Hydraulic Structures-15CV832

Module -1- Gravity Dams

A **gravity dam** is a dam constructed from concrete or stone masonry and designed to hold back water by primarily utilizing the weight of the material alone to resist the horizontal pressure of water pushing against it. Gravity dams are designed so that each section of the dam is stable, independent of any other dam section

FORCES ACTING ON GRAVITY DAM:

In the design of a dam, the first step is the determination of various forces which acts on the structure and study their nature. Depending upon the situation, the dam is subjected to the following forces:

1. Water pressure
2. Earthquake forces
3. Silt pressure
4. Wave pressure
5. Ice pressure
6. Self weight of the dam.

The forces are considered to act per unit length of the dam.

For perfect and most accurate design, the effect of all the forces should be investigated. Out of these forces, most common and important forces are water pressure and self weight of the dam.

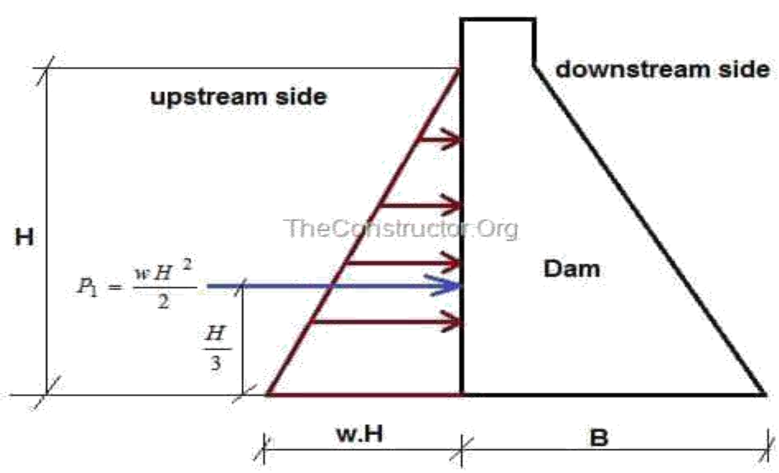
1. Water Pressure

Water pressure may be subdivided into the following two categories:

I) External water pressure:

It is the pressure of water on the upstream face of the dam. In this, there are two cases:

(I) Upstream face of the dam is vertical and there is no water on the downstream side of the dam (figure 1).



The total pressure is in horizontal direction and acts on the upstream face at a height $\frac{H}{3}$ from the

$$P_1 = \frac{wH^2}{2}$$

bottom. The pressure diagram is triangular and the total pressure is given by
Where w is the specific weight of water. Usually it is taken as unity.
H is the height upto which water is stored in m.
(ii) Upstream face with batter and there is no water on the downstream side (figure 2).

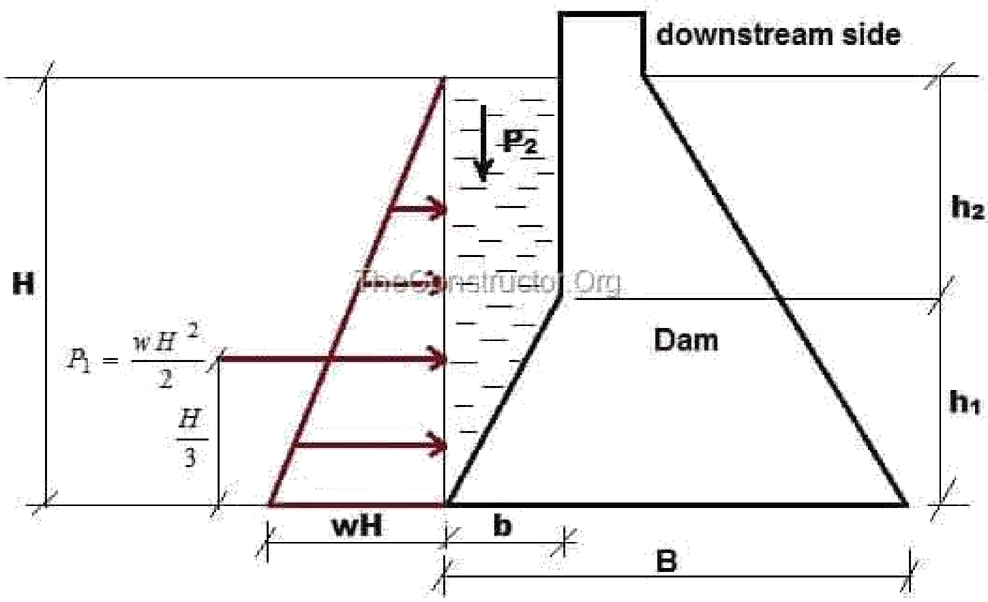


Figure 2

Here in addition to the horizontal water pressure P_1 as in the previous case, there is vertical pressure of the water. It is due to the water column resting on the upstream sloping side.

The vertical pressure P_2 acts on the length 'b' portion of the base. This vertical pressure is given by

$$P_2 = (b \times h_2 \times w) + \left(\frac{1}{2} b \times h_1 \times w\right)$$

Pressure P_2 acts through the centre of gravity of the water column resting on the sloping upstream face.

If there is water standing on the downstream side of the dam, pressure may be calculated similarly. The water pressure on the downstream face actually stabilizes the dam. Hence as an additional factor of safety, it may be neglected.

II) Water pressure below the base of the dam or Uplift pressure

When the water is stored on the upstream side of a dam there exists a head of water equal to the height upto which the water is stored. This water enters the pores and fissures of the foundation material under pressure. It also enters the joint between the dam and the foundation at the base and the pores of the dam itself. This water then seeps through and tries to emerge out on the downstream end. The seeping water creates hydraulic gradient between the upstream and downstream side of the dam. This hydraulic gradient causes vertical upward pressure. The upward pressure is known as uplift. Uplift reduces the effective weight of the structure and consequently the restoring force is reduced. It is essential to study the nature of uplift and also some methods will have to be devised to reduce the uplift pressure value.

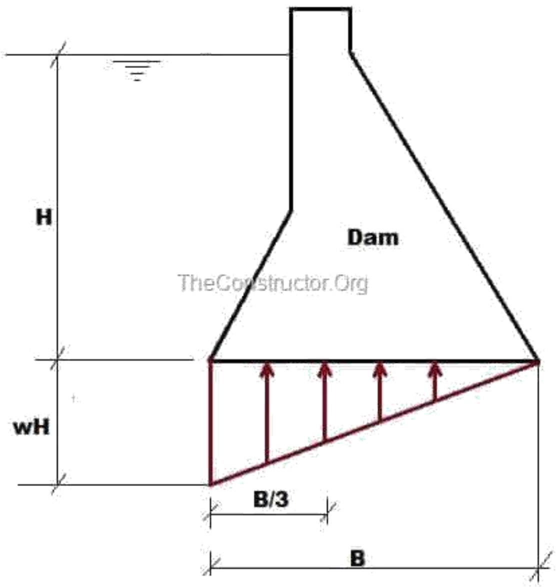


Figure 3

$$P_u = \frac{wH \times B}{2}$$

With reference to figure 3, uplift pressure is given by

Where P_u is the uplift pressure, B is the base width of the dam and H is the height upto which water is stored.

This total uplift acts at $\frac{B}{3}$ from the heel or upstream end of the dam.

Uplift is generally reduced by providing drainage pipes or holes in the dam section.

Self weight of the dam is the only largest force which stabilizes the structure. The total weight of the dam is supposed to act through the centre of gravity of the dam section in vertically downward direction. Naturally when specific weight of the material of construction is high, restoring force will be more. Construction material is so chosen that the density of the material is about 2.045 gram per cubic meter.

2. Earthquake Forces

The effect of earthquake is equivalent to acceleration to the foundation of the dam in the direction in which the wave is travelling at the moment. Earthquake wave may move in any direction and for design purposes, it is resolved into the vertical and horizontal directions. On an average, a value of 0.1 to 0.15g (where g = acceleration due to gravity) is generally sufficient for high dams in seismic zones. In extremely seismic regions and in conservative designs, even a value of 0.3g may sometimes by adopted.

Vertical acceleration reduces the unit weight of the dam material and that of water is to $(1 - k_v)$ times the original unit weight, where k_v the value of g accounted against earthquake forces, i.e. 0.1 is when 0.1g is accounted for earthquake forces. The horizontal acceleration acting towards the reservoir causes a momentary increase in water pressure and the foundation and dam accelerate towards the reservoir and the water resists the movement owing to its inertia. The extra pressure exerted by this process is known as hydrodynamic pressure.

3. Silt Pressure

If h is the height of silt deposited, then the forces exerted by this silt in addition to the external water pressure, can be represented by Rankine formula

$P_{\text{silt}} = \frac{1}{2}\gamma_s h^2 k_a$ acting at $\frac{h}{3}$ from the base.

Where,

k_a = coefficient of active earth pressure of silt = $\frac{1 - \sin\phi}{1 + \sin\phi}$ ϕ = angle of internal friction of soil, cohesion neglected.

γ_s = submerged unit weight of silt material.

h = height of silt deposited.

4. Wave Pressure

Waves are generated on the surface of the reservoir by the blowing winds, which exert a pressure on the downstream side. Wave pressure depends upon wave height which is given by the equation

$h_w = 0.032\sqrt{PV} + 0.763 - 0.271 \times (F)^{1/4}$ for F < 32 km, and

$h_w = 0.032\sqrt{VF}$ for F > 32 km

Where h_w is the height of water from the top of crest to bottom of trough in meters.

V – wind velocity in km/hour

F – fetch or straight length of water expanse in km.
The maximum pressure intensity due to wave action may be given by

$P_w = 2.4\gamma_w h_w$ and acts at $\frac{h_w}{2}$ meters above the still water surface.

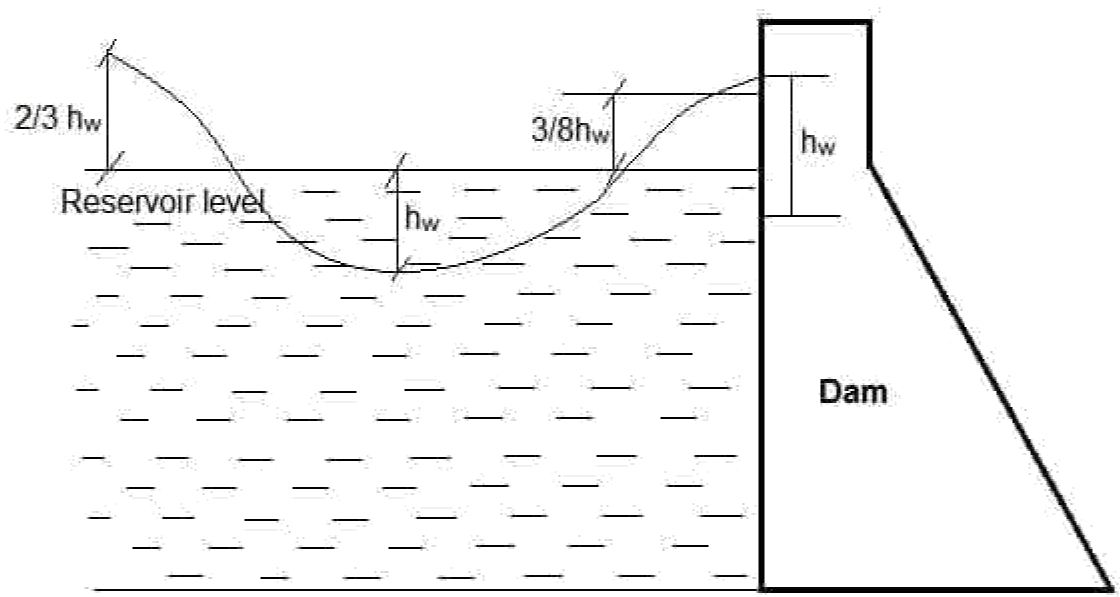


Figure 4

The pressure distribution may be assumed to be triangular of height $\frac{5h_w}{3}$ as shown in figure 4. Hence total force due to wave action P_w

= $\frac{1}{2} \times (2.4\gamma_w h_w) \times \frac{5}{3} h_w$ acting at $\frac{3}{8} h_w$ above the reservoir surface.

5. Ice Pressure

The ice which may be formed on the water surface of the reservoir in cold countries may sometimes melt and expand. The dam face is subjected to the thrust and exerted by the expanding ice. This force acts linearly along the length of the dam and at the reservoir level. The magnitude of this force varies from 250 to 1500 kN/sq.m depending upon the temperature variations. On an average, a value of 500 kN/sq.m may be taken under ordinary circumstances.

6. Weight of dam

The weight of dam and its foundation is a major resisting force. In two dimensional analysis of dam

FAILURES OF GRAVITY DAM

Failure of gravity dam occurs due to overturning, sliding, tension and compression. A gravity dam is designed in such a way that it resists all external forces acting on the dam like water pressure, wind pressure, wave pressure, ice pressure, uplift pressure by its own self-weight. Gravity dams are constructed from masonry or concrete. However, concrete gravity dams are preferred these days and mostly constructed. The advantage of gravity dam is that its structure is most durable and solid and requires very less maintenance.

Causes of failure of a Gravity Dam:

A gravity dam may fail in following modes:

1. Overturning of dam about the toe
2. Sliding – shear failure of gravity dam
3. Compression – by crushing of the gravity dam
4. Tension – by development of tensile forces which results in the crack in gravity dam.

Overturning Failure of Gravity Dam:

The horizontal forces such as water pressure, wave pressure, silt pressure which act against the gravity dam causes overturning moments. To resist this, resisting moments are generated by the self-weight of the dam.

If the resultant of all the forces acting on a dam at any of its sections, passes through toe, the dam will rotate and overturn about the toe. This is called overturning failure of gravity dam. But, practically, such a condition does not arise and dam will fail much earlier by compression.

The ratio of the resisting moments about toe to the overturning moments about toe is called the factor of safety against overturning. Its value generally varies between 2 and 3.

Factor of safety against overturning is given by

FOS = sum of overturning moments/ sum of resisting moments

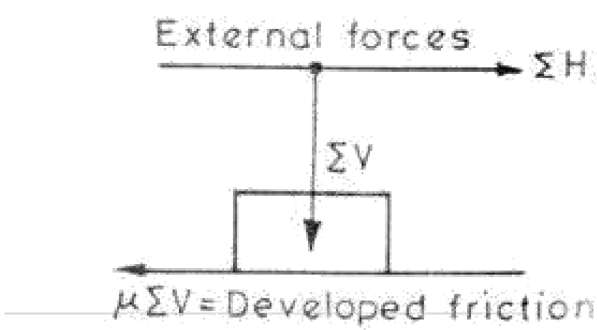
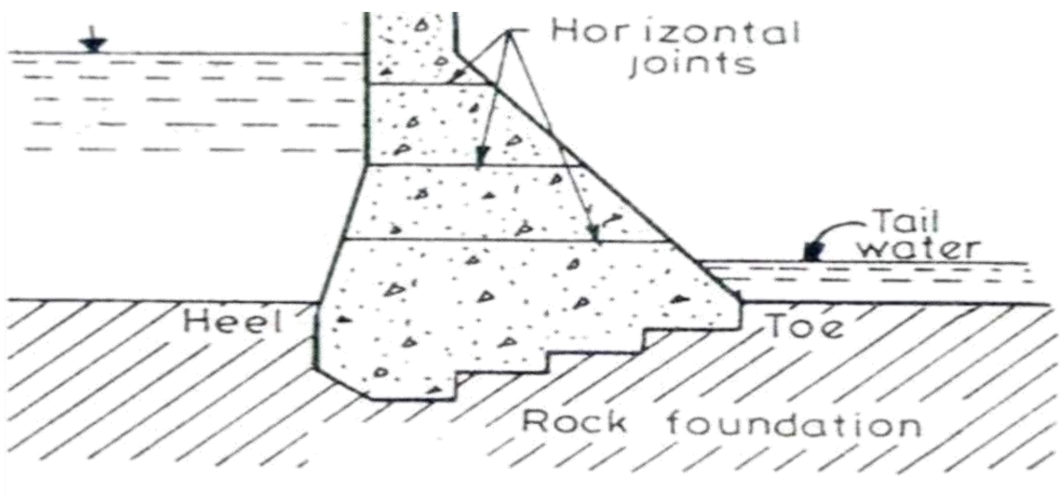


Fig: sum of external horizontal forces greater than vertical self-weight of dam (overacting, sliding occurs)

Sliding Failure of Gravity Dam: When the net horizontal forces acting on gravity dam at the base exceeds the frictional resistance (produced between body of the dam and foundation), The failure occurs is known as sliding failure of gravity dam.
In low dams, the safety against sliding should be checked only for friction, but in high dams, for economical precise design, the shear strength of the joint is also considered
Factor of safety against sliding can be given based on Frictional resistance and shear strength of the dam
Factor of safety based on frictional resistance:

$$\text{FOS against sliding} = \text{FOS} = \frac{\mu \sum V}{\sum H}$$

μ=co-efficient of friction between two surfaces
$\sum V$ = sum of vertical forces acting on dam
$\sum H$ = sum of vertical forces acting on dam



Gravity Dam Failure due to Tension Cracks: Masonry and concrete are weak in tension. Thus masonry and concrete gravity dams are usually designed in such a way that no tension is developed anywhere. If these dams are subjected to tensile stresses, materials may develop tension cracks. Thus the dam loses contact with the bottom foundation due to this crack and becomes ineffective and fails. Hence, the effective width B of the dam base will be reduced. This will increase pmax at the toe. Hence, a tension crack by itself does not fail the structure, but it leads to the failure of the structure by producing excessive compressive stresses.
For high gravity dams, certain amount of tension is permitted under severest loading conditions in order to achieve economy in design. This is permitted because the worst condition of loads may occur only momentarily and may not occur frequently.

Gravity Dam Failure due to Compression: A gravity dam may fail by the failure of its material, i.e. the compressive stresses produced may exceed the allowable stresses, and the dam material may get crushed.

Design principles

Principal and shear stresses

Stability analysis of gravity dams

The stability analysis of gravity dams may be carried out by various methods. In gravity method, the dam is considered to be made up of a number of vertical cantilevers which act independently for each other. The resultant of all horizontal and vertical forces including uplift should be balanced by an equal and opposite reaction at the foundation consisting of the total vertical reaction and the total horizontal shear and friction at the base and the resisting shear and friction of the passive wedge, if any. For the dam to be in static equilibrium, the location of this force is such that the summation of moments is equal to zero. The distribution of the vertical reaction is assumed as trapezoidal. Otherwise, the problem of determining the actual stress distribution at the base of a dam is complicated by the horizontal reaction, internal stress relations, and other theoretical considerations. Moreover, variation of foundation materials with depth, cracks and fissures which affect the resistance of the foundation also make the problem more complex. The internal stresses and foundation pressures should be computed both with and without uplift to determine the worst condition.

