Hydraulic Structures

Theories of Seepage

Causes of failure of weir or barrage on permeable foundation:

I- **FAILURE DUE TO SUBSURFACE FLOW**

A. Failure by Piping or undermining

The water from the upstream side continuously percolates through the bottom of the foundation and emerges at the downstream end of the weir or barrage floor. The force of percolating water removes the soil particles by scouring at the point of emergence. As the process of removal of soil particles goes on continuously, a depression is formed which extends backwards towards the upstream through the bottom of the foundation. A hollow pipe like formation thus develops under the foundation due to which the weir or barrage may fail by subsiding. This phenomenon is known as failure by piping or undermining.

B. Failure by Direct uplift

The percolating water exerts an upward pressure on the foundation of the weir or barrage. If this uplift pressure is not counterbalanced by the self-weight of the structure, it may fail by rapture.

II- **FAILURE BY SURFACE FLOW**

1. By hydraulic jump

When the water flows with a very high velocity over the crest of the weir or over the gates of the barrage, then hydraulic jump develops. This hydraulic jump causes a suction pressure or negative pressure on the downstream side which acts in the direction uplift pressure. If the thickness of the impervious floor is sufficient, then the structure fails by rapture.

2. By scouring

During floods, the gates of the barrage are kept open and the water flows with high velocity. The water may also flow with very high velocity over the crest of the weir. Both the cases can result in scouring effect on the downstream and on the upstream side of the structure. Due to scouring of the soil on both sides of the structure, its stability gets endangered by shearing.

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 HL 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. HL/L) is called the hydraulic gradient. Further, Bligh makes no distinction between horizontal and vertical creep.

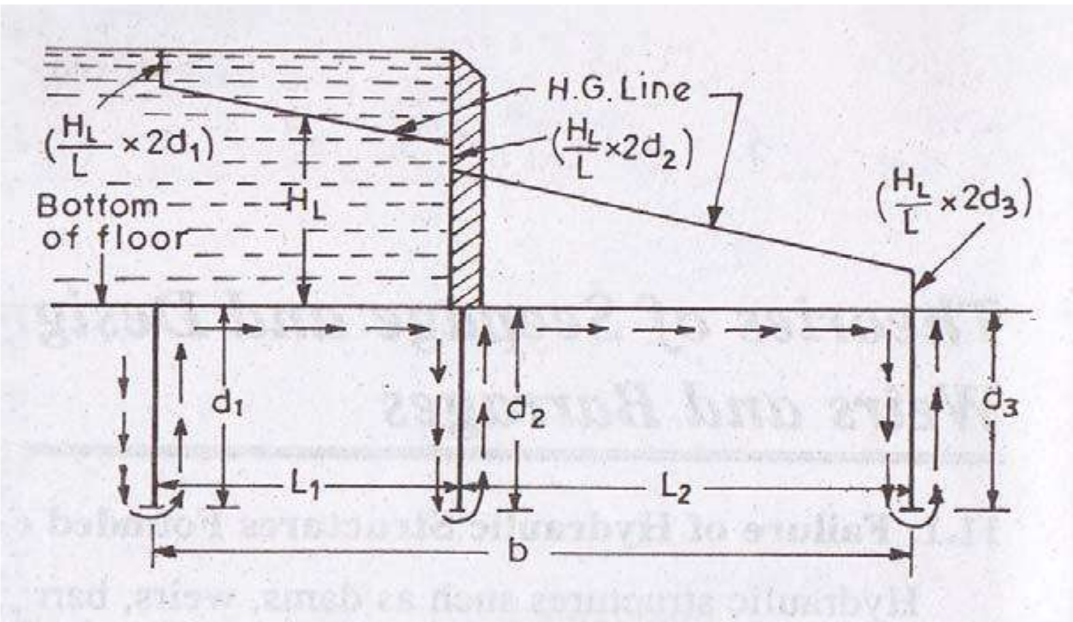


Fig-1: Bligh's Creep

Consider a section as shown in Fig above. Let H_L be the difference of water levels between upstream and downstream ends. 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

$L = d_1 + d_1 + L_1 + d_2 + d_2 + L_2 + d_3 + d_3$

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

$= b + 2(d_1 + d_2 + d_3)$

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( ),( ), ( ) ; 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 figure above.

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 the Bligh's Coefficient for the soil. Different values of C for different types of soils are tabulated in Table 1 below:

SL No.	Type of Soil	Value of C	Safe Hydraulic gradient should be less than
1	Fine micaceous sand	15	1/15
2	Coarse grained sand	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. HL/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.

1. 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′ meters is this ordinate, then water pressure equal to h′ meters will act at this point, and has to be counterbalanced by the weight of the floor of thickness say t.

Uplift pressure = $\gamma_w \times h'$ [where γ_w is the unit weight of water]

Downward pressure = $(\gamma_w \times G).t$ [Where G is the specific gravity of the floor material] For equilibrium,

$\gamma_w \times h' = \gamma_w \times G.$

$\therefore h' = G \times t$

Subtracting t on both sides, we get

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

Where, $h' - t = h$ = Ordinate of the H.G line above the top of the floor.

$G - 1$ = Submerged specific gravity of the floor material.

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 weight age factor of 1/3 for the horizontal creep, as against 1.0 for the vertical creep.

Thus in Fig–1, the total Lane's creep length (L_l) is given by

$L_l = (d_1 + d_1) + (1/3)\,L_1 + (d_2 + d_2) + (1/3)\,L_2 + (d_3 + d_3)$

= (1/3) (L1 + L2) + 2(d1 + d2 + d3)

= (1/3) b + 2(d1 + d2 + d3)

To ensure safety against piping, according to this theory, the creep length Ll must no be less than C1HL, where HL is the head causing flow, and C1 is Lane's creep coefficient given in table 2.

Table 2: Values of Lane's Safe Hydraulic Gradient for different types of Soils

SL No.	Type of Soil	Value of Lane's Coefficient C_1	Safe Lane's Hydraulic gradient should be less than
1	Very fine 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

KHOSLA'S THEORY AND CONCEPT OF FLOW NETS

Many of the important hydraulic structures, such as weirs and barrage, were designed on the basis of Bligh's theory between the periods 1910 to 1925. In 1926 – 27, the upper Chenab canal siphons, designed on Bligh's theory, started posing undermining troubles. Investigations started, which ultimately lead to Khosla's theory. The main principles of this theory are summarized below:

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

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

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

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

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 follows smooth curves transiting slowly from the outline of the foundation to a semi-ellipse, as shown below.

Equipotential Lines: (1) Treating the downstream bed as datum and assuming no water on the downstream side, it can be easily started that every streamline possesses a head equal to h1 while entering the soil; and when it emerges at the down-stream end into the atmosphere, its head is zero. Thus, the head h1 is entirely lost during the passage of water along the streamlines.

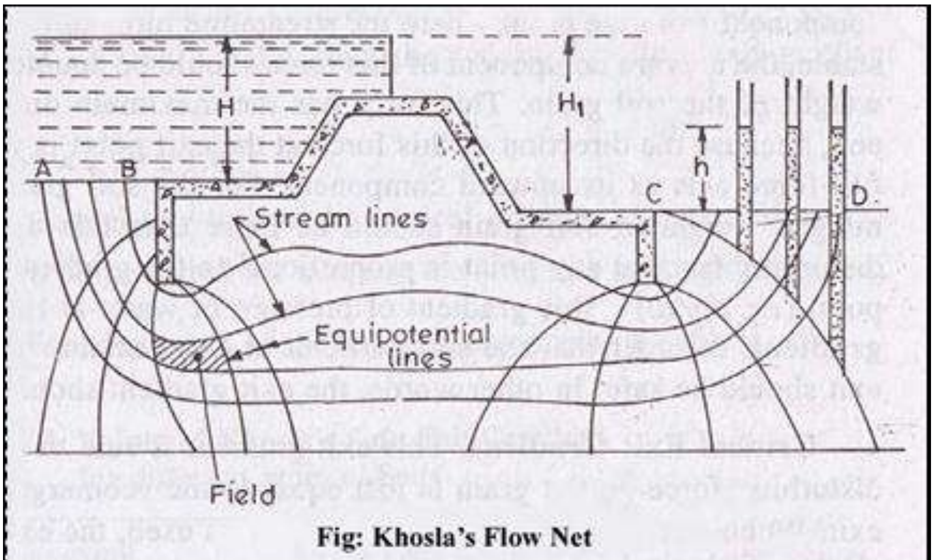
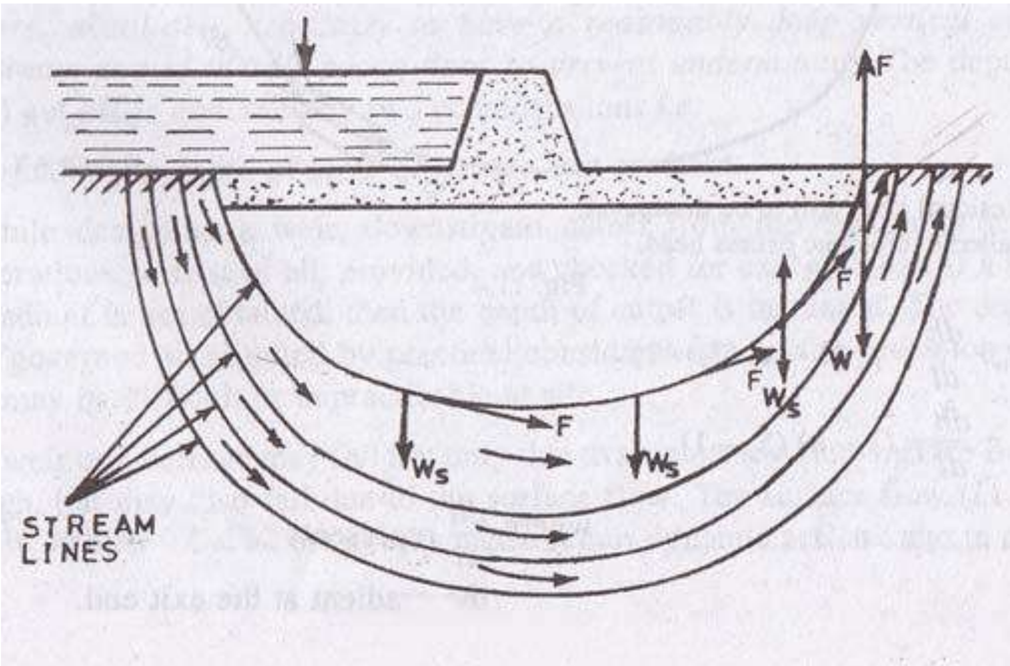


Fig: Khosla's Flow Net

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

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



(b) The seepage water exerts a force at each point in the direction of flow and tangential to the streamlines as shown in figure above. This force (F) has an upward component from the point where the streamlines turns upward. 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 to 5 is used, the exit gradient can then be taken as safe. In other words, an exit gradient equal to ¼ to 1/5 of the critical exit gradient is ensured, so as to keep the structure safe against piping.

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

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

Where, γ_w = unit weight of water.

Ss = Specific gravity of soil particles

n = Porosity of the soil material

For critical conditions to occur at the exit point

F = Ws

Where F is the upward disturbing force on the grain

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

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 with respect to the boundary conditions. These are complicated methods and are time consuming. Therefore, for designing hydraulic structures such as weirs or barrage or pervious foundations, *Khosla* has evolved a simple, quick and an accurate approach, called *Method of Independent Variables.*

In this method, a complex profile like that of a weir is broken into a number of simple profiles; each of which can be solved mathematically. Mathematical solutions of *flownets* for these simple standard profiles have been presented in the form of equations given in Figure and curves given in Plate, which can be used for determining the percentage pressures at the various key points. The simple profiles which hare 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.

(ii) A straight horizontal floor depressed below the bed but without any vertical cut-offs.

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

The key points are the junctions of the floor and the pole 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 the thickness of floor

(c) Correction for the slope of the floor

(a) Correction for the Mutual interference of Piles:

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

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

Where:

b' = the distance between two pile lines.

D = the depth of the pile line, the influence of which has to be determined on the neighboring pile of depth.

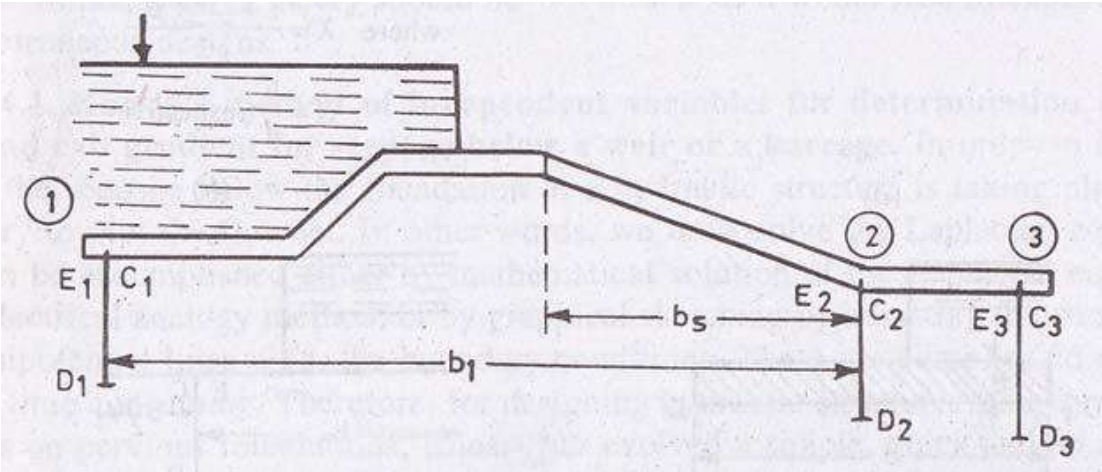
d. D is to be measured below the level at which interference is desired.

d = the depth of the pile on which the effect is considered

b = Total floor length

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

Suppose in the above figure, 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 positive. 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 pressures at the actual points are calculated by assuming a straight line pressure variation.

Since the corrected pressure at E1 should be less than the calculated pressure at E1′, the correction to be applied for the joint E1 shall be negative. Similarly, the pressure calculated C1′ is less than the corrected pressure at C1, and hence, the correction to be applied at point C1 is positive.

(c) Correction for the slope of the floor

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

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.

Exit gradient (GE)

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

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

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

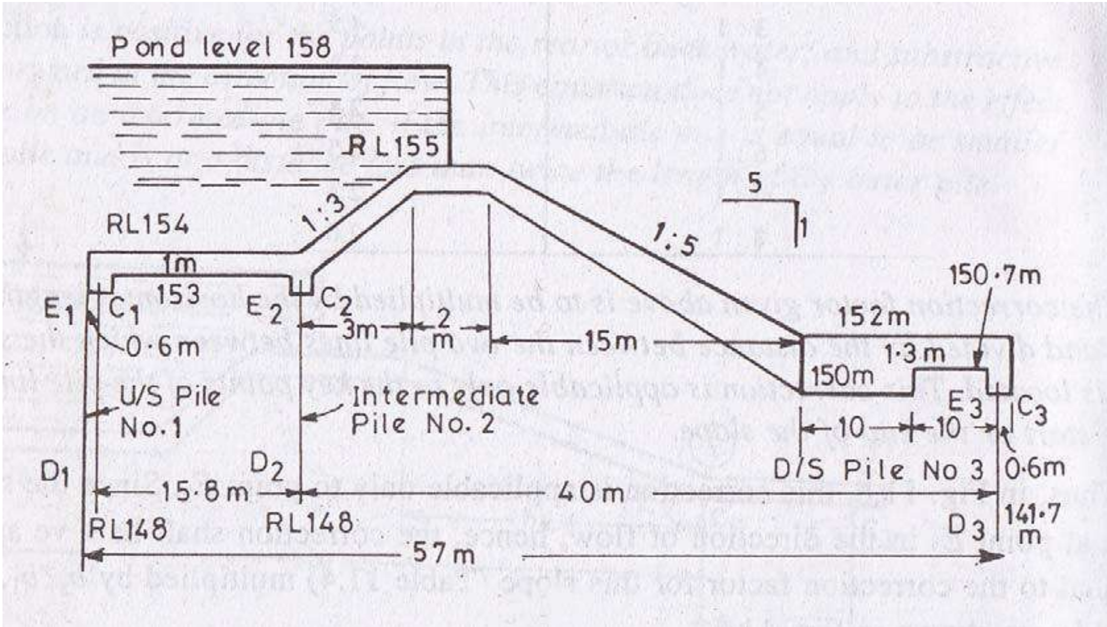
$$\alpha = b/d$$

H = Maximum Seepage Head

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

Example

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



***Solution*:**

For upstream Pile Line No. 1

Total length of the floor, b = 57.0 m Depth of u/s pile line, d = 154 – 148 = 6 m

α = b/d = 57/6 = 9.5

1/α = 1/9.5 = 0.105

From curve plate 11.1 (a)

$\varphi C1 = 100 - 29 = 71\ \%$

$\varphi D1 = 100 - 20 = 80\ \%$

These values of φC1 must be corrected for three corrections as below:

Corrections for φC1

(a) Correction at C1 for Mutual Interference of Piles (φC1) is affected by intermediate pile No.2

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

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

$$= 1.88\ \%$$

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

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

b′ = Distance between two piles = 15.8 m

b = Total floor length = 57 m

Since the point C1 is in the rear in the direction of flow, the correction is (+) ve.

Correction due to pile interference on C1 = 1.88 % (+ ve)

(b) Correction at C_1 due to thickness of floor:

Pressure calculated from curve is at C_1', (Fig. 7.1) but we want the pressure at C_1. Pressure at C_1 shall be more than at C_1' as the direction of flow is from C_1 to C_1' as shown; and hence, the correction will be + ve and

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

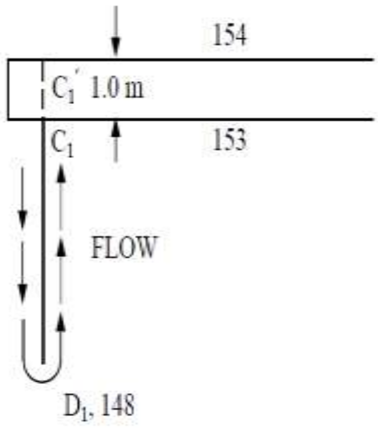


Fig: 7.1

(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

$$\therefore \text{ Corrected } (\varphi_{C1}) = 71\ \% + 1.88\ \% + 1.5\ \%$$
$$= 74.38\ \%\ (\text{ans})$$
$$\text{And } \quad (\varphi_{D1}) = 80\ \%$$

(2) For intermediate Pile Line No. 2

d = 154 – 148 = 6 m

b = 57 m

α = b/d = 57/6 = 9.5

Using curves of plate 11.1 (b), we have b1 in this case b1 = 0.6 + 15.8 = 16.4

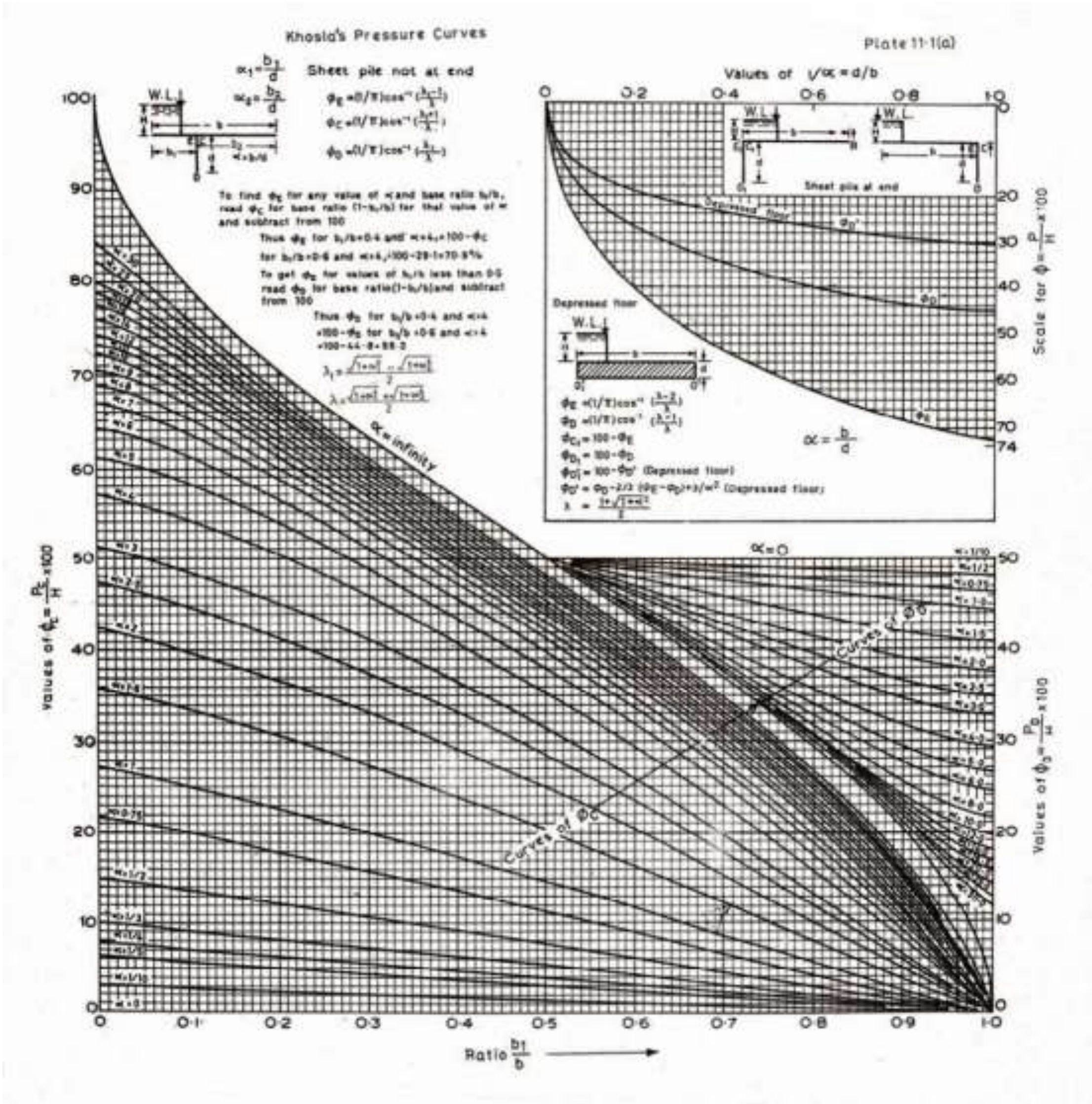
b = 57 m

b1/b = 16.4/57 = 0.298 (for φC2) 1 – b1/b = 1 – 0.298 = 0.702

φE2 = 100 – 30 = 70 % (Where 30 % is φC for a base ratio of 0.702 and α = 9.5)

φC2 = 56 % (For a base ratio 0.298 and α = 9.5)

φD2 = 100 – 37 = 63 % (Where 37 % is φD for a base ratio of 0.702 and α = 9.5)



Corrections for φE2

(a) Correction at E2 for sheet pile lines. Pile No. (1) Will affect the pressure at E2 and since E2 is in the forward direction of flow, this correction shall be – ve. The amount of this correction is given as:

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

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

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

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

(b) Correction at E_2 due to floor thickness

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

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

Since the pressure observed is at E_2' and not at E_2, (Fig. 7.2) and by looking at the direction of flow, it can be stated easily that pressure at E_2 shall be less than that at E_2', hence, this correction is negative,

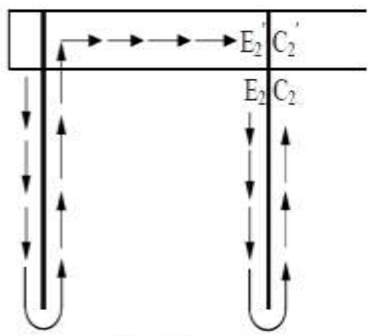


Fig: 7.2

Correction at E2 due to floor thickness = 1.17 % (- ve)

(b) Correction at E2 due to slope is nil, as the point E2 is neither situated at the start of a slope nor at the end of a shape

Hence, corrected percentage pressure at E2 = Corrected φE2 = (70 – 1.88 – 1.17) % = 66.95 %

Corrections for φC2

(a) Correction at C2 due to pile interference. Pressure at C2 is affected by pile No.(3) and since the point C2 is in the back water in the direction of flow, this correction is (+) ve. The amount of this correction is given as:

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

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

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

Where, D = Depth of pile No.3, the effect of which is considered below the level at which interference is desired = 153 – 141.7 = 11.3 m

d = Depth of pile No. 2, the effect on which is considered = 153 – 148= 5 m

b′ = Distance between two piles (2 &3) = 40 m

b = Total floor length = 57 m

(b) Correction at C_2 due to floor thickness. From Fig. 11.10, it can be easily stated that the pressure at C_2 shall be more than at C_2', and since the observed pressure is at C_2', this correction shall be + 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 m

∴ Actual correction = 4.5 × (3/40) = 0.34 % (- ve)

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

(3) Downstream Pile Line No. 3

d = 152 – 141.7 = 10.3 m

b = 57 m

1/α = 10.3/57 = 0.181

From curves of Plate 11.1 (a), we get

φD3 = 26 %

φE3 = 38 %

Corrections for φE3

(a) *Correction due to piles*. The point E3 is affected by pile No. 2, and since E3 is in the forward direction of flow from pile No. 3, this correction is negative and its amount is given by

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

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

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

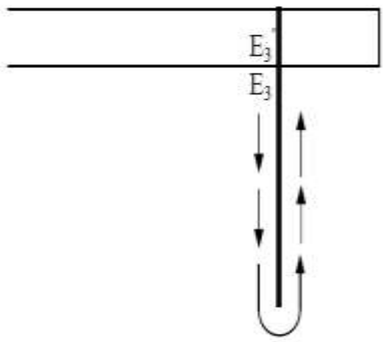
Where, D = Depth of pile No.2, the effect of which is considered = 150.7 – 148 = 2.7 m
d = Depth of pile No. 3, the effect on which is considered = 150 – 141.7 = 9 m
b′ = Distance between two piles = 40 m
b = Total floor length = 57 m

(b) Correction due to floor thickness

From Fig. 7.3, it can be stated easily that the pressure at E_3 shall be less than at E_3', and hence the pressure observed form curves is at E_3'; this correction shall be – ve and its amount

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

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



(a) Correction due to slope at E3 is nil, as the point E3 is neither situated at the start nor at the end of any slope

Hence, corrected φE3 = (38 – 1.02 – 0.76) % = 36.22 %

Exit gradient

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

The maximum seepage head, H = 158 – 152 = 6 m
The depth of d/s cur-off, d = 152 – 141.7 = 10.3 m
Total floor length, b = 57 m

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

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

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

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

Culvert

A **culvert** is a structure that allows water to flow under a road, railroad, trail, or similar obstruction from one side to the other side. Typically embedded so as to be surrounded by soil, a culvert may be made from a pipe, reinforced concrete or other material. A structure that carries water above land is known as an aqueduct.

Culverts are commonly used both as cross-drains for ditch relief and to pass water under a road at natural drainage and stream crossings. A culvert may be a bridge-like structure designed to allow vehicle or pedestrian traffic to cross over the waterway while allowing adequate passage for the water. Culverts come in many sizes and shapes including round, elliptical, flat-bottomed, pear-shaped, and box-like constructions. The culvert type and shape selection is based on a number of factors including: requirements for hydraulic performance, limitation on upstream water surface elevation, and roadway embankment height.

