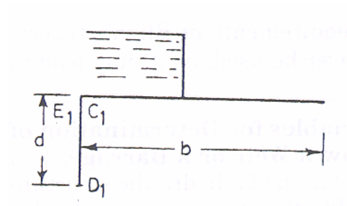
$$P_{uHD} = \frac{H_s}{\pi} \cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right)$$

$$P_{uHC} = 0$$

where $\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2}$; and $\alpha = \frac{b}{d}$.

(ii) Pile at upstream end

The uplift pressure head at key points E_1, D_1 and C_1 as shown in Fig is given by



$$P_{uHC1} = \frac{H_s}{\pi} \cos^{-1}\left(\frac{2 - \lambda}{\lambda}\right)$$

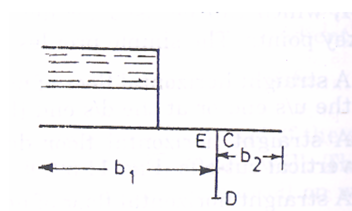
$$P_{uHD1} = \frac{H_s}{\pi} \cos^{-1}\left(\frac{1 - \lambda}{\lambda}\right)$$

$$P_{uHE1} = H_s$$

where $\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2}$; and $\alpha = \frac{b}{d}$.

(iii) Intermediate Pile

For this case the uplift pressure head at key points E, D and C as shown in Fig is given by



$$P_{uHE} = \frac{H_s}{\pi} \cos^{-1}\left(\frac{\lambda_2 - 1}{\lambda_1}\right)$$

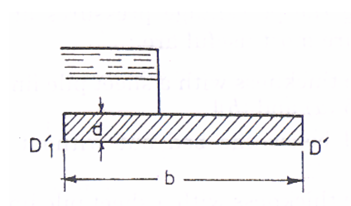
$$P_{uHD} = \frac{H_s}{\pi} \cos^{-1}\left(\frac{\lambda_2}{\lambda_1}\right)$$

$$P_{uHC} = \frac{H_s}{\pi} \cos^{-1}\left(\frac{\lambda_2 + 1}{\lambda_1}\right)$$

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

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

$$\lambda_2 = \frac{\sqrt{1 + \alpha_1^2} - \sqrt{1 + \alpha_2^2}}{2}$$



(iv) Depressed floor

The uplift pressure head at key points D'_1 and D' as shown in Fig is given by

$$P_{uHD'} = P_{uHD} - \frac{2}{3}\left(P_{uHE} - P_{uHD}\right) + \frac{3H_s}{\alpha^2} = \frac{5P_{uHD} - 2P_{uHE}}{3} + \frac{3H_s d^2}{b^2}$$

$$P_{uHD_1'} = H_s - P_{uHD'}$$

Principle of reversibility

It may be observed from different expressions for elementary cases that some of them are similar except sign. For example u/s pile and d/s pile cases if

$$\cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right) = \theta \text{ then } P_{uHE} = \frac{H_s}{\pi}\theta \text{ and } P_{uHC1} = \frac{H_s}{\pi}(\pi - \theta) = H_s - P_{uHE}$$

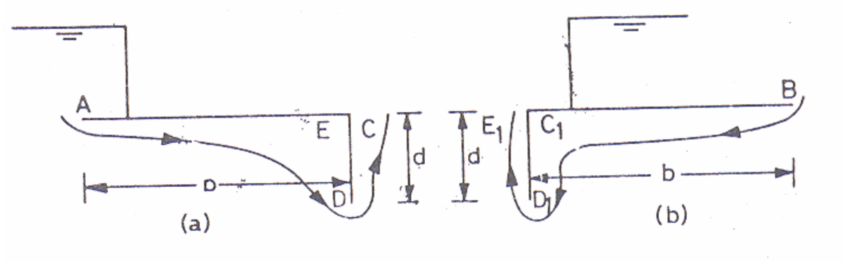
or $\quad P_{uHE} = H_s - P_{uHC1}$

Similarly if

$$\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right)=\theta \text{ then } P_{uHD}=\frac{H_s}{\pi}\theta \text{ and } P_{uHD1}=\frac{H_s}{\pi}(\pi-\theta)=H_s-P_{uHD}$$

or $\quad P_{uHD}=H_s-P_{uHD1}$

Also $\quad P_{uHE1}=H_s-P_{uHC}$



Thus the pressure at key points in pile at u/s case can be obtained from the pressure at key points in pile at d/s case by subtracting from the seepage head provided the length of floor and the depth of pile are same in both cases. This is based on the principle of reversibility. According to this principle if the direction of flow is reversed but other conditions remain the same, the flow pattern below the structure does not change as shown in Fig.