The stability analysis of a dam section is carried out to check the safety with regard to

1. Rotation and overturning
2. Translation and sliding
3. Overstress and material failure

Stability against overturning

Before a gravity dam can overturn physically, there may be other types of failures, such as cracking of the upstream material due to tension, increase in uplift, crushing of the toe material and sliding. However, the check against overturning is made to be sure that the total stabilizing moments weigh out the de-stabilizing moments. The factor of safety against overturning may be taken as 1.5. As such, a gravity dam is considered safe also from the point of view of overturning if there is no tension on the upstream face.

Stability against sliding

Many of the loads on the dam act horizontally, like water pressure, horizontal earthquake forces, etc. These forces have to be resisted by frictional or shearing forces along horizontal or nearly-horizontal seams in foundation. The stability of a dam against sliding is evaluated by comparing the minimum total available resistance along the critical path of sliding (that is, along that plane or combination of plans which mobilizes the least resistance to sliding) to the total magnitude of the forces tending to induce sliding.

Sliding resistance is also a function of the cohesion inherent in the materials at their contact and the angle of internal friction of the material at the surface of sliding. The junction plane between the dam and rock is rarely smooth. In fact, special efforts are made during construction to keep the interface as rough as possible. There may, however be some lower plane in the foundation where sliding is resisted by friction alone especially if the rock is markedly stratified and horizontally bedded. The factor of safety against sliding (F) along a plane may be computed from

$$F = \frac{\frac{Net\ shear\ force\ along\ the\ plane}{F_{\varphi}} + \frac{Net\ cohesive\ force\ along\ the\ plane}{F_{c}}}{Net\ horizontal\ destabilizing\ force\ along\ the\ plane}$$

Where,

F_{φ} is the Partial Factor of Safety of friction

F_{c} is the Partial Factor of Safety of cohesion.

Failure against overstressing

A dam may fail if any of its part is overstressed and hence the stresses in any part of the dam must not exceed the allowable working stress of concrete. In order to ensure the safety of a concrete gravity dam against this sort of failure, the strength of concrete shall be such that it is more than the stresses anticipated in the structure by a safe margin. The maximum compressive stresses occur at heel (mostly during reservoir empty condition) or at toe (at reservoir full condition) and on planes normal to the face of the dam.

The calculation of the stresses in the body of a gravity dam follows from the basics of elastic theory, which is applied in a two-dimensional vertical plane, and assuming the block of the dam to be a cantilever in the vertical plane attached to the foundation. Although in such an analysis, it is assumed that the vertical stresses on horizontal planes vary uniformly and horizontal shear stresses vary parabolically, they are not strictly correct. Stress concentrations develop near heel and toe, and modest tensile stresses may develop at heel. The basic stresses that are required to be determined in a gravity dam analysis are discussed below:

Normal stresses on horizontal planes

On any horizontal plane, the vertical normal stress (σ_z) may be determined as:

$$\sigma_z = \frac{\sum V}{T} \pm \frac{12 \sum V_e}{T^3} y$$

Where,

$\sum V$ is the resultant vertical load above the plane considered

T is the thickness of the dam block i.e., the length measured from heel to toe

E is the eccentricity of the resultant load

Y is the distance from the neutral axis of the plane to the point where σ_z is being determined

At the heel, y= -T/2 and at the toe, y=+T/2. Thus, at these points, the normal stresses are found out as:

$$\sigma_{z_{heel}} = \frac{\sum V}{T}(1 - \frac{6e}{T})$$

$$\sigma_{z_{toe}} = \frac{\sum V}{T}(1 + \frac{6e}{T})$$

The eccentricity e may be found out as:

$$e = \frac{Net\ moment}{Net\ vertical\ force}$$

Naturally, there would be tension on the upstream face if the overturning moments under the reservoir full condition increase such that e becomes greater than T/6. The total vertical stresses at the upstream and downstream faces are obtained by addition of external hydrostatic pressure.

Shear stresses on horizontal planes

Horizontal stresses (τ_{zy}) and the shear stresses (τ_{yz}) are developed at any point as a result of the variation in vertical normal stress over a horizontal plane. The following relation can be derived relating the stresses with the distance y measured from the centroid

$$\tau_{zy} = \tau_{yz} = \tau_{yzD} - \frac{2}{T}\left[\frac{3H}{T} + \tau_{yzU} + 2\tau_{yzD}\right]y + \frac{3}{T^2}\left[\frac{2H}{T}\tau_{yzD} + \tau_{yzU}\right]y^2$$

Where,

$\tau_{yzD} = (\sigma_{zD} - p_D)\tan\phi_{D;}$ is the shear stress at downstream face

$\tau_{yzU} = -(\sigma_{zU} - p_U)\tan\phi_{D;}$ is the shear stress at upstream face

H is the height of the dam

The shear stress is seen to vary parabolically from τ_{yzU} at the upstream face up to τ_{yzD} at the downstream face.

Principal and shear stresses on vertical planes

The vertical stress intensity, P_{max} or P_{min} is determined using the vertical direct stress distribution at base

$$P_{\frac{min}{max}} = \frac{\sum V}{B}(1 \pm \frac{6e}{B})$$

It is not the maximum direct stress produced anywhere in the dam. The maximum normal stress will, indeed, be the major principal stress that will be generated on the major principal plane. The principal (σ) and shear (τ) stresses at the toe and heel of gravity dam can be expressed by

$$\sigma_{toe} = p_v \sec^2\alpha - (p' - p_e')\tan^2\delta$$

$$\sigma_{heel} = p_v \sec^2\varphi - (p + p_e)\tan^2\theta$$

$$\tau_{toe} = [p_v - (p' - p_e')]\tan\delta$$

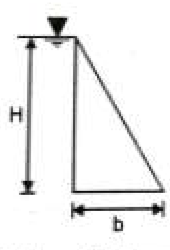
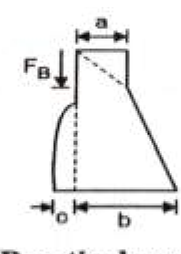
$$\tau_{heel} = [p_v - (p + p_e)]\tan\theta$$

Where,

δ is the angle which the downstream face of the dam makes with the vertical,
θ is the angle which the upstream face makes with the vertical,
p_v is the intensity of uplift pressure,
p is the minor principal stress at the heel
p_e is the hydrodynamic pressure exerted by the head water
p_e' is the hydrodynamic pressure exerted by the tail water during an earthquake

In a gravity dam, stability is secured by making it of such a size and shape that it will resist overturning, sliding and crushing at the toe. The dam will not overturn provided that the moment around the turning point, caused by the water pressure is smaller than the moment caused by the weight of the dam. This is the case if the resultant force of water pressure and weight falls within the base of the dam. However, in order to prevent tensile stress at the upstream face and excessive compressive stress at the downstream face, the dam cross section is usually designed so that the resultant falls within the middle at all elevations of the cross section (the core). For this type of dam, impervious foundations with high bearing strength are essential.

Elementary profile and practical profile of a gravity dam

Theoretical profile	**Practical Profile**
Provision of free board is not provided.	Provision of free board is provided.
Road way at top is not possible.	Road way at top is possible.
For reservoir empty condition it will provide maximum possible stability.	For reservoir empty condition tension is developed at toe and hence some masonry is provided on u/s side.
 Theoretical profile	 Practical profile

Drainage Galleries in Gravity Dams

Galleries are the horizontal or sloping openings or passages left in the body of the dam. • They may run longitudinally (i.e. parallel to dam axis) or traversely (i.e. normal to the dam axis) and are provided at various elevations. All the galleries are interconnected by steeply sloping passages or by vertical shafts fitted with stairs or mechanical lifts.

Function and types of galleries in Dams

(i) Foundation Gallery

A gallery provided in a dam may serve one particular purpose or more than one purpose. For example, a gallery provided near the rock foundation, serves to drain off the water which percolates through the foundations. This gallery is called a foundation gallery or a drainage gallery.

1. It runs longitudinally and is quite near to the upstream face of the dam. Drain holes are drilled from the floors of this gallery after the foundation grouting has been completed. Seepages is collected through these drain holes.
2. Besides draining off seepage water, it may be helpful for drilling and grouting of the foundations, when this can not be done from the surface of the dam.

(ii) Inspection Galleries

The water which seeps through the body of the dam is collected by means of a system of galleries provided at various elevations and interconnected by vertical shafts, etc. All these galleries, besides draining off seepage water, serves inspection purpose. They provide access to the interior of the dam and are, therefore, called inspection purposes. They generally serve other purposes along with this purpose.

1. They intercept and drain off the water seeping through the dam body
2. They provide access to dam interior for observing and controlling the behavior of the dam.
3. They provide enough space for carrying pipes, etc. during artificial cooling of concrete
4. They provide access to all the outlets and spillway gates, valves, etc. by housing their electrical and mechanical controls. All these gates, valves, etc, can hence be easily controlled by men, from inside the dam itself.
5. They provide space for drilling and grouting of the foundations, then it cannot be done from the surface of the dam.

MODULE -2- EARTH DAMS

Introduction

An **embankment dam** is a large artificial dam. It is typically created by the placement and compaction of a complex semi-plastic mound of various compositions of soil, sand, clay, or rock. It has a semi-pervious waterproof natural covering for its surface and a dense, impervious core. This makes such a dam impervious to surface or seepage erosion. Such a dam is composed of fragmented independent material particles. The friction and interaction of particles binds the particles together into a stable mass rather than by the use of a cementing substance.

Embankment dams come in two types: the **earth-filled dam** (also called an earthen dam or terrain dam) made of compacted earth, and the **rock-filled dam**. A cross-section of an embankment dam shows a shape like a bank, or hill. Most have a central section or core composed of an impermeable material to stop water from seeping through the dam. The core can be of clay, concrete, or asphalt concrete. This dam type is a good choice for sites with wide valleys. They can be built on hard rock or softer soils. For a rock-fill dam, rock-fill is blasted using explosives to break the rock. Additionally, the rock pieces may need to be crushed into smaller grades to get the right range of size for use in an embankment dam.
The building of a dam and the filling of the reservoir behind it places a new weight on the floor and sides of a valley. The stress of the water increases linearly with its depth. Water also pushes against the upstream face of the dam, a nonrigid structure that under stress behaves semiplastically, and causes greater need for adjustment (flexibility) near the base of the dam than at shallower water levels. Thus the stress level of the dam must be calculated in advance of building to ensure that its break level threshold is not exceeded.

Causes of failure of earth dams

Stability and Failure of Earth Filled Dams
Failure of earth dams may be:

1. Hydraulic Failure
2. Seepage Failure
3. Structural Failure

1. Hydraulic Failure:

1. Overtopping of dams
2. Erosion of the Upstream Surface
3. Erosion of the Downstream Surface
4. Erosion of the Downstream toe

i. Overtopping of dams:

This type of dam is made up of only one type of material. Usually porous materials is used. These dams are easy and cheap to construct but cannot be used to make multipurpose large dams. For large multipurpose dams zoned type method is used. Over topping failures result from the erosive action of water on the embankment. Erosion is due to un-controlled flow of water over, around, and adjacent to the dam. Earth embankments are not designed to be over-topped and therefore are particularly susceptible to erosion. Once erosion has begun during over-topping, it is almost impossible to stop. A well vegetated earth embankment may withstand limited over topping if its crest is level and water flows over the crest and down the face as an evenly distributed sheet without becoming concentrated. The owner should closely monitor the reservoir pool level during severe storms.

ii. Erosion of the Upstream Surface:

Here zones of different materials are made.
Shell is used to give support and stability to the structure of dam. It is made of coarse materials and is pervious in nature.
Core is used to make the dam water tight and to reduce the seepage. Fine material is used here.
Used in large dams.

iii. Erosion of the Downstream Surface:

Due to rainfall, snow and winds the downstream surface of the dam also erodes. By providing a section of coarse materials here, this erosion can be reduced or prevented.

2. Seepage Failure:

All earth dams have seepage resulting from water permeating slowly through the dam and its foundation. Seepage must be controlled in both velocity and quantity. If uncontrolled, it can progressively erode soil from the embankment or its foundation, resulting in rapid failure of the dam. Erosion of the soil begins at the downstream side of the embankment, either in the dam proper or the foundation, progressively works toward the reservoir, and eventually develops a direct connection to the reservoir. This phenomenon is known as "piping." Piping action can be recognized by an increased seepage flow rate, the discharge of muddy or discolored water, sinkholes on or near the embankment, or a whirlpool in the reservoir. Once a whirlpool (eddy) is observed on the reservoir surface, complete failure of the dam will probably follow in a matter of minutes. As with over topping, fully developed piping is virtually impossible to control and will likely cause failure. Seepage can cause slope failure by creating high pressures in the soil pores or by saturating the slope. The pressure of seepage within an embankment is difficult to determine without proper instrumentation. A slope which becomes saturated and develops slides may be showing signs of excessive seepage pressure.

Seepage failure of the dams is of the following types

1. Piping through the dam
2. Piping through the foundation
3. Conduit Leakage

1. **Piping thorugh the dam:** There are two kinds of forces acting on the downstream face of the dam:

1. Weight of the material
2. Seepage Force

If the seepage force exceeds the weight of the material the water washes away the soil from the plate and creates a hole in the ground. This hole deepens as more and more mateial is taken away from it and extends longitudinally, making a pipe hole called "Piping in the dam".

3. Structural Failure:

Structural failures can occur in either the embankment or the appurtenances. Structural failure of a spillway, lake drain, or other appurtenance may lead to failure of the embankment. Cracking, settlement, and slides are the more common signs of structural failure of embankments. Large cracks in either an appurtenance or the embankment, major settlement, and major slides will require emergency measures to ensure safety, especially if these problems occur suddenly. If this type of situation occurs, the lake level should be lowered, the appropriate state and local authorities notified, and professional advice sought. If the observer is uncertain as to the seriousness of the problem, the Division of Water should be contacted immediately. The three types of failure previously described are often interrelated in a complex manner. For example, uncontrolled seepage may weaken the soil and lead to a structural failure. A structural failure may shorten the seepage path and lead to a piping failure. Surface erosion may result in structural failure.

Failure of downstream face during steady seepage conditions
1. Failure of upstream face during sudden draw down
2. Failure due to sliding of foundation
3. damage due to burrowing animals
4. Failure of dam due to earthquake

1. Usually upper part of the dam is dry and the lower is saturated with water which gives rise to pore water pressure within the voids. Dam body is saturated - All pores / voids are filled with water, pore water pressure is induced. Effective pressure reduces and shear strength of soil decreases

2. When water is suddenly withdrawn or in other words if the level of water in the reservoir reduces suddenly, the soil on the upstream face of the dam body may be highly saturated and has pore water pressure that tries to destabilise the dam and if this force is high enough, it can fail the dam.

3. If the shear strength of the soil on which the foundation is built is weak though the foundation itself may be strong but due to weakness of the soil foundation may slide on the sides and in some cases the foundation itself may be not able to resist the shear force that may have increased from normal due to any reason.

4. Burrowing animals - Small animals living in the holes and pits may have dug theri holes anywhere in te dam body which may widen with the passage of time and can be dangerous.

5. Earthquake

Minor defects such as cracks in the embankment may be the first visual sign of a major problem which could lead to failure of the structure. The seriousness of all deficiencies should be evaluated by someone experienced in dam design and construction. A qualified professional engineer can recommend appropriate permanent remedial measures.

Preliminary section of Earthen Dam

The various components of an earthen dam are shown in Fig.

1. **Shell, Upstream Fill, Downstream Fill or Shoulder:** These components of the earthen dam are constructed with pervious or semi-pervious materials upstream or downstream of the core. The upstream fill is called the upstream shell and the downstream portion is the downstream shell.
2. **Upstream Blanket**: It is a layer of impervious material laid on the upstream side of an earthen dam where the substratum is pervious, to reduce seepage and increase the path of flow. The blanket decreases both the seepage flow and excess pressure on the downstream side of the dam. A natural blanket is a cover of naturally occurring soil material of low permeability.
3. **Drainage Filter**: It is a blanket of pervious material constructed at the foundation to the downstream side of an earthen dam, to permit the discharge of seepage and minimize the possibility of piping failure.
4. **Cutoff Wall or Cutoff**: It is a wall, collar or other structure intended to reduce percolation of water through porous strata. It is provided in or on the foundations.
5. **Riprap**: Broken stones or rock pieces are placed on the slopes of embankment particularly the upstream side for protecting the slope against the action of water, mainly wave action and erosion.
6. **Core Wall, Membrane or Core**: It is a centrally provided fairly impervious wall in the dam. It checks the flow of water through the dam section. It may be of compacted puddled clay, masonry, or concrete built inside the dam.

7. **Toe Drain**: It is a drain constructed at the downstream slope of an earthen dam to collect and drain away the seepage water collected by the drain filters.
8. **Transition Filter**: It is a component of an earthen dam section which is provided with core and consists of an intermediate grade of material placed between the core and the shells to serve as a filter and prevent lateral movement of fine material from the core.

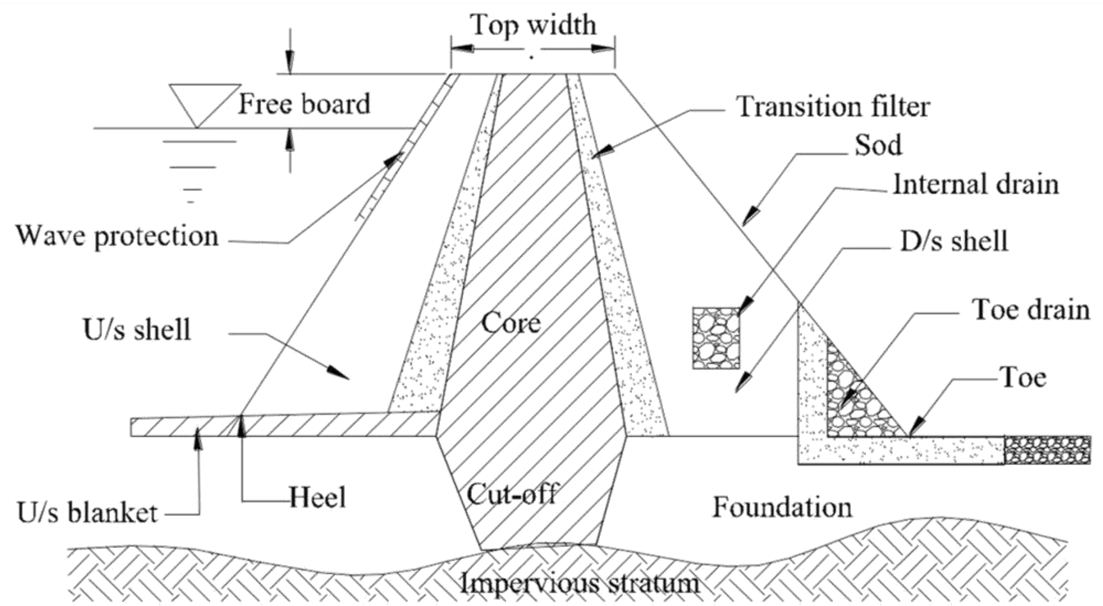


Fig.. Cross-section of an Earthen Dam with Various Components. (Source: Michael and Ojha, 2012)

Advantages

1. Design procedures are straightforward and easy.
2. Local natural materials are used.
3. Comparatively small establishment and equipment are required.
4. Earth fill dams resist settlement and movement better than more rigid structures and can be more suitable for areas where earth movements are common.