Design of Culvert

- Definition - A structure used to convey surface runoff through embankments.
- It may be a round pipe, rectangular box, arch, ellipse, bottomless, or other shapes.
- And it may be made of concrete, steel, corrugated metal, polyethylene, fiberglass, or other materials.

Design of culvert needs the following studies:

1. Hydrological studies
2. Hydraulic studies
3. Structural studies

1. Hydrological studies

$Q = 0.278\ C\ A\ I$

Q: maximum discharge in $\frac{m^3}{sec}$

C: Runoff Coefficient

A: Catchment area in Km^2

I: Rainfall intensity in $\frac{mm}{hr}$

Example:

Estimate the maximum design discharge for a pipe culvert if the catchment area is (150 Km^2) and the rainfall intensity is (10 mm/hr) and the surface runoff coefficient of the area is (5%).

$Q = 0.278\ C\ A\ I$

$$Q = 0.278 * \frac{5}{100} * 150 * 10\frac{mm}{hr} = 20.85\ \frac{m^3}{sec}$$

2. Hydraulic studies

The main objectives for the hydraulic design of the culvert are:

- Select the type of Culvert
- Calculate the culvert dimensions
- Estimate the maximum permissible height of water upstream (Headwater of the culvert)

2.1 Hydraulic design of culvert

The most important consideration in culvert hydraulics is whether the flow is subject to inlet or outlet control. Figures 1 and 2 show the range of flow types commonly encountered in culverts. For inlet control two distinct regimes exist, depending on whether the inlet is submerged or not submerged. Outlet control occurs in long culverts, laid on flat grades and with high tailwater depths. In designing culverts, the type of control is determined by the greater of the headwater depths calculated for both inlet control and outlet control. For the two types of control, different factors and formulae are used to calculate the hydraulic capacity of a culvert. Under inlet control, the cross-sectional area of the culvert cell, the inlet geometry and the amount of headwater or ponding at the entrance are of primary importance. Outlet control involves the additional consideration of the elevation of the tailwater in the outlet channel and the slope, roughness and length of the culvert cell.

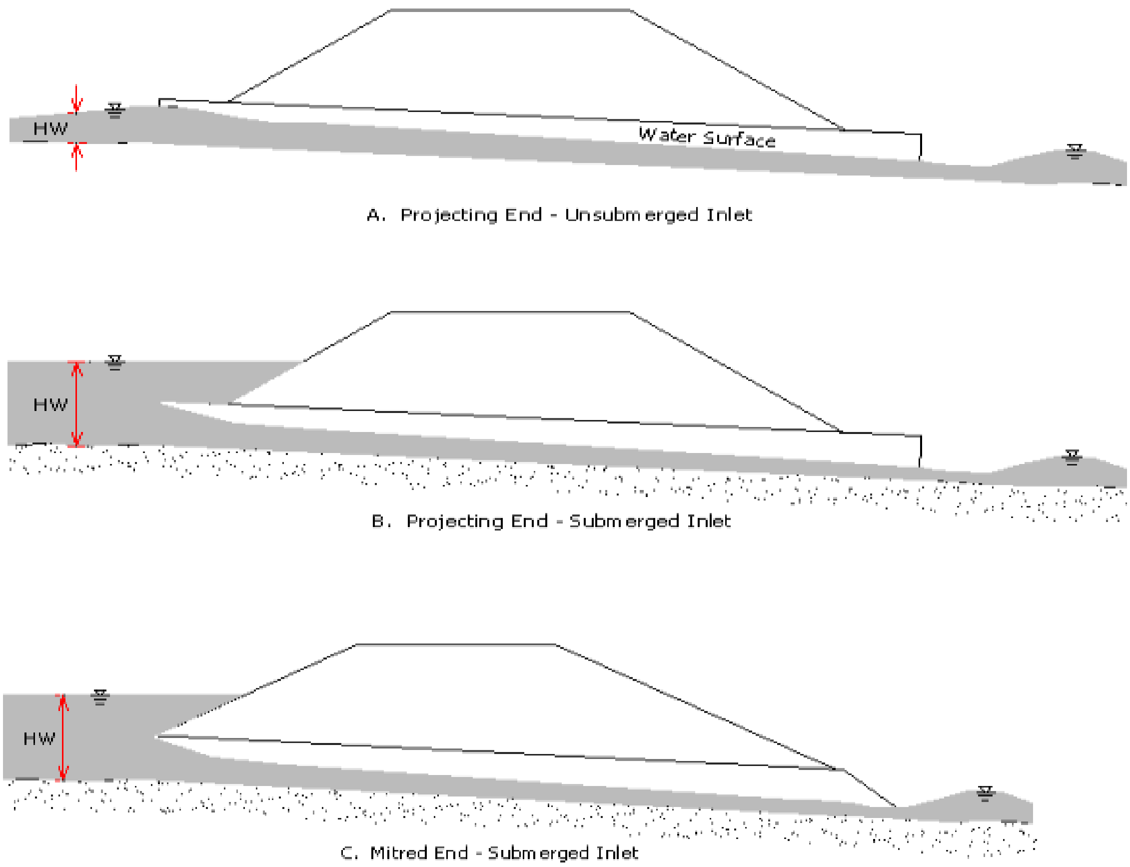


Figure (1) Flow Profiles for Culvert under Inlet Control

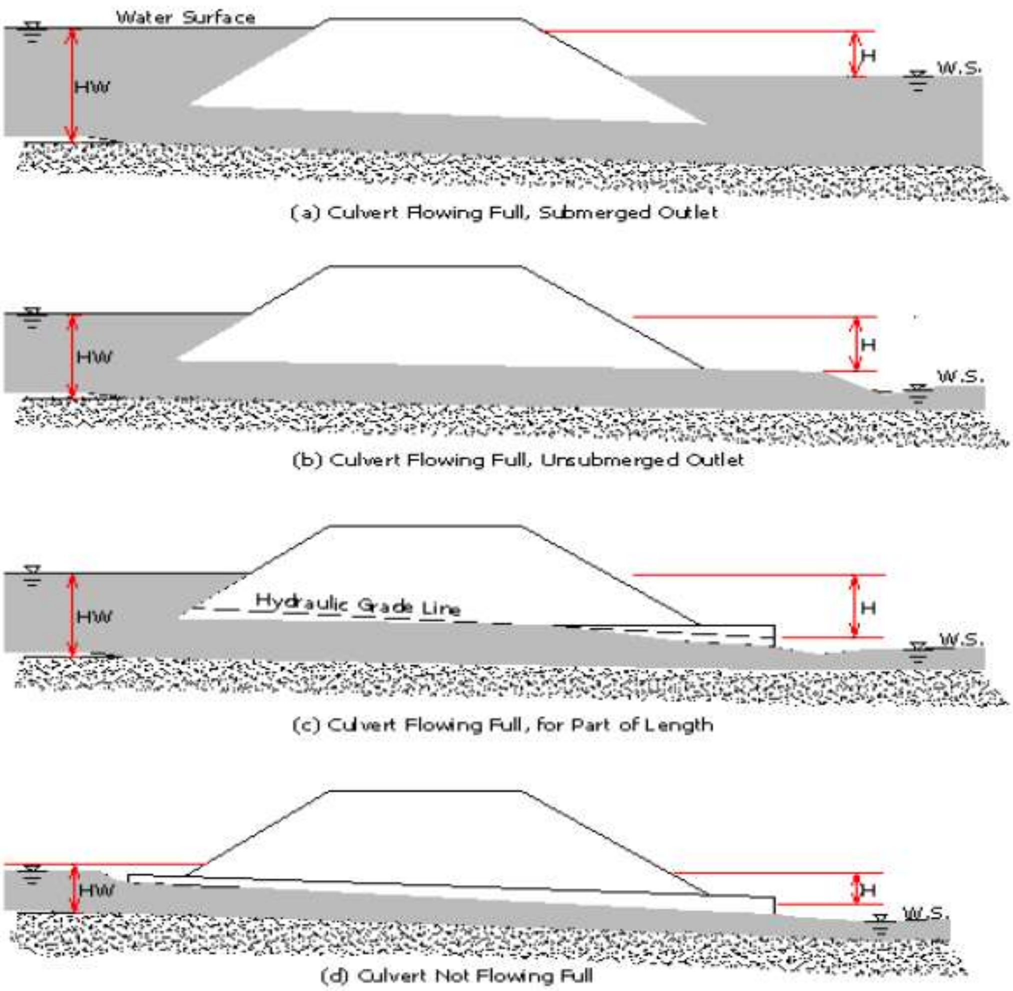


Figure (2) Flow Profiles for Culvert under Outlet Control

2.1.1 Culvert operating with outlet control

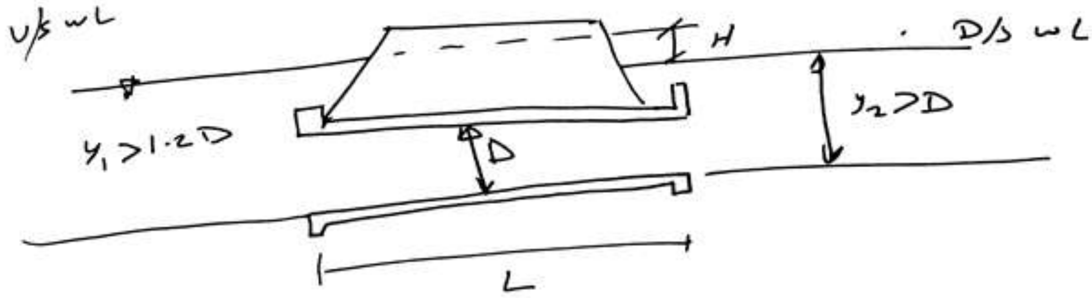


Figure (3) Outlet Control

In this case as shown in figure 3, the total head loss can be calculated as follows:

$$H = h_e + h_f + h_o + \cdots h_n \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$H = k_1 \frac{v^2}{2g} + k_2 \frac{v^2}{2g} + k_3 \frac{v^2}{2g} + \cdots + k_n \frac{v^2}{2g} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Where:

h_e : Entrance head loss

k_1: Entrance Coefficient:

= 0.5 for square edge entrance

= 0.05 for well-rounded entrance

hf: head loss due to friction through pipe culvert. This depend on k_2 which in tern depend on Manning Roughness and the length and diameter of culvert barrel

h_o = head loss due to velocity head this depend on k_3

Usually taken as: $k_3 = 1.0$

2.1.1.1 Flow velocity through culvert

Flow velocity through culvert can be calculated using Manning Formula:

$$\boldsymbol{V = \frac{1}{n} R^{\frac{2}{3}}\ S^{\frac{1}{2}}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..(\mathbf{3})$$

For circular cross section with pipe diameter (D) then:

$$Area = A = \frac{\pi D^2}{4} \quad and\ Wetted\ perimeter = W_p = \pi D$$

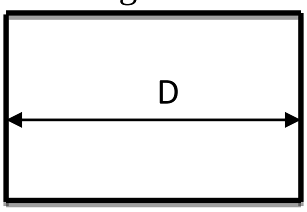
Then: the hydraulic radius $R = \frac{A}{W_p} = \frac{D}{4}$ therefore $V = \boldsymbol{\frac{1}{n}(\frac{D}{4})^{\frac{2}{3}}(\frac{h_l}{l})^{\frac{1}{2}}}$

In which (h_l is the difference between water level upstream and downstream of culvert).

And: $Q = \boldsymbol{\frac{\pi}{10n}\ D^{\frac{8}{3}} \left(\frac{h_l}{l}\right)^{\frac{1}{2}}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(\mathbf{4})$

Note: for rectangular cross section or square cross section, use the largest dimension for (D)

Rectangular culvert cross section



$k_2\ can\ be\ estimated\ using:$ $$k_2 = \frac{123\ L\ n^2}{D^{\frac{4}{3}}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots.(5)$$

2.1.1.2 Flow formula for the flow discharge through culvert

$$H = \frac{V^2}{2g}\ \{k_1 + k_2 + k_3 + \cdots \ldots\ldots\ldots. + k_n \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..(6)$$

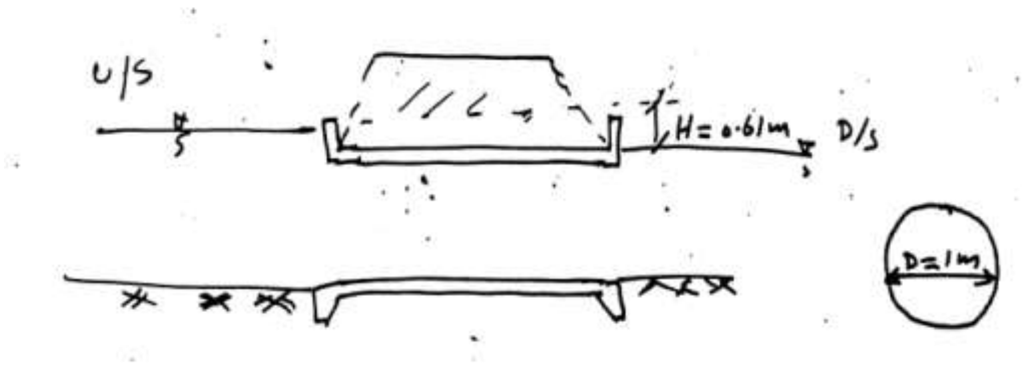
$$V = \frac{\sqrt{2gH}}{\sqrt{\sum k}} \qquad , \qquad Q = A\,V = A\,\frac{\sqrt{2gH}}{\sqrt{\sum k}}$$

$$for\ circular\ pipe\ with\ diameter = D\ and\ for\ g = 9.81\ , then$$

$$Q = \frac{3.48\ D^2\sqrt{H}}{(\sqrt{k_1 + k_2 + k_3 + \cdots k_n})} \quad \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots. (7)$$

Example:

Calculate the required discharge for a box culvert with circular cross section and square edge. The pipe material is concrete with Manning roughness n= 0.013. Given that the pipe diameter for culvert barrel is equal to (1 m) and the culvert barrel length is equal to (16 m) with total head difference (H) is equal to (0.61 m).



Solution

Use: k_1= 0.5 for square edge, k_3= 1.0,

and k_2 can be estimated using :

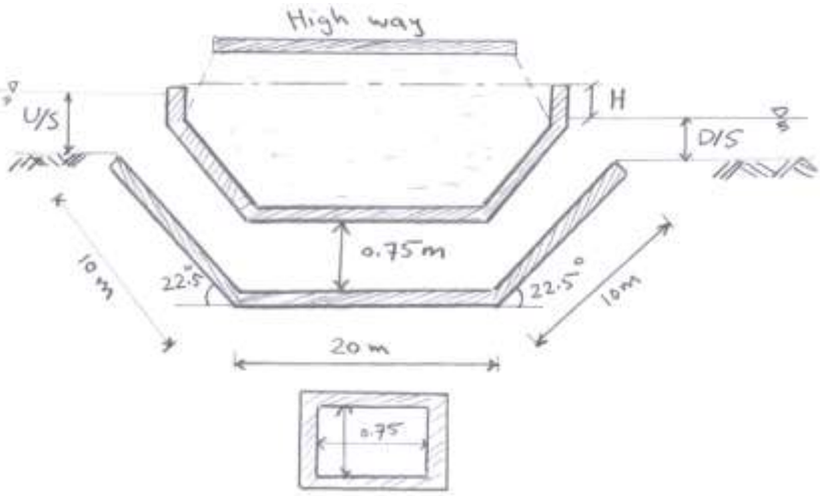
$$k_2 = \frac{123\ L\ n^2}{D^{\frac{4}{3}}} = \frac{123\ (\ 16)(0.013)^2}{1^{\frac{4}{3}}} = 0.33259$$

$then$

$$Q = \frac{\pi\ D^2\sqrt{2gH}}{4\ \sqrt{k_1+k_2+k_3+\cdots k_n}} \qquad Q = 2\ \frac{m^3}{sec}$$

Example

Box culvert constructed from pre-stressed concrete under traffic highway. Calculate the total head a cross. The culvert (H) for the case of flow discharge (Q = 0.5 m^3/ sec) and culvert dimension (0.75 m by 0.75 m). Manning roughness is 0.012 and with well rounded edges (i.e. k_1 = 0.05). The bending coefficient (k_n) for angle = 22.5^o is (k_n= 0.4)



Solution

Use k_1= 0.05 , k_3= 1.0 , estimate $k_2 = \frac{123\, L\, n^2}{D^{\frac{4}{3}}}$

L= 20 + 2 (10)= 40 m then k_2= 1.04

Use k_n= 2 * 0.4 = 0.8 then $\sqrt{k_1 + k_2 + k_3 + \cdots k_n} = 1.6882$

$$Q = \frac{(.75 * .75)\sqrt{2g * H}}{\sqrt{k_1 + k_2 + k_3 + \cdots k_n}} \quad with = 0.5\, \frac{m^3}{sec}$$

$then$

$H = 0.116m \quad H = 12\; cm$

2.1.2 Culvert operating with inlet control

For culverts subject to inlet control, the important factors are entrance conditions, including the entrance type, existence and angle of headwalls and wing walls and the projection of the culvert into the headwater pond. Inlet control can occur with the inlet submerged and the outlet not submerged (figure 1).

In this case the entrance of culvert act as an orifice, Then:

$$Q = C_d A \sqrt{2gh} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(8)}$$

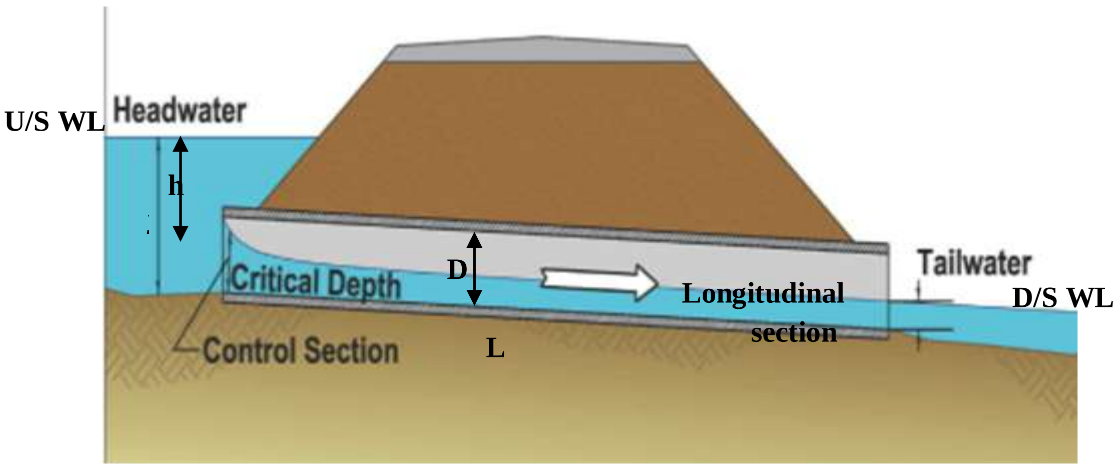


Figure (4) Inlet control

In which: h = height of water upstream (U/S) measured from longitudinal axis.

C_d = Coefficient of discharge

for square edged entrance = 0.62 and

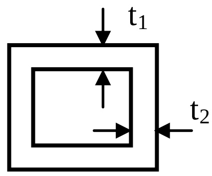
for well rounded entrance = 1.0)

A= area of culvert barrel

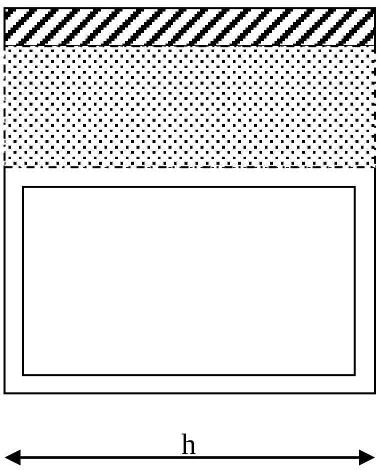
Structural Design of Culvert

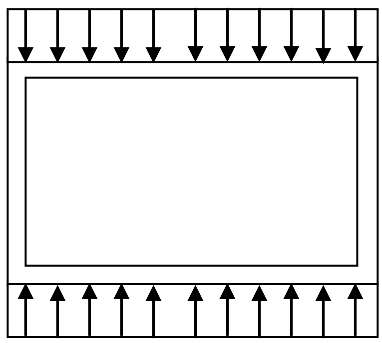
Design steps

1. Establish the following data
   a. Ground level for culvert upstream and downstream
   b. Road level
   c. X-section of culvert for V= Q/A
   d. Depth of earth fill over the top of culvert (usually not less than 0.5)
2. Determination of the internal dimension of culvert X-section after the determination of culvert flow X-section
3. Estimation of culvert wall and slabs thickness $t = \frac{L}{12}$

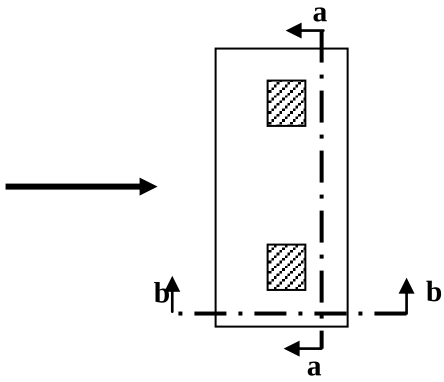


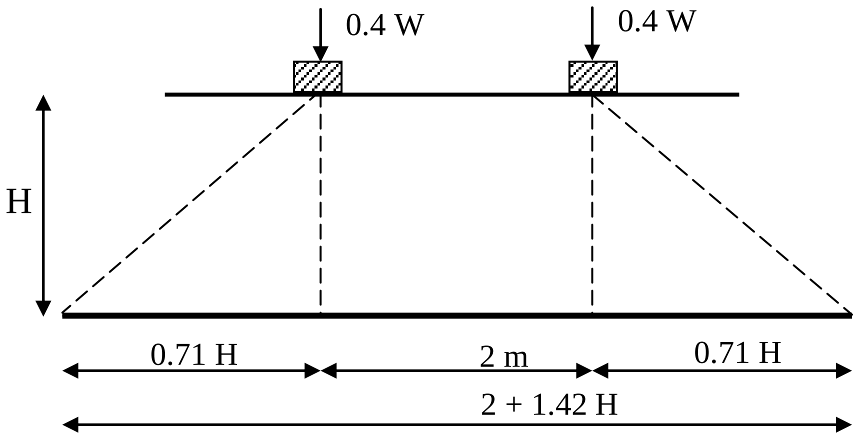
4. Estimation of effection loading
   a. Uniform vertical loads acting over the top of culvert because the following:
   1. Live load impact
   2. Weight of soil above culvert
   3. Weight of pavement
   4. Weight of concrete slab on top

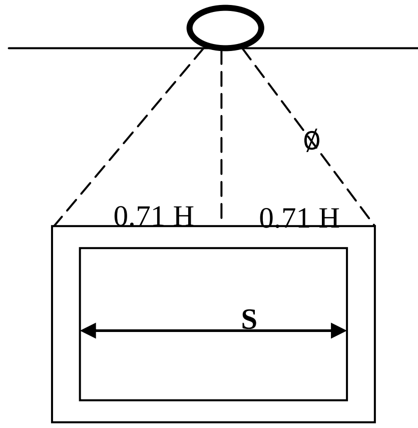




I. L.L on Top of culvert



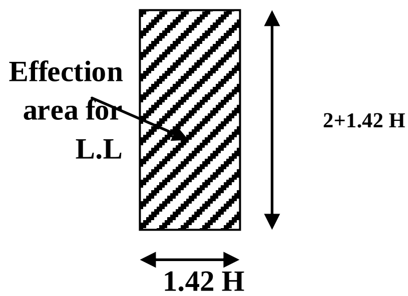




Section (a-a)

Section (b-b)

$$L.L = \frac{0.8\,W}{(2+1.42H)(1.42H)}$$



$$Impact \quad I\% = \frac{16}{S+40} \leq 50\%$$

Impact load = *I% x L.L*

Note: if the earth fill depth H > 3 m the force (load) of L.L and its impact (ignored)

a. Weight of soil above the culvert

$W_1 = \gamma_{soil} * H =$ Tone/m^2

b. Weight of pavement

$W_2 = \gamma_{Asphalt} * t_{Asphalt} =$ Tone/m^2

Note: $\gamma_{Asphalt} = 2.15 \text{ Tone/m}^3$

c. Weight of concrete slab on top

$W_3 = \gamma_{concrete} * t =$ T/m^2

t: thickness of culvert wall

$\gamma_{concrete}$: specific weight of concrete ≅ 2.5 T/m^3

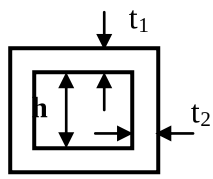
Total weight of culvert (W)

W = (L.L + I. L.L) + W_1 +W_2 +W_3

II. Reaction unit load (Soil Reaction)

Forces acting on culvert base

$$\overline{W} = W + \frac{weight\ of\ two\ walls}{b*1} = W + \frac{2h*t_2*\gamma_{concrete}}{b*1}$$

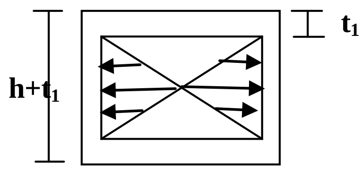


$\overline{W}$ = total (dead + L.L)

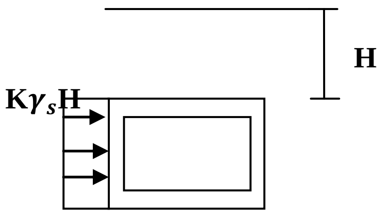
III. Water pressure inside the culvert

$= \gamma_w * h$

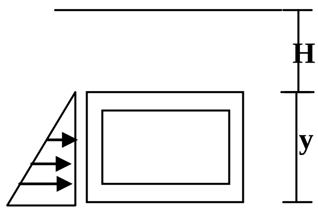
Note: weight of water downward is ignored since there is counter force upward from water is soil



IV. Uniform soil pressure on sides



V. Triangular lateral soil pressure



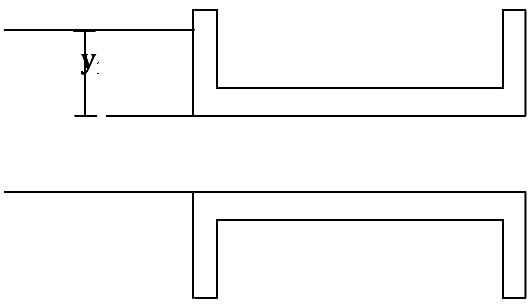
$y = (h + t_1)$

$= \gamma_s * k * y$

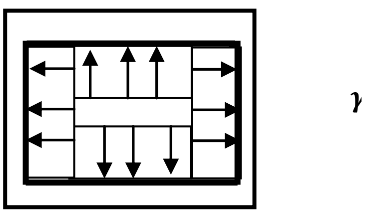
$k = 0.333$ *for cohesive soil*

VI. Additional hydrostatic pressure of water head

$= \gamma_w * y_1$



This distribution uniformly along the interior sides of culvert

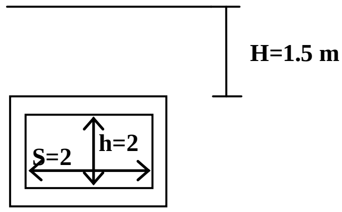


For the case of conservative design: culvert will be takes empty from water.