Assemble/superposition of uplift pressures at key points

Correction and interpolation

In actual structure the floor has some thickness and it may not be horizontal and also there may be more than one line of piles, so the following corrections have to be applied to the superposed values of the uplift pressures

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

Correction for thickness of floor

Simple linear interpolation is adopted for this correction e.g.

$$P_{uHE1}=P_{uHE}-\frac{P_{uHE}-P_{uHD}}{d}t$$

$$P_{uHC1}=P_{uHC}+\frac{P_{uHD}-P_{uHC}}{d}t$$

Correction for mutual interference of piles

Interference is local, only on facing side of interfering pile given by

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

where C is the percentage correction, b' is the distance between the two piles, b is the total length of impervious floor, D is the depth of the pile whose effect is required to be determined on the adjacent pile of depth d. The following points may be noted: (i) the correction is positive if the point is u/s to the interfering pile and negative if the point is u/s to the interfering pile; (ii) both D and d are measured below the level at which the interference is required; (iii) the effect of interference of a pile is determined only for that face of the pile which is towards the interfering pile; (iv) the mutual interference does not apply to find the effect of an outer pile on an intermediate pile if the intermediate pile is equal to or less than the outer pile and is at a distance less than the twice the length of the outer pile.

Correction for sloping floor

In the derivation of the expressions for the uplift pressure at the key points, the floor has been assumed to be horizontal. If the floor is sloping, the correction is applicable to key points at beginning and end of sloping floor as given by

$$C = \pm\left(\frac{b_s}{b'}\right)C' \quad \% \text{ of } H_s$$

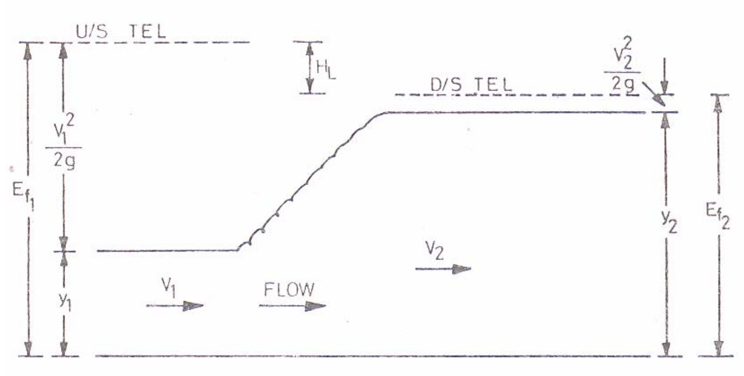
where b_s is the horizontal length of the sloping floor, b is the distance between the two piles in between which the sloping floor lies, C' varies from 2 for slope 8:1 to 11.2 for slope 1:1. The correction is negative for the upslope and is positive for the downslope. It may be noted that the correction is applicable only to the key points of the pile lines which lie either at the beginning or at the end of the sloping floor.

Uplift at Points other than Key Points

The corrected pressures at all key points can be determined as described above. The uplift at any point on the floor between the two key points/pile lines is obtained by linear interpolation of the pressures at key points of these two piles.

SURFACE FLOW

Hydraulic jump is formed to dissipate energy on d/s glacis. A suction pressure occurs on the floor due to hydraulic jump. For the determination of the suction pressure, the location of hydraulic jump and the profile of water surface on the upstream and downstream of the point at which the jump is formed are required. The basic relations for a hydraulic jump as shown in Fig are



$$E_{f1} = y_1 + \frac{V_1^2}{2g}$$

$$E_{f2} = y_2 + \frac{V_2^2}{2g}$$

$$E_{f2} = E_{f1} + H_L$$

The conjugate depths before and after the jump (y_1 and y_2) are related as

$$y_1 y_2 (y_1 + y_2) = \frac{2q^2}{g}$$

$$H_L = \frac{(y_2 - y_1)^3}{4 y_1 y_2}$$

The computation of y_1 and y_2 for known values of q and HL is very cumbersome and inconvenient. To simply the process Blench, Montague and IS code provided curves. To overcome these curves Swamee's presented the following method