Disadvantages

1. An earthen embankment is easily damaged or destroyed by water flowing on, over or against it. Thus, a spillway and adequate upstream protection are essential for any earthen dam.
2. Designing and constructing adequate spillways is usually the most technically difficult part of any dam building work. Any site with a poor quality spillway should not be used.
3. If it is not adequately compacted during construction, the dam will have weak structure prone to seepage.
4. Earthen dams require continual maintenance to prevent erosion, tree growth, subsidence, animal and insect damage and seepage.

Types of Earthen Dam

1. Based on the method of construction:

(a) Rolled Fill Earthen Dams: In this type of dams, successive layers of moistened or damp soils are placed one above the other. Each layer not exceeding 20 cm in thickness is properly consolidated at optimum moisture content maintained by sprinkling water. It is compacted by a mechanical roller and only then the next layer is laid.

(b) Hydraulic Fill Earthen Dam: In this type of dams, the construction, excavation and transportation of the earth are done by hydraulic methods. Outer edges of the embankments are kept slightly higher than the middle portion of each layer. During construction, a mixture of excavated materials in slurry condition is pumped and discharged at the edges. This slurry of excavated materials and water consists of coarse and fine materials. When it is discharged near the outer edges, the coarser materials settle first at the edges, while the finer materials move to the middle and settle there. Fine particles are deposited in the central portion to form a water tight central core. In this method, compaction is not required.

2. Based on the mechanical characteristics of earth materials used in making the section of dam:

(a) **Homogeneous Earthen Dams:** It is composed of one kind of material (excluding slope protection). The material used must be sufficiently impervious to provide an adequate water barrier, and the slopes must be moderately flat for stability and ease of maintenance (Fig.).

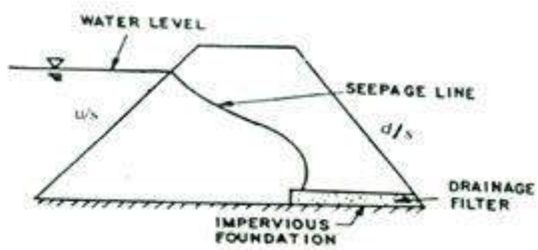


Fig. Homogenous Earthen Dam. (Source: Michael and Ojha, 2012)

(b**) Zoned Earthen Dams**: It contains a central impervious core, surrounded by zones of more pervious material, called shells. These pervious zones or shells support and protect the impervious core (Fig).

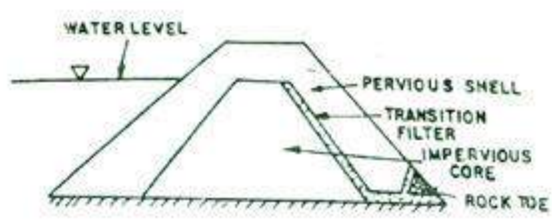


Fig.. Zoned Earthen Dam. (Source: Michael and Ojha, 2012)

(c) **Diaphragm Earthen Dam**: This type of dam (Fig. 11.4) is a modified form of homogenous dam which is constructed with pervious materials, with a thin impervious diaphragm in the central part to prevent seepage of water. The thin impervious diaphragm may be made of impervious clayey soil, cement concrete or masonry or any impervious material. The diaphragm can be constructed in the central portion or on the upstream face of the dam. The main difference in zoned and diaphragm type of dams depends on the thickness of the impervious core or diaphragm. The thickness of the diaphragm is not more than 10 m.

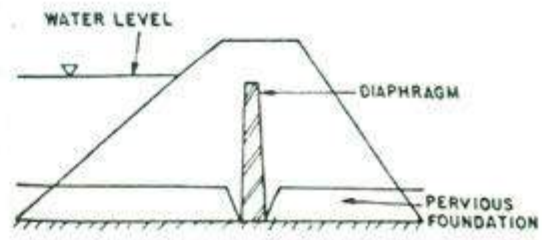


Fig.. Diaphragm Earthen Dam. (Source: Michael and Ojha, 2012)

Design Criteria

Following main design criteria may be laid down for the safety of an earth dam:

1. To prevent hydraulic failures the dam must be so designed that erosion of the embankment is prevented. For this purpose, the following steps should be followed:

(a) Spillway capacity is sufficient to pass the peak flow.

(b) Overtopping by wave action at maximum water level is prevented.

(c) The original height of structure is sufficient to maintain the minimum safe freeboard after settlement has occurred.

(d) Erosion of the embankment due to wave action and surface runoff does not occur.

(e) The crest should be wide enough to withstand wave action and earthquake shock.

2. To prevent the failures due to seepage:

(a) Quantity of seepage water through the dam section and foundation should be limited.

(b) The seepage line should be well within the downstream face of the dam to prevent sloughing.

(c) Seepage water through the dam or foundation should not remove any particle or in other words cause piping.

(d) There should not be any leakage of water from the upstream to the downstream face. Such leakage may occur through conduits, at joints between earth and concrete sections or through holes made by aquatic animals.

3. To prevent structural failures:

(a) The upstream and downstream slopes of the embankment should be stable under all loading conditions to which they may be subjected including earthquake.

(b) The foundation shear stresses should be within the permissible limits of shear strength of the material.

Design of Earthen Dam

The preliminary design of earthen dam is done on the basis of past experiences. For designing purpose several parameters, given below should be considered.

1. Top Width
2. Free Board
3. Settlement Allowance
4. Casing or Outer Shell
5. Cut-off Trench
6. Downstream Drainage System

1. Top Width: Minimum top width (*W*) should be such that it can enhance the practicability and protect it against the wave action and earth wave shocks. Sometimes it is also used for transportation purposes. It depends upon the height of the earthen dam and can be calculated as follows:

$$W = \frac{H}{5} + 3 \quad \text{(for very low dam)}$$

$$W = 0.55\sqrt{H} + 0.2H \quad (H \leq 30)$$

$$W = 1.65\sqrt[3]{H + 1.5} \quad (H \geq 30)$$

where H = the height of the dam (m), for Indian conditions it should not be less than 6 m.

Free board: It is the vertical distance between the top of the dam and the full supply level of the reservoir or the added height. It acts as a safety measure for the dam against high flow condition that is waves and runoff from storms greater than the design frequency from overtopping the embankment. The Recommended values of free board for different heights of earthen dams, given by U.S.B.R., are given in Table.

Table . Recommended Values of Free Board given by U.S.B.R.

Nature of spillway	Height of dam	Free board
Free	Any	Minimum 2 m and maximum 3 m over the maximum flood level
Controlled	< 60 m	2.5 m above the top of the gate
Controlled	> 60 m	3 m above the top of the gate

If fetch length or exposure is given then the free board can also be calculated by Hawksley's formula:

$$h_w = 0.014 D_m^{0.5}$$

where, = wave height (m); D_m = fetch or exposure (m).

2. Settlement Allowance: It is the result of the settlement of the fill and foundation material resulting in the decrease of dam storage. It depends upon the type of fill material and the method and speed of construction. It varies from 10% of design height for hand compacted to 5% for machine compacted earthfill.

3. Casing or Outer Shell: Its main function is to provide stability and protection to the core. Depending upon the upstream and downstream slopes, a recommendation for the casing and outer shell slopes for different types of soils given by Terzaghi is presented in Table 1.

Table. Recommended Slopes of Earthen Dam (Sources: S.K. Garg, 2008)

Sl. No.	Types of material	u/s slope	d/s slope
1.	Homogenous well graded material	$2\frac{1}{2}:1$	$2:1$
2.	Homogenous coarse silt	$3:1$	$2\frac{1}{2}:1$
3.	Homogenous silty clay or clay a) Height less than 15 m b) Height more than 15 m	$2\frac{1}{2}:1$ $3:1$	$2:1$ $2\frac{1}{2}:1$
4.	Sand or sand and gravel with clay core	$3:1$	$2\frac{1}{2}:1$
5.	Sand or sand and gravel with R.C. core wall	$2\frac{1}{2}:1$	$2:1$

Cutoff Trench: It is provided to reduce the seepage through the foundation and also to reduce the piping in the dam. It should be aligned in a way that its central line should be within the upstream face of the impervious core. Its depth should be more than 1 m. Bottom width of cutoff trench (B) is calculated as:

$$B = h - d$$

where h = reservoir head above the ground surface (m); and d = depth of cutoff trench below the ground surface (m).

4. Downstream Drainage System: It is performed by providing the filter material in the earthen dam which is more pervious than the rest of the fill material. It reduces the pore water pressure thus adding stability to the dam.

Three types of drains used for this purpose are:

a) Toe Drains
b) Horizontal Blanket
c) Chimney Drains.

Determination of parametric line by Casagrande's method

Phreatic Line in Earth Dam

Phreatic line is also known as seepage line or saturation line. It is defined as an imaginary line within a dam section, below which there is a positive hydrostatic pressure and above it there is a negative hydrostatic pressure. The hydrostatic pressure represents atmospheric pressure which is equal to zero on the face of phreatic line. Above the phreatic line, there is capillary zone, also called as capillary fringe, in which the hydrostatic pressure is negative. The flow of seepage water, below the phreatic line, reduces the effective weight of the soil; as a result shear strength of a soil is reduced due to increased intergranular pressure in earth fill material.

1. Derivation of Phreatic Line with Filter

In this case, before going directly for derivation, the important features of phreatic line must be known. From the experimental evidence, it has been found that, the seepage line is pushed down by the toe filter and is very close to parabolic shape except at the junction point of the upstream face. The upstream face of the dam represents 100% equipotential line when it is covered by the water; under this condition the seepage line should be drawn perpendicular to this face at the junction point.

Casagrande method is used for deriving the phreatic line (Fig); the procedure is described as follows:

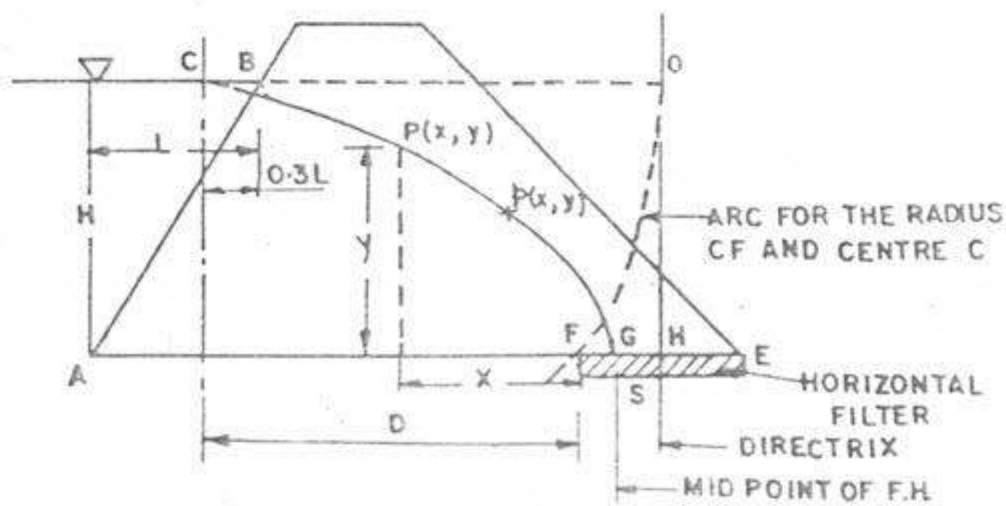


Fig.. Derivation of phreatic line in earth dam. (Source: Suresh, 2002)

a) Let the phreatic line is assumed to be a base parabola with its focus at point *F*, *i.e*. at the starting point of the filter, *FE*.

b) *AB* is the upstream face of the earth dam and *L* is the horizontal projection of face *AB* on the water surface. Measure the distance *BC* equal to 0.3*L*. Count point *C* as a starting point of base of parabola.

c) For deciding the position of directrix of the parabola, the principle which states that every point of the parabola is at equidistance from the focus as well as directrix. Hence, considering *C* as a center and *CF* as radius, an arc is drawn which cut the horizontal line *CB* at point O. Since *CO*= *CF*, hence vertical line *OH* will be directrix of parabola.

d) The last point G of the parabola will fall at the middle of the points *F* and *H*.

e) The intermediate points of parabola are located on the principle that their distance from the focus and directrix are the same. Here to locate the point *P* as an intermediate point, a vertical line *DP* is drawn at any distance *x* from the *F*. Now considering the distance *DH* as radius with *F* as a center, an arc is drawn which cuts the vertical *DP* at point P.

f) Now all there obtained points are joined by free hand to get the base parabola. However, this needs to be corrected at the entry point, for the feature that phreatic line must be started from the point *B* only, not from *C*. It should be sketched perpendicular to the upstream face *AB*, as it is 100 percent equipotential line. Now phreatic line is sketched by free hand in such way that, it should be perpendicular to face *AB* and meets to rest of the points of the parabola tangentially. In addition, the base parabola should also be met perpendicular, to the downstream face of the dam at point*G*.

13.3 Equation of Parabola

The equation of base parabola can be derived from its basic properties *i.e.* the distance of any point *P(x, y)* on the parabola from its focus is the same as the distance of the point *P*(*x*, *y*) from directrix.

Thus we have,

$$PF = DH$$

$$(x^2 + y^2)^{1/2} = DF + FH = x + s$$

Where, s = focal distance (*FH)*
From equation,

$$x^2 + y^2 = x^2 + s^2 + 2xs$$

$$y^2 = s^2 + 2xs \qquad 1$$

This is the desired equation of base parabola.
For deriving the expression of discharge (q) for the earth dam equipped with horizontal filter, the Darcy's law is used. According to which, the discharge (q) through vertical section *PD*, is equal to:

$$q = kiA = k.\frac{\partial y}{\partial x}(y \times 1) \qquad 2$$

Partial differentiation of Eqn.2, resulted

$$\frac{\partial y}{\partial x} = \frac{s}{(2xs+s^2)^{\frac{1}{2}}} \qquad 3$$

Substituting the value of in Eqn. 3, the rate of seepage flow through the dam is given by:

$$q = k\,\frac{s}{(2xs+s^2)^{\frac{1}{2}}} \times (2xs+s^2)^{\frac{1}{2}}$$

or, $$q = k \times s$$

This is the expression for computing the rate of seepage discharge through the body of earthen dam, in terms of focal distance *s*. The distance s can be determined either graphically or analytically. Considering *C* as co-ordinate, the value of *s* can be obtained as:
From Eqn: 1

$$s = \sqrt{x^2 + y^2} - x$$

At point *C*, $x = D$ and $y = H$

Therefore, $$s = \sqrt{D^2 + H^2} - D$$

Thus, $$q = k \times s$$

$$q = k\,[(D^2 + H^2)^{1/2} - D]$$

By using this equation, if the value of coefficient of permeability (*k*) and focal distance (*s*) are known, the discharge (*q*) can be calculated. This gives an accurate value of seepage rate and is applicable to such dams, which are provided with horizontal drainage (filter) system but can also be used for other types of dam section.

2. Phreatic Line in Earthen Dam without Filter

The position of phreatic line in an earth dam without filter can be determined using the same manner, as in previous case i.e. with a filter. In this case, the focal point (*F*) of the parabola will be the lowest point of the downstream slope (Fig. 2). The base of the parabola *BJC* cut at a point *J*on downstream slope and is extended beyond the limit of the dam, as indicated by dotted line, but the seepage line should be emerged at point *K*, tangential to downstream face. In this way, the phreatic line should be shifted to the point *K* from *J*. The distance *KF* is known as discharge face, which always remains under saturation condition. The correction *JK* (say) by which the base of parabola need to be shifted downward, can be determined by graphical and analytical methods.

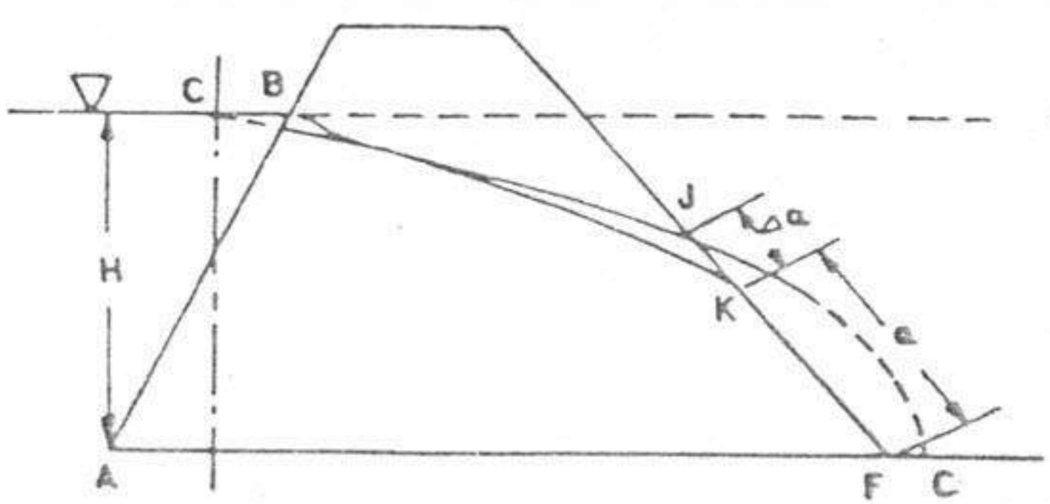


Fig. 2. Phreatic line without filter.
(Source: Suresh, 2002)

1. Graphical Method

Casagrande has given a general solution to determine the value of   for various degrees of inclination of the discharge face. The inclination angle may be more than 90°, especially in case of rock fill dam.