Design Example of Culvert

It is required to design a square X-section box culvert, the following data is established:

- Water X-section area = 4.5 m^2 (no water inside culvert)
- Earth fill H = 1.5 m
- Live load type (H-10)
- No water exist up the top level of culvert
- Specific weight of earth fill = $\gamma_s = 1.6\ Tone/m^3$
- Coefficient of soil lateral pressure K= 1/3 = 0.333
- Specific weight of concrete $= \gamma_c = 2.5\ Tone/m^3$
- Net height of culvert S = 2 m
- Thickness of road pavement t = 0.15 m
- Specific weight of asphalt $= \gamma_{asphalt} = 2.15\ Tone/m^3$
- Steel tensile stress f_s = 12500 Tone/m^2
- Concrete compressive stress f_c = 800 Tone/m^2



Solution

$L_1 = L_2 = \sqrt{4.5} = 2.12\ m$ say 2 m

Thickness for slab and side walls is taken to be homogenous: $t = \frac{L}{12} = \frac{2}{12} = 0.176\ m \approx 0.2\ m = 20\ cm$

I- Vertical load acting on culvert

1- Live load (L.L)

$$\text{L.L} = \frac{\mathbf{0.8\ w}}{(\mathbf{1.42\ H+2})(\mathbf{1.42H})} = \frac{\mathbf{0.8*10}}{(\mathbf{1.42*1.5+2})(\mathbf{1.42*1.5})} = 0.909\ Tone/m^2$$

Calculation of load impact = I %

$I\% = \frac{16}{S+40} = \frac{16}{2+40} = 38\% \ < 50\%$ Ok

Impact load $= 38\% * 0.909 = 0.345\ Tone/m^2$

L.L+Impact = $0.909 + 0.345 = 1.254\ Tone/m^2$

2- Weight of soil above culvert

$= \gamma_s H = 1.6 * 1.5 = 2.4\ Tone/m^2$

3- Weight of road pavement

$= \gamma_{asphalt} * t_{asphalt} = 2.15 * 0.15 = 0.323\ Tone/m^2$

4- Weight of culvert slab

$= \gamma_{con.} * t_{con.} = 2.5 * 0.2 = 0.5\ Tone/m^2$

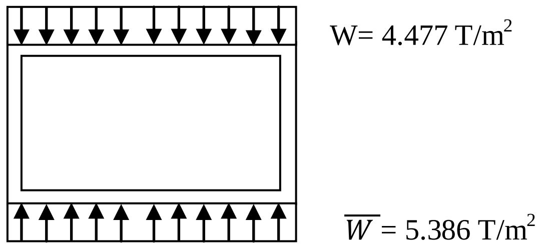
5- Total weight acting on Top of culvert

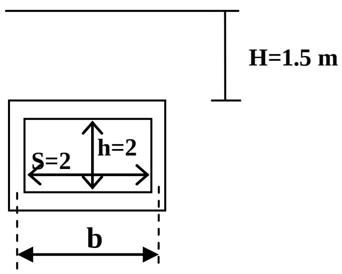
$W = 1.254 + 2.4 + 0.5 + 0.323 = 4.477\ Tone/m^2$

II- Soil Reaction ($\overline{W}$)

$\overline{\mathrm{W}} = \mathrm{W} + \frac{\text{weight of 2-side wall}}{\mathrm{b}}$ $\quad\quad$ $\mathrm{b} = \mathrm{L} + \mathrm{t} = 2 + 0.2 = 2.2$

$= 4.477 + \frac{2 * 0.2 * 2.5 * 2}{2.2} = 5.386\ \text{Tone/m}^2$

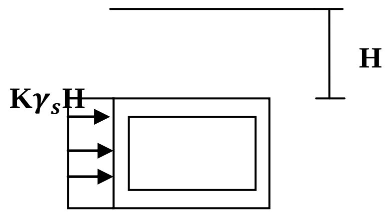




III- No water inside the box culvert

IV- Lateral homogenous pressure on side of culvert

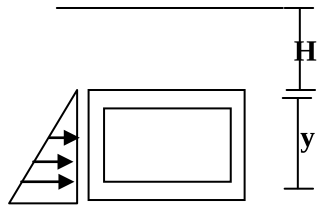
= 1.6 * 0.333 * 1.5 = 0.799 Tone /m^2



V- Triangular lateral soil pressure on side

$= \gamma_s * k * y = 1.6 * 0.33 * 2.2 = 1.172$

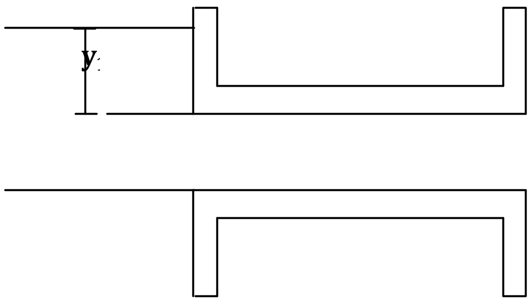
$y = 2 + t = 2 + 0.2 = 2.2$



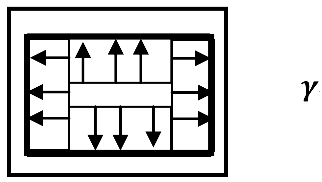
The value of k depend on soil characteristics k = 1/3 = 0.333 for cohesive and non-cohesive soil.

Additional hydrostatic pressure of water head

$= \gamma_w * y_1$

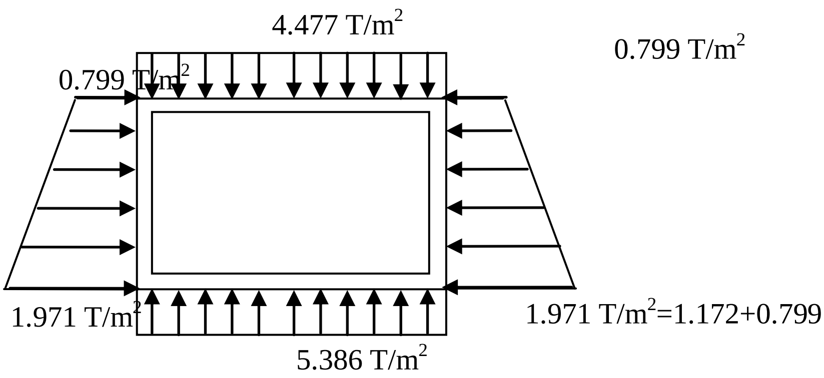


This pressure is distributed uniformly on all side of culvert

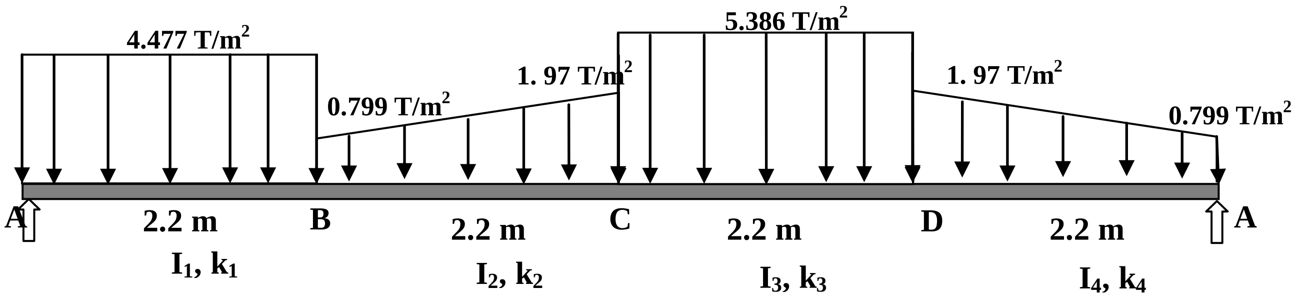


For case of conservative design culvert will be takes empty from water

Finally:- Draw the loading diagram



Bending Moment Calculation



This required calculating the moment of inertia

1- $I = \frac{bt^3}{12}$

2- Calculation of Rigidity coefficient (k)

$k_1 = \frac{I_2}{I_1} * \frac{l_1}{l_2}$, $k_2 = \frac{I_3}{I_2} * \frac{l_2}{l_3}$, $k_3 = \frac{I_4}{I_3} * \frac{l_3}{l_4}$

3- Coefficient of moment distribution

$\frac{k_1}{k_1+k_2}$, $\frac{k_2}{k_2+k_3}$, $\frac{k_3}{k_3+k_4}$

Since all box culvert dimensions are the same and since thickness of all walls, slabs and foundation are the same then:

$k_1 = k_2 = k_3$

$I_1 = I_2 = I_3 = I_4$

$I = \frac{bt^3}{12} = \frac{1*0.2^3}{12} = 6.667 * 10^{-4} m^4$

$$k_1 = \frac{I_2}{I_1} * \frac{l_1}{l_2} = \frac{6.667 * 10^{-4}}{6.667 * 10^{-4}} * \frac{2.2}{2.2} = 1$$

$k_1 = k_2 = k_3 = 1$ rigidity coefficient

$\frac{k_1}{k_1+k_2} = \frac{1}{1+1} = 0.5$ Coefficient of moment distribution

$$\frac{k_2}{k_2 + k_3} = \frac{1}{1 + 1} = 0.5$$

a) Calculation of Fixed End Moment (-ve moment)

$$M_{AB} = M_{BA} = -\frac{wL^2}{12} = -\frac{4.477(2.2)^2}{12} = -1.81$$

$$M_{CD} = M_{DC} = -\frac{wL^2}{12} = -\frac{5.386(2.2)^2}{12} = -2.17$$

$$M_{BC} = M_{AD} = -\left[\frac{wL^2}{12} + \frac{wL^2}{15} * \frac{1}{2}\right] =$$

$$-\left[\frac{0.799(2.2)^2}{12} + (1.971 - 0.799) * \frac{2.2}{2} * \frac{2.2}{15}\right] = -0.51\, T/m$$

$$M_{CB} = M_{DA} = -\left[\frac{0.799(2.2)^2}{12} + (1.971 - 0.799) * \frac{2.2}{2} * \frac{2.2}{10}\right] = -0.61\, T/m$$

Note: Use the Moment Distribution Method to find the equilibrium moment on ends.

A		B		C		D		A
k = 0.5		**k = 0.5**		**k = 0.5**		**k = 0.5**		
+1.81	**-1.8**	**+0.51**	**-0.61**	**+2.17**	**-2.17**	**+0.61**	**-0.51**	**+1.81**
-0.65(1)	**+0.65**	**+0.65**	**-0.78**	**-0.78(2)**	**+0.78**	**+0.78**	**-0.65**	**-0.65**
0.33	**-0.33**	**-0.39**	**+0.33**	**+0.39**	**-0.39**	**-0.33**	**+0.39**	**+0.33**
-0.36	**(3)+0.36**	**+0.36**	**-0.36**	**-0.36**	**+0.36**	**+0.36**	**-0.36**	**-0.36**
-1.13(4)	**-1.13**	**-1.13**	**-1.42**	**-1.42**	**-1.42**	**-1.42**	**-1.13**	**-1.13**

<u>Calculation</u>

1- M = -1.81+ 0.51 = -1.3 km = -1.3 * 0.5 = -0.65
2- M = +2.17 – 0.61 = +1.56 km = +1.56 * 0.5 = +0.78
3- M = -0.33 – 0.39 = -0.72 km = -0.72 * 0.5 = -0.36
4- M = 1.81 – 0.65 +0.33 -0.36 = 1.13

Mid Span Moment

a) Top slab Bending Moment

$$\text{B.M} = \frac{WL^2}{8} = \frac{4.477(2.2)^2}{8} = +2.71$$

b) Bottom slab Moment

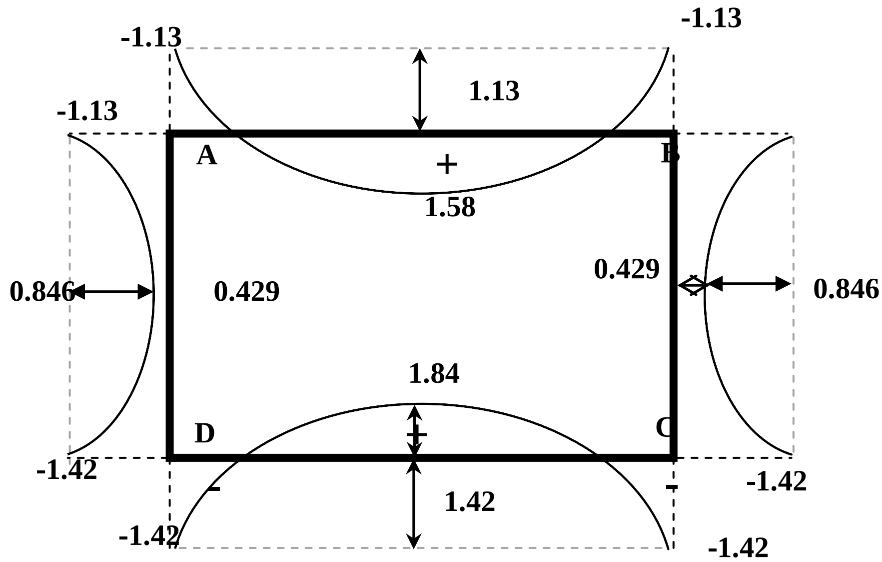
$$\text{B.M} = \frac{WL^2}{8} = \frac{5.386(2.2)^2}{8} = +3.26$$

c) Side walls Moment

B.M

$$= [\frac{WL^2}{8} + 0.128 * \frac{WL^2}{2}] = \frac{0.799(2.2)^2}{8} + 0.128 * \frac{(1.971-0.799)2.2^2}{2} =$$

$$+0.483 + 0.363 = +0.846\ Tone.m$$



Reinforcement Calculation

$$A_s = C_2 \frac{B.M}{d}$$

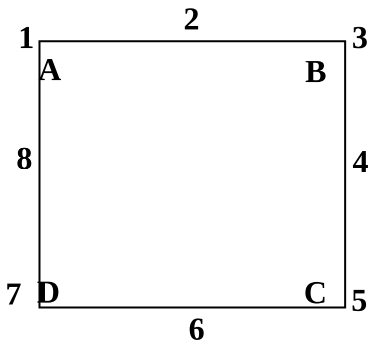
$$d = C_1 * \sqrt{\frac{B.M_{max}}{b}}$$

For longitudinal and thermal reinforcement

$$A'_s = \frac{30}{100} As_{max}$$

For check assumed (t) value, find (d) from Max. Moment

Point	**B.M (T.m)**
1	-1.13
2	+1.58
3	-1.13
4	-0.43
5	-1.42
6	+1.84
7	-1.42
8	-0.43



To calculate reinforcement area, and check for (d) value:

B.M =1.84 Tone.m ………… Max. Moment to calculate (d)

f_s = 12500 T/m^2 , f_c = 800 T/m^2 c_1 =0.087, c_2 = 0.92

$$d = c_1 * \sqrt{\frac{B.M}{b}} = 0.087 * \sqrt{\frac{1.84}{1}} = 0.118\, m = 11.8\, cm$$

$t = d + cover = 11.8 + 8 = 19.8\, cm \approx 20\, cm$ Ok *d = 20 – 8 = 12 cm*

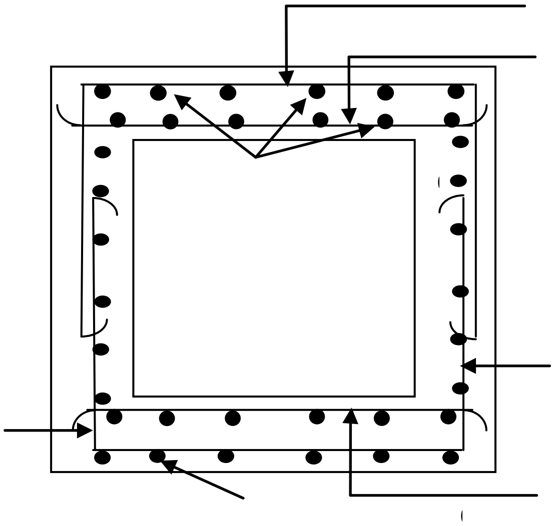
$$A_s = C_2 \frac{B.M}{d} = 0.92 * \frac{B.M}{0.12} = 7.67 * (B.M)$$

B.M (T.m)	***A_s (cm^2/m)***	***Bar diameter and Space***
-1.13	***-8.67***	$\boldsymbol{\emptyset 16\ mm@20cm\ c/c}$
+1.58	***+12.12***	$\boldsymbol{\emptyset 20\ mm@25cm\ c/c}$
-1.13	***-8.67***	$\boldsymbol{\emptyset 16\ mm@20cm\ c/c}$
-0.43	***-3.3***	$\boldsymbol{\emptyset 10\ mm@20cm\ c/c}$
-1.42	***-10.89***	$\boldsymbol{\emptyset 20\ mm@25\ cm\ c/c}$
+1.84	***+14.11***	$\boldsymbol{\emptyset 20\ mm@20cm\ c/c}$
-1.42	***-10.89***	$\boldsymbol{\emptyset 20\ mm@25cm\ c/c}$
-0.43	***-3.3***	$\boldsymbol{\emptyset 10\ mm@20cm\ c/c}$

Longitudinal Reinforcement

$$As' = \frac{30}{100} * As_{max} = \frac{30}{100} * 14.11 = 4.233\ cm^2/m$$

Use $\emptyset 12\ mm@25cm\ c/c$



Regulators

Regulator and control structures: they are used in open channels systems to control and regulate the discharge and /or water levels within the system, structures which control discharge are head regulator, structure controlling water levels are cross regulator.

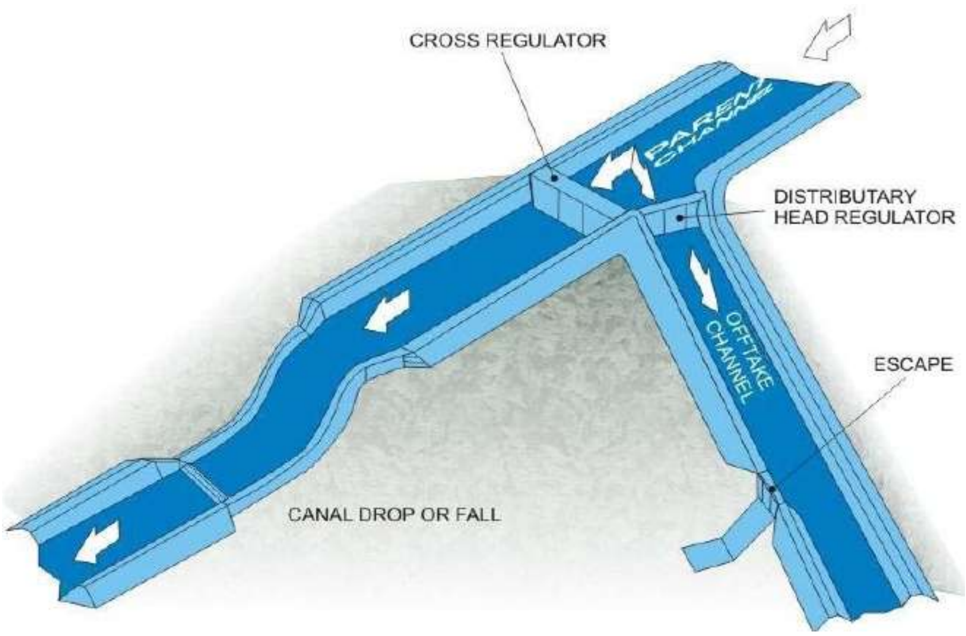
Hydraulic Design

Type of Regulators

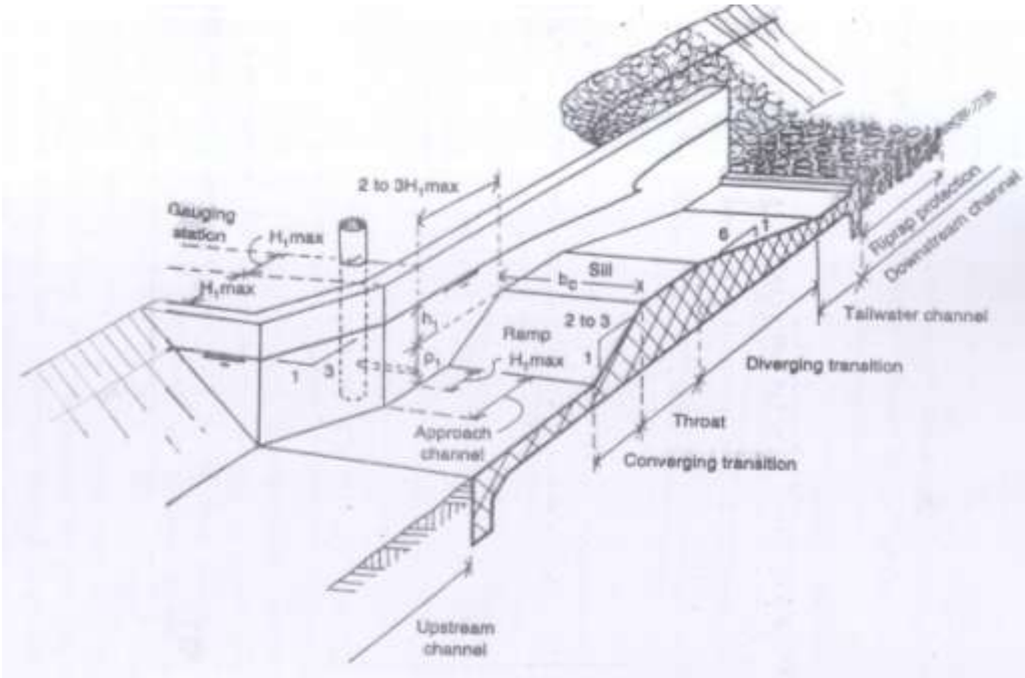
1- Main Regulators: located at the beginning of an irrigation project. It regulates the amount of discharge passing into the projects main canal. Such as Al-Sudur Regulator in Diyala.

2- Head Regulators: located at the beginning of a canal within the irrigation project such as at the beginning of secondary canals. Used to regulate discharge downstream.

3- Cross Regulators: it is located across the flow (i.e. perpendicular to the flow direction usually used to regulate the water level upstream.

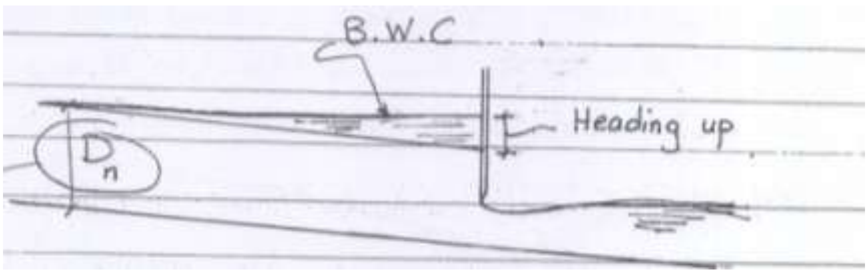


Longitudinal Section in head regulator



Heading up (Afflux) and Normal depth:

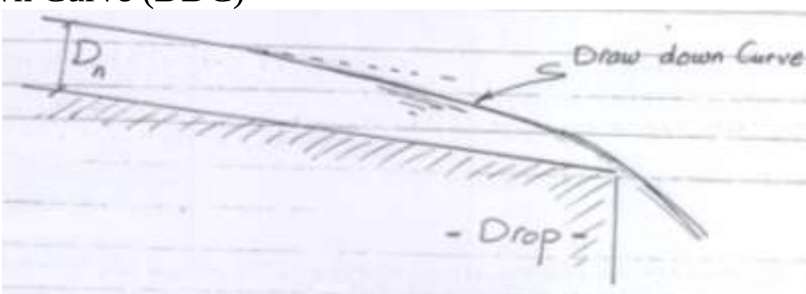
The rise in the water level above the normal water level in a channel (or a river) on the U/S of the structure that constructed on the channel.



B.W.C: Back Water Curve

D_n: Normal depth at which uniform will occur in an open channel. Usually determined by using Manning equation $Q = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} A$

Draw Down Curve (DDC)



Hydraulic Calculation of Regulators

Velocity and Discharge

For efficient hydraulic design the following should be met:

1- Losses are to be kept at minimum
2- Hydraulic jump must be contained within the structure
3- The flow pattern must be as uniform as possible

Maximum and Minimum discharge flow

Q_n : Normal discharge

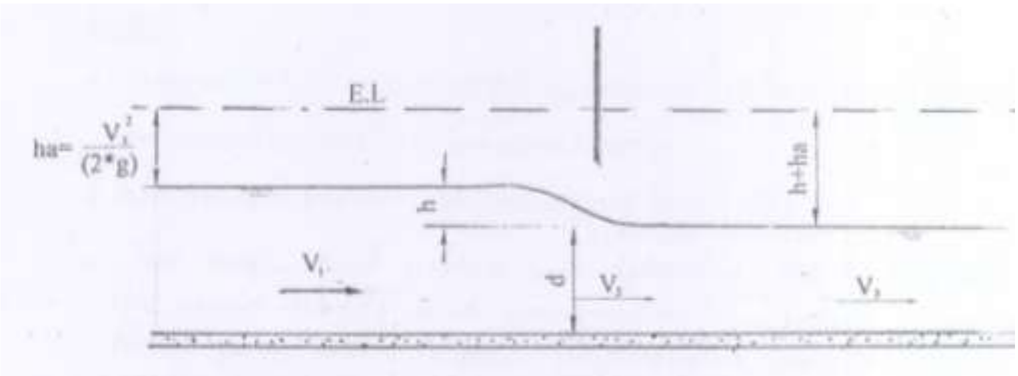
$Q_{max.}$: 1.2 Qn (in order to find $V_{max.}$)

$Q_{min.}$: 0.6 Qn (in order to find $V_{min.}$)

For the case of regulator consist of an opening controlled by radial or vertical lift gates;

The discharge is determined by the orifice equation

$$Q = C\ A\sqrt{2gh}$$



A: Area of water way through the regulator (net area = d*S)

S: Regulator net width

d: actual height of gate above sill in partially closed opening

d: D/S W.L – Sill level under gate, for fully open regulator

V: velocity through the regulator opening

V_a: Approach velocity

h_a: $V_a^2/2g$

h: heading up (afflux)

Afflux: the rise in U/S due to construction on the structure across river

The design purpose the full discharge Q is given and consequently the regulator should be fully opened at this discharge. The discharge equation through fully opened regulator is:

$$Q = C\ S\ d\ \sqrt{2g(h + h_a)}$$

Cases of gate discharge

1- Fully opened gate

Flow is subcritical with free surface

$F_r < 1$, Q = max.

$$Q = C\ S\ d\ \sqrt{2g(h + h_a)}$$

Water way: the max. design discharge for canal structure is (1.2 Qn), where Q_n = Normal full supply discharge .