Let $y = \frac{y_2}{y_c}$; $x = \frac{y_1}{y_c}$; and $z = \frac{H_L}{y_c}$ then

$$xy(x + y) = 2$$

$$z = \frac{(y - x)^3}{4xy}$$

where critical depth

$$y_c = \left(q^2/g\right)^{1/3}$$

These two equations involve x and y as function of z. Swamee solved these equation varying z and then fitted into explicit equation for y

$$y = 1 + 0.93556(z)^{0.368} \quad \text{for z < 1}$$

$$y = 1 + 0.93556(z)^{0.24} \quad \text{for z > 1}$$

Once y is known other parameters x, y_1, y_2, E_{f1} and E_{f2} are calculated

$$E_{f1} = \left(x + \frac{1}{2x^2} \right) y_c$$

$$E_{f2} = \left(y + \frac{1}{2y^2} \right) y_c$$

Design of vertical drop weir

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

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

Lacey's silt factor (f) for the silt at the river site is generally decided by experience. It can also be determined from the average size of the particles as $f = 1.76\sqrt{d_s}$, where d_s is the average size (not radius) of particle (mm).

Afflux is the rise in water level on the upstream of the structure after the construction of the weir. The high flood level on the upstream is higher than that at the downstream. The area of submergence and the top levels of the marginal banks and guide banks depend upon the afflux. The location of the hydraulic jump on the downstream glacis also depends upon u/s TEL which is a function of afflux. If the afflux is very large, the length of the weir will be small because of high discharge intensity (q) over the crest. However, the cost of the river training works (guide banks, marginal bunds, etc) will increase. Moreover, the risk of the failure of the structure due to outflanking will also increase. Further, the scour depth will be large and it will increase the cost of protection works on the upstream and downstream of the impervious floor. The afflux is usually limited to 1 m. IS: 6933-1973 recommends an afflux of 1 m for the alluvial rivers in the upper and middle reaches of the river and of 0,3 m in the lower reaches.

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

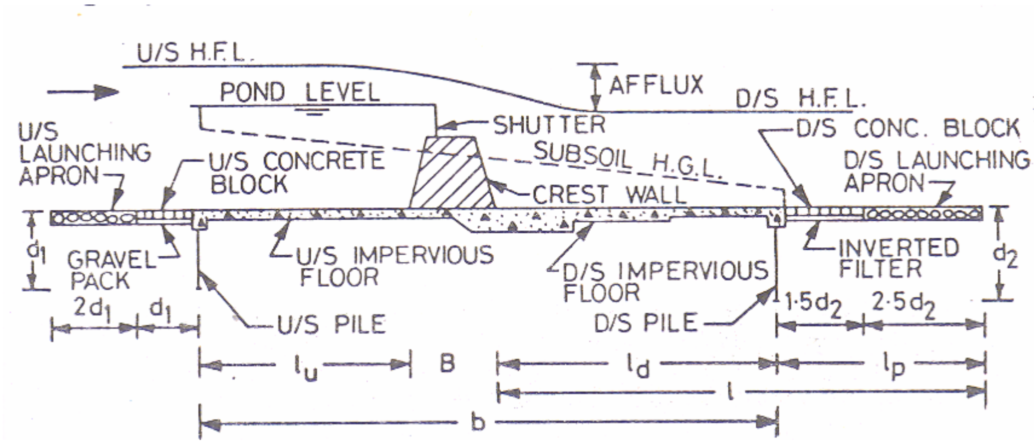
$$Q = C_d L_e H_e^{3/2}$$

where C_d is the coefficient of discharge which depends upon the type of crest, L_e is the effective length, and H_e is the head over the crest, including the head due to velocity of approach. For sharp crested weir (top width < 2 H_e/3) C_d = 1.84 and for broad crested weir (top width > 2 H_e/3) C_d = 1.1.703.