Let, if a is the slope angle of the discharge face with the horizontal is known, and then various values of $\frac{\Delta a}{a+\Delta a}$ corresponding to a are given by Casagrande (Table).

$$a + \Delta a = JF \text{ (From Fig.}$$

Here, *JF* indicates the distance of the focus from the point, where base of parabola cuts downstream face. The values of and can be obtained by Eqn and Table.

Table. Values of $\frac{\Delta a}{a+\Delta a}$ for various slope angles (a)

Slope angle α (in degree)	$\frac{\Delta a}{a+\Delta a}$	Remarks
30	0.36	**Note**: Intermediate values of $\frac{\Delta a}{a+\Delta a}$ can be computed by interpolation method
60	0.32	
90	0.26	
120	0.18	
135	0.14	
150	0.10	
180	0	

Estimation of seepage

Example:
An earth dam made of a homogeneous material has a horizontal filter and other parameters as shown in the figure. Determine the phreatic line and the seepage quantity through the body of the dam.

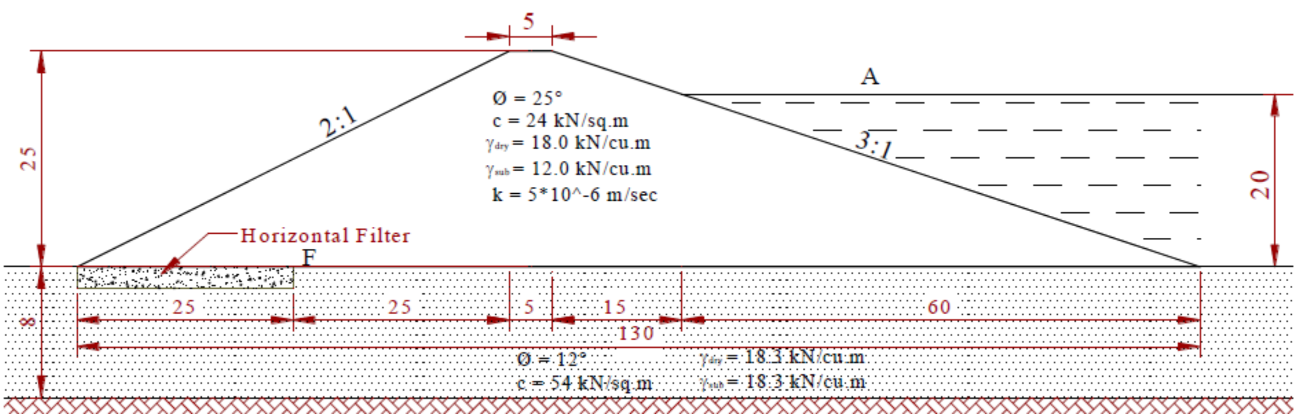


Figure 5 Section of a homogenous earth dam

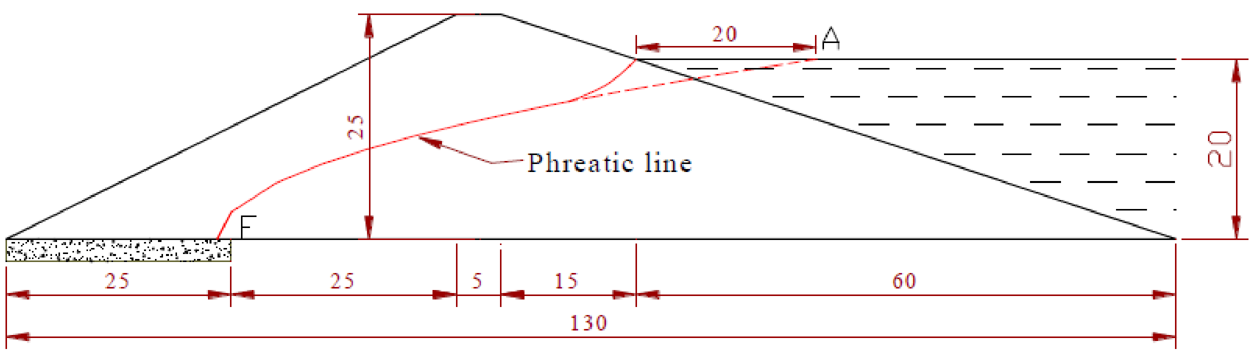
.

For the origin of the Cartesian co-ordinate system at the face of the filter (point F), the equation of the parabola of the seepage line can be expressed as:

$$\sqrt{x^2 + y^2} = x + S$$

At point A, x = 65m, and y = 20m. Inserting into the parabola equation, S = 3.07m. Working out a few more points from the equation, the parabola can be easily drawn and corrected for the curve at the upstream face of the dam, so as to get the seepage line.

x	-1.51	0	10	15	25	30	40	45	55	65
y^2	0	9.06	69.26	99.36	159.56	189.66	249.86	279.96	340.16	400.36
y	0	3.01	8.32	9.97	12.63	13.77	15.81	16.73	18.44	20.01



The amount of seepage flow is

Q = kS

$= 5 * 10^{-6} * 3.07$

$= 15.35 * 10^{-6}\ m^3$/sec per meter width of dam

B. *Homogeneous dam section without horizontal filter*

The focus (F) of the parabola will be the lowest point of the downstream slope as shown in Figure 5-8. The base parabola BIJC will cut the downstream slope at J and extend beyond the dam toe up to the point C i.e. the vertex of the parabola.

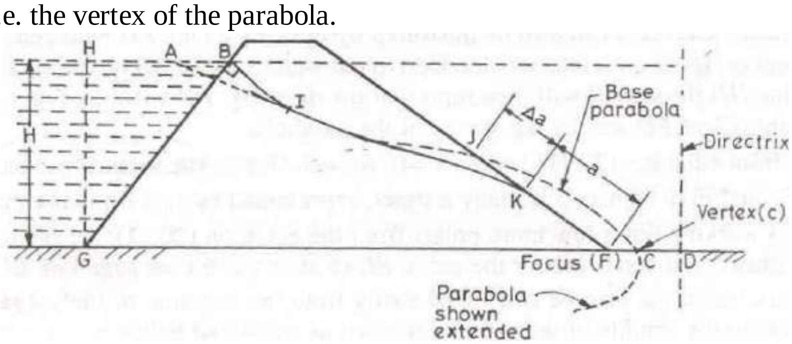


Figure Homogeneous dam section without filter

The seepage line will, however, emerge out at K, meeting the downstream face tangentially there. The portion KF is known as discharge face and always saturated. The correction JK (say Δa) by which the parabola is to be shifted downward can be determined as follows:

$\alpha^{\ddagger}$ in degrees	$\frac{\Delta a}{a + \Delta a}$
30^o	0.36
60^o	0.32
90^o	0.26
120^o	0.18
135^o	0.14
150^o	0.10
180^o	0.0

α is the angle which the discharge face makes with the horizontal. a and Δa can be connected by the general equation;

$$\Delta a = (a + \Delta a)\left[\frac{180^{\circ} - \alpha}{400^{\circ}}\right]$$

Module 3

Spillways

Spillway is the most important component of the dam which serves to release excess flood from a reservoir efficiently and safely. It is the most expensive of all the appurtenances structure. Its capacity is determined from the hydrological studies over the drainage area.

Spillway components include;

a. Entrance channel: to minimize head loss and to obtain uniform distribution of flow over the spillway crest

b. Control structure: to regulate and control the outflow. It may consist of a sill, weir, orifice, tube, or pipe.

c. Discharge channel: to convey the discharge from the control structure to the terminal structure/stream bed. The conveyance structure may be the downstream face of a concrete dam, an open channel excavated along the ground surface, a closed cut-and-cover conduit placed through or under a dam, or a tunnel excavated through an abutment.

d. Terminal structure: to dissipate excess energy of the flow in order to avoid scouring of the stream bed

e. Outlet channel: to safely convey the flow from the terminal structure to the river channel.

Types of spillway taking the hydraulic as criteria are broadly

a. **Controlled (Gated) spillway:** a spillway having a certain means to control the outflow from the reservoir.

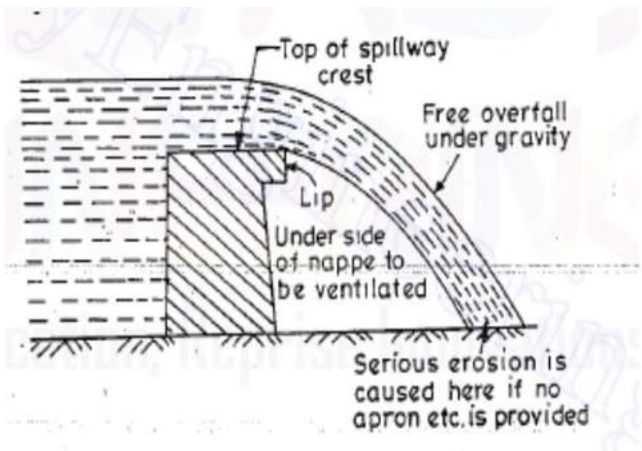
b. **Uncontrolled (Ungated) spillway:** is a spillway, the crest of which permits water to escape automatically, as the water level in the reservoir rises above the crest.

Types of Spillway

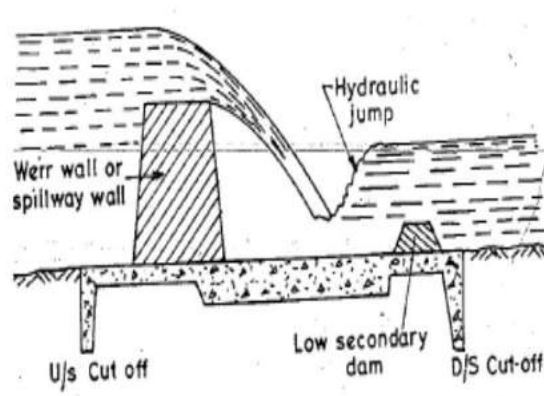
1. Free overfall (straight drop) spillway
2. Ogee (overflow) spillway
3. Chute (open channel or trough) spillway
4. Side channel spillway
5. Drop inlet (shaft or morning glory) spillway
6. Siphon spillway

1) **Free Overfall / Straight drop Spillway**:

In this type of spillway, the water freely drops down from the crest, as for an arch dam (Figure 1) also for a decked over flow dam with a vertical or adverse inclined downstream face (Figure 2). Flows may be free discharging, as will be the case with a sharp-crested weir or they may be supported along a narrow section of the crest. Water freely falls from crest under *the action of gravity*. Since vacuum is created in the under-side portion of the falling jet, sufficient ventilation of nappe is required in order to avoid *pulsating and fluctuating effects of the jet*.



(Without D/s protection)



(With D/s protection)

2) Overflow (Ogee) Spillway:

This type of spillway is the most common type adopted in the field. It divides naturally into three zones i.e. Crest, spillway face and the toe. The concept evolves from replacing the lower nappy of the flow over thin plate weir by solid boundary. The overflow type spillway has a crest shaped in the form of *an ogee or S-shape*. The upper curve of the ogee is made to conform closely to the profile of the lower nappy of a ventilated sheet of water falling from a sharp crested weir (figure 3). Flow over the crest of an overflow spillway is made to adhere to the face of the profile by preventing access of air to the underside of the sheet of flowing water.

Naturally, the shape of the overflow spillway is designed according to the shape of the lower nappe of a free flowing weir conveying the discharge flood any discharge higher than the design flood passing through the overflow spillway would try to shoot forward and get detached from the spillway surface, which reduces the efficiency of the spillway due to the presence of negative pressure between the sheet of water and spillway surface. For discharges at designed head, the spillway attains near-maximum efficiency.

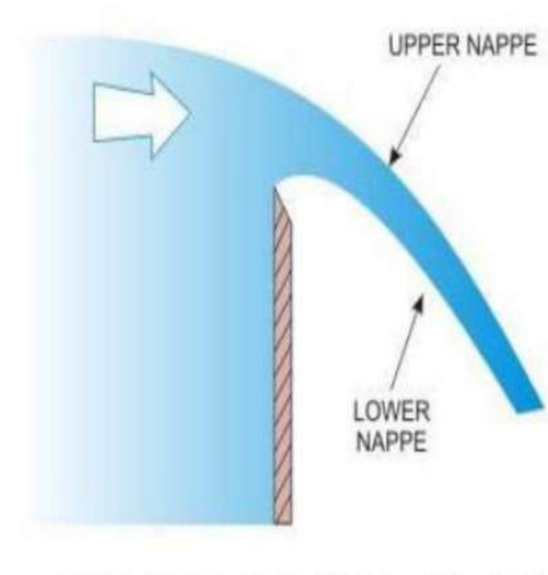


Fig- Outflow from a freely falling weir properly ventilated from below,

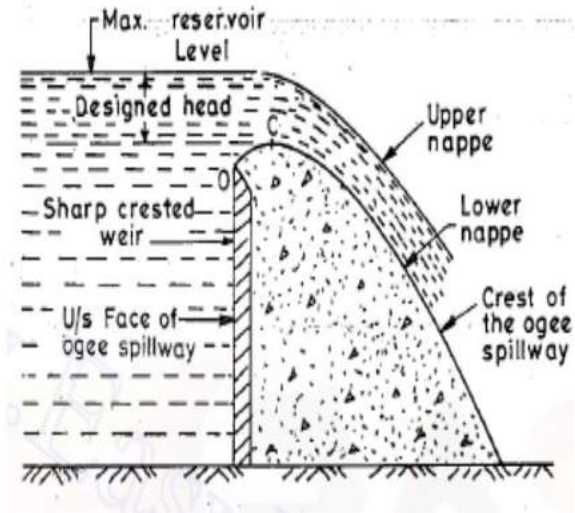


Fig Section of an Ogee spillway with vertical u/s face

3. Chute (Open Channel/Trough) Spillway:

A chute spillway, variously called as open channel or trough spillway, is one whose discharge is conveyed from the reservoir to the downstream river level through an open channel, placed either along a dam abutment or through a saddle. The control structure for the chute spillway need not necessarily be an overflow crest, and may be of the side-channel type, as has been shown in Figure 5. Generally, the chute spillway has been mostly used in conjunction with embankment dams, like the Tehri dam. Chute spillways are simple to design and construct and have been constructed successfully on all types of foundation materials, ranging from solid rock to soft clay. Chute spillways ordinarily consist of an entrance channel, a control structure, a discharge channel, a terminal structure, and an outlet channel.

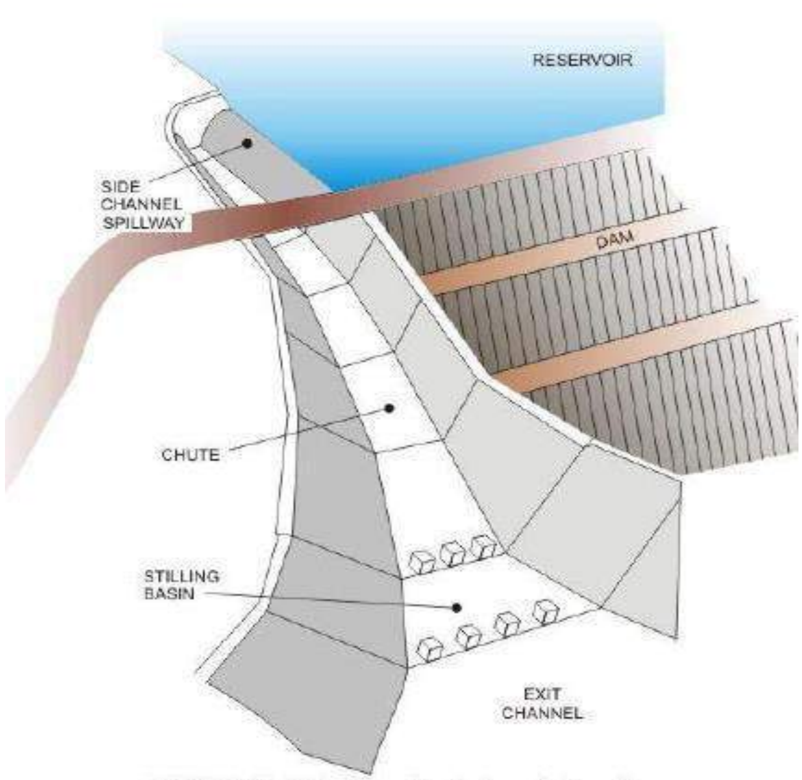


Fig- side channel entry to a Chute spillway

4. Side Channel Spillway:

A side channel spillway is one in which the control weir is placed approximately parallel to the upper portion of the discharge channel, as may be seen in fig 6. The flow over the crest falls into a narrow trough opposite to the weir, turns an approximate right angle, and then continues into the main discharge channel. The side channel design is concerned only with the hydraulic action in the upstream reach of the discharge channel and is more or less independent of the details selected for the other spillway components.

Discharge characteristics of a side channel spillway are similar to those of an ordinary overflow spillway and are dependent on the selected profile of the weir crest. Although the side channel is not hydraulically efficient, nor inexpensive, it has advantages which make it adoptable to spillways where a long overflow crest is required in order to limit the afflux (surcharge held to cause flow) and the abutments are steep and precipitous.