The formula to be used for calculates the discharge at full gate opening (is given by Serge Leliavsky) as:

$$Q = C\ L\ d\ \sqrt{2g(H + H_a)}$$

C: discharge coeff. = 0.82 for square entrance

= 0.9 for round entrance

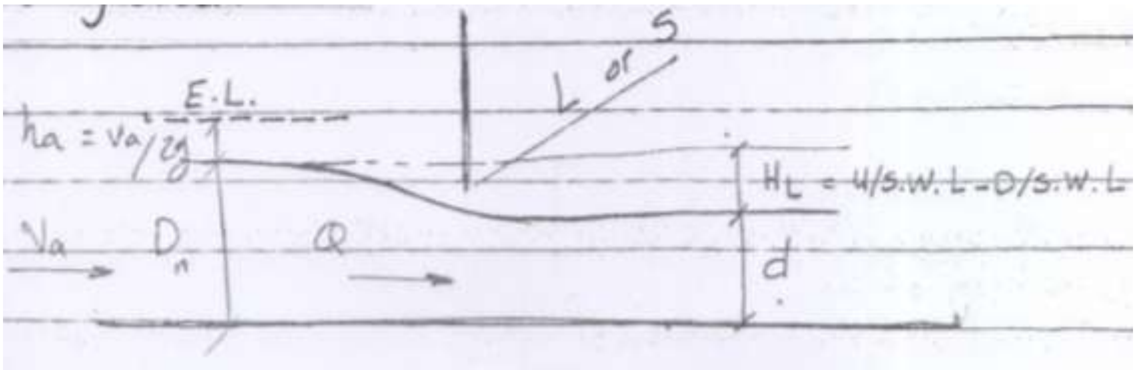
L: width of gate (m)

d: tail water depth

h_a: head due to velocity of approach = $V_a^2/2g$

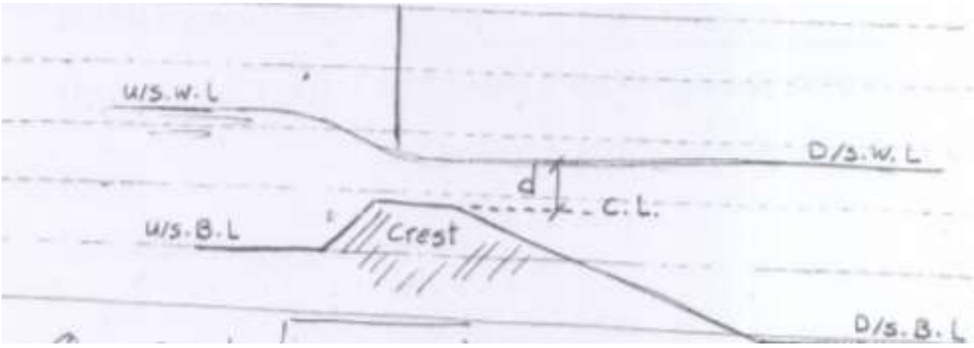
H_L: difference in U/S and D/S water levels

The velocity head (ha) of approaching flow is to be considered for in-line structure (cross regulator) but in case of head regulator where the gate opening is at an angle to the main stream, the term ($h_a = V_a^2/2g$) is ignored.



(Fully gate opening, Horizontal floor)

For the case of sill exist under gate



(Fully gate opening, floor with crest)

$$Q = C\ S\ d\ \sqrt{2g(H + h_a)}$$

C: 0.82 or 0.9

d: D/S W.L – (sill level, or crest level, or floor level) under gate

H: U/S W.L - D/S W.L

For partially opening for the gate one can a (h_a) ignore.

2- Gate is partially closed

Free floor condition

Discharge is smaller; (h_a) is so small (Ignored)

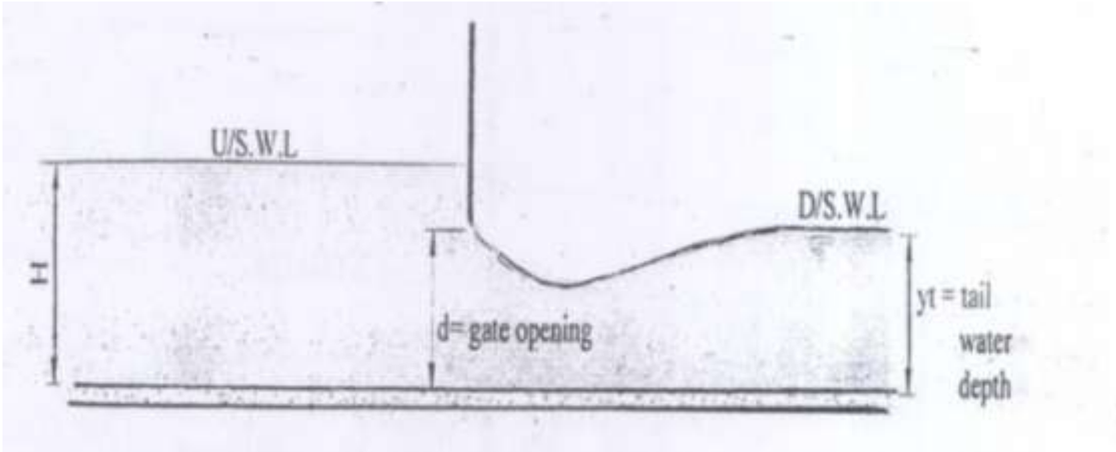
Submerged orifice formula is used

$$Q = C_d\ A\ \sqrt{2gH}$$

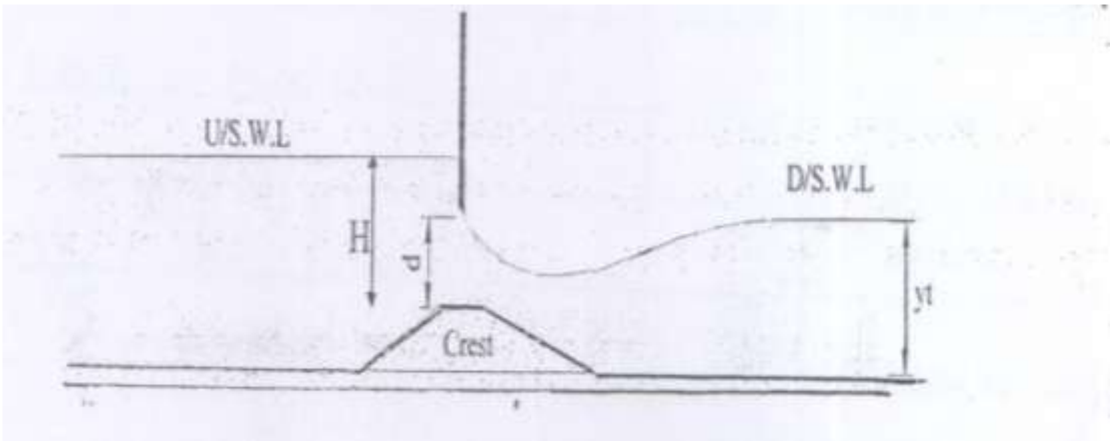
A: Flow area = width of gate (L)* gate opening (d)

H: operating head (U/S water depth)

C_d: coeff. Of discharge = 0.62



(Partially Closed gate, Horizontal floor)



(Partially Closed gate, floor with crest)

Example 1

Find the width of distributary canal head regulator, given the following data:-

	Main canal	Distributary canal
Normal discharge m3/sec	22.1	3
Normal full supply water level (m)	51.9	51.8
Bed level (m)	49.51	50.6
Bed width (m)	5	2.5
Side Slope	1 1/2 H:1V	1 1/2 H:1V
Depth of flow (m)	2.39	1.2

Neglect the effect of approaching velocity

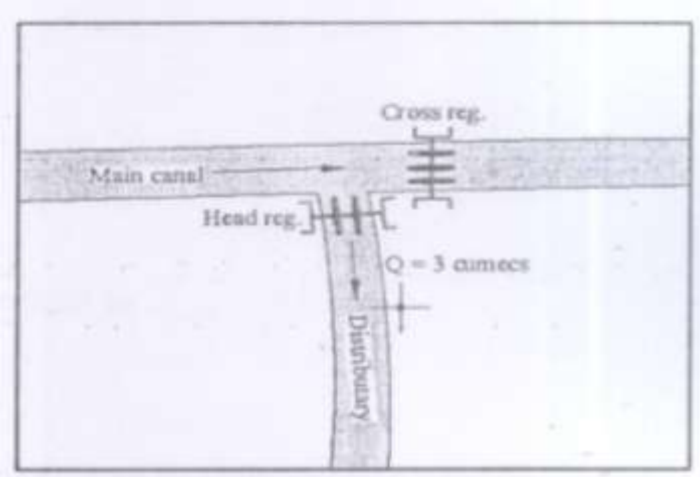
C = 0.82 (square opening)

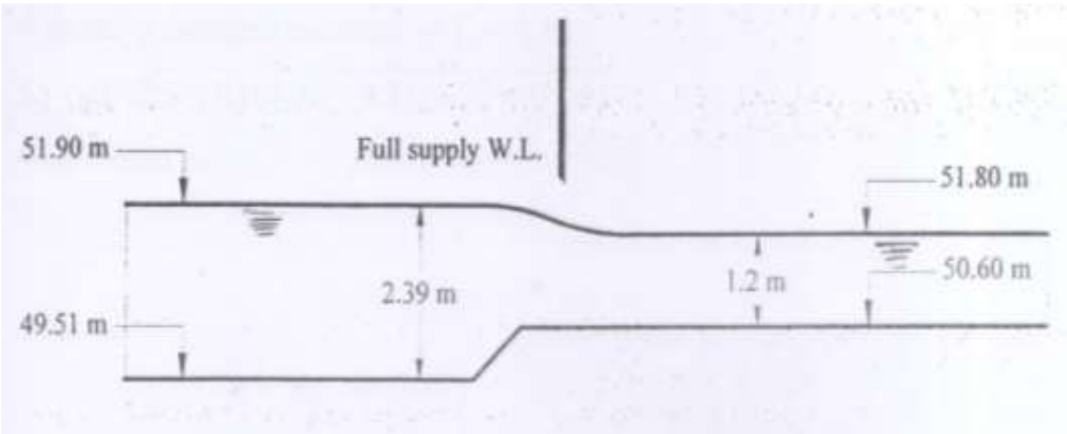
D = D/S W.L – Sill level = 51.8 – 50.6 = 1.2 m

H = 51.9 -51.8 = 0.1 m

$$Q = C\ S\ D\sqrt{2g(H + H_a)}$$

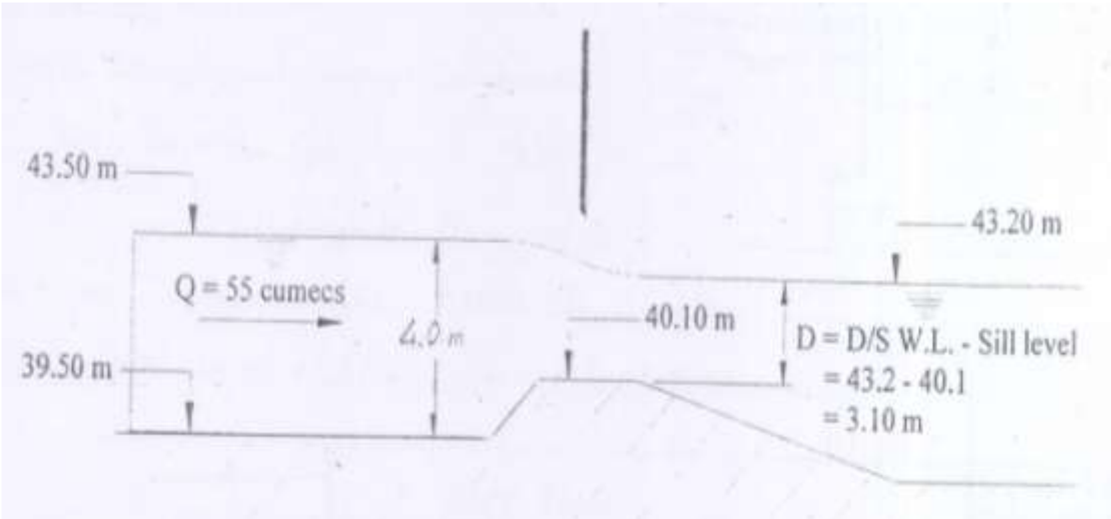
$$3 = 0.82 * S * 1.2\sqrt{2 * 9.8(0.1)} \quad \rightarrow S = 2.2\ m$$





Example 2

A cross regulator is to be constructed on a canal of 10 m bed width, 1:1 side slope, U/S W.L of 43.50 m, U/S B.L 39.50 m, D/S W.L 43.20 m, sill level under gate 40.10 m, full supply discharge Q = 55 cumecs, calculate the required width of regulator taking velocity of approach into consideration.



Solution:

To find the approaching velocity:

$$A_{U/S} = (B + Z * y) * y = (10 + 1 * 4) * 4 = 56.00\ m^2$$

$$V = \frac{Q}{A} = \frac{55}{56.00} = 0.98\ m/sec$$

$$h_a = \frac{V_a^2}{2g} = \frac{(0.98)^2}{2 * 9.81} = 0.05\ m$$

D = 3.1 m (as shown above)

H=U/S W.L –D/S W.L = 43.5 – 43.2 = 0.3 m

Now $\mathrm{Q = C\ S\ D\sqrt{2g(H + H_a)}}$

Assuming rounded entrance C = 0.9

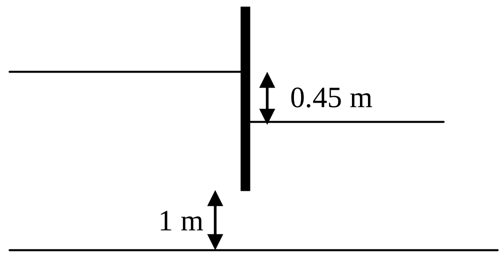
$55 = 0.9 * S * 3.1\sqrt{2 * 9.81(0.3 + 0.05)}$

S = 7.75 m

Example:

The head regulator of a canal has 3 opening each (3 m) wide. The vertical opening of the gate is (1 m). The head on the regulator is 0.45 m (Afflux). If the U/S W.L rises by 0.2 m, find how much the gates must be lowered to maintain the canal discharge unaltered.

Solution:



The width of regulator openings = 3 m spans of 3m each = 9 m when the gate opening is 1 m, the discharge can be calculated by:

$$Q = C_d\, A \sqrt{2g\, H} = 0.62\ (1 * 9)\sqrt{2 * 9.81(0.45)} \ldots\ldots\ldots..(1)$$

In the second case, when the U/S W.L rises by 0.2 m, let the gate opening be (d m) to keep the discharge unaltered

$$Q = 0.62\ (d) * (9)\sqrt{2 * 9.81(0.65)} \ldots\ldots\ldots\ldots..(2)$$

Equ. (1) = Equ. (2)   (Constant Q)

$\Rightarrow\ d = 0.82\ m$

Hence the gate must be lowered by an amount

1 – 0.83 = 0.17 m

Example:

Given the following data for a regulator site:

Upstream conditions

U/S water level = 31.78 m

U/S bottom level = 28 m

U/S canal bed width = 12 m

U/S side slope = 1.5 H: 1 V

U/S manning's (n) = 0.021

U/S discharge = 56 m^3/sec

Silt factor (f) = 0.7

Downstream conditions

Water level at full discharge = 31.25 m

Other data are the same as U/S canal

Lane's coefficient of creep C_w = 8

Total length of concrete floor L_H = 32.6 m

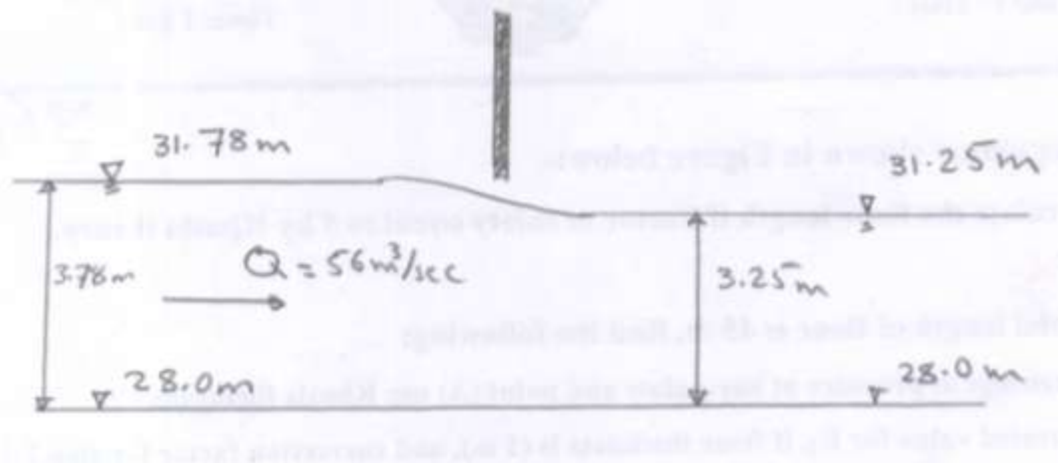
Using the above information, solve the following

1- How much is the velocity of approach when the regulator gate is fully opened?
2- How much is the required regulator width (S) for a fully opened regulator to pass the given discharge at the given water levels?
3- Assume a regulator opening of (6 m) width (S) calculate the required U/S and D/S cutoff depths?
4- Check by Lane's method to show whether your selected depths are sufficient (safe) or not?
5- Use D/S and U/S cutoff depths of 10 m and 5 m respectively, plot the uplift pressure diagram showing values at important points.

6- Calculate the required floor thickness at :

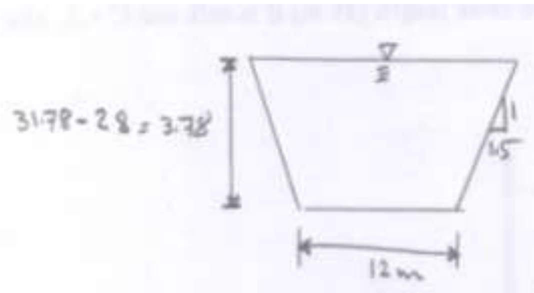
   a) U/S of gate
   b) Under the gate
   c) At the U/S side of D/S cutoff

7- If the gate width (S) is 6 m and the opening height is 0.31 m, calculate the velocity through the gate and the total discharge Q, using full supply U/S water level (neglect h_a).



Solution

U/S cross section of canal



A = (B + Z Y) Y

A = (12 + 1.5 (3.78))* 3.78

A = 66.79 m^2

1- $V_a = \frac{Q}{A} = \frac{56}{66.79} = 0.838\ m/sec$

$$h_a = \frac{V_a^2}{2g} = \frac{0.838^2}{2 * 9.81} = 0.036\ m$$

2- $Q = C\ S\ D\sqrt{2g(H + H_a)}$

$H = 3.78 - 3.25 = 0.53\ m$ $\qquad$ $H_a = 0.04\ m$

$D = 31.25 - 28 = 3.25\ m$

$56 = 0.9 * S * 3.25\sqrt{2g(0.53 + 0.04)}$ $\qquad$ *Solve for S = 5.6 m*

3- The depth of cutoff $D_{cutoff} = XR - Y$

Where X = factor of Safety = 1.25 for U/S and 1.5 for D/S

Y = water depth

R = scouring depth in (m), measured below the highest flood level

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

Where q: discharge per unit width $m^3/sec/m$

f = silt factor obtained the equation

$f = 1.76\sqrt{d_{mm}}$ d_{mm}: medium size of bed material particles

$q = \frac{56}{6} = 9.333\ m^3/sec/m$

$R = 1.35\,(\frac{9.333^2}{0.7})^{1/3} = 6.73\,m$

$D_{cutoff(D/S)} = XR - Y = 1.5(6.73) - 3.25 = 6.85\ m$

$D_{cutoff(U/S)} = 1.25\,(6.73) - 3.78 = 4.63\ m$

4- Required $L_w = C_w * H = 8 * 3.78 = 30.24\ m$

Available $L_w = \frac{32.6}{3} + 2(4.63 + 6.85) = 33.83\,m$

Since the available L_w > Required L_w, the design is accepted.

H = max. Static head = 3.78 m

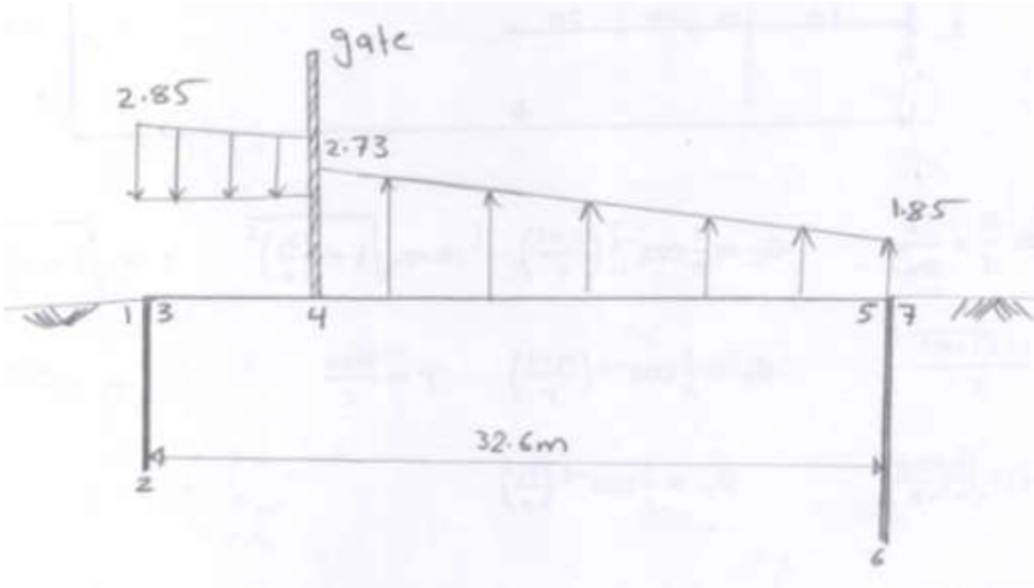
5- $L_w = \frac{32.6}{3} + 20 + 10 = 40.86\,m$

Uplift pressure at point 3 $= 3.78 - \left(\frac{3.78}{40.86}\right) * 10 = 2.85\,m$

at point 4 $= 3.78 - \left(\frac{3.78}{40.86}\right) * 11.33 = 2.73\,m$

at point 5 $= 3.78 - \left(\frac{3.78}{40.86}\right) * 20.86 = 1.85\,m$

at point 7 = zero



6- Thickness of floor (under the gate) = $t_{max.}$

$$= \frac{h_4}{G-1} = \frac{2.73}{2.4-1} = 1.8\,m \;\approx 2\,m$$

Thickness of floor at the end = $\frac{1.85}{2.4-1} = 1.25\,m$

Min. Thickness of floor at the beginning = $\frac{t_{max}}{3} = \frac{1.8}{3} \approx 0.6\,m$

7- If $S = 6\ m$, $D = 0.31\ m$, h_a is negligible

$$Q = C\,S\,D\sqrt{2gH} = 0.62 * 6 * 0.31\sqrt{2g * 3.78} = 10.3\ m^3/sec$$

Minimum depth of U/S and D/S cutoff

Canal Discharge or (capacity)	Min. depth of cutoff below G-level
Up to 3 m^3/sec	1- 1.25 m
3.1 - 30 m^3/sec	1.25 – 1.5 m
30.1 – 150 m^3/sec	1.5 - 1.75 m
Above 150 m^3/sec	Use 2 m

Structural Design of Regulators

Cases of structural Design:-

1- Regulators with fully opened gates

In this case the water head (H) is small also small uplift pressure.

2- Regulators with fully closed gates

In this case we practice very high water head (H) and very high uplift pressure.

3- Regulators under dry condition

In this case the downward weight of regulators is high and the soil reaction equal to downward force per unit area.

Note that: in case of water exist the behavior of structure floor is as floating body (submerged).

$$R_s \leq \sum \text{vertical Loads downward}$$

R_s: Soil Reaction

The design should be at critical condition (i.e. no water exists).

In this case the soil Reaction is calculated at dry condition and the floor is design according to it.

The design should be checked as follows:

1- Check for structure stability

In this case the following should be:

Uplift pressure ≈ weight of submerged structure

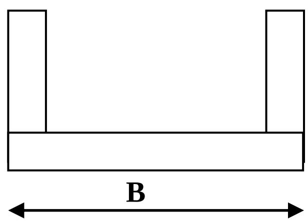
2- For structural design the structure is taken as dry and

$$\sum \text{vertical Loads} = R_s$$

$$\text{Soil Reaction } (R_s) = \frac{\text{Total weight of structure+L.L}}{\text{Foundation area}} = \frac{\sum \text{Dead Load+L.L}}{\text{A}}$$

$$R_s = \frac{\sum D.L + L.L}{B * 1}$$

(Per one meter of footing length)



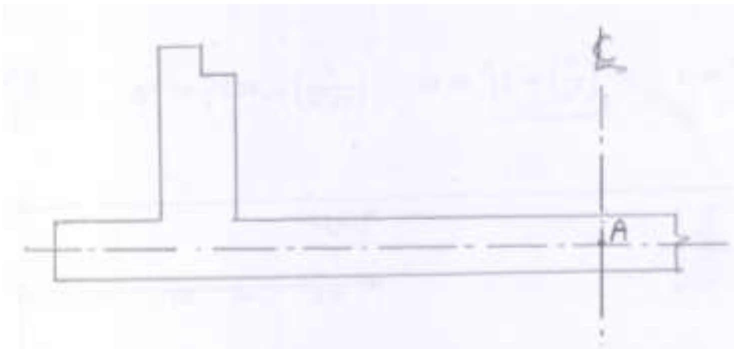
Moments Calculation

1- Take moment around point (A) at the center of floor this will be done in two cases

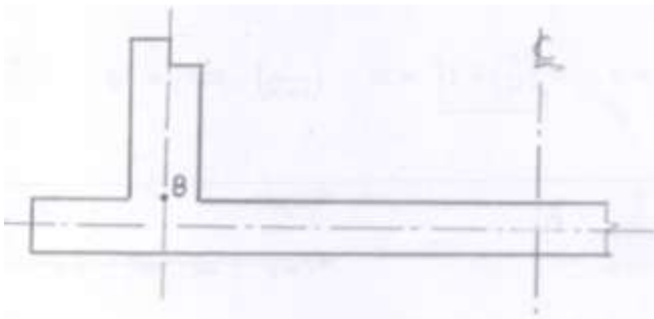
   a- Water exist around the structure (Soil surrounding the structure is saturated)

   b- Water does not exist around the structure (dry Soil)

   = M_a



2- Take moment around point B at the center of regulator column (B) as shown in figure below = M_B

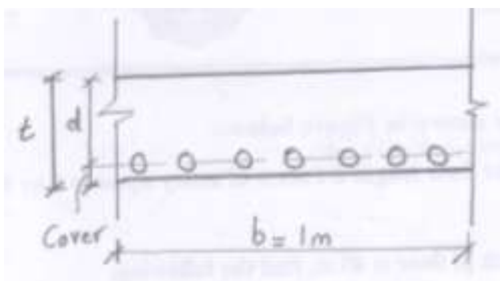


3- From (1) and (2) choose the max. Positive Bending Moment and the max. Negative Bending Moment

   - From max. (+Ve) B.M we estimate floor thickness
   - From max. (+Ve) & (-Ve) B.M we calculate the steel reinforcement.