The effective length L_e. is determined as follows: $L_e = L' - 2(NK_p + K_a)H_e$, where L' = clear length excluding total width of piers, N is the number of piers, K_p is the pier contraction coefficient and K_a is the abutment contraction coefficient. Coefficients K_p and K_a *d*epend on the shape of piers and abutment respectively.

There are FOUR broad design steps

A. hydraulic calculation to fix various levels
B. design for weir wall
C. design for impervious floor and piles
D. design for u/s and d/s protection



A. *Hydraulic calculation to fix various levels*

1. Length of waterway (L) = regime perimeter P. From Lacey's regime theory, $L = P = (4.5\ to\ 6.3)\sqrt{Q}$ where Q is the design discharge.
2. Discharge intensity $q = Q/L$
3. Using Lacey's theory Normal Scour depth $R = 1.35\left(q^2/f\right)^{1/3}$, where f Lacey's silt factor
4. Determine the regime velocity of flow $V = q/R$ and then the velocity head, $h_a = V^2/2g$
5. d/s TEL = HFL before construction + h_a
6. u/s TEL = d/s TEL + afflux
7. u/s HFL. = u/s TEL - h_a
8. Determine the head required over the crest (H_e) for passing the design intensity q, assuming that the weir acts as a broad-crested weir $q = 1.705(H_e)^{3/2} \Rightarrow \quad H_e = (q/1.705)^{2/3}$
9. Crest level = u/s TEL - H_e (If the crest level is lower, afflux will be less because the head over the crest is increased and consequently, the discharge intensity is also increased. However, a low crest gives rise to an increased depth of water over the crest upto the pond level. It results in the increased height of gates, thickness of floor and the overall cost of the structure).
10. Pond level = FSL of offtaking canal + Head loss through head regulator (The head loss through the head regulator is usually taken between 0.5 m to 1.0 m, depending upon the type of regulator). The full supply level (FSL) of the canal depends upon a number of factors, such as the water requirements of crops, the topography, the head loss in the canal system and the cumulative fall in the water surface levels from the head to the tail of the canal.
11. Height of shutters *(s)* = Pond level - Crest level

B. Design for weir wall

1. The top level of the weir wall is kept at the required crest level therefore Height of Weir H = Crest level - Bed level of river bed.
2. Top width (a) and Base width (B) of weir can be estimated considering elementary profile of a gravity dam. The top width of the weir wall is fixed as the largest of the three values $a = d/\sqrt{S_c}$; $d/\mu S_c$; $s\,(height\ of\ shutters) + 1$ where d = water depth above crest = u/s HFL level - Crest level $= H_e$ - h_a ; S_c = specific gravity of weir wall material; μ = friction factor. The top. width should be sufficient so that when the shutter is laid over it during floods, it does not project beyond the wall.
3. Bottom/Base width (B) should be sufficient so that the maximum compressive stresses are within the allowable limits and the tension does not develop. For preliminary design, the base width may be taken as the largest of the three values $B = (H + d)/\sqrt{S_c}$; $(H + d)/\mu S_c$; $a + 0.8H$. The last criteria is based on the slopes on u/s (1:0.3) and d/s (1:0.5) faces.
4. Stability analysis assuming weir a gravity dam should be performed under different conditions (eg No flow, High flood, Normal water at pond level etc) to determine the developed stresses, those must be within permissible values.