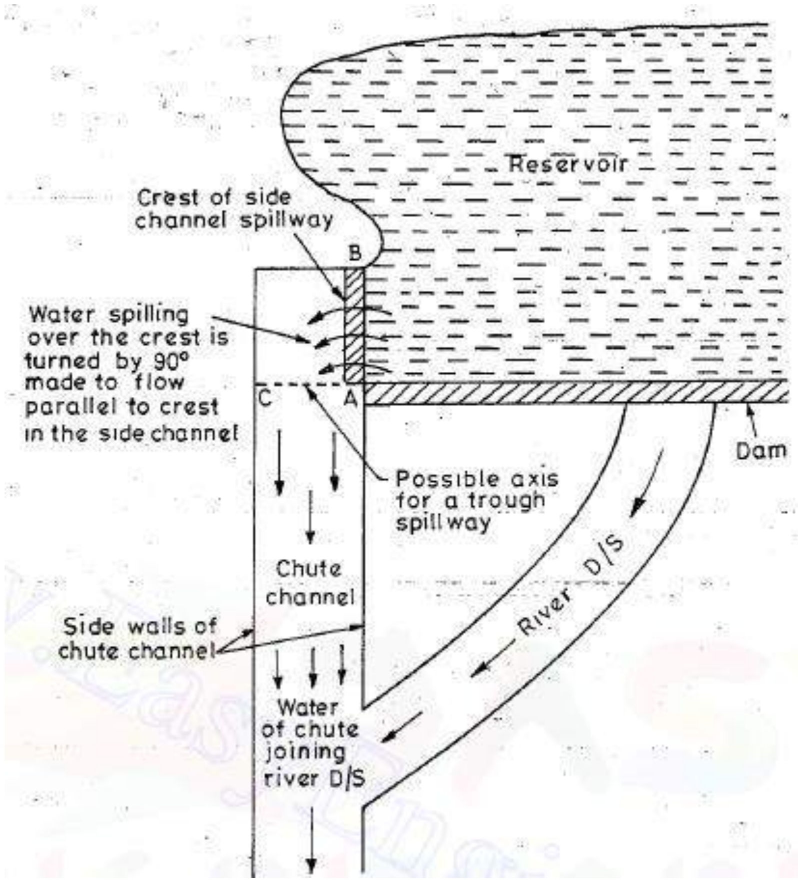


Fig- sketch of a side-channel spillway

5. Shaft (Drop Inlet/Morning Glory) spillway:

A Shaft Spillway is one where water enters over a horizontally positioned lip, drops through a vertical or sloping shaft, and then flows to the downstream river channel through a horizontal or nearly horizontal conduit or tunnel. A drop inlet spillway can be used advantageously at dam sites that are located in narrow gorges where the abutments rise steeply.

Discharge characteristics of the drop inlet spillway may vary with the range of head. The head increases, the flow pattern would change from the initial weir flow over crest to tube flow and then finally to pipe flow in the tunnel. This type of spillway attains maximum discharging capacity at relatively low heads. However, there is little increase in capacity beyond the designed head, should a flood larger than the selected inflow design flood occur.

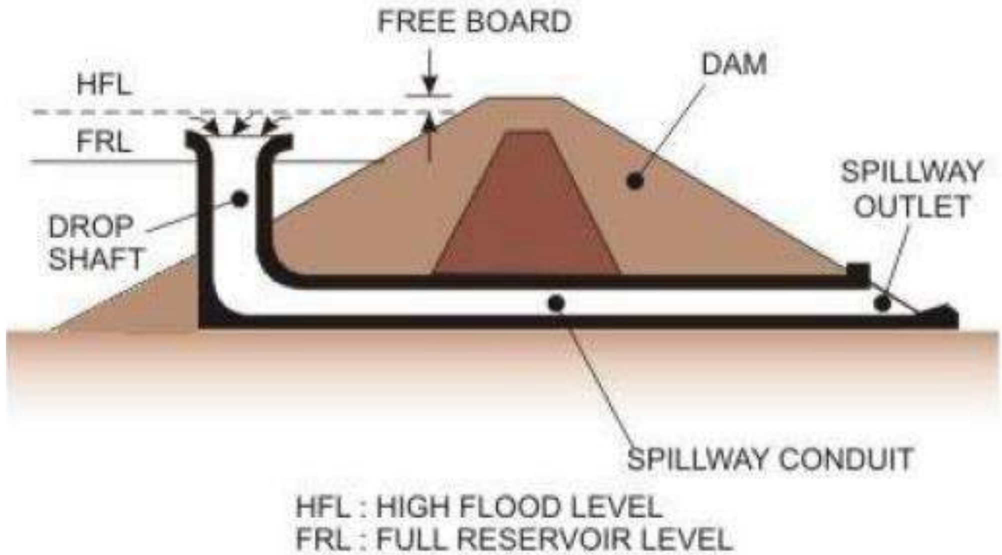


Fig- Section through a shaft spillway

6. Tunnel (Conduit) spillway:

Where a closed channel is used to convey the discharge around a dam through the adjoining hill sides, the spillway is often called a tunnel or conduit spillway. The closed channel may take the form of a vertical or inclined shaft, a horizontal tunnel through earth or rock, or a conduit constructed in open cut and backfilled with earth materials. Most forms of control structures, including overflow crests, vertical or inclined orifice entrances, drop inlet entrances, and side channel crests, can be used with tunnel spillways. Tunnel spillways are advantageous for dam sites in narrow gorges with steep abutments or at sites where there is danger to open channels from rock slides from the hills adjoining the reservoir. Conduit spillways are generally most suited to dams in wide valleys as in such cases the use of this types of spillway would enable the spillway to be located under the dam very close to the stream bed.

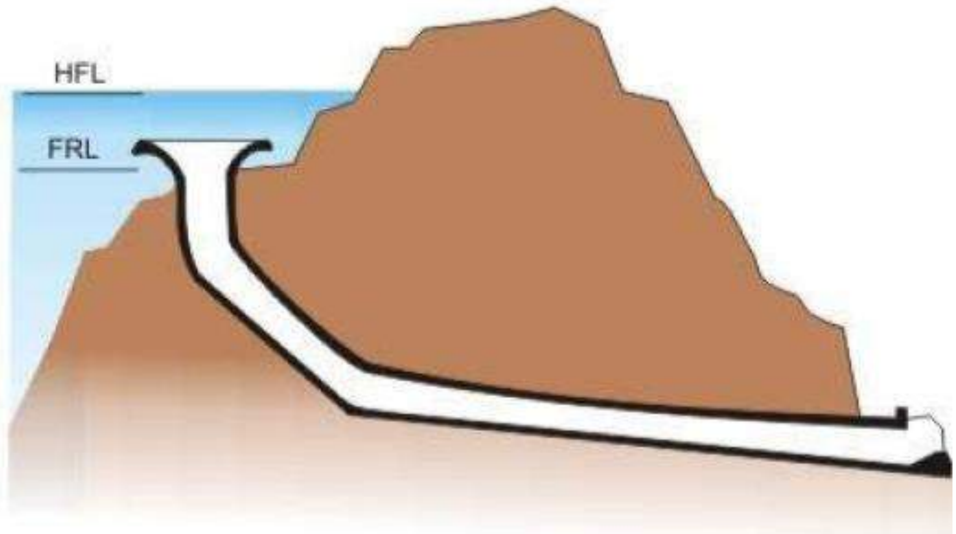


Fig-8 Tunnel spillway with a morning glory entrance

7. Siphon spillway:

A siphon spillway is a closed conduit system formed in the shape of an inverted U, positioned so that the inside of the bend of the upper passageway is at normal reservoir storage level. This type of siphon is also called a Saddle siphon spillway. The initial discharges of the spillway, as the reservoir level rises above normal, are similar to flow over a weir. Siphonic action takes place after the air in the bend over the crest has been exhausted. Continuous flow is maintained by the suction effect due to the gravity pull of the water in the lower leg of the siphon.

Siphon spillways comprise usually of five components, which include an inlet, an upper leg, a throat or control section, a lower leg and an outlet. Another type is hooded type of siphone spillway in which reinforced concrete hood is constructed over an ordinary overflow section of a gravity dam. The inlet of this hood is kept submerged so as to prevent entry of debris and ice. A small depriving hood is kept above the main hood and both these hoods are connected by air vent and head of the depriver hood is kept at normal pool level.

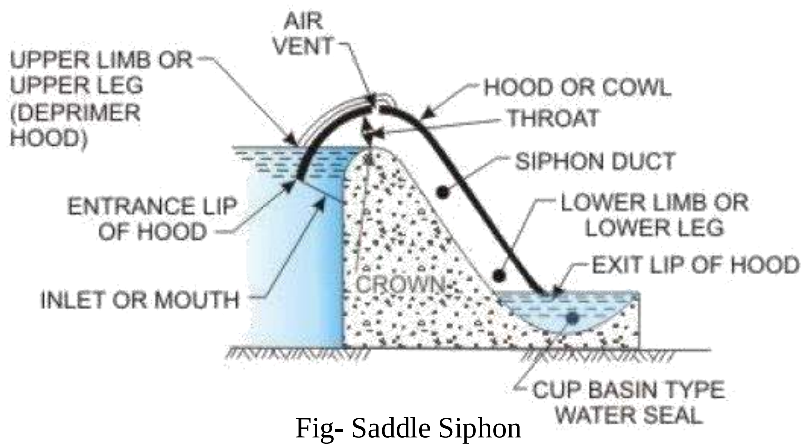


Fig- Saddle Siphon

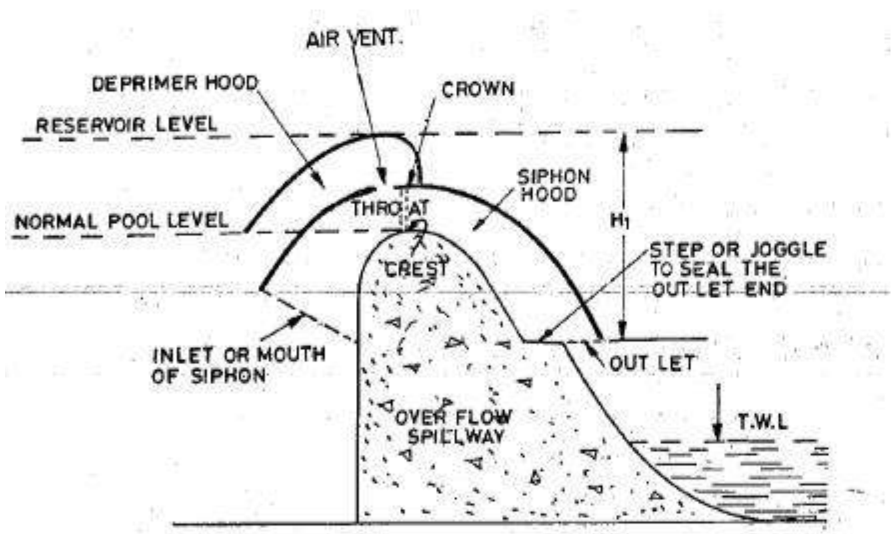


Fig- Siphon installed over the overflow spillway

Design of Ogee spillway

Free overflow ogee spillway.

For the free overflow ogee a sound rock foundation is assumed to exist for the construction of the gravity dam and a ski jump is found to be satisfactory at the toe of the ogee for the dissipation of energy. From the topography it is observed that there is no need for the construction of an approach channel.

Design data

Design discharge (Q) = 1410 m^3/s

River bed elevation = 1390 m

The design head is 6m, but a negative pressure head of 1.0 m is assumed to develop in the crest of the spillway for economic reasons and the workmanship is assumed to be good enough not to create rough surface for this negative head to result in cavitation problem. The vapor pressure of water for the spillway site is 3.595m

Therefore, from the negative pressure head (h_u) specified the corresponding design head (h_{des}) is

$h_u = h(1-h/h_{des})$ -1. = $6(1- 6/h_{des})$ $h_{des} = 5.14$ m

P/h = 6

This value (P/h = 6) hence the effect of approach velocity is too small and can be neglected. But a case where the dam is filled by sediment is considered and P is decreased. Therefore P is assumed to be 2m.

P/h = 2/6

= 0.333

The respective value of C_o (coefficient of discharge) from chart is

Co = 2.175

$q_o = C_oH^{1.5} = 2.175 * 6^{1.5} = 32$ m^3/s/m $v_o = q/(P + h) = 32/(2+6) = 4$ m/sec

Velocity head (h_a)

ha = $v_o^2/2*g = 16/19.62 = 0.81$ m adding 10% of ha for entrance and other losses $h_a = 0.9$ m

Therefore, $H_e = 6.9$ m

Correction for the coefficient of discharge

$P/H_e = 0.29$

$C_o = 2.18$ hence, no appreciable change from the previous value.

For an upstream slope of 2:3

$C_i/C_o = 1.026$

Submergence effect is not considered here because the downstream apron is much below the crest level for any submergence to occur for the design discharge. For similar reason the correction for downstream apron is not carried out.

Therefore, the final corrected value of the coefficient of discharge for the ogee is

C = 2.18 *1.026 = 2.23

From the discharge equation by Polini

Q = CL‟‟H_e1.5 1410 = 2.23 *L‟‟* 6.9*1.5 L‟‟ = 35.00m

For the provision of round nosed piers ($k_p = 0.01$) at every 8m interval along the ogee

Number of piers required = 4

Pier thickness is 2m

Rounded abutments with headwalls at 900 to the direction of flow are used ($k_a = 0.1$)

The effective length of the crest will then be

$L = L' + 2(nkp + ka)H = 35.0 + 2(4*0.01 + 0.1) * 6.9 = 36.93 \quad 37.0m$

Adding the pier width the total width of the crest will be B = 37.0 + 8 = 45.0 m

The profile of the nape is determined based on the charts available on USBR design of small dams.

Ha/He = 0.9/6.9 = 0.13

For an upstream slope of 1:1 crest position

$X_c/H_e = 0.195$ $X_c = 1.35m$ $Y_c/H_e = 0.07$ $Y_c = 0.49m$

Profile upstream of the crest

$R_1/H_e = 0.465$

$R_1 = 3.21m$ $R_2/H_e = 0.367$

$R_2 = 2.53m$

Down stream of the crest

$Y/H_e = -k(X./H_e)^n$ values of the constants are found(from charts on USBR) to be

K = 0.52

n = 1.763

$y = -0.119\ x^{1.763}$

Tabulating values for the above equation,

The point of tangency in the downstream for a slope of m= 0.6

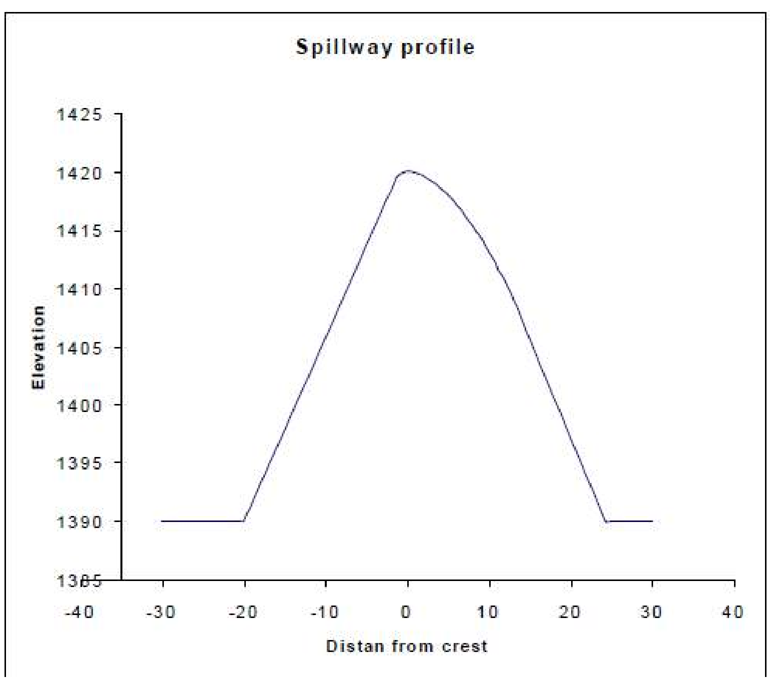
The value of a is obtained from table (a = 1.80)

$Y_T = -H_eK(mkn)^{n/(1-a)}$

= -9.4m

The coordinate values obtained so far for the ogee nape profile are tabulated and plotted as follows.

X	Elevation
-30	1390
-20.5	1390
-20	1390
-1.35	1419.5
-1	1419.7
0	1420
1.3	1419.8
2	1419.6
2.5	1419.4
3	1419.2
3.5	1418.9
4	1418.6
4.5	1418.3
5	1418
5.5	1417.6
6	1417.2



6.5	1416.8
7	1416.3
7.5	1415.8
8	1415.3
8.5	1414.8
9	1414.3
9.5	1413.7
10	1413.1
10.5	1412.5
11	1411.8
11.5	1411.2
12	1410.5
13	1409.1
24.28	1390
25	1390
30	1390

Example 21.1. *Design a suitable section for the overflow portion of a concrete gravity dam having the downstream face sloping at a slope of 0.7 H : 1 V. The design discharge for the spillway is 8,000 cumecs. The height of the spillway crest is kept at RL 204.0 m. The average river bed level at the site is 100.0 m. The spillway length consists of 6 spans having a clear width of 10 m each. Thickness of each pier may be taken to be 2.5 m.*

Solution. Since the given spillway looks like a high weir, the coefficient of discharge may be assumed to be 2.2.

Now $$Q = C \cdot L_e H_e^{3/2}$$

where $L_e = L - 2\,[N K_p + K_a]\, H_e$

Let us first work out the approximate value of H_e for a value of

$$L_e \approx L = \text{clear waterway} = 6 \times 10 = 60 \text{ m.}$$

$$\therefore \quad 8{,}000 = 2.2 \times 60\, H_e^{3/2}$$

or $$H_e^{3/2} = \frac{8{,}000}{2.2 \times 60} = 60.6$$

or $$H_e = (60.6)^{2/3} = 15.5 \text{ m.}$$

The height of the spillway above the river bed (see Fig. 21.15)

$$= h = 204 - 100 = 104.0 \text{ m}$$

Since $\frac{h}{H_d}$, *i.e.* $\frac{104}{15.5} > 1.33$, it is a high spillway, the effect of velocity head can, therefore, be neglected.

Since $$\frac{h_d+d}{H_e}=\frac{H_e+h}{H_e}=\frac{15.5+104}{15.5}>1.7\,;$$

the discharge coefficient is not affected by tail water conditions, and the spillway remains a high spillway.

U/s Slope. The upstream face of the dam and spillway is proposed to be kept vertical. However, a batter of 1 : 10 will be provided from stability considerations in the lower part. This batter is small and will not have any effect on the coefficient of discharge.

Effective length of spillway (L_e) can now be worked out as

$$L_e=L-2\,[N.K_p+K_a]\,H_e$$

Assuming that 90° cut water nose piers and rounded abutments shall be provided, we have

$$K_p=0.01$$

and $$K_a=0.1$$

No. of piers $=N=5$.

Also assuming that the actual value of H_e is slightly more than the approximate value worked out (*i.e.* 15.5 m), say, let it be 16.3 m, we have

$\therefore$ $$L_e=60-2\,[5\times0.01+0.1]\times16.3=\mathbf{55.1\ m.}$$

Hence $$Q=2.2\times55.1\times H_e^{3/2}$$

or $$8{,}000=2.2\times55.1\times H_e^{3/2}$$

or $$H_e^{3/2}=\frac{8{,}000}{2.2\times55.1}\cong66.0$$

or $$H_e=(66.0)^{2/3}=\mathbf{16.4\ m}\cong16.3\text{ (assumed)}$$

Hence, the assumed H_e for calculating L_e is all right. The crest profile will be designed for $H_d=16.4$ m (neglecting velocity head).