$d = C_1 * \sqrt{\frac{M}{b}}$ $\qquad t = d + cover$

$A_s = Area\ of\ Steel = C_2 * \frac{M}{d}$



Where:

M: Bending Moment T.m

d: effective depth m

b: section width m

f_s: steel stress T/m^2

f_c: concrete compressive strength T/m^2

As: Area of steel required cm^2

C_1 & C_2: coefficient (from tables)

- Area of Temp. Steel

$A_{sTemp.}$ = 0.2 % * t* 1

- Cover for steel reinforcement According to USBR (United States Bureau of Reclamation)

For side facing water not less than 8 cm for other sides (5-6 cm)

Example

Calculate the steel reinforcement required for regulator floor (0.6 m in Thickness) given the following data:

Density of concrete = γ_c= 2.4 T/m^3

Wt of dry soil = γ_{sdry}= 2 T/m^3

Wt of submerged soil = γ_{ssub}= 1 T/m^3

Earth pressure coefficient = k_a = 0.4

Regulator clear span = S = 4 m

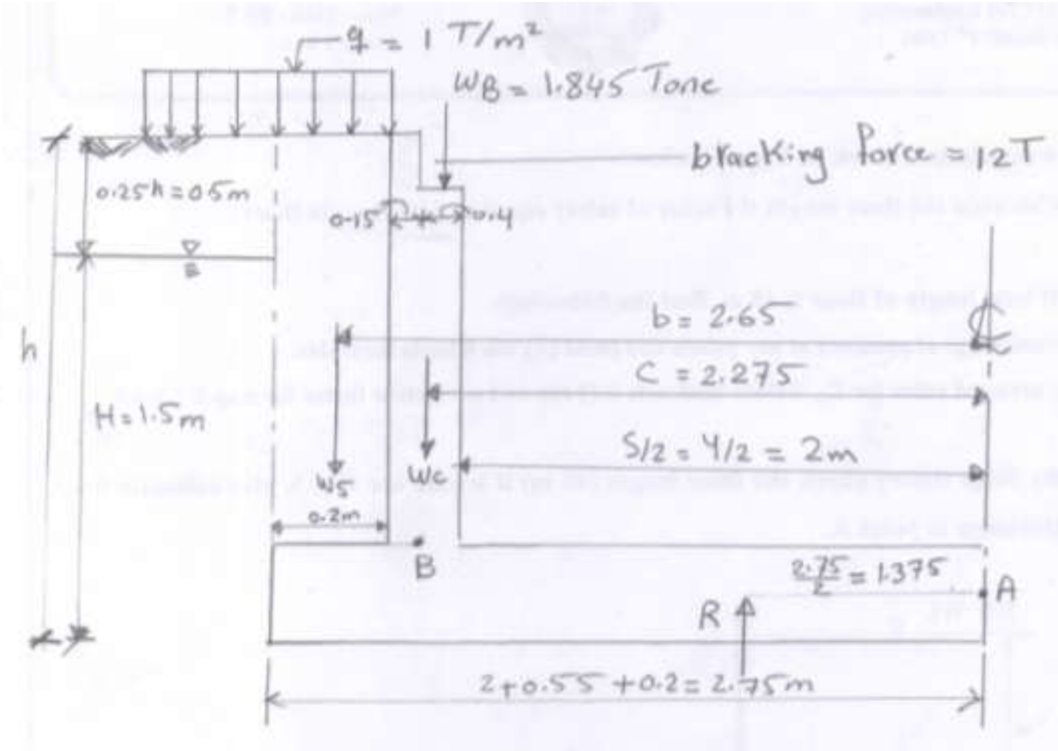
Height of side wall = h = 2 m

Water level surrounded the structure from ground surface = 0.25 h

Live Load Bracing force = 12 t

Steel tensile stress = f_s = 12500 T/m^2

Concrete compressive stress = f_c = 950 T/m^2



Solution:

1) Determination of bending moment around (A) at mid span of floor.

1- Water exists around the structure and no water inside.

Weight	Amount of Load (Tone)	Arm (m)	Moment M_A (Tone.m)
W_s	0.2 h*γ_{sdry}=0.2*2*2 = 0.8	$\frac{0.2}{2}$+0.55+2=2.65	2.12
W_c	(0.18+0.4)*2*2.4 = 2.64	$\frac{0.55}{2} + 2 = 2.275$	6.006
W_B	Bridge load (Dead +Live)	$\frac{0.4}{2} + 2 = 2.2$	4.059
$\sum V = 5.285$ Tone		$\sum M_A$=-12.186 T.m	

W_s: weight of soil behind wall

W_c: weight of wall

W_B: Loads come from bridge (Dead + Live Loads)

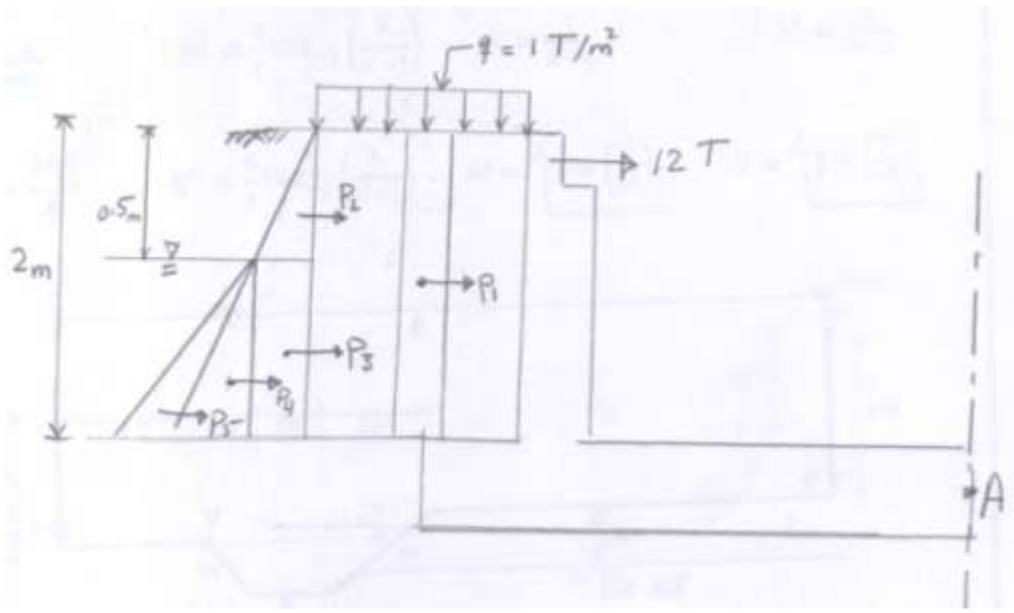
q: surcharge force

- Moment of Reaction (R):

$$M = 5.285 * \left(\frac{2 + 0.55 + 0.2}{2}\right) = +7.267$$

$$\sum M_A = -12.185 + 7.267 = -4.918\ T.m$$

1- Determination of Moment around (A) for horizontal forces

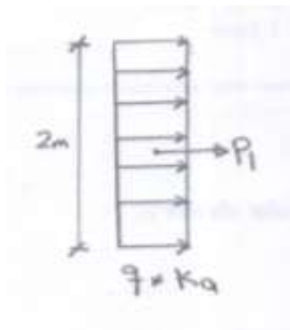


a- Force due to extra loading (q)

P$_1$ = Force due to surcharge load

P$_1$ = $q * k_a * h$ = 1 * 0.4 * 2 = 0.8 Tone

Arm of P$_1$ = (2/2 + 0.6/2) = 1.3 m



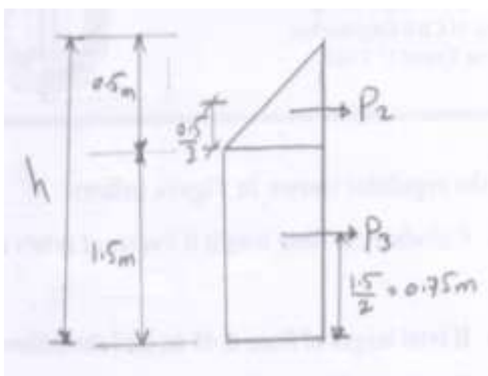
b- Force due to dry soil

$$P_2 = \gamma_{sdry} * k_a * \bar{h} * \frac{\bar{h}}{2}$$

= 2 * 0.4 * (0.5)^2/2 = 0.1 Tone

Arm of p_2 = 0.5 /2 + 1.5 + 0.6/2

= 1.967 m



c- Force due to dry soil

$P_3 = \gamma_{sdry} * k_a * \bar{h} * (h - \bar{h})$

= 2 * 0.4 *0.5 (2 – 0.5) = 0.6 Tone

Arm of P_3 = 0.75 + 0.6 /2 = 1.05 m

d- Force due to submerged soil

$$P_4 = \gamma_{ssub} * k_a * \left(\frac{h - \bar{h}}{2}\right)^2$$

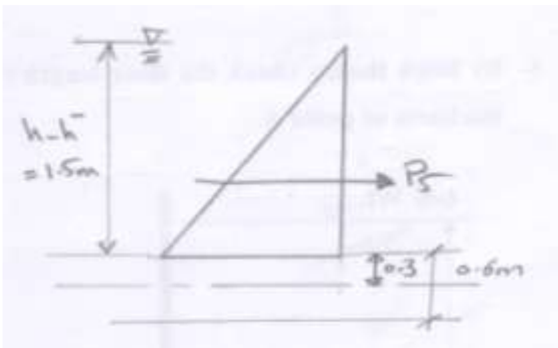
$$= 1 * 0.4 * \left(\frac{2 - 0.5}{2}\right)^2 = 0.45\ Tone$$

Arm of $P_4 = \left(\frac{h-\bar{h}}{3}\right) + \frac{0.6}{2} = \frac{1.5}{3} + 0.3 = 0.8\ m$

e- Force due to water pressure

$$P_5 = \gamma_w * \frac{(h - \bar{h})^2}{2} = 1 * \frac{1.5^2}{2} = 1.125\ Tone$$

Arm of $P_5 = \left(\frac{h-\bar{h}}{3}\right) + \frac{0.6}{2} = \frac{1.5}{3} + 0.3 = 0.8\ m$



- Calculation of bending moment around (A):

Force (Horizontal)	Tone	Arm (m)	Moment around A (T.M)
P_1	0.8	1.3	1.04
P_2	0.1	1.967	0.197
P_3	0.6	1.05	0.63
P_4	0.45	0.8	0.35
P_5	1.125	0.8	0.9
$\sum$	3.075		3.127

$\sum M_A$ = +3.127 Tone.m

- Moment due to braking force

$$M_B = F_b \frac{(h+\frac{t}{2})}{(1+2h+t)} = 12\frac{2+0.6/2}{(1+2*2+0.6)} = 4.929\ T.m$$

$\sum M_A$ = 3.127 + 4.929 = + 8.056 T.m

Sum of Moment due to both horizontal & vertical forces around (A) = $\sum M_A$ = - 4.018 + 8.056 = 3.138 T.m

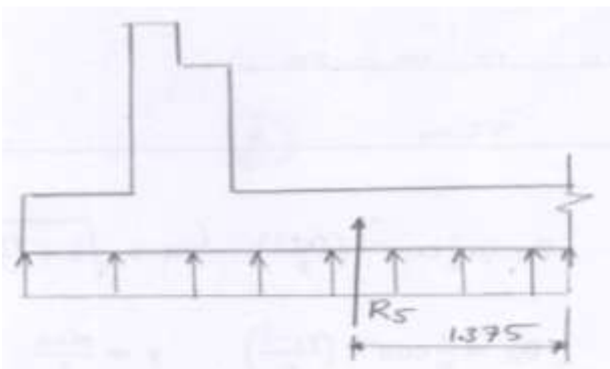
2- Calculation of Moment in case no water exist around structure (Before operation or just after the construction end)

a- Moment due to vertical forces

$\sum M_A$ = -12.185 T.m

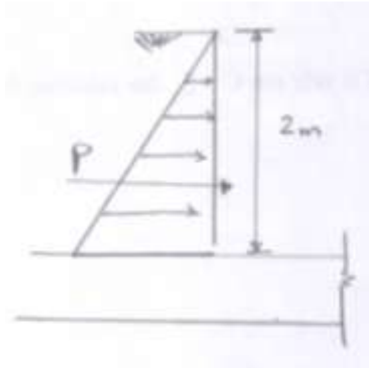
b- Moment due to soil reaction

$M_{Rs} = \sum V$ * 1.375 = 5.285 * 1.375 = 7.267 T.m



c- Moment due to Horizontal forces

1- Due to dry soil



P = $\gamma_{sdry} * k_a * h * \frac{\bar{h}}{2} = 2 * 2 * 0.4 * \frac{2}{2} = 1.6\ T$

M_A = 1.6 * (2/2 + 0.6/2) = 1.547 T.m

2- Moment due to surcharge load

$F_s = q * k_a * h$

= 1 * 0.4 *2 = 0.8 Tone

M_A = 0.8 * (2/2 + 0.6/2) = 1.04 Tone.m

$\sum M_A$ = -12.185 + 7.267 + 1.547 +1.04 = -2.331 T.m

Summary:

- Water exist

M_A = + 3.138 T.m

- No water exist

M_A = - 2.332 T.m

t = d + cover = 0.6 = d + 0.08   d = 0.52 m

For C_1 = 0.08 C_2 = 0.82

Then:

$A_s = C_2 \frac{M}{d} = 0.82 * \frac{+3.138}{0.52} \approx 5\ {cm^2}/{m}$

$As = 0.82 * \frac{-2.331}{0.52} \approx 4\ {cm^2}/{m}$

Area for $\emptyset(12\ mm) \approx 1.1\ cm^2$

Use 5 bars for $+V_e$ moment

4 bars for $-V_e$ moment

Hydraulic Jumps

Hydraulic jumps mark the flow transition from supercritical to subcritical flow. When subcritical flow accelerates into the supercritical state the transition often is smooth with gradually increasing velocity and decreasing depth bringing about a smooth drop in the water surface until the alternate depth is achieved. Any disturbance to the water surface is smoothed out by the surface or gravity wave propagation mechanism. In these circumstances energy losses are not great and the Bernoulli equation does a credible job of describing the changes to the flow. When supercritical flow changes to subcritical flow, however, there is no smoothing of the water surface upstream of the transition because the high downstream velocity prevents upstream diffusion of the water-surface deformation. As a result the transition to subcritical flow is sudden and marked by an abrupt discontinuity, or hydraulic jump, in the water surface (Figure1). The greater the difference between the alternate depths the more severe the hydraulic jump.

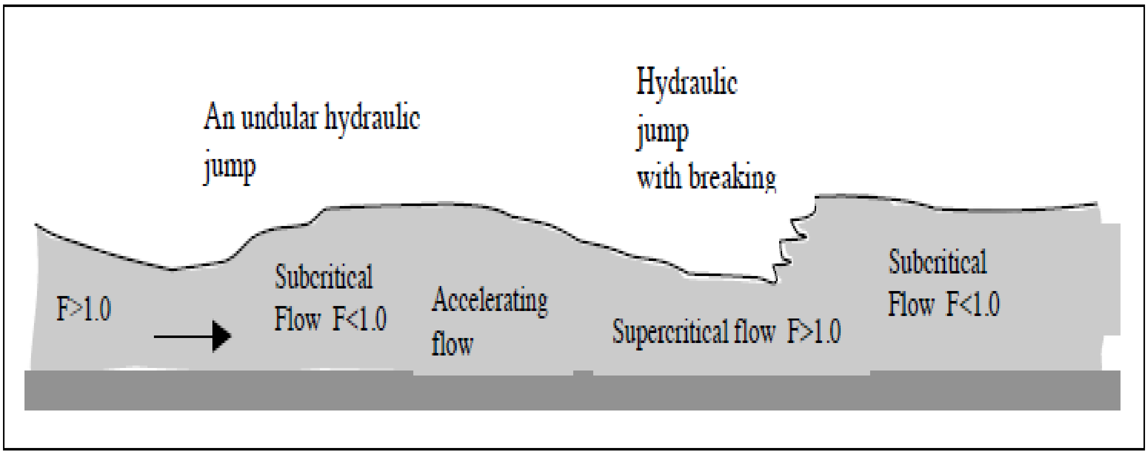


Figure (1) Undular and breaking hydraulic jumps in open-channel flow

Hydraulic jumps appear in the surface of rivers as standing waves or surges. In their most fully developed state they appear as a line of breaking waves normal to the river bank which remains stationary with respect to an observer on the bank. Upstream of the jump the flow is supercritical and downstream it is subcritical. Hydraulic jumps may occur as a single event in a reach of channel or they may occur in trains as flow alternates back and forth across the two flow states. They may be steady-state features or they may be intermittent or even periodic, forming and collapsing, and reestablishing again, in response to the highly unsteady non-uniform flow. As we might expect, hydraulic jumps are more likely to be found disrupting flows in steep mountain streams rather than those in large low-slope rivers although even the latter may exhibit this flow phenomenon at high flood discharges. Because the flow at the hydraulic jump often is surging and highly agitated, there is a great deal of energy lost here, precisely the circumstance we must avoid if the Bernoulli equation is the only analytical tool at our disposal.

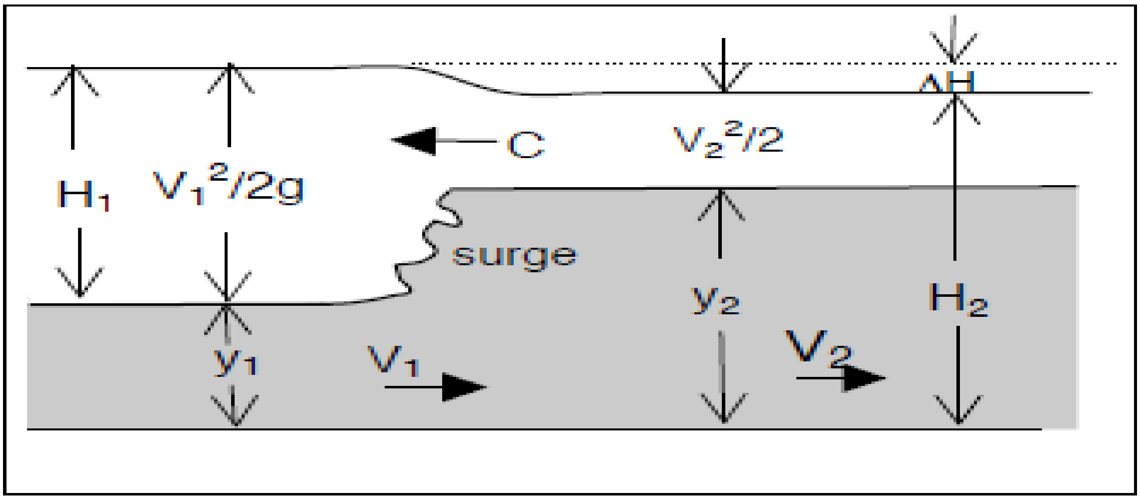


Figure (2) Flow characteristics at hydraulic jump

y_1: the initial depth of the jump (depth of water before the jump)

y_2: the sequent depth of the initial depth (depth of water after the jump).

The following relations can be used for the calculations of hydraulic jump:

$$\frac{V_1}{\sqrt{gy_1}} = F_1 = \sqrt{\frac{1}{2}\frac{y_2}{y_1}\left(\frac{y_2}{y_1}+1\right)}$$

$$\frac{V_1^2}{gy_1} = \frac{1}{2}\frac{y_2}{y_1}\left(\frac{y_2}{y_1}+1\right)$$

$$\frac{y_2}{y_1} = \frac{1}{2}\left(\sqrt{8F_1^2+1}-1\right)$$

$$\frac{y_1}{y_2} = \frac{1}{2}\left(\sqrt{8F_2^2+1}-1\right)$$

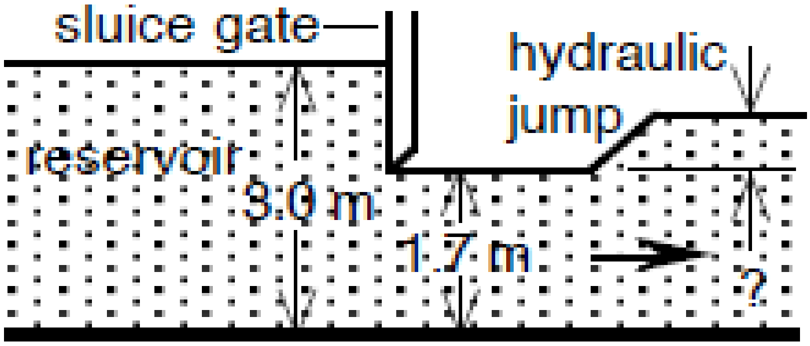
Problem (1): Water flows along a 10-m wide rectangular channel and through a hydraulic jump. If the flow depth just before the jump is 2.0 m, and 3.0 m after it, what is the discharge through the channel?

We get $\frac{V_1^2}{9.806y_1} = \frac{1}{2} x \frac{3}{2}\left(\frac{3}{2}+1\right)$

So that $V_1^2 = 36.7725$ and $V_1 = 6.064\ ms^{-1}$

Thus discharge, $Q = wV_1y_1 = 10x6.064\ x\ 2 = 121.3\ m^3s^{-1}$

Problem (2): A sluice gate at the base of a large reservoir is raised 1.7 m, as shown opposite, and the water discharges through this 5.0 m-wide rectangular orifice into a rectangular channel of the same width. If a hydraulic jump forms in the channel, what will be its height?



Solution:

Since the reservoir is large, we can assume that the water depth will not change rapidly and that the depth of water in the tank represents the total head E1 = 3.0 m. Thus we can write the Bernoulli equation for this case as

$$E_1 = y_1 + \frac{V_1^2}{2g}$$

or

$$3 = 1.7 + \frac{V_1^2}{2x\ 9.8} \quad \text{Therefore,} \quad V1 = 5.049\ \frac{m}{sec}$$

The flow discharging from under the sluice gate is supercritical since $F = \dfrac{5.049}{\sqrt{9.806 \text{ x } 1.70}} = 1.237$

$$\frac{y2}{y1} = \frac{1}{2}\left(\sqrt{(8 \text{ x } 1.237^2) + 1} - 1\right) = 1.319$$

Since $y_1 = 1.70$ m, $y_2 = 2.242$ m so that the hydraulic jump must be 0.542 m high

Energy Loss in Hydraulic Jumps

To obtain the energy loss across a hydraulic jump in a rectangular channel, we have to compute E1-E2 when M1 = M2. It will be found that unless F12 and y2/y1 are quite large, the difference E1-E2 is substantially smaller than either E1 or E2, making it difficult to calculate the difference with reasonable accuracy .This is a common problem in computing work; the obvious but clumsy remedy is to calculate E1and E2 to a higher degree of precision than is required for the difference E1-E2. A much better way out of this difficulty, one suggested by the influential river engineer professor Frank Henderson (1966), is outlined below. He recommended rearranging the algebra so that E1-E2 is expressed in a form involving products rather than differences. We have

$$E_1\text{-}E_2 = (y_1 + \frac{v_1^2}{2g}) - (y_2 + \frac{v_2^2}{2g}) = (y_1 - y_2) + \frac{q^2}{2g}\left[\frac{1}{y_1^2} - \frac{1}{y_2^2}\right]$$

So, making the substitution $F_1^2 = \dfrac{q^2}{gy_1^3}$, we take out the factor $(y_1 - y_2)$ leading to

$$E_1 - E_2 = (y_1 - y_2)\left[1 - \frac{F_1^2}{2}\frac{y_1(y_1+y_2)}{y_2^2}\right] \ldots\ldots\ldots\ldots\ldots\ldots$$

Now we impose the condition $M_1 = M_2$ by using the hydraulic jump equation:

$F_1^2 = \dfrac{1}{2}\dfrac{y_2}{y_1}\left(\dfrac{y_2}{y_1}+1\right)$. Setting $r = \dfrac{y_2}{y_1}$ and simplifying the expression within the bracket yields:

and finally, $$\frac{E_1 - E_2}{y_1} = \frac{(r-1)^3}{4r}$$

For example, in a hydraulic jump where $y_1 = 1.000$ and $y_2 = 1.100$ ($r = \dfrac{y_2}{y_1} = 1.100$)

$$E_1 - E_2 = (1.000)\frac{(1.100-1)^3}{4(1.100)} = 0.00023 \text{ m.}$$

In other words, energy loss through this mild hydraulic jump amounts to about two fifths of one millimeter of head

In the case of severe hydraulic jumps, energy losses may be very large indeed.

In a hydraulic jump where $y_1 = 1.000$ and $y_2 = 2.000$ ($r = \dfrac{y_2}{y_1} = 2.0$)

$$E_1 - E_2 = (1.000)\frac{(2.000-1.000)^3}{4(2.00)} = 0.125 \text{ m.}$$

Yields F_1=1.73 for these flow conditions, associated with a head loss amounting to 12.5% of the energy of the approach flow. As we noted earlier, hydraulic jumps can be very efficient dissipaters of flow energy and are sometimes designed by engineers to form in outlet flows where the approach flow might otherwise cause damaging scour to the channel bed.

Types of jump:

Depending upon the incoming Froude No. F1, the jump on horizontal floor can be classified as follows:-

No.	Froude N0.	Type of jump	Efficiency	Length
1	1	Critical flow (no jump)		
2	1 – 1.7	Undular jump	0 % - 5 %	
3	1.7 – 2.5	Weak jump	5 % - 16 %	4 – 4.9 y_2
4	2.5 – 4.5	Oscillating jump	16 % - 45 %	4.9 – 5.9 y_2
5	4.5 – 9	Well developed jump(steady jump)	45 % - 70%	5.9 – 6.1 y_2
6	9 - 20	Strong jump	70 % - 85 %	6.1 – 8.4 y_2

Note:

Type 1, 2, and 3 not useful for energy dissipation
Type 4 should be avoided
Type 5 is the best

References

Henderson, F.M., 1966, Open Channel Flow, Macmillan, New York.

Example:

In order to dissipate energy below the spillway, it is proposed to form a hydraulic jump in the stilling basin. Due to this, the depths of flow changes from 1.0 m to 3.8 m. Calculate the discharge over the spillway, the crest length of which is 110 m.

Solution:

From the equation: $\frac{y_2}{y_1} = \frac{1}{2}\left(\sqrt{8F_1^2 + 1} - 1\right)$

F_1 = 3.02 then solve for V_1 = initial velocity of flow = $F_1\sqrt{gy_1}$

V_1 = 9.45 m /sec

Hence the discharge over spillway $Q = V_1\ y_1\ L = 9.45\ (1)\ (110) = 1040\ m^3/sec$

Example:

Water flows at a rate of 10 m^3/sec in a rectangular channel 5.5 m wide at a depth of 30 cm. What is the total energy loss in a hydraulic jump which has occurred from this flow?

Solution:

$$V_1 = \frac{Q}{By_1} = \frac{10}{5.5\ (0.3)} = 6.06\ m/sec$$

$$F_1 = \frac{V_1}{\sqrt{gy_1}} = \frac{6.06}{\sqrt{9.81\ (0.3)}} = 3.53$$

Use $\frac{y_2}{y_1} = \frac{1}{2}\left(\sqrt{8F_1^2 + 1} - 1\right)$

Solve for y_2 = 1.357 m and $V_2 = \frac{Q}{By_2} = \frac{10}{5.5\ (1.357)} = 1.34\ m/sec$

Use $E_1 - E_2 = y_1\ \frac{(r-1)^3}{4r}$

$r = \frac{y_2}{y_1} = \frac{1.357}{0.3} = 4.52$ then $E_1 - E_2 = 0.721\ m$

Or $\Delta E = \left(y_1 + \frac{V_1^2}{2g}\right) - \left(y_2 + \frac{V_2^2}{2g}\right) = 2.17 - 1.449 = 0.721\ m$

Example:

A hydraulic jump occurs in a rectangular open channel. The water depths before and after the jump are 0.6 m and 1.5 m, respectively. Calculate the depth and the loss of head in the jump.