C. Design for impervious floor and piles

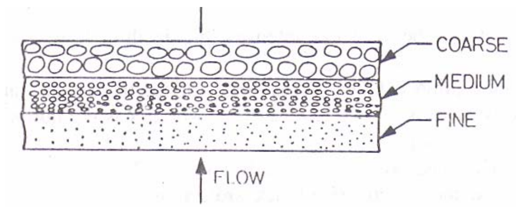
1. Seepage head is the difference between water levels u/s and d/s of weir due to which seepage takes place. During high flood the head difference is equal to Afflux. During lean period gates are raised and all the river water is diverted into canal so water level is at pond level at u/s and no tail water at d/s and hence seepage head is equal to pond level minus river bed level. Normally the worst condition occurs during lean period. Thus the maximum seepage head (H_s) = Max. of (Afflux, Pond level - river bed level) or H_s = Pond level - river bed level = $H + s$.
2. Total length of impervious floor $L_T = C\, H_s$ from Bligh's theory where C = Bligh's creep coefficient.
3. Depths of u/s and d/s piles are fixed based upon the maximum scour depth, which is 1.25 R to 1.5 R for u/s pile and 1.5 R to 2 R for u/s pile. Thus d/s Max scour depth = d/s HFL – 2R and u/s Max scour depth = u/s HFL – 1.5R . Therefore d_2 = 2R – (d/s HFL - d/s bed level) and d_1 = 1.5R – (u/s HFL - u/s bed level). where d_1. and d_2 are the depths of the u/s and d/s piles below the bed levels, respectively.
4. The length of the horizontal floor *(b)* = $l_u + B + l_d = L_T - 2d_1$. - $2d_2$ where l_u = length of impervious floor u/s of weir and l_d = length of impervious floor d/s of weir.
5. Certain minimum length of impervious floor d/s of weir is always required to dissipate energy and avoid scouring, which is from Bligh's consideration $l_d = 2.21C\sqrt{H_s/13}$
6. Length of u/s impervious floor $l_u = L_T - 2d_1$. - $2d_2$ - B - l_d
7. Thickness of u/s floor: The upstream floor is provided with a nominal thickness of about 0.6 m to 1.0 m, as the net uplift force is zero on the u/s floor.
8. The thickness of the d/s floor in l_d length is determined by computing uplift pressures at selected points. The section just d/s of weir is critical where

maximum thickness has to be provided. In the remaining part the thickness may be provided in suitable number of steps. *Bligh's theory* may be used to determine uplift pressures at selected points eg just d/s of weir $P_{uH1} = (2d_1 + l_u + B)H_s/L_T$ and just before d/s pile $P_{uH2} = (2d_1 + l_u + B + l_d)H_s/L_T$ and hence thicknesses at these points are $t_1 = \frac{4}{3}\frac{P_{uH1}}{S_c - 1}$ and $t_2 = \frac{4}{3}\frac{P_{uH2}}{S_c - 1}$ respectively. The thickness at 3 to 5 points is generally found; depending upon the length of d/s floor. The thickness of floor from the weir wall to d/s end is reduced in steps for ease in construction.

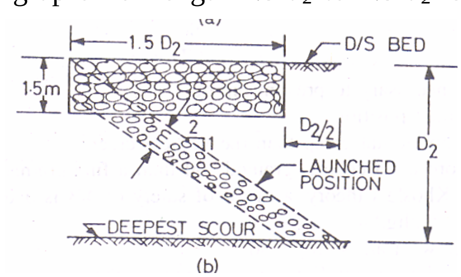
9. Use Khosla's method to determine uplift pressures and corresponding thicknesses and also exit gradient for tentative dimensions fixed using Bligh's theory. The final dimensions of the impervious floor and piles must not be unsafe as well over safe.

D. *Design for u/s and d/s protection*

1. An inverted filter is provided immediately downstream of the d/s impervious floor beyond the d/s pile to relieve the pressure along with filtering out foundation material so that washing out of fine particles does not occur. The filter is properly graded, with the finer layer at the bottom. The total thickness of filter is usually between 50 to 75 cm. The length of the inverted filter is generally kept equal to 1.5 d_2 to 2 d_2.



2. To prevent the damage and dislocation of the inverted filter due to surface flow and counteract uplift, it is generally loaded with concrete blocks or block stones of size 90 to 120 cm cube, generally 1 m × 1 m × 1 m. The joints between the concrete blocks are 10 cm thick filled with sand or bajri.
3. On the d/s on the inverted filter, a launching apron of length 1.5 d_2 to 2.5 d_2 is provided. It consists of loosely packed stones. The apron is initially laid horizontal at the river bed level but when scouring occurs, it settles and takes an inclined position. The launching apron protects the impervious floor, d/s pile and inverted filter, as it forms a protective covering of stones over a certain slope below the river bed. It is generally assumed that the aprons launch at a slope of 2:1 to 3:1. The thickness of the apron in the launched position is usually specified as 0.9 m to 1.0 m. The thickness of the apron in the horizontal position can be found from the volume of stone in the launched position. 'For example. for slope of 3: 1 and the launched thickness of 1 m, the thickness in horizontal position = $\sqrt{10}d_2/2d_2$.



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