Note. The velocity head (H_a) can also be calculated as follows :

$$\text{Velocity of approach}=V_a=\frac{8{,}000}{(60+5\times2.5)\,(104+16.4)}$$

$$=\frac{8{,}000}{72.5\times120.4}=0.917\text{ m/sec.}$$

$$H_a=\text{Velocity Head}=\frac{V_a^2}{2g}=\frac{(0.917)^2}{2\times9.81}=0.043\text{ m.}$$

This is very small and was, therefore, neglected.

Downstream profile. The W.E.S. d/s profile for a vertical u/s face is given by equation (21.2) as :

$$x^{1.85}=2\cdot H_d^{0.35}\cdot y$$

or $$y=\frac{x^{1.85}}{2\,(H_d)^{0.85}}=\frac{x^{1.85}}{2\times(1.64)^{0.85}}$$

or $$y = \frac{x^{1.85}}{2 \times 10.8}$$

or $$y = \frac{x^{1.85}}{21.6} \qquad \ldots(21.7)$$

Before we determine the various co-ordinates of the d/s profile, we shall first determine the tangent point.

The d/s slope of the dam is given to be 0.7 H : 1 V.

Hence, $$\frac{dy}{dx} = \frac{1}{0.7}$$

Differentiating the equation of the d/s profile w.r. to x, we get

$$\frac{dy}{dx} = \frac{1.85x^{1.85-1}}{21.6} = \frac{1}{0.7}$$

or $$x^{0.85} = \frac{21.6}{1.85 \times 0.7} = 16.7$$

or $$x = 22.4 \text{ m.}$$

$\therefore$ $$y = \frac{(22.4)^{1.85}}{21.6} = 14.6 \text{ m.}$$

The co-ordinates from $x = 0$ to $x = 22.4$ m are worked out in Table 21.5.

Table 21.5

x *metres*	$y = \frac{x^{1.85}}{21.6}$ *metres*
1	0.046
2	0.166
3	0.354
4	0.60
5	0.905
6	1.274
7	1.710
8	2.162
9	2.684
10	3.240
12	4.575
14	6.020
16	7.88
18	9.74
20	11.85
22	14.35
22.4	14.60

The u/s profile. The u/s profile may be designed as per equation (21.3), as :

$$y = \frac{0.724\,(x + 0.27\,H_d)^{1.85}}{H_d^{0.85}} + 0.126\,H_d - 0.4315\,H_d^{0.375}\,(x + 0.27\,H_d)^{0.625}$$

Using $H_d = 16.4$ m, we get

$$y = \frac{0.724\,[x + 0.27 \times 16.4]^{1.85}}{(16.4)^{0.85}} + 0.126\,(16.4)$$

$$- 0.4315\,(16.4)^{0.375}\,(x + 0.27 \times 16.4)^{0.625}$$

or $$y = 0.07\,(x + 4.44)^{1.85} + 2.07 - 1.234\,(x + 4.432)^{0.625} \qquad ...(21.8)$$

This curve should go upto $x = -\,0.27\,H_d$

or $$x = -\,0.27 \times 16.4 = -\,4.443 \text{ m.}$$

Various values of x such as, $x = -\,0.5, x = -\,1.0, x = -\,2.0, x = -\,3.0, x = -\,4.0, x = -\,4.443$ are substituted in equation (21.8) and corresponding values of y are worked out, as given below in Table 21.6.

Table 21.6

x in metres	*y in metres*
− 0.5	0.020
− 1.0	0.063
− 2.0	0.27
− 3.0	0.65
− 4.0	1.34
− 4.443	2.07

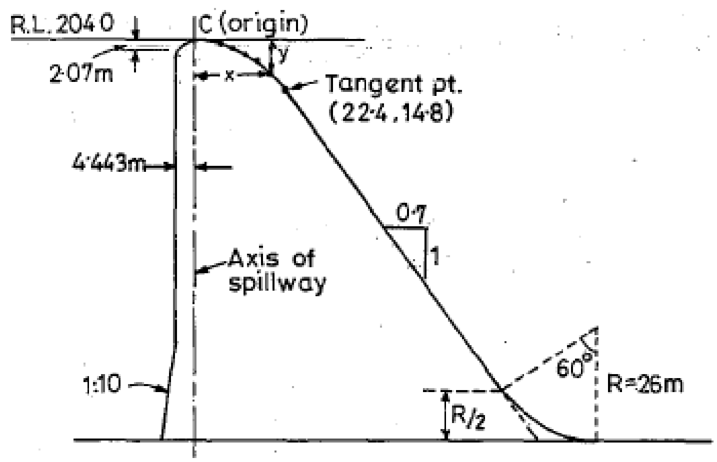


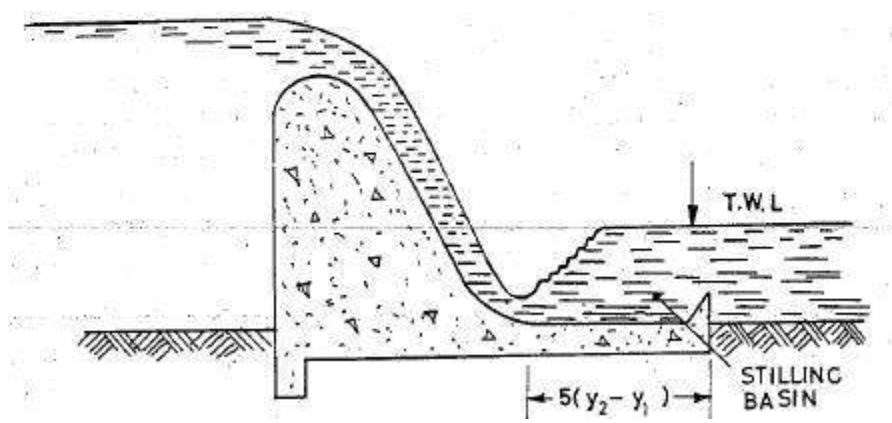
Fig. 21.17

The profile of the spillway has been determined and plotted in Fig. 21.17. A reverse curve at the toe with a radius equal to $\frac{h}{4} = \frac{104}{4} = 26$ m can be drawn at angle 60°, as shown in Fig. 21.17. Aeration pipes (say 25 mm pipes at 3 m c/c) can be installed along the spillway face below the gate lip, so as to prevent the development of negative pressures. The energy dissipation arrangements have not been shown. They should be designed depending upon the position of the jump height curve and tail water curve, as explained afterwards. A sky jump bucket or an apron may be provided as per the prevailing conditions.

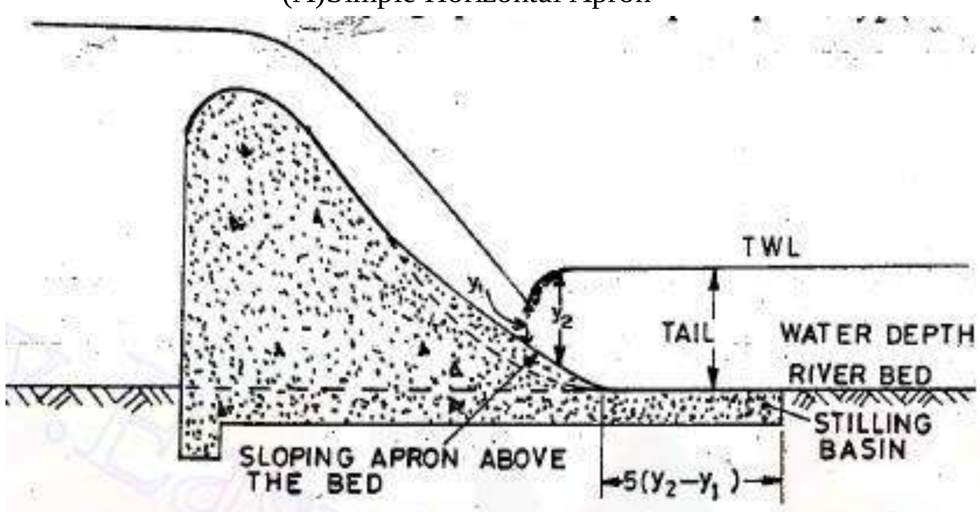
Energy dissipation devices

The flood water discharging through the spillway has to flow down from a higher elevation at the reservoir surface level to a lower elevation at the natural river level on the downstream through a passage, which is also considered a part of the spillway. At the bottom of the channel, where the water rushes out to meet the natural river, is usually provided with an energy dissipation device that kills most of the energy of the flowing water. These devices, commonly called as Energy Dissipators, are required to prevent the river surface from getting dangerously scoured by the impact of the out falling water.

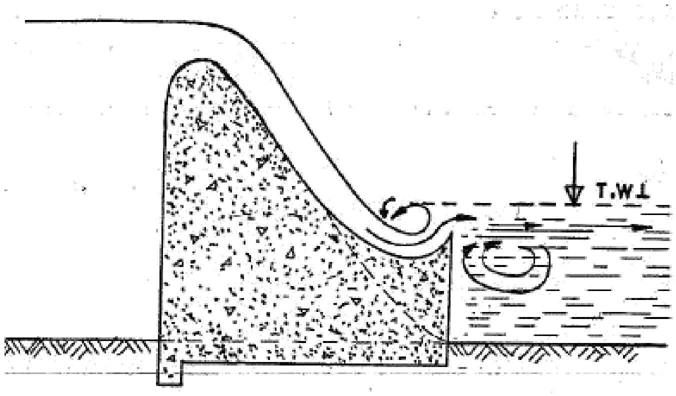
Types as per cases



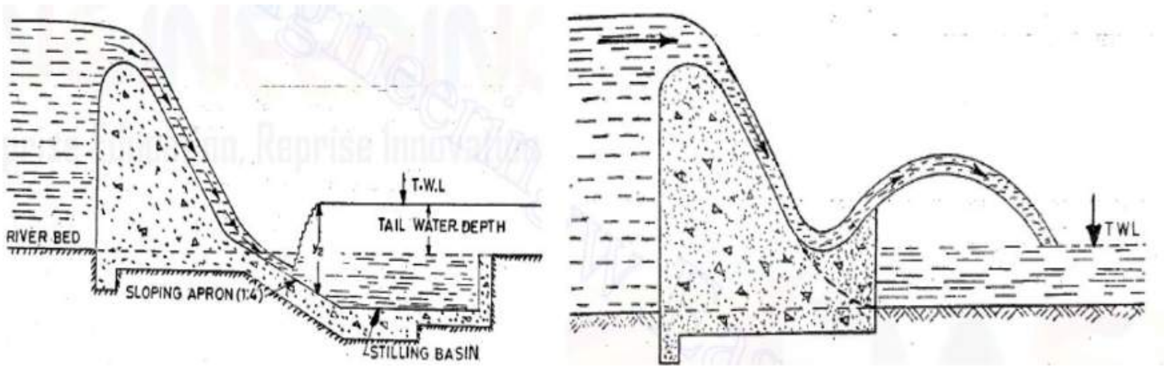
(A)Simple Horizontal Apron



(B) Sloping Apron above the bed



(C) Roller Bucket Types



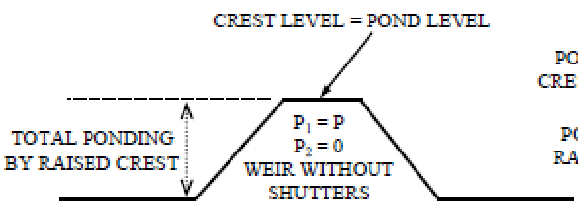
(D)Sloping apron partly above and partly below ground level

Diversion Headwork's

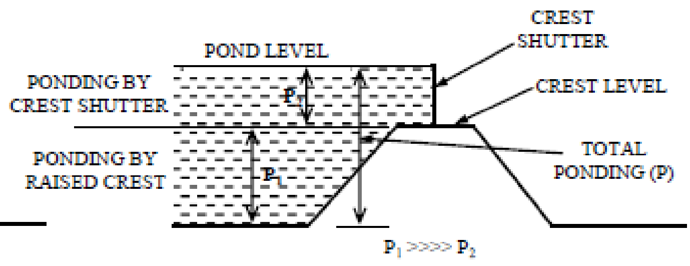
The works, which are constructed at the head of the canal, in order to divert the river water towards the canal, so as to ensure a regulated and continuous supply of silt-free water with a certain minimum -head into the canal, are known as Diversion Head Works

Weir and Barrage

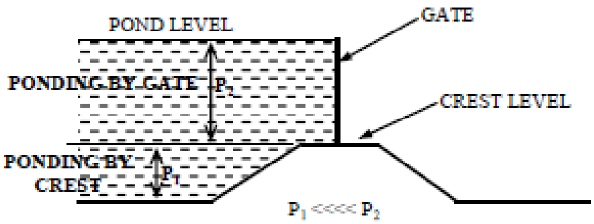
If the major part or the entire ponding of water is achieved by a raised crest and a smaller part or nil part of it is achieved by the shutters, then this barrier is known as a weir [Fig]. On the other hand, if most of the ponding is done by gates and a smaller or nil part of it is done by the raised crest, then the barrier is known as a Barrage or River Regulator [Fig.].



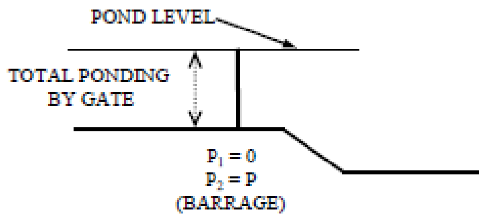
(a) Fig: Weir without shutters



(b) Fig: Weir with shutters



(c) Fig: Barrage with a small raised crest



(d) Fig: Barrage without any raised crest

Gravity and Non-Gravity Weirs

When the weight of the weir *(I.e.* its body and floor) balances the uplift pressure caused by .the head of the water seeping below the weir, it is called a ***Gravity weir.*** On the other hand, if the weir floor is designed continuous with the divide piers as reinforced structure, such that the weight of concrete slab together with the weight of divide piers, keep the structure safe against the uplift ; then the structure may be called as a ***Non gravity Weir.***

Layout of a Diversion Head Works and its Components

A typical layout of a canal head-works is shown in Fig. Such a head-works consists of:

(1) Weir proper.

(2) Under-sluices.

(3) Divide wall, dividing the river width into two portions; one is called the weir portion, and the other portion from which the canal takes off, is - having openings and called the 'under-sluice-pocket' or 'under sluices' or 'weir scouring sluices'. If there are two canals, taking off from each flank, then there will be two divide walls and two under sluices.

(4) River training works, such as marginal bunds, guide banks, groynes, etc

(5) Fish Ladder.

(6) Canal Head Regulator.

(7) Weir's ancillary works, such as shutters, gates, etc.

(8) Silt Regulation Works.

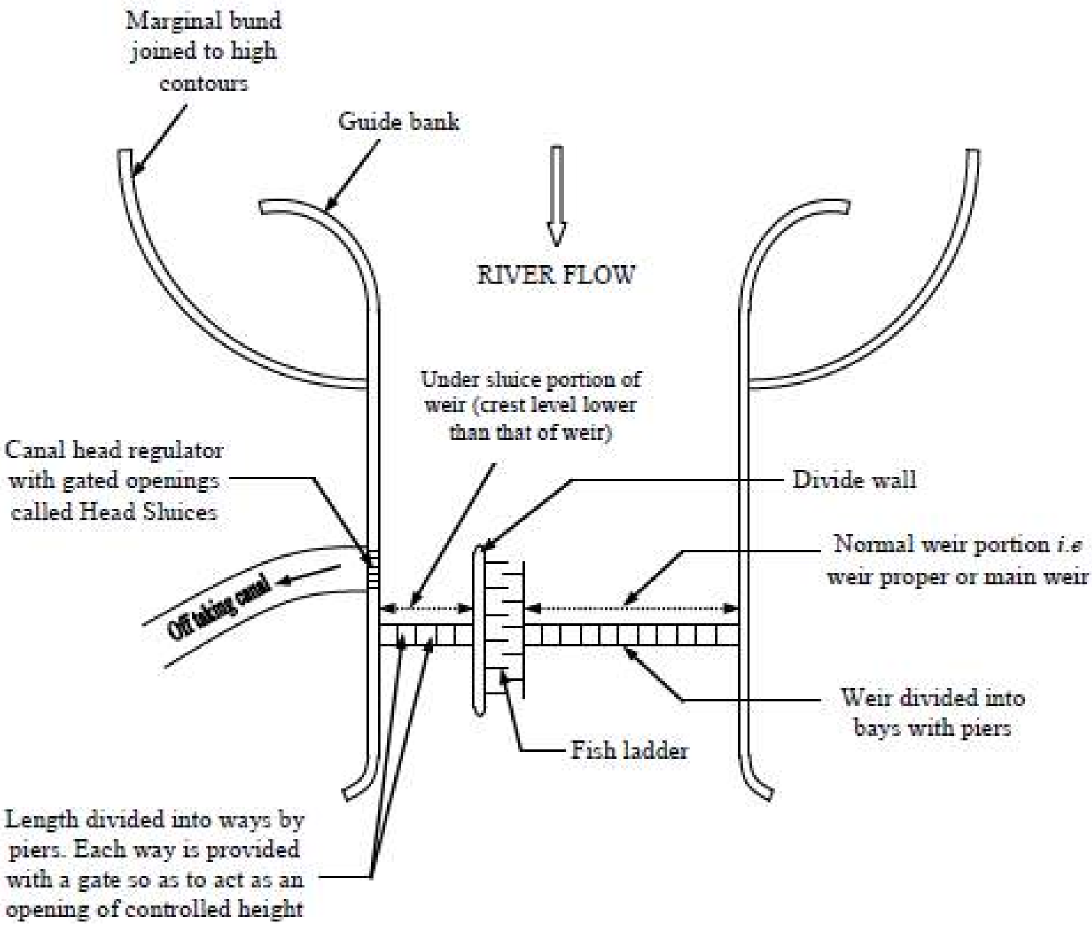


Fig. Typical Layout of Diversion Head-Works.

The Diversion Weir and its Types

The weirs may be divided into the following three classes:

(i) Masonry weirs with vertical drop ;

(ii) Rock-fill weirs with sloping aprons ; and

(iii) Concrete weirs with sloping glacis.