Solution:

$$\frac{y_2}{y_1} = -\frac{1}{2} + \sqrt{\frac{1}{4} + \frac{2q^2}{gy_1}} \quad \rightarrow \quad \frac{1.5}{0.6} = -\frac{1}{2} + \sqrt{\frac{1}{4} + \frac{2q^2}{9.81x0.6}}$$

$$q = 3.045\ \frac{m^3}{s.m}$$

Or $\frac{q^2}{g} = \frac{1}{2}\, y_1 y_2 (y_1 + y_2) \quad \rightarrow \quad \frac{q^2}{9.81} = \frac{1}{2} x 0.6 x 1.5 x (0.6 + 1.5) \quad \rightarrow$

$q = 3.045$

$$y_c = \sqrt[3]{\frac{q^2}{g}} = \sqrt[3]{\frac{3.045^2}{9.81}} = 0.981\ m$$

$$E_1 = y_1 + \frac{q^2}{2gy_1^2} = 0.6 + \frac{(3.045)^2}{19.62x0.6^2} = 1.913\ m$$

$$E_2 = y_2 + \frac{q^2}{2gy_2^2} = 1.5 + \frac{(3.045)^2}{19.62(1.5)^2} = 1.71\ m$$

$$\Delta E = E_1 - E_2 = 1.913 - 1.71 = 0.203\ m$$

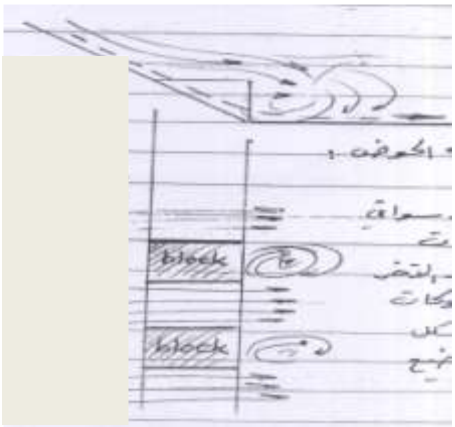
Or $\Delta E = \frac{(y_2 - y_1)^3}{4y_1 y_2} = \frac{(1.5-0.6)^3}{4x1.5x0.6} = 0.2025\ m$

Standard Stilling Basins

A stilling basin is a short length of paved channel placed at the end of any source of super critical flow. The aim of the designer is to make a hydraulic jump from within the basin; so that the flow is converted to subcritical flow before it reaches the exposed and unpaved riverbed D/S. The basin thus designed is usually provided with special appurtenances, including chute blocks, sills, and baffle blocks.

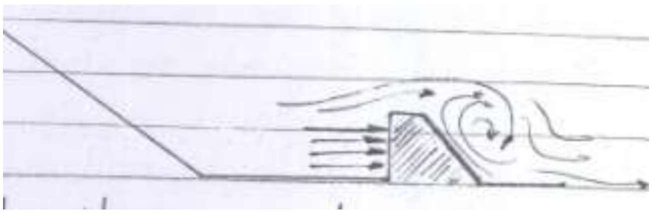
1- Chute blocks:

Their function is to separates the supercritical flow into lower and upper layers at the blocks, or to furrow the incoming jet and lift a portion of it from the floor. In this action the created turbulence dissipate a big amount of energy.

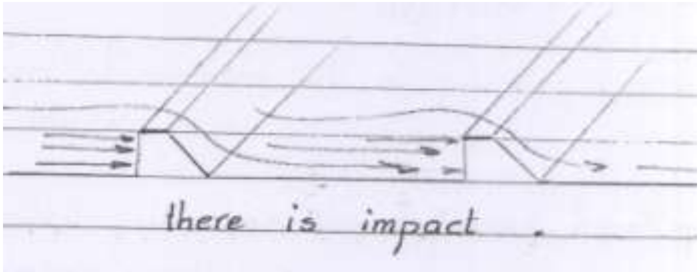


2- Impact blocks (Baffle piers)

Are blocks placed in intermediate position across the basin floor. Their function is to dissipate energy mostly by impact action; a big percentage of excessive energy is dissipated through impact with the blocks. Thus the blocks should be strong enough to resist this force.

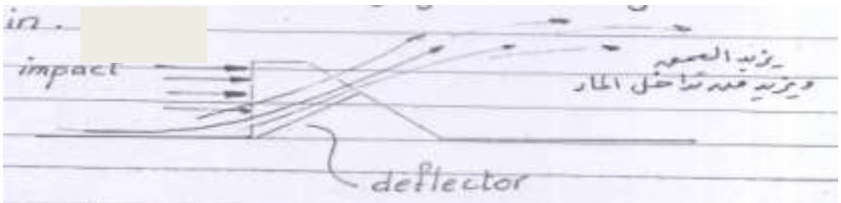


In supercritical flow, if the blocks are continuous or near to each other, the supercritical flow may overshoot the blocks.

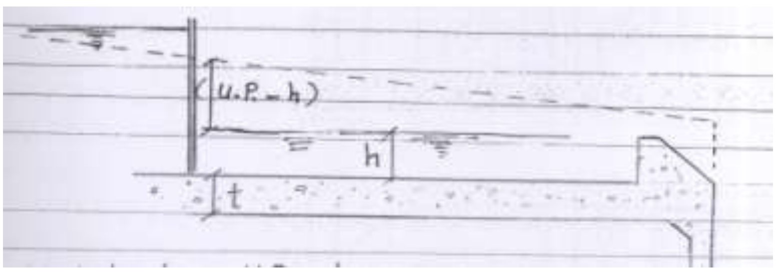


3- End Sills

Either dentate or solid, is usually provided at the end of the stilling basin. Its function is to reduce further the length of the jump and to control scour. For large basins that are designed for high incoming velocities, the sill is usually dentate to perform the additional function of diffusing the residual portion of the high –velocity jet that may reach the end of the basin.



The effect of dentate sill is practically same as chute and impact blocks.
Continuous sill: has the following function:
1- As impact wall.
2- It keeps the water at its U/S as a stilling pool.
3- The depth of water at its U/S help reducing the floor thickness.



Needed $t = \frac{u.p.-h}{G-1}$

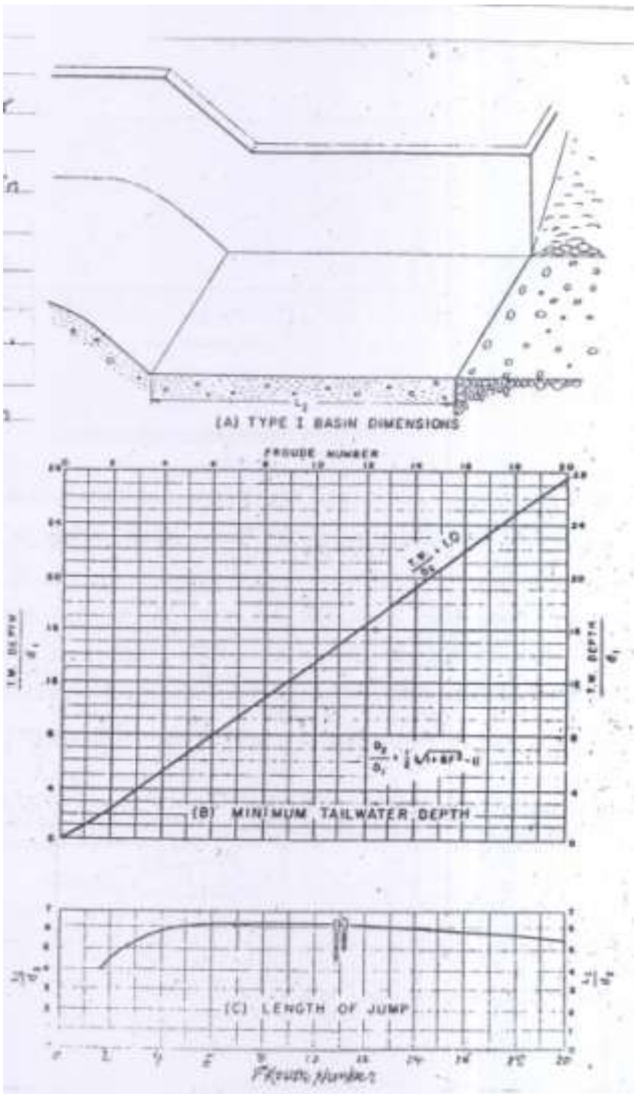
Type Stilling Basins and Energy Dissipaters

Using the U.S.B.R. (U.S. Bureau of Reclamation) 1987 classification, there are five basic hydraulic jump type basins that are briefly described:-

Basin I

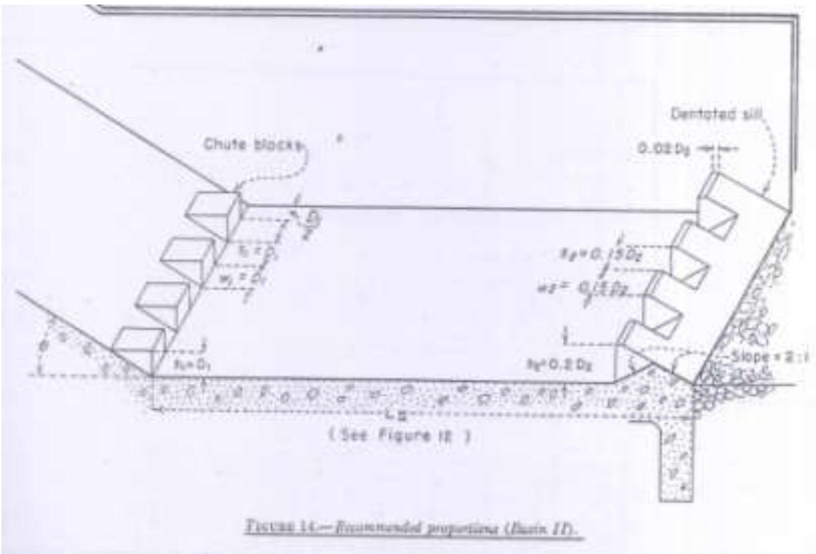
For Froude number less than 1.7, no special stilling basin is required.

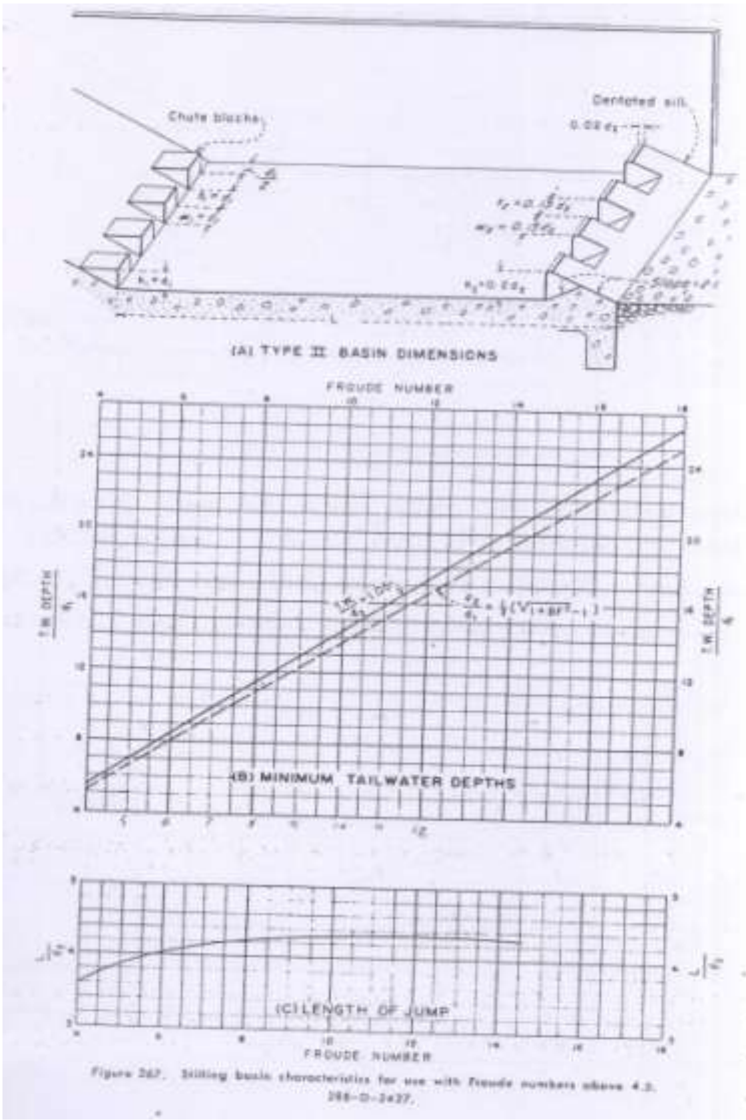
This basin do not required baffle or dissipation devices. For (Fr) between 1.7 and 2.5 the type I-basin also applies.



U.S.B.R. Stilling Basin No. II

It is used when the incoming velocities exceed 15 m/sec, and for high spillways or high head and large structures, $Fr_1 > 4.5$. The basin contains chute blocks at the U/S end and a dentate sill near the D/S end. No baffle piers are used because the relatively high velocities entering the jump might cause cavitations on piers.





U.S.B.R Stilling Basin No. III

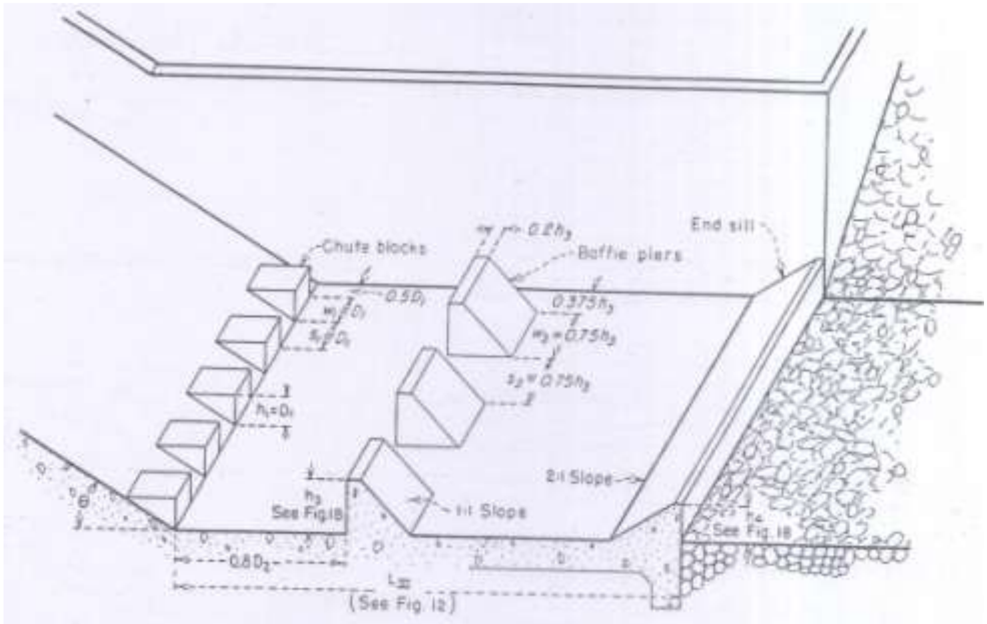
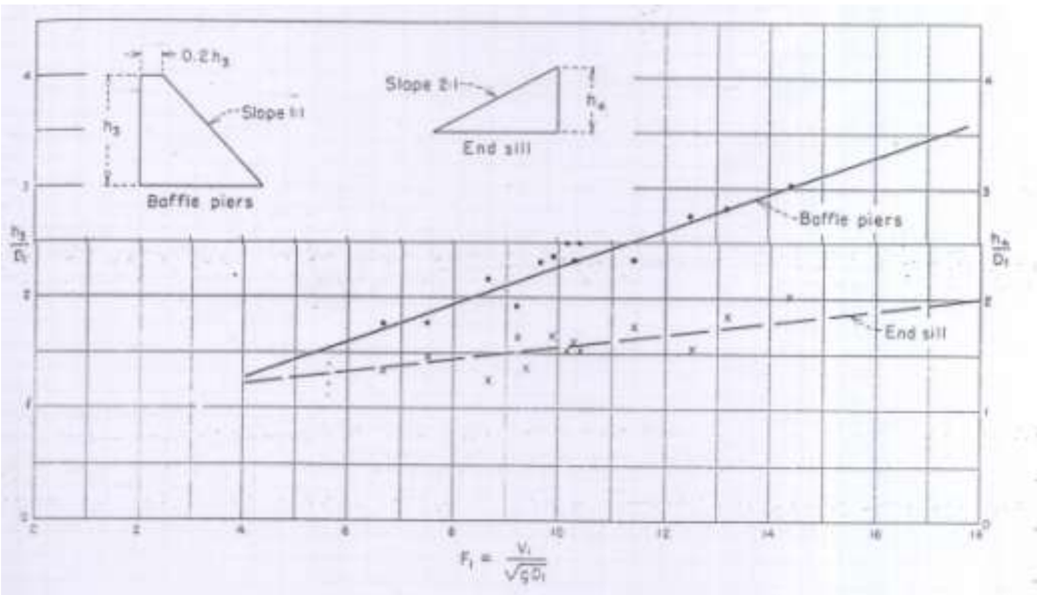
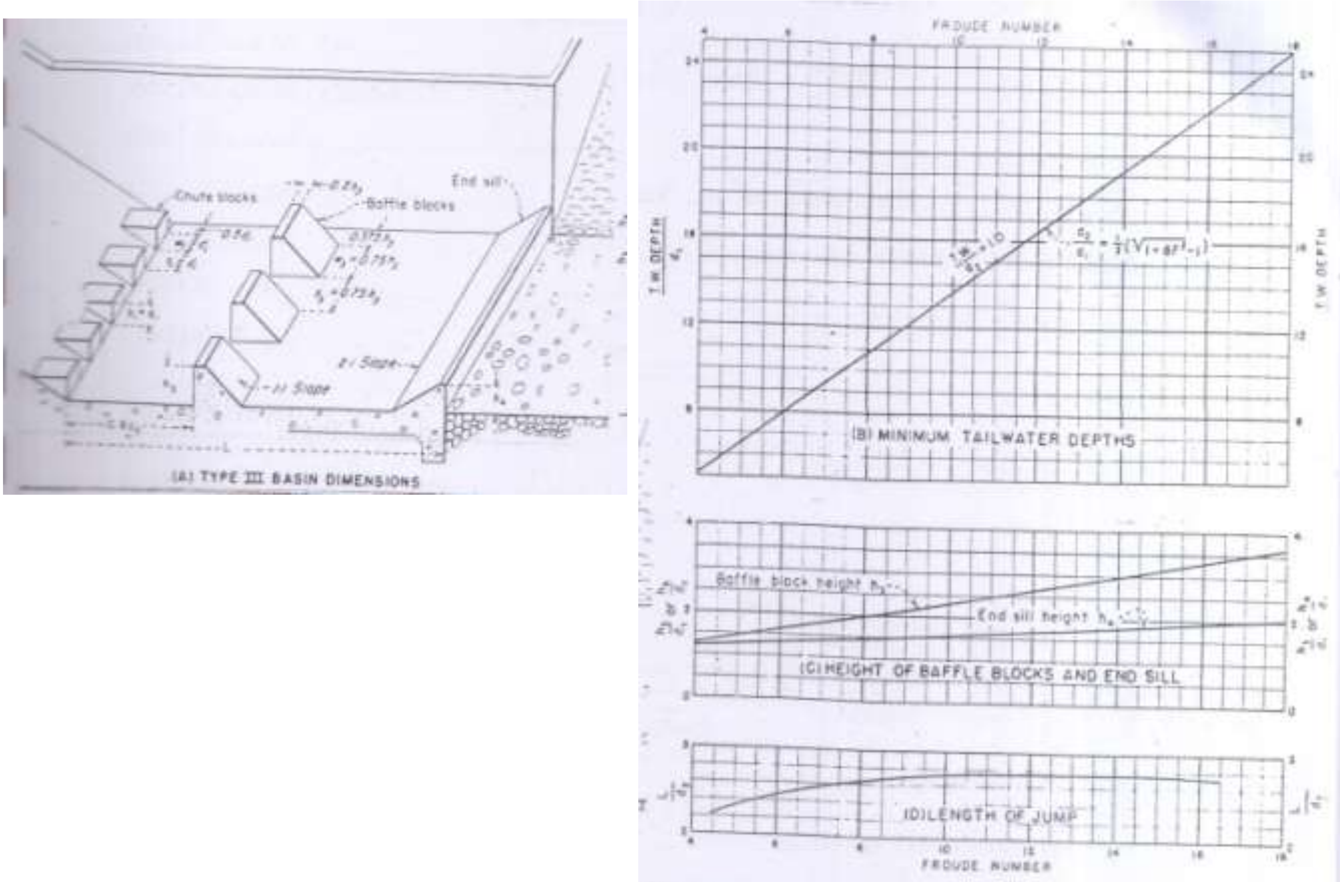
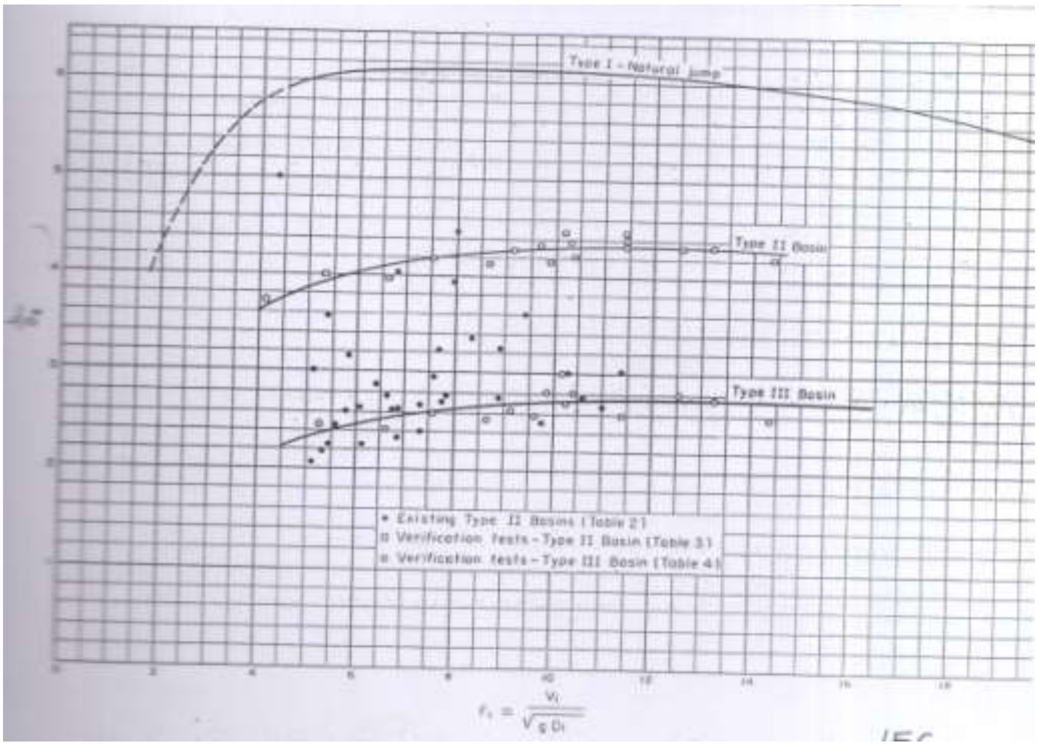


FIGURE 17.—Recommended proportions (Basin III).

This basin may be used when the incoming velocity do not exceeds 15 m/sec, and for $Fr_1 > 4.5$ but small structures. It is the same as No. II but with additional impact blocks and continuous end sill.

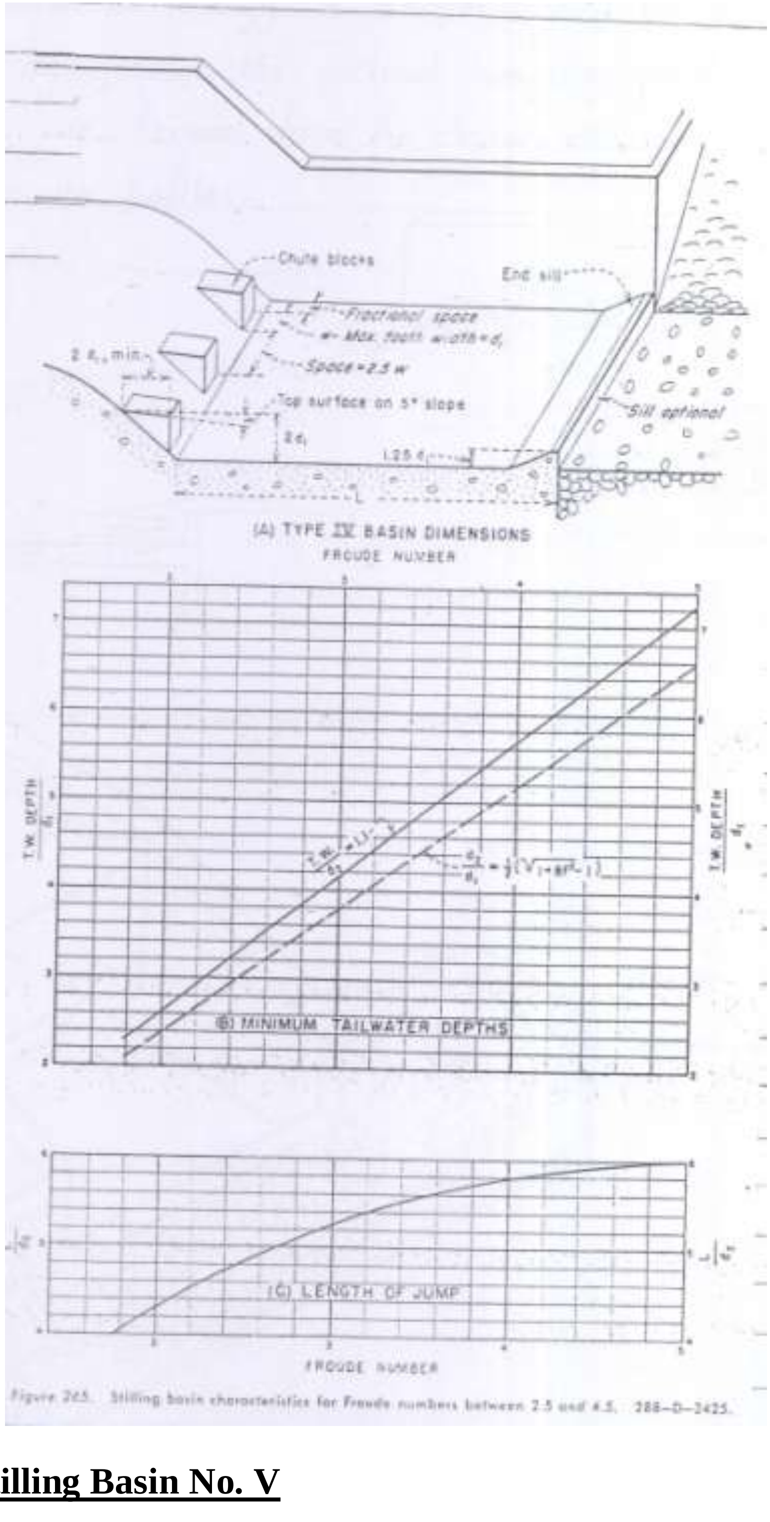






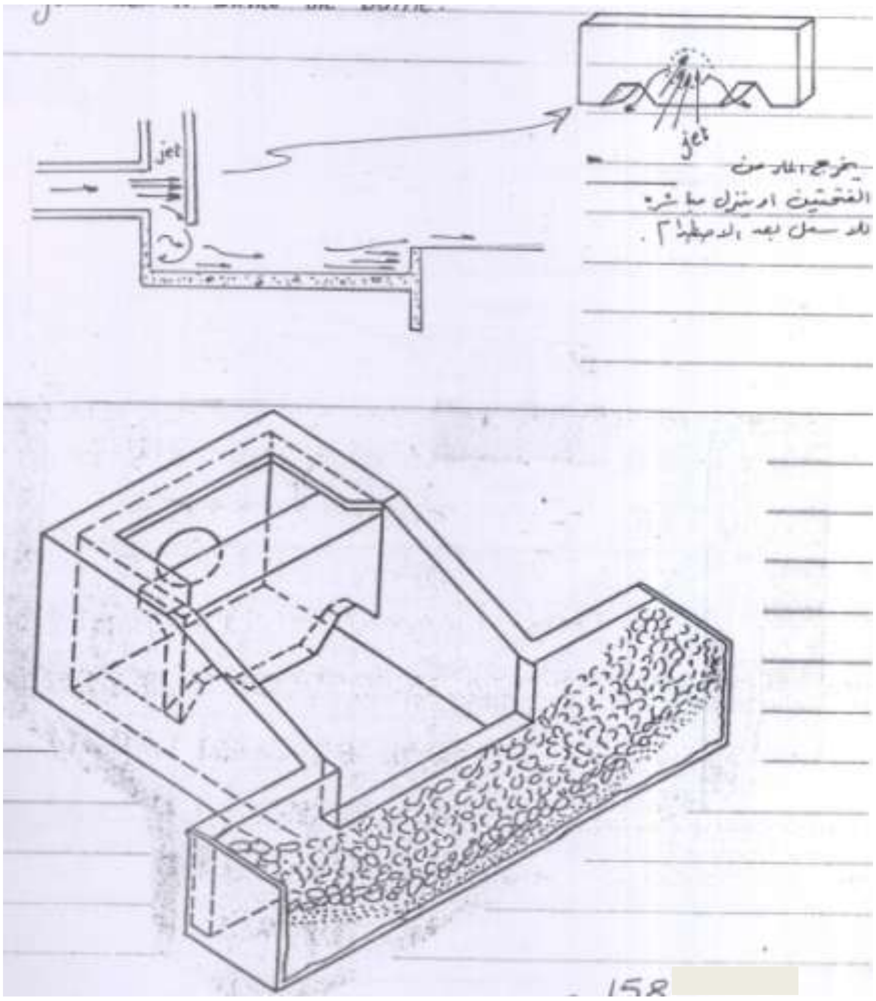
U.S.B.R Stilling Basin No. IV

This is recommended for use with jump of Fr_1 = 2.5 to 4.5 (Oscillating jump) which usually occur on canal structures and diversion dams. This basin is applicable to rectangular cross section only, it is designed to eliminating the wave at its source.



U.S.B.R Stilling Basin No. V

It can be used for small structures and closed conduit structures, dissipation of energy is accomplished by the impact of the incoming jet on the vertical hanging baffle and by eddies which are formed from the change direction of the jet after it strike the baffle.



U.S.B.R Stilling Basin No. VI

Providing energy dissipation independent of a tail water curve or tail water of any kind.

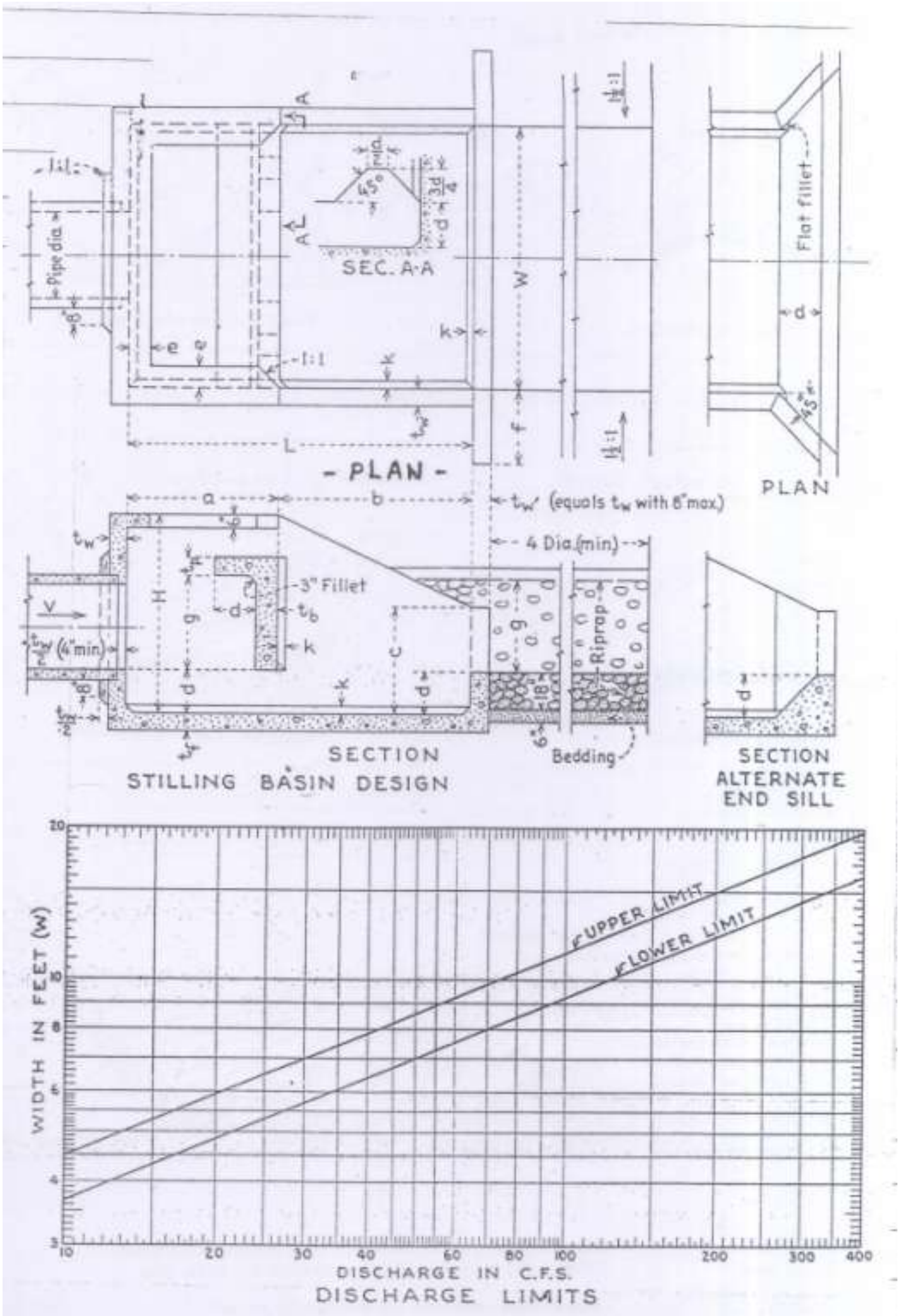
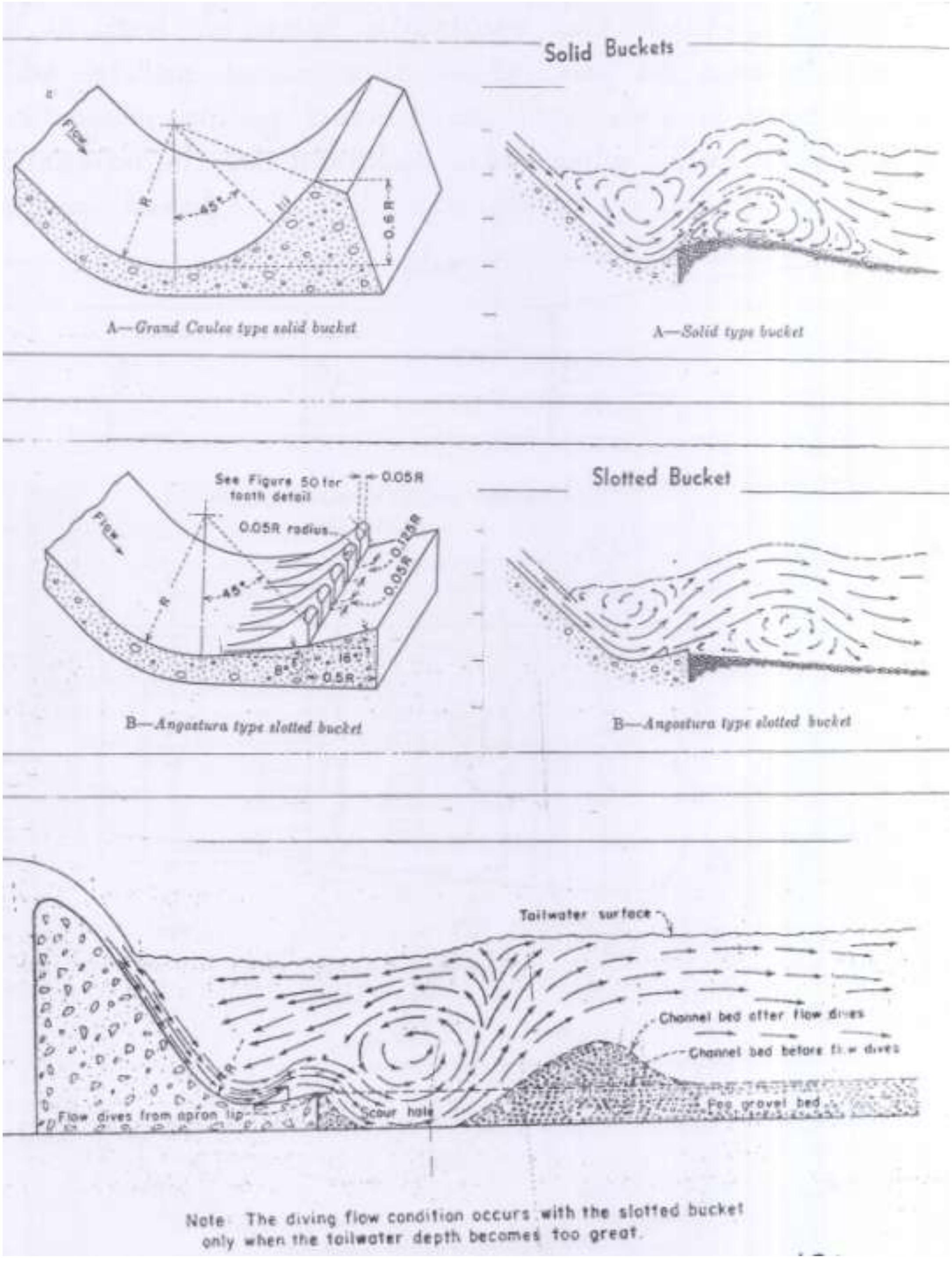


FIGURE 42.—Impact-type energy dissipator (Basin VI).

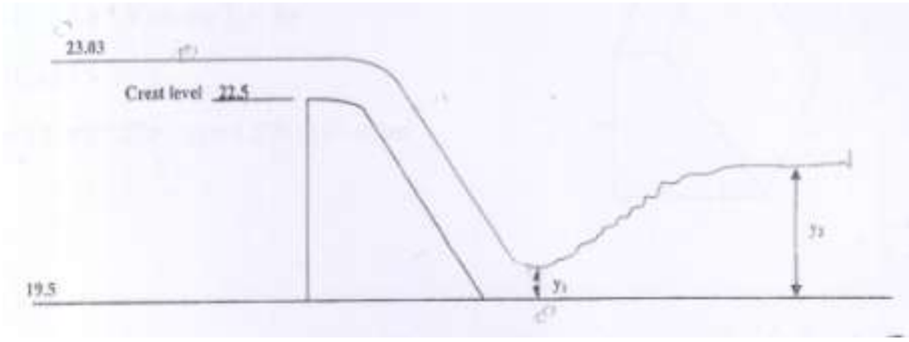
U.S.B.R Stilling Basin No. VII

This type of energy dissipater is suitable where the stream bed is composed of hard rock and the tail water depth is less than D_2 curve.



Example 1:

Design U.S.B.R. stilling basin for the following hydraulic structures, Q= 5.15 m^3/sec, crest width = 6 m, neglect the effect of approaching velocity.



Solution

$$V_1 = \frac{Q}{By_1} = \frac{5.15}{6y_1} = \frac{0.858}{y_1}$$

The energy equation (Bernoulli)

$$Z_o + y_o + \frac{V_o^2}{2g} = Z_1 + y_1 + \frac{V_1^2}{2g}$$

$$3.35 = y_1 + \frac{V_1^2}{2g}$$

$$3.35 = y_1 + \left(\frac{q^2}{y_1^2}\right) \cdot \frac{1}{2g}$$

$3.35 = y_1 + \frac{(0.858)^2}{19.62y_1^2}$ by trial and error $\rightarrow y_1 = 0.11\ m \rightarrow V_1 = 7.8\ m/sec$

$V_1 = 7.8\ m/\sec < 15\ m/sec$

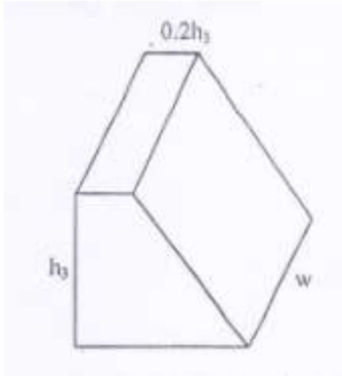
$F_{r1} = \frac{7.8}{\sqrt{9.81(0.11)}} = 7.57\ > 4.5$ from the curve on page 14

Therefore use stilling basin No. III

Now for $F_{r1} = 7.5\ \rightarrow \frac{L}{y_2} = 2.6$

$$y_2 = \frac{y_1}{2}\left(\sqrt{8F_{r1}^2 + 1} - 1\right) = 1.12\ m$$

$L = 2.6(1.12) = 2.93\ m\ use\ L = 3\ m$

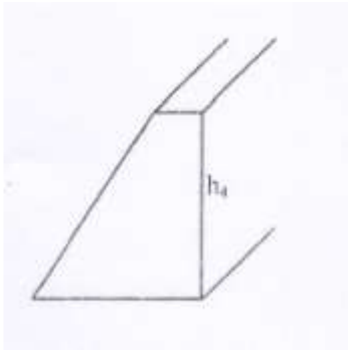


Baffle Blocks

$\frac{h_3}{y_1} = 1.8 \rightarrow h_3 = 1.8(0.11) = 0.2\ m$

End Sill

$\frac{h_4}{y_1} = 1.4 \rightarrow h_4 = 1.4(0.11) = 0.154\ m$



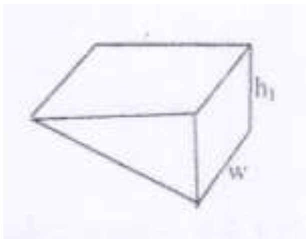
Use $h_4 = 0.15m$

Chute Blocks

$h_1 = y_1 = 0.11m$

$w_1 = y_1 = 0.11m$

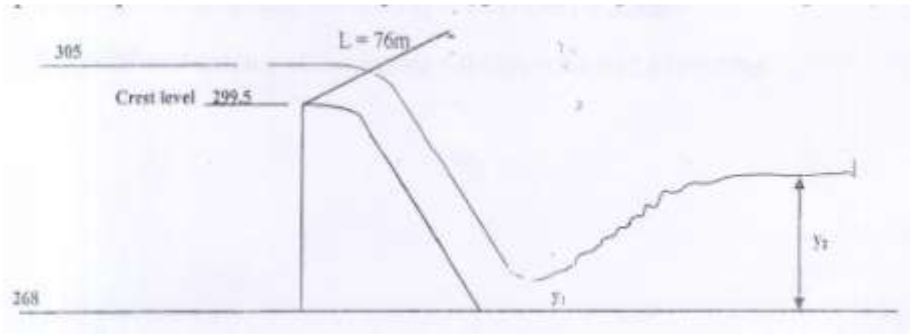
$S_1 = 0.11m$



Example 2

Proportion a U.S.B.R. stilling basin type II for the given flow spillway.

Q= 2125.4 m^3/sec, assuming a high over flow spillway, the approaching of velocity is negligible.



Solution: assuming a high over flow spillway, the effect of approach velocity is negligible or you can calculate it as following:-

$V_1 = \frac{Q}{By_1} = \frac{2125.4}{76y_1}$ The energy equation (Bernoulli)

$$Z_o + y_o + \frac{V_o^2}{2g} = Z_1 + y_1 + \frac{V_1^2}{2g}$$

$$37 = y_1 + \left(\frac{2125.4^2}{(76y_1)^2}\right) \cdot \frac{1}{2g} \quad \rightarrow \quad y_1 = 1.05\,m \rightarrow V_1 = 26.63\,\frac{m}{sec} > 15m/sec$$

$$F_{r1} = \frac{26.63}{\sqrt{9.81(1.05)}} = 8.3 > 4.5$$

$$y_2 = \frac{y_1}{2}\left(\sqrt{8F_{r1}^2 + 1} - 1\right) = 11.81\,m$$

$$\frac{L}{y_2} = 4.25 \rightarrow L = 4.25(11.81) = 50.2\,m$$

- The height, width and spacing of the chute blocks as recommended are y_1=0.105m.

- The height of the dentate sill is 0.2 y_2 = 0.2(11.81) = 2.36 m.

-The width and spacing of the dentate =0.15 y_2 = 0.15(11.81) = 1.8 m.

Dams

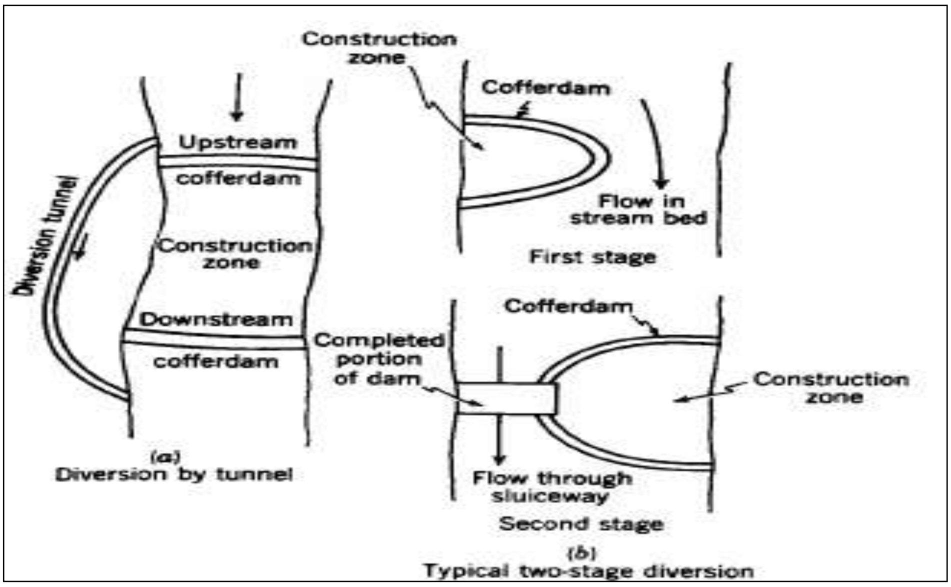
Introduction

A dam is a hydraulic structure of fairly impervious material built across a water way to create a reservoir on its upstream side for impounding water for various purposes. Such as storing water and or controlling flood. Dams are generally constructed in the mountainous reach of the river where the valley is narrow and the foundation is good. Dams are probably the most important hydraulic structure. These are very huge structure and require huge money, manpower and time to construct.

Classification

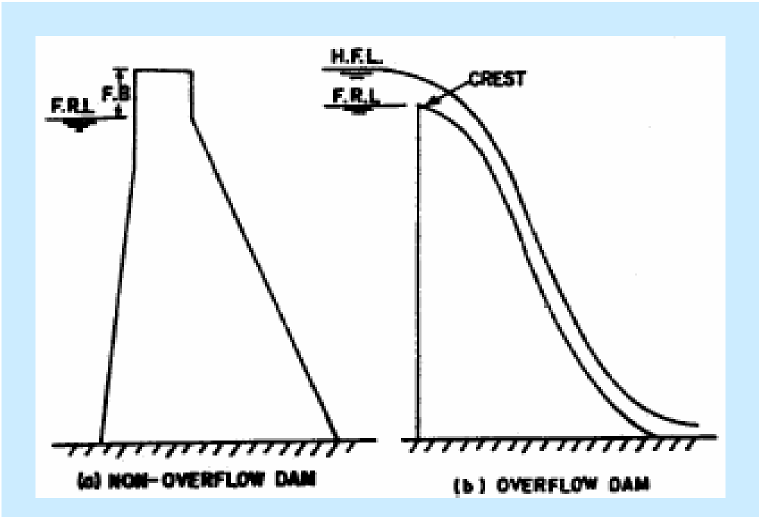
1. Based on Function Served

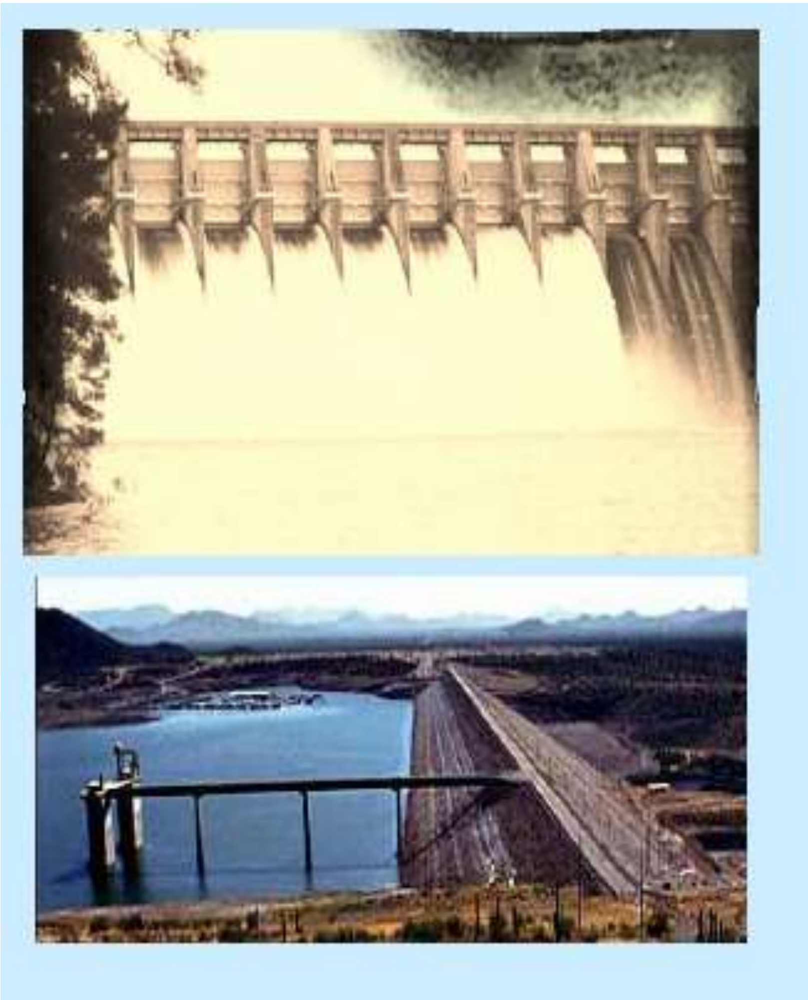
- Storage dams
- Detention dams
- Diversion dams
- Debris dams
- Coffer dams (a temp dam constructed for facilitating construction. It is an enclosure constructed around a site to exclude water so that the construction can be done in dry)



2. Based on Hydraulic Design

- Overflow dams
- Non-overflow dams





3. Based on Rigidity

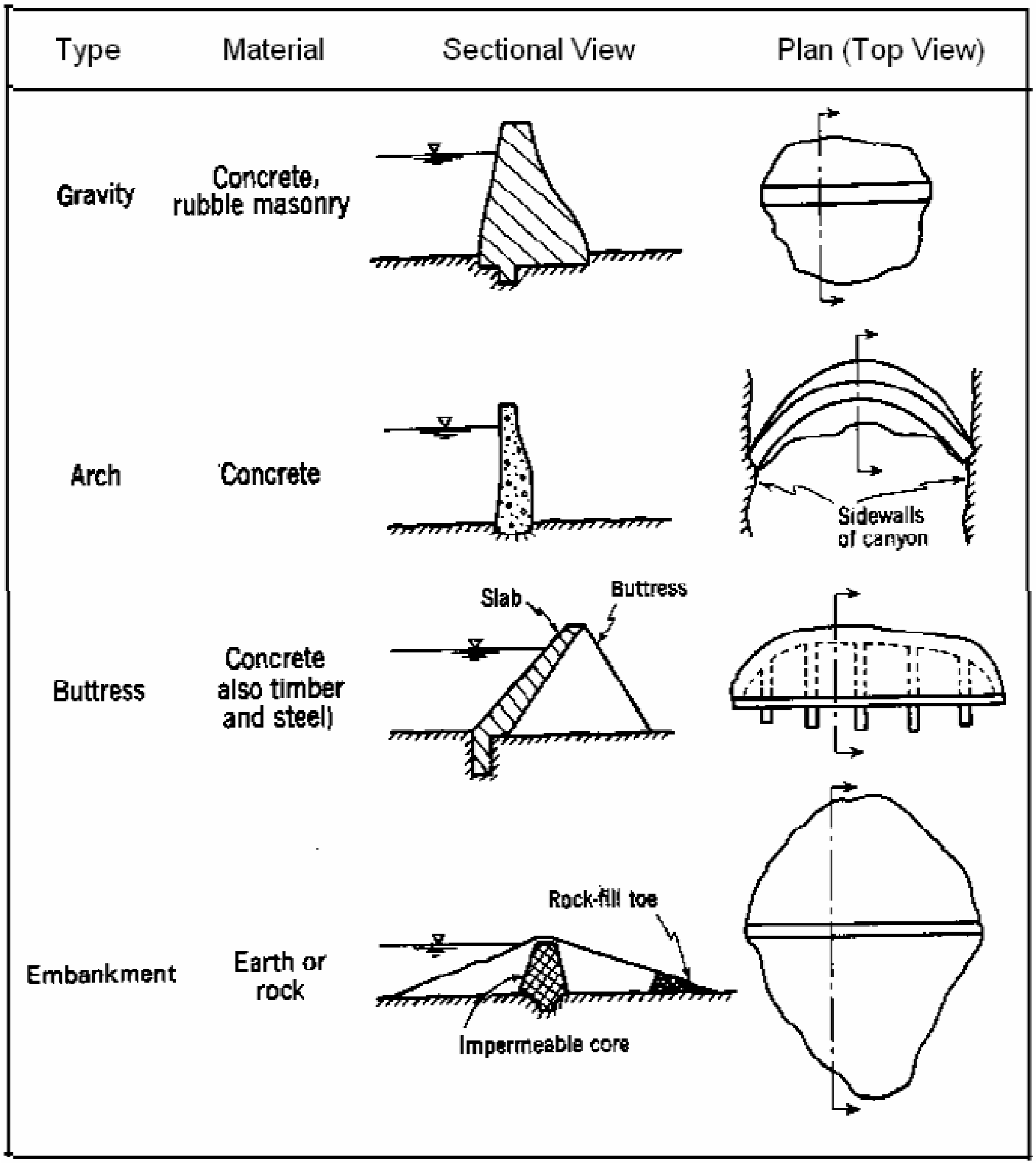
- Rigid dams: A rigid dam is quite stiff. It is constructed of stiff materials such as concrete, masonry, steel and timber. These dams deflect and deform very little when subjected to water pressure and other forces.