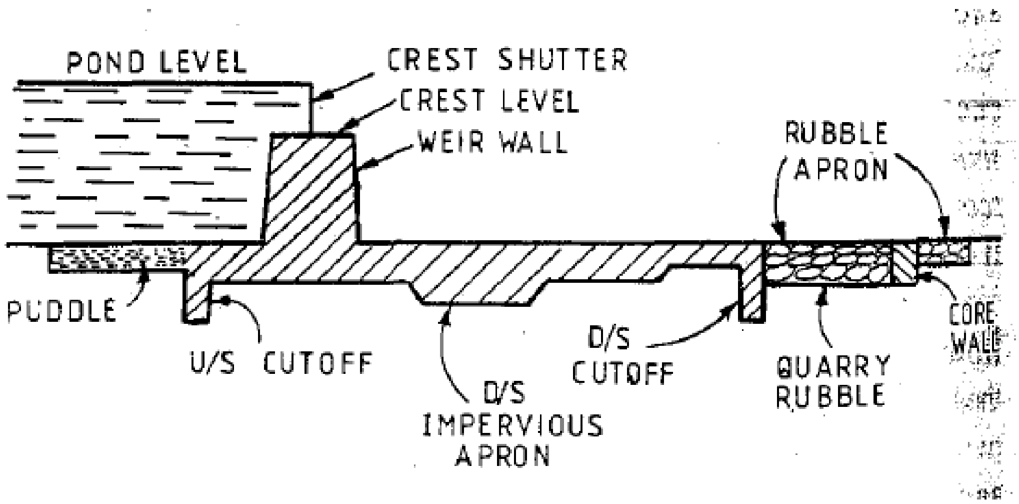


Fig. Masonry Weir

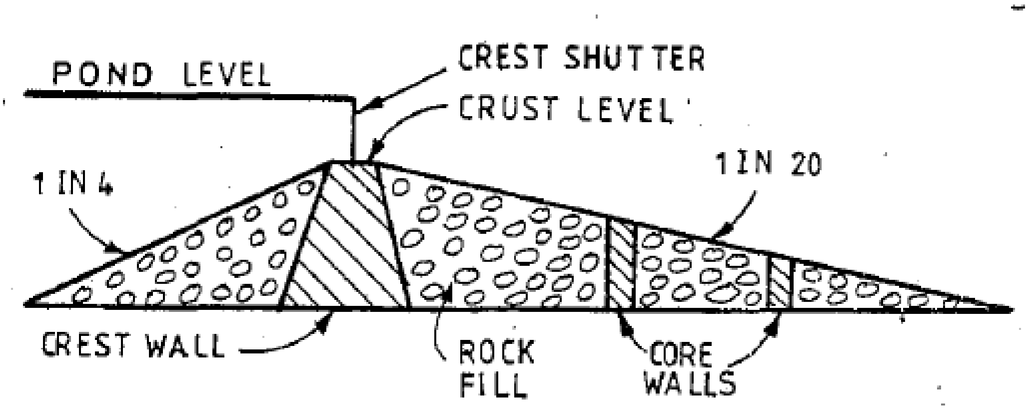


Fig. Rock-fill weir

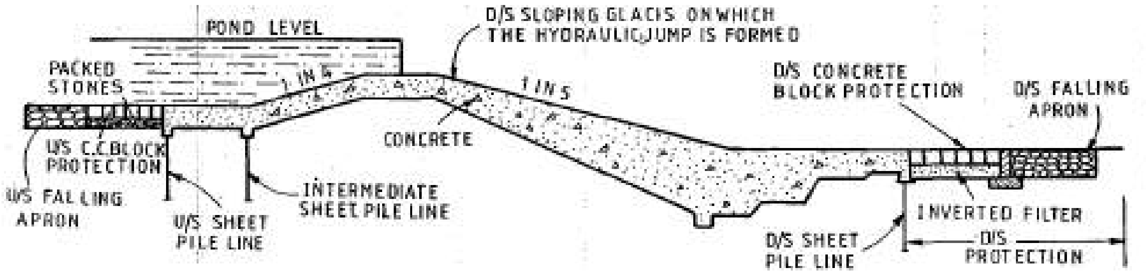
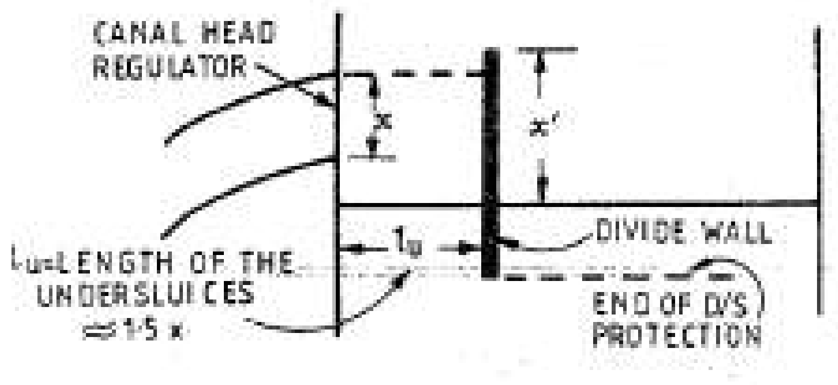


Fig. Concrete weirs with sloping glacis

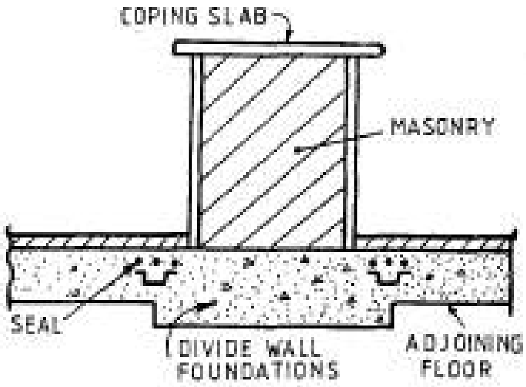
(2) Under-sluices

A comparatively less turbulent pocket of water is created near the canal head regulator by constructing under-sluice portion of the weir. A divide wall separates the main weir portion from the under-sluice portion of the weir. *The crest of the under-sluice portion "of the weir is kept at a lower level than, the crest of the normal portion of the weir.* Normally, the crest level of the under-sluices is kept equal to the deepest bed level of the river during non-monsoon season; whereas, the crest level of the 'weir' is kept higher by about 1 to 1.5 m.

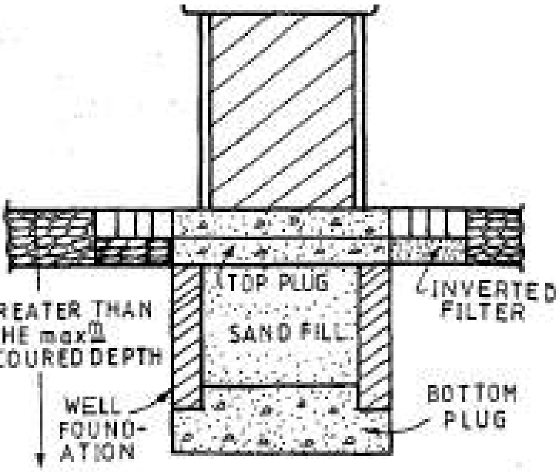


(3) Divide wall

The 'divide wall' is a masonry or a concrete wall constructed at right angle to the axis of the weir, and separates the 'weir proper' from the 'under-sluices'. The divide wall extends on the upstream side beyond the beginning of the canal head regulator; and on the downstream side, it extends up to the end of loose protection of the under sluices. The top width of divide wall is about 1.5 to 2.5 metres. These walls are founded on wells closely spaced beyond, the pucca floor upto the end. The wells are taken well below the deepest possible scour. Typical cross-section of the divide wall on pucca floor and beyond the pucca floor are shown in Fig. *(a)* and *(b)*.



a) Cross-section of Divide Wall on Pucca floor



b) Cross-section of Divide Wall beyond Pucca floor.

(4) River training works

River training works are required near the weir site in order to ensure a smooth and an axial flow of water, and thus, to prevent the river from outflanking the works due to a change in its course. The river training work required on a canal headworks, are

(i) *Guide banks*

(ii) Marginal bunds and

(iii) Spurs or groynes.

(i) Guide Bank

When a barrage is constructed across a river which flows through the alluvial soil, the guide banks must be constructed on both the approaches to protect the structure from erosion.

Guide bank serves the following purposes:

It protects the barrage from the effect of scouring and erosion.

It provides a straight approach towards the barrage.

It controls the tendency of changing the course of the river.

It controls the velocity of flow near the structure.

(ii)Marginal Bunds

The marginal bunds are earthen embankments which are constructed parallel to the river bank on one or both the banks according to the condition. The top width is generally 3 m to 4 m. The side slope on the river side is generally 1.5: 1 and that on the country side is 2:1.

The marginal bunds serve the following purposes:

It prevents the flood water or storage water from entering the surrounding area which may be submerged or may be water logged.
It retains the flood water or storage water within a specified section.
It protects the towns and villages from devastation during the heavy flood.
It protects valuable agricultural lands.

(iii)Spurs or groynes

These are temporary structures permeable in nature provided on the curve of a river to protect the river bank from erosion. These are projected from the river bank towards the bed making angles 60^{o} to 75^{o} with the bank of the river. The length of the spurs depends on the width of the river and the sharpness of the curve. The function of the spurs is to break the velocity of flow and to form a water pocket on the upstream side where the sediments get deposited. Thus the reclamation of land on the river bank can be achieved. *The spurs may be of the following types*:

Bamboo Spur
Timber Spur
Boulder Spur

(5) Fish Ladder

Large rivers are generally inhabited by several types of fish, many of which are migratory. Such migratory type of-fish ca1led-anadromous *fish,* move from one part of the river to another part, according to the season. In India, only one such migratory fish is found, and this. Specie is known as Hilsa. *Salman, Steel head trout,* etc. are the other species of such anadromous fish, found in other countries.

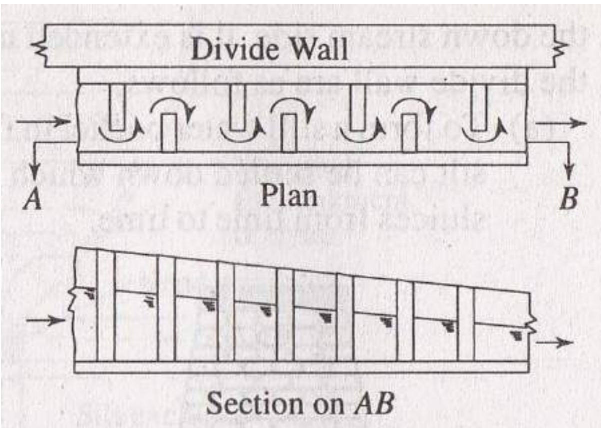


Fig: Fish Ladder

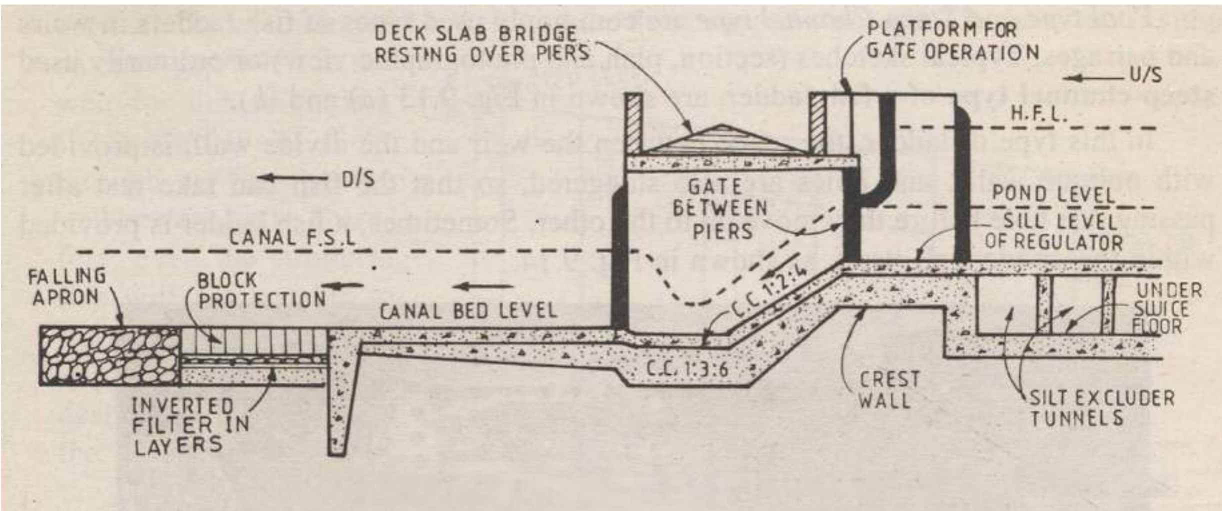
(6) Canal Head Regulator

A canal head regulator (C.H.R.) is provided at the head of the off-taking canal, and serves the following functions:

(i) It regulates the supply of water entering the canal.

(ii) It controls the entry of silt in the canal.

(iii) It prevents the river floods from entering the canal.



(8) Silt Regulation Works

The entry of silt into a canal, which takes off from a Head-Works, can be reduced by constructing, certain special Works, called silt control works. These works may be classified into the following two types ;

(a) **Silt Excluders**. Silt excluders are those works which are *constructed on the bed of the river,* upstream of the head regulator. The clearer water enters the head regulator and the silted water enters the silt excluder. In this type of works, the silt is, therefore, removed from the water before it enters the canal.

(b) **Silt Ejectors.** Silt ejectors, also called silt extractors, are those devices which extract the silt from the canal-water after the silted water has travelled a certain distance in the off-take canal. These works are, therefore, constructed on the bed of the canal, and a little distance downstream from the head regulator:

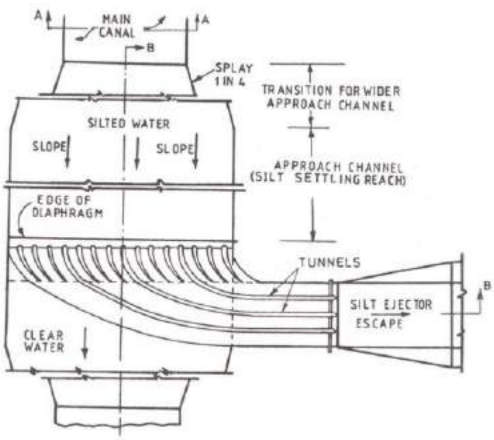


Fig: Plan of Silt Ejector

Bligh's Creep Theory for Seepage Flow

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

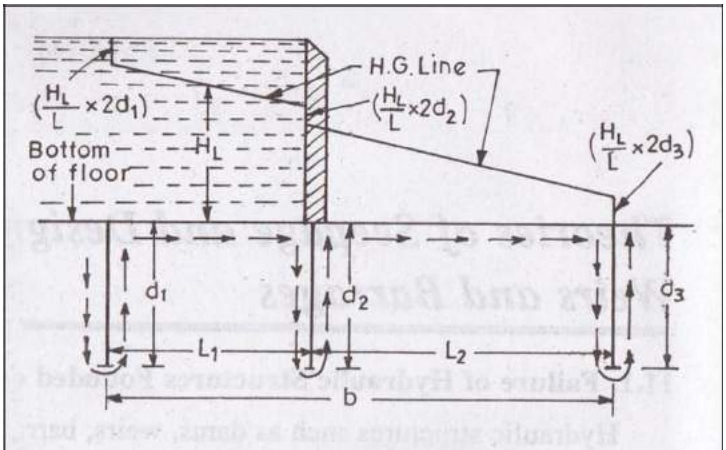


Fig-1: Bligh's Creep

Consider a section a 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 = $\left[\dfrac{H_L}{b + 2(d_1 + d_2 + d_3)}\right] = \dfrac{H_L}{L}$

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

(i) Safety against piping or undermining:

According to Bligh, the safety against piping can be ensured by providing sufficient creep length, given by $L = C.H_L$, where C is 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. H_L/L is then equal to 1/C. Hence, it may be stated that the hydraulic gradient must be kept under a safe limit in order to ensure safety against piping.

(ii) Safety against uplift pressure:

The ordinates of the H.G line above the bottom of the floor represent the residual uplift water head at each point. Say for example, if at any point, the ordinate of H.G line above the bottom of the floor is 1 m, then 1 m head of water will act as uplift at that point. If h′ 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,

$$\therefore \gamma_w \times h' = \gamma_w \times G.\, t$$

$$h' = G \times t$$

Subtracting t on both sides, we get

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

$$\Rightarrow t = \left(\frac{h' - t}{G - 1}\right) = \left(\frac{h}{G - 1}\right)$$

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

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.

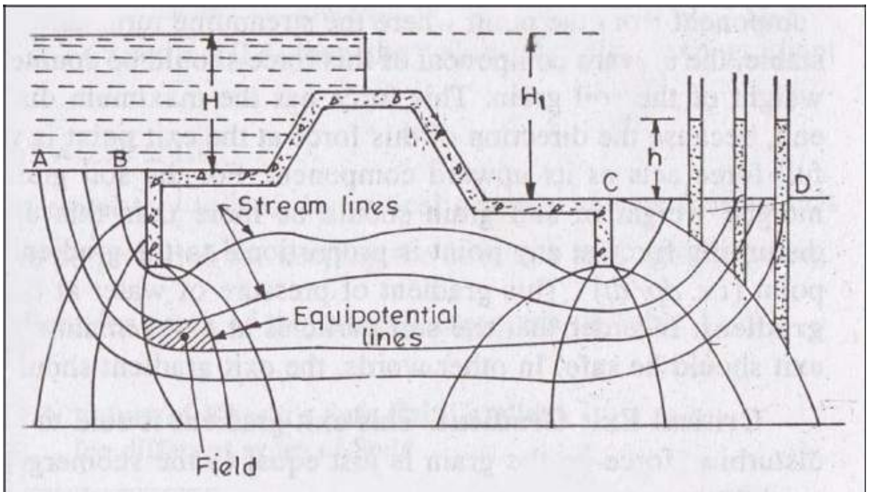


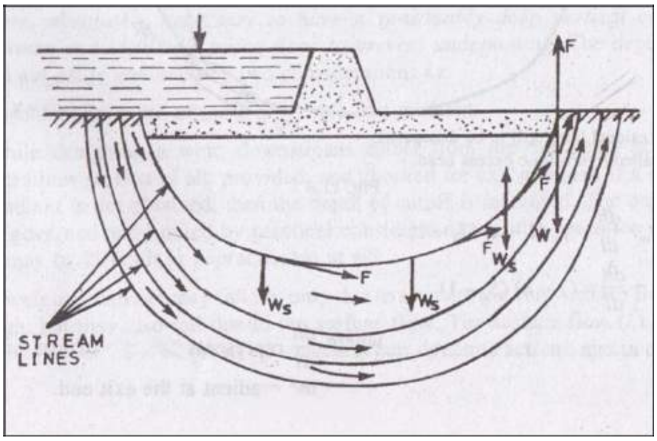
Fig: Khosla's Flow Net

Equipotential Lines: (1) Treating the downstream bed as datum and assuming no water on the downstream side, it can be easily started that every streamline possesses a head equal to h_1 while entering the soil; and when it emerges at the down-stream end into the atmosphere, its head is zero. Thus, the head h_1 is entirely lost during the passage of water along the streamlines.