- Non-rigid dams: A non-rigid dam is relatively less stiff compared to a rigid dam. The dams constructed of earth and rockfill are non-rigid dams. There are relatively large settlements and deformations in a non-rigid dam.

- Rockfill dams are actually neither fully rigid nor fully nonrigid. These are sometimes classified as semi-rigid dams.

4. Based on structural action

- Gravity dams
- Embankment dams
- Earth dams
- Rockfill dams
- Arch dams
- Buttress dams
- Others
- Steel dams
- Timber dams

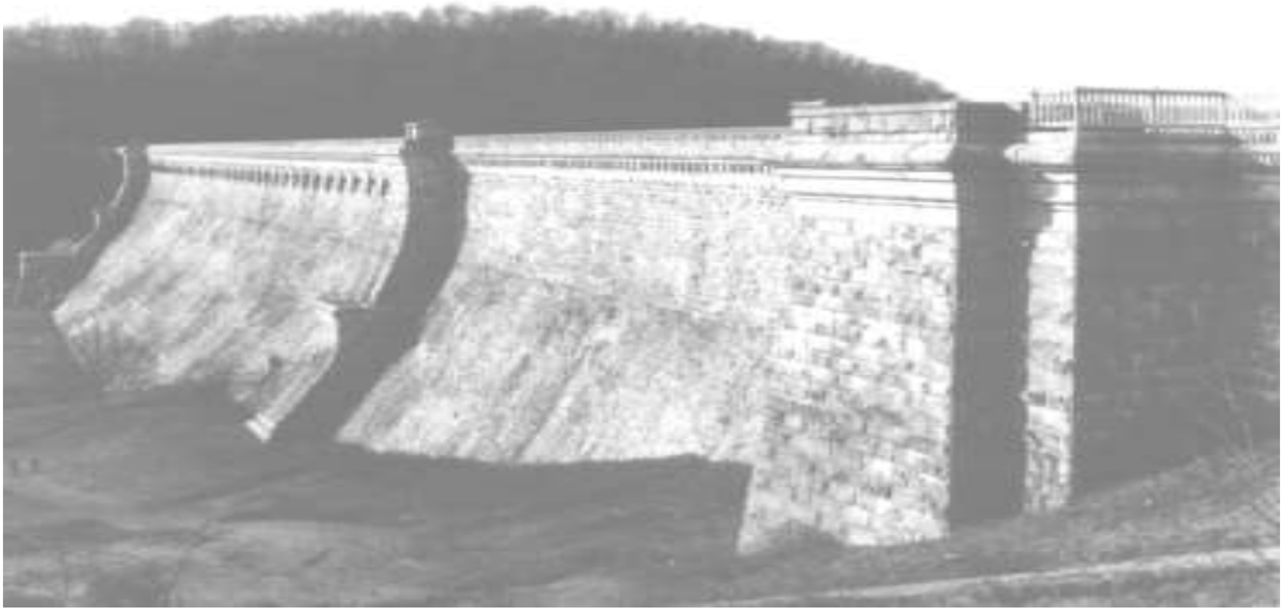


GRAVITY DAMS

A gravity dam resists the water pressure and other forces due to its weight (or gravitational forces). Usually made of cement concrete and straight in plan are approximately triangular in cross-section, with apex at the top. In the past, the gravity dams were made of stone masonry,

* Bhakra dam (structural height of 226 m) was the highest concrete gravity dam of the world when built (surpassing 221 m high Hoover dam in USA). At present, it is second highest after Grand Dixence Dam in Switzerland (284 m high).

*Nagarjuna Sagar Dam (125 m) is highest masonry dam of the world.



Concrete Gravity Dam with Overflow Section

Advantages

1. Gravity dams are quite strong, stable and durable.
2. are quite suitable across moderately wide valleys and gorges having steep slopes where earth dams, if constructed, might slip.
3. Can be constructed to very great heights, provided good rock foundations are available.
4. are well adapted for use as an overflow spillway section. Earth dams cannot be used as an overflow section. Even in earth dams, the overflow section is usually a gravity dam.
5. are specially suited to such areas where there is very heavy downpour. The slopes of the earth dams might be washed away in such an area.

Disadvantages

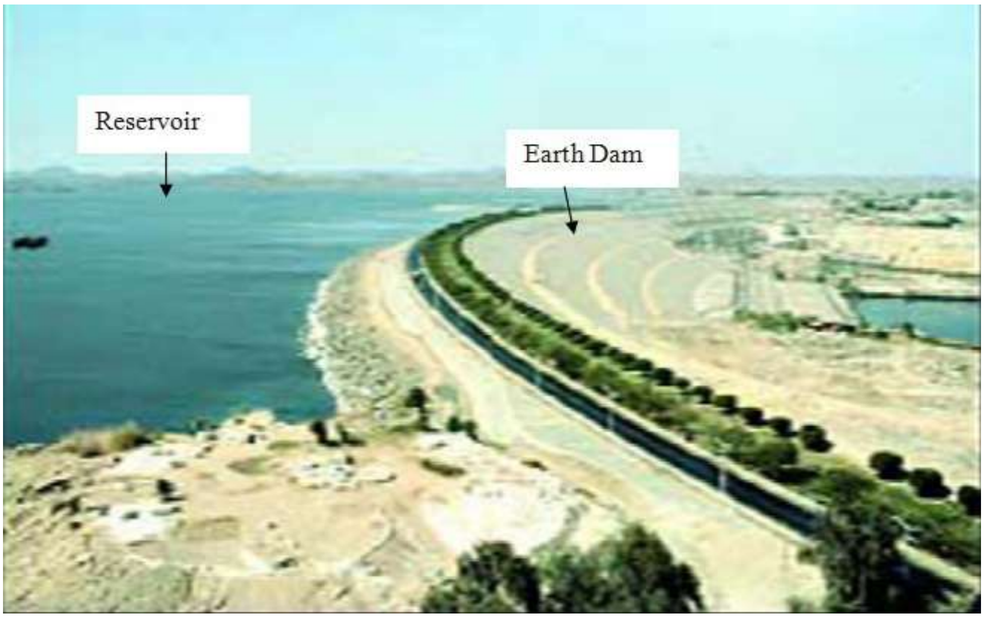
1. Gravity dams of great height can be constructed only on sound rock foundations. These cannot be constructed on weak or permeable foundations on which earth dams can be constructed.
2. Initial cost of a gravity dam is usually more than that of an earth dam. At the sites where good earth is available for construction and funds are limited, earth dams are better.
3. Usually take a longer time in construction than earth dams, especially when mechanized plants for batching, mixing and transporting concrete are not available.
4. Require more skilled labour than that in earth dams.
5. Subsequent raising is not possible in a gravity dam.

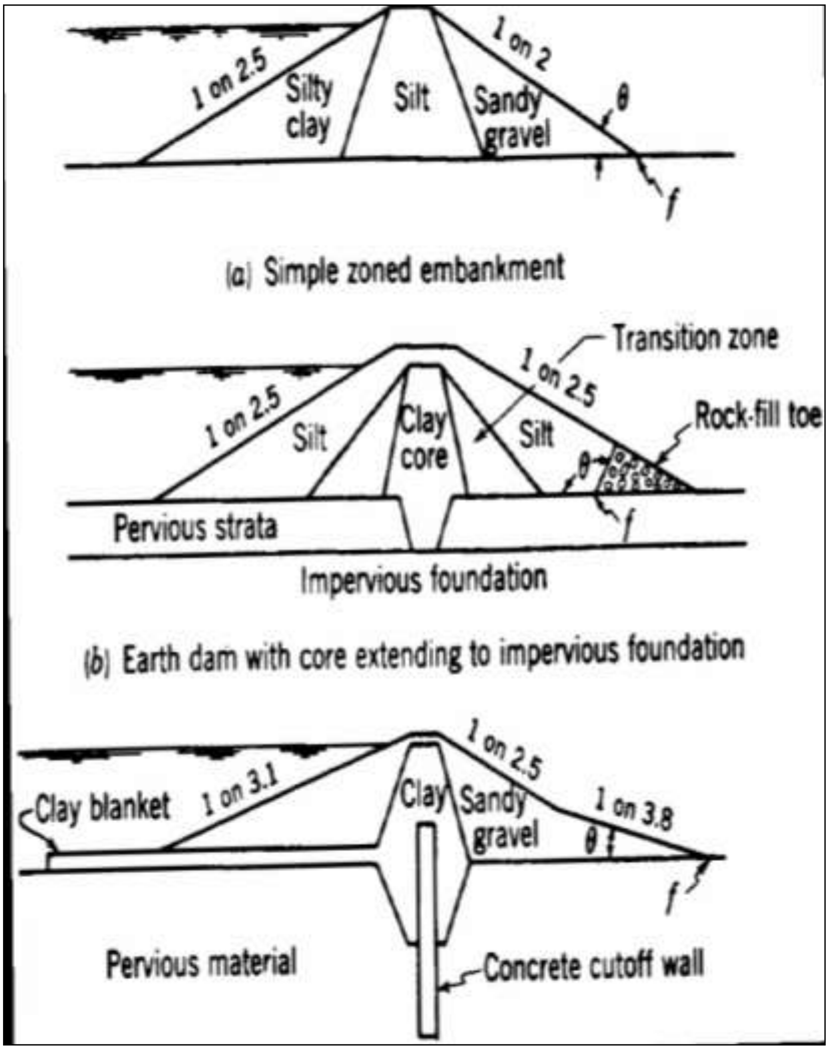
EARTH DAMS

- An earth dam is made of earth (or soil) and resists the forces exerted upon it mainly due to shear strength of the soil.
- are usually built in wide valleys having flat slopes at flanks (abutments).
- can be homogeneous when the height of the dam is not great.
- are of zoned sections, with an impervious zone (called core) in the middle and relatively pervious zones (called shells or shoulders) enclosing the impervious zone on both sides.

Nowadays majority of dams constructed are of this type.

-The highest dams of the world are earth dams (Rongunsky dam Rusia, 325 m and Nurek dam, Rusia, 317 m) as well as the largest capacity dams (New Cornelia dam, USA and Tarbela dam, Pakistan).





Advantages

1. are usually cheaper than gravity dams if suitable earth for construction is available near the site.
2. can be constructed on almost all types of foundations, provided suitable measures of foundation treatment and seepage control are taken.
3. can be constructed in a relatively short period.
4. skilled labor is not required in construction of an earth dam.
5. can be raised subsequently.
6. are aesthetically more pleasing than gravity dams.
7. are more earthquake-resistant than gravity dams.

Disadvantages

1. are not suitable for narrow gorge with steep slopes.
2. Cannot be designed as an overflow section. A spillway has to be located away from the dam.
3. Cannot be constructed in regions with heavy downpour, as the slopes might be washed away.
4. Maintenance cost of an earth dam is quite high. It requires constant supervision.
5. Sluices cannot be provided in a high earth dam to remove slit.
6. Fails suddenly without any sign of imminent failure. A sudden failure causes havoc and untold miseries.

ROCKFILL DAMS

-A rockfill dam is built of rock fragments and boulders of large size.

- An impervious membrane (cement concrete or asphaltic concrete or earth core) is placed on the rockfill on the upstream side to reduce the seepage through the dam.
- A dry rubble cushion is placed between the rockfill and the membrane for the distribution of water load and for providing a support to the membrane.
- Side slopes of rockfill are usually kept equal to the angle of repose of rock (1.4:1 or 1.3:1).

-Rockfill dams are quite economical when a large quantity of rock is easily available near the site.

Mica dam (242 m, Canada), and Chicoasen dam (240 m, Maxico) are highest rockfill dams.

Advantages

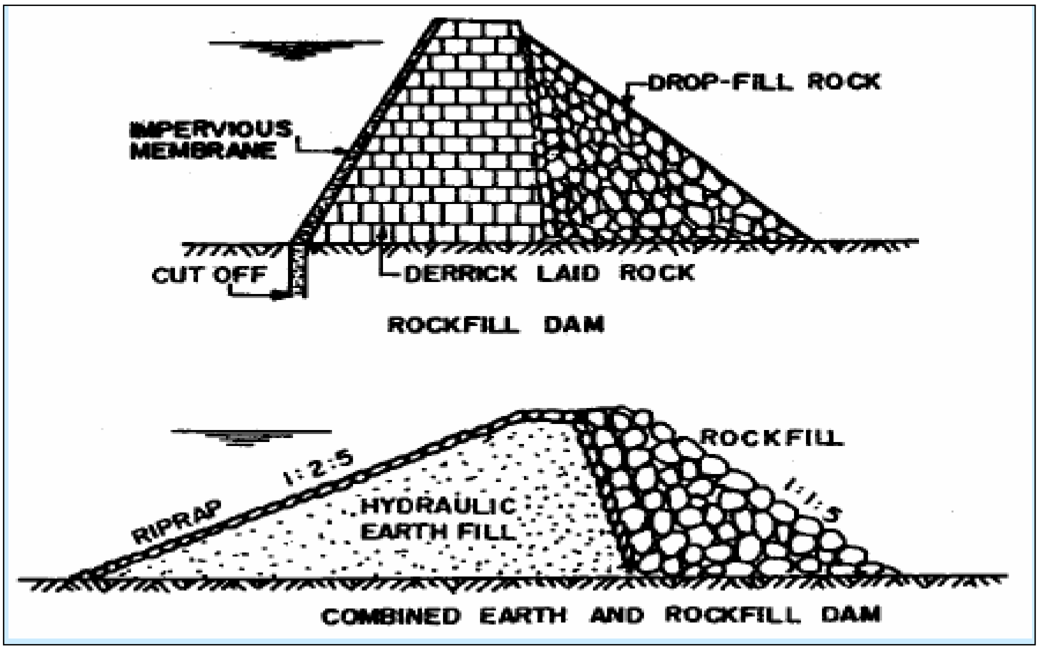
Rockfill dams have almost the same advantages and disadvantages over gravity dams as discussed for earth dams.

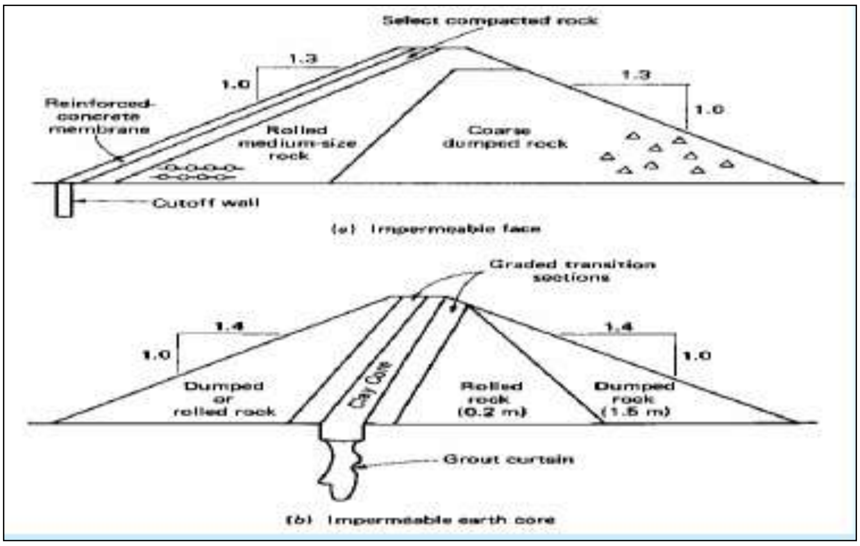
Particular advantages and disadvantages over earth dams.

1. are quite inexpensive if rock fragments are easily available.
2. can be constructed quite rapidly.
3. can better withstand the shocks due to earthquake than earth dams.
4. can be constructed even in adverse climates

Disadvantages

1. Rockfill dams require more strong foundations than earth dams.
2. Rockfill dams require heavy machines for transporting, dumping and compacting rocks.

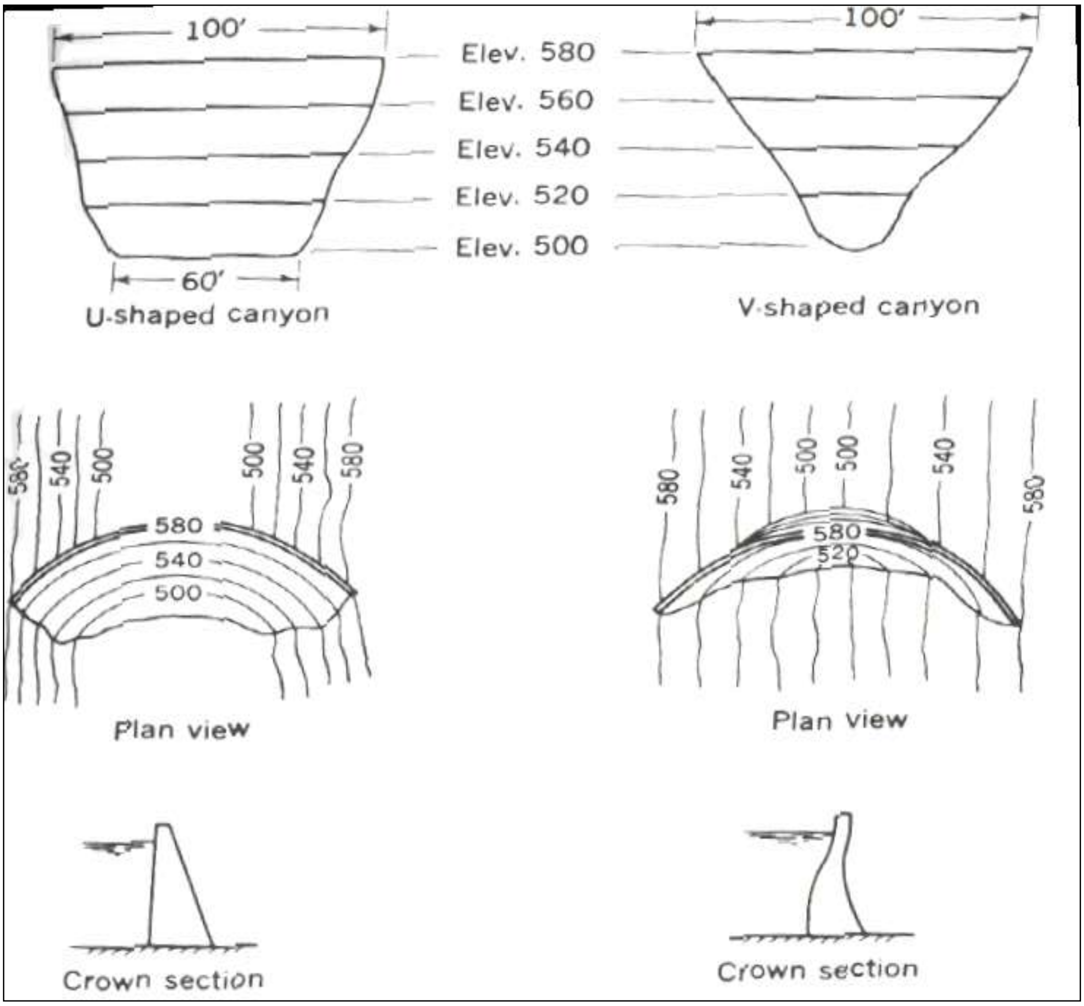




(a) Impermeable face

(b) Impermeable earth core

ARCH DAMS


100′
Elev. 580
Elev. 560
Elev. 540
Elev. 520
Elev. 500
60′
U-shaped canyon
100′
V-shaped canyon
580
540
500
580
540
500
Plan view
580
520
Plan view
Crown section
Crown section

Advantages

1. An arch dam requires less concrete as compared to a gravity dam as the section is thinner.
2. Arch dams are more suited to narrow, V-shaped valley, having very steep slopes.
3. Uplift pressure is not an important factor in the design of an arch dam because the arch dam has less width and the reduction in weight due to uplift does not affect the stability.
4. An arch dam can be constructed on a relatively less strong foundation because a small part of load is transferred to base, whereas in a gravity dam full load is transferred to base.

Disadvantages

1. An arch dam requires good rock in the flanks (abutments) to resist the thrust. If the abutments yield, extra stresses develop which may cause failure.
2. The arch dam requires sophisticated formwork, more skilled labour and richer concrete.
3. The arch dam cannot be constructed in very cold climates because spalling of concrete occurs due to alternate freezing and thawing.
4. The arch dams are more prone to sabotage.
5. The speed of construction is relatively slow.

BUTTRESS DAMS

-Buttress dams are of three types: (i) Deck type, (ii) Multiple arch-type, and (iii) Massive-head type.
- A deck type buttress dam consists of a sloping deck supported by buttresses.
-Buttresses are triangular concrete walls which transmit the water pressure from the deck slab to the foundation.
-Buttresses are compression members.

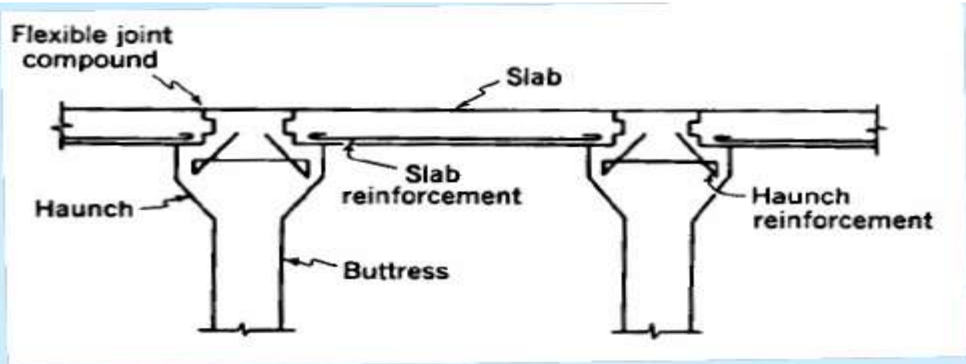
-The deck is usually a reinforced concrete slab supported between the buttresses, which are usually equally spaced.

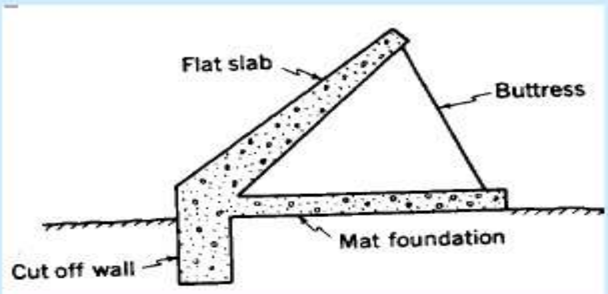
-In a multiple-arch type buttress dam the deck slab is replaced by horizontal arches supported by buttresses. The arches are usually of small span and made of concrete.

-In a massive-head type buttress dam, there is no deck slab.

Instead of the deck, the upstream edges of the buttresses are flared to form massive heads which span the distance between the buttresses.

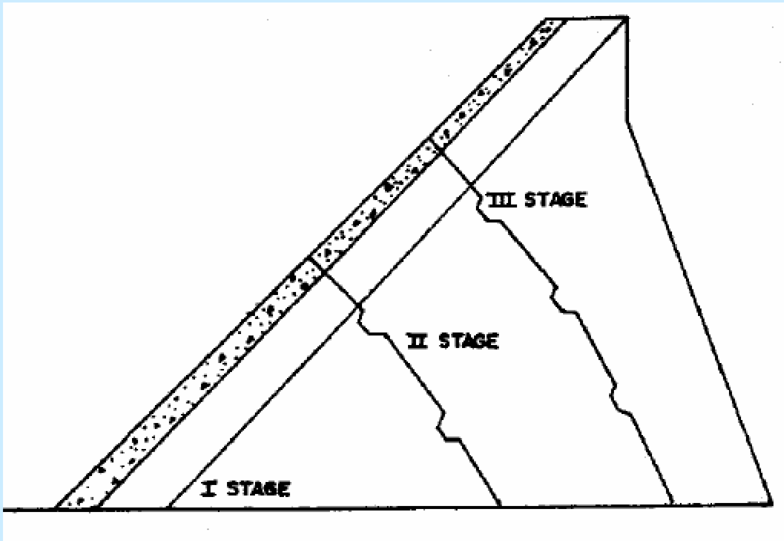
Buttress Dam (d/s side)





Advantages

1. Buttress dams require less concrete than gravity dams.
2. Uplift/ice pressure is generally not a major factor
3. Can be constructed on relatively weaker foundations.
4. Power house and water treatment plants, etc. can be housed between buttresses.
5. Vertical component of the water pressure on deck prevents the dam against overturning and sliding failures.
6. Can be designed to accommodate moderate movements of foundations without serious damages.
7. Heat dissipation is better in buttress dams.
8. Back of the deck and the foundation between buttresses are accessible for inspection.
9. Can be easily raised subsequently by extending buttresses and deck slabs.



RAISING HEIGHT OF BUTTRESS DAM

Disadvantages

1. Buttress dams require costlier formwork, reinforcement and more skilled labor. Consequently, the overall cost of construction may be more than that of a gravity dam.
2. Buttress dams are more susceptible to damage and sabotage.

3. Buttress dams cannot be constructed in very cold climates because of spalling of concrete.
4. Because the upstream deck slab is thin, its deterioration may have very serious effect on the stability.

SITE SELECTION FOR A DAM

- A dam is a huge structure requiring a lot of funds.
-Extreme care shall be taken while selecting the site of a dam.
-A wrong decision may lead to excessive cost and difficulties in construction and maintenance.
-Various factors should be considered when selecting the site of a dam.

1. Topography
2. Suitable Foundation
3. Good Site for reservoir –
   (i) Large storage capacity
   (ii) Shape of reservoir basin
   (iii) Watertightness of the reservoir
   (iv) Good hydrological conditions
   (v) Deep reservoir
   (vi) Small submerged area
   (vii) Low silt inflow
   (viii) No objectionable minerals
5. Spillway site
6. Availability of materials
7. Accessibility
8. Healthy surroundings
9. Minimum overall cost
10. Other considerations

SELECTION OF TYPE OF DAM

- Selection of the most suitable type of dam for a particular site requires a lot of judgment and experience.
- It is only in exceptional cases that the most suitable type is obvious.
- Preliminary designs and estimates are usually required for several types of dams before making the final selection on economic basis.

-The salient features of different types of dams discussed in the preceding sections should be kept in mind while selecting the type of dam.

- Various factors govern the selection of type of dam

1. Topography and valley shape
2. Geology and foundation conditions
3. Availability of construction materials
4. Overall cost
5. Spillway size and location
6. Earthquake hazards
7. Climatic conditions
8. Diversion problems
9. Environmental considerations
10. Roadway
11. Length and height of dam
12. Life of dam
13. Miscellaneous considerations