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

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

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:

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

Where, γ_w = unit weight of water.
S_s = Specific gravity of soil particles
n = Porosity of the soil material

For critical conditions to occur at the exit point

$$F = W_s$$

Where F is the upward disturbing force on the grain
Force F = pressure gradient at that point = $dp/dl = \gamma_w \times dh/dl$

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 flow net. In other words, we must solve the *Laplacian* equations. This can be accomplished either by mathematical solution of the Laplacian equations, or by 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 (11.5) and curves given in Plate (11.1), which can be used for determining the percentage pressures at the various key points. The simple profiles which 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:

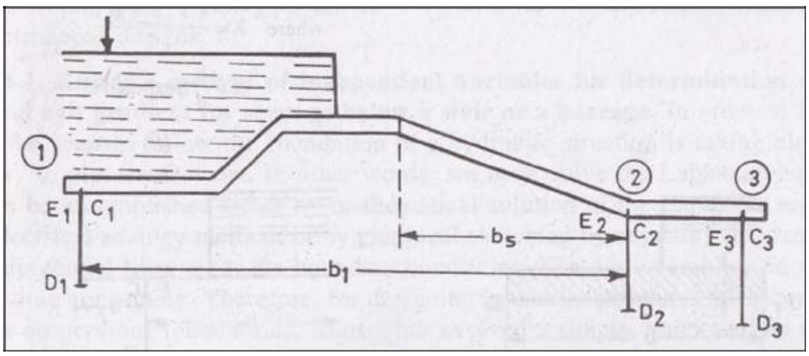
The correction *C* to be applied as percentage of head due to this effect, is given by

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

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

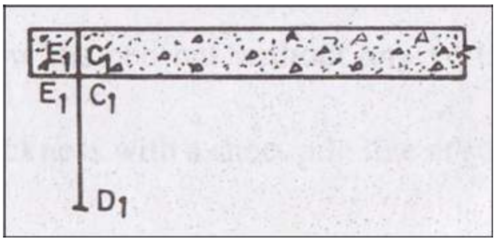
The correction is positive for the points in the rear of back water, and subtractive for the points forward in the direction of flow. This equation does not apply to the effect of an outer pile on an intermediate pile, if the intermediate pile is equal to or smaller than the outer pile and is at a distance 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 E_1 should be less than the calculated pressure at E_1', the correction to be applied for the joint E_1 shall be negative. Similarly, the pressure calculated C'_1 is less than the corrected pressure at C_1, and hence, the correction to be applied at point C_1 is 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. are tabulated in Table 7.4

Slope (H : V)	Correction Factor
1 : 1	11.2
2 : 1	6.5
3 : 1	4.5
4 : 1	3.3
5 : 1	2.8
6 : 1	2.5
7 : 1	2.3
8 : 1	2.0

The correction factor given above is to be multiplied by the horizontal length of the slope and divided by the distance between the two pile lines between which the sloping floor is located. This correction is applicable only to the key points of the pile line fixed at the start or the end of the slope.

Exit gradient (G_E)

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

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

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

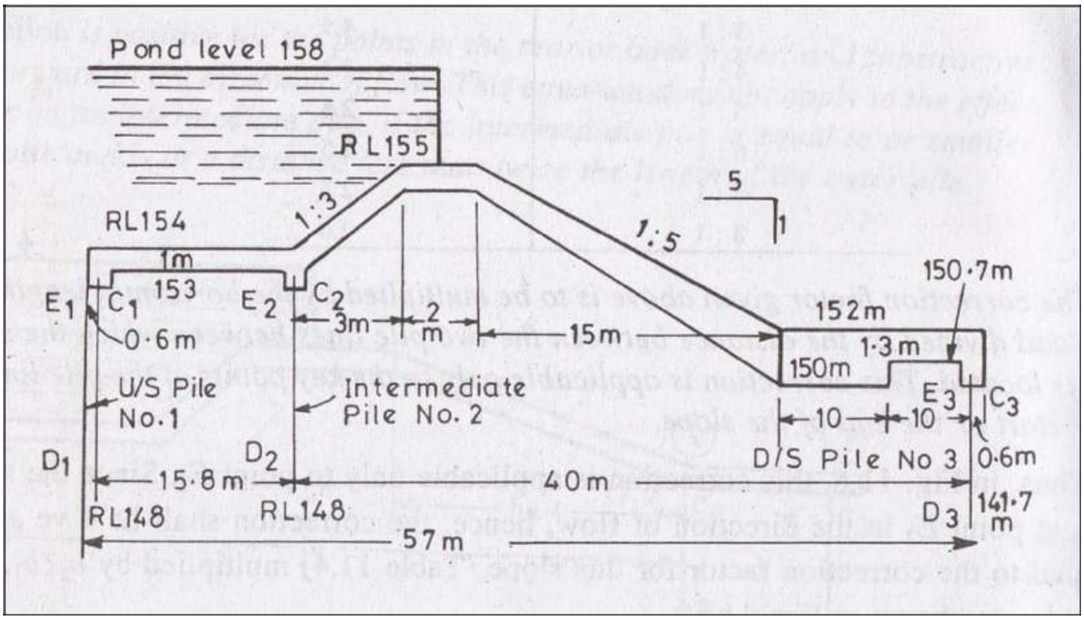
$\alpha = b/d$

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)

Problem-2

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*:**

(1) For upstream Pile Line No. 1

Total length of the floor, b = 57.0 m

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

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

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

From curve plate 11.1 (a)

$\varphi_{C1} = 100 - 29 = 71\ \%$

$\varphi_{D1} = 100 - 20 = 80\ \%$

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

Corrections for φ_{C1}

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

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

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

$$= 1.88\ \%$$

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

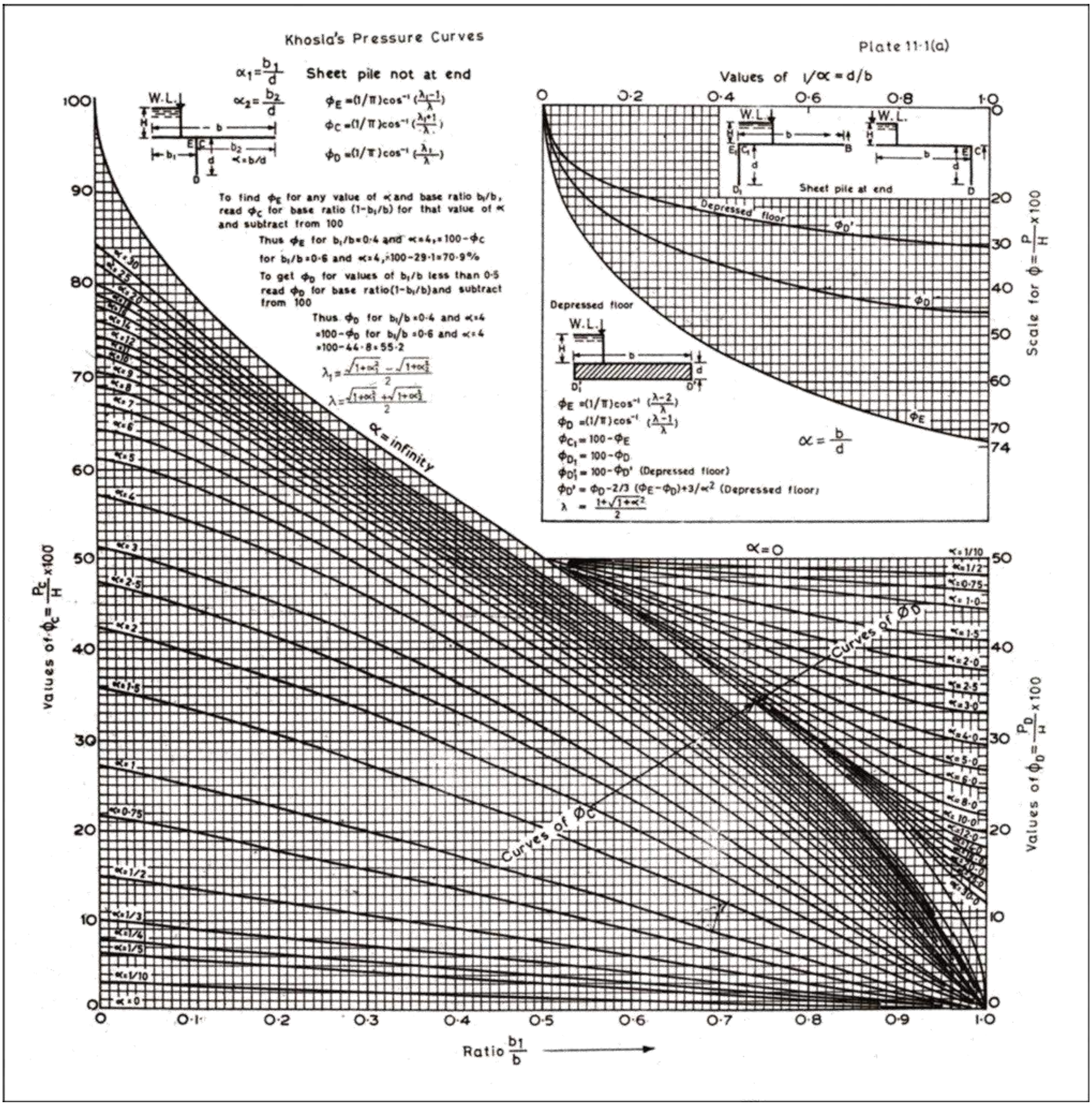
d = Depth of pile No. 1 = 153 – 148 = 5 m

b′ = Distance between two piles = 15.8 m

b = Total floor length = 57 m

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

∴ Correction due to pile interference on C_1 = 1.88 % (+ ve)


Khosla's Pressure Curves
Plate 11·1(a)
Sheet pile not at end
To find ϕ_E for any value of α and base ratio b_1/b, read ϕ_C for base ratio $(1-b_1/b)$ for that value of α and subtract from 100
Thus ϕ_E for $b_1/b=0.4$ and $\alpha=4$, $=100-\phi_C$ for $b_1/b=0.6$ and $\alpha=4$, $=100-29.1=70.9\%$
To get ϕ_D for values of b_1/b less than 0·5 read ϕ_D for base ratio $(1-b_1/b)$ and subtract from 100
Thus ϕ_D for $b_1/b=0.4$ and $\alpha=4$ $=100-\phi_D$ for $b_1/b=0.6$ and $\alpha=4$ $=100-44.8=55.2$
Values of $1/\alpha = d/b$
Sheet pile at end
Depressed floor
$\alpha = \frac{b}{d}$
α = infinity
α=0
Curves of ϕ_C
Curves of ϕ_D
Values of $\phi_C = \frac{P_C}{H} \times 100$
Values of $\phi_D = \frac{P_D}{H} \times 100$
Scale for $\phi = \frac{P}{H} \times 100$
Ratio $\frac{b_1}{b}$

(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

$$= \left[\frac{80\% - 71\%}{154 - 148}\right] \times (154 - 153)$$

$= (9/6)\times 1$

$= 1.5\%$ (+ ve)

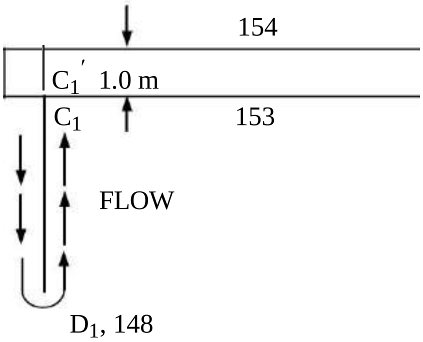


Fig: 5.1

(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

$\therefore$ slope Corrected (φ_{C1}) = 71 % + 1.88 % + 1.5 %

= 74.38 % (ans)

And (φ_{D1}) = 80 %

(2) For intermediate Pile Line No.

2 d = 154 – 148 = 6

m b = 57 m

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

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

$b_1 = 0.6 + 15.8 = 16.4$

b = 57 m

$\therefore b_1/b = 16.4/57 = 0.298$ (for φ_{C2})

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

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

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

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

Corrections for φ_{E2}

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

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

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

$= 1.88\%$ (– 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

$$= \frac{\text{Obs}\,\varphi_{E2} - \text{Obs}\,\varphi_{D2}}{\text{Distance between } E_2 D_2} \times \text{Thickness of floor}$$

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

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,

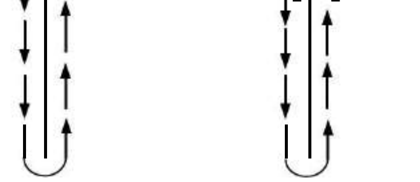


Fig: 5.2

$\therefore$ Correction at E_2 due to floor thickness = 1.17 % (- ve)

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

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

Corrections for φ_{C2}

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

o 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)

p 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 %

p 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

φ_{D3} = 26 %

φ_{E3} = 38 %

Corrections for φ_{E3}

p Correction due to piles. The point E_3 is affected by pile No. 2, and since E_3 is in the forward direction of flow from pile No. 3, this correction is negative and its amount is given by

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

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

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

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

(1) $\left[\frac{38\%\_32\%}{152-141.7}\right]\times 1.3 = (16/10.3)\times 1.3$

(2) 0.76 % (– ve)

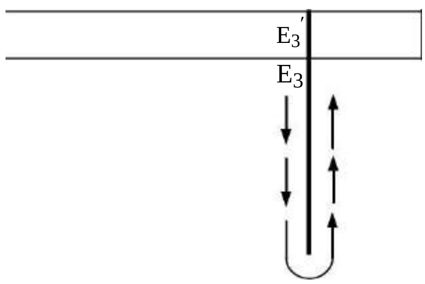


Fig:5.3

- *Correction due to slope at* E_3 is nil, as the point E_3 is neither situated at the start nor at the end of any slope

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

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

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

Exit gradient

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

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

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

Total floor length, b = 57 m

o = b/d = 57/10.3 = 5.53

For a value of α = 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.

$\frac{o}{\pi\sqrt{\lambda}} = 0.18$

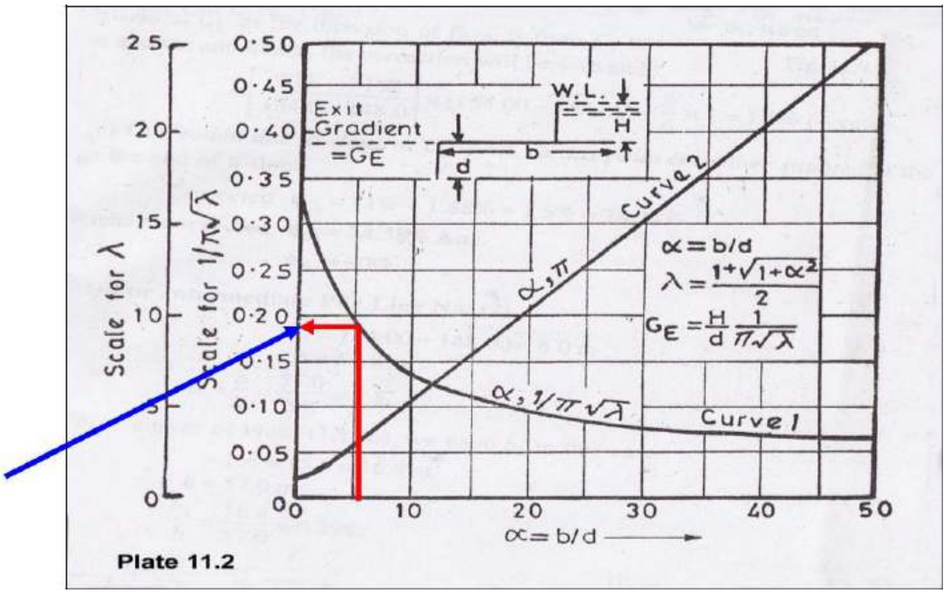


Plate 11.2

Module 4

Cross Drainage Works

Introduction

In an irrigation project, when the network of main canals, branch canals, distributaries, etc. are provided, then these canals may have to cross the natural drainages like rivers, streams, nallahs, etc at different points within the command area of the project. The crossing of the canals with such obstacle cannot be avoided. So, suitable structures must be constructed at the crossing point for the easy flow of water of the canal and drainage in the respective directions. These structures are known as ***cross-drainage works***.

<u>Necessity of Cross-drainage works</u>:

- The water-shed canals do not cross natural drainages. But in actual orientation of the canal network, this ideal condition may not be available and the obstacles like natural drainages may be present across the canal. So, the cross drainage works must be provided for running the irrigation system.
- At the crossing point, the water of the canal and the drainage get intermixed. So, far the smooth running of the canal with its design discharge the cross drainage works are required.
- The site condition of the crossing point may be such that without any suitable structure, the water of the canal and drainage can not be diverted to their natural directions. So, the cross drainage works must be provided to maintain their natural direction of flow.

<u>Types of Cross-Drainage Works</u>:

(3) Type I (Irrigation canal passes over the drainage)

(a) Aqueduct
(b) Siphon aqueduct

(4) Type II (Drainage passes over the irrigation canal)

(a) Super passage
(b) Siphon super passage

(5) Type III (Drainage and canal intersection each other of the same level)

(a) Level Crossing
(b) Inlet and outlet

<u>Selection of type of cross-drainage works</u>

- Relative bed levels
- Availability of suitable foundation
- Economical consideration
- Discharge of the drainage
- Construction problems

Aqueduct

The aqueduct is just like a bridge where a canal is taken over the deck supported by piers instead of a road or railway. Generally, the canal is in the shape of a rectangular trough which is constructed with reinforced cement concrete. Sometimes, the trough may be of trapezoidal section.

An inspection road is provided along the side of the trough.

The bed and banks of the drainage below the trough is protected by boulder pitching with cement grouting.

- The section of the trough is designed according to the full supply discharge of the canal.
- A free board of about 0.50 m should be provided.
- The height and section of piers are designed according to the highest flood level and velocity of flow of the drainage.
- The piers may be of brick masonry, stone masonry or reinforced cement concrete.
- Deep foundation (like well foundation) is not necessary for the piers. The concrete foundation may be done by providing the depth of foundation according to the availability of hard soil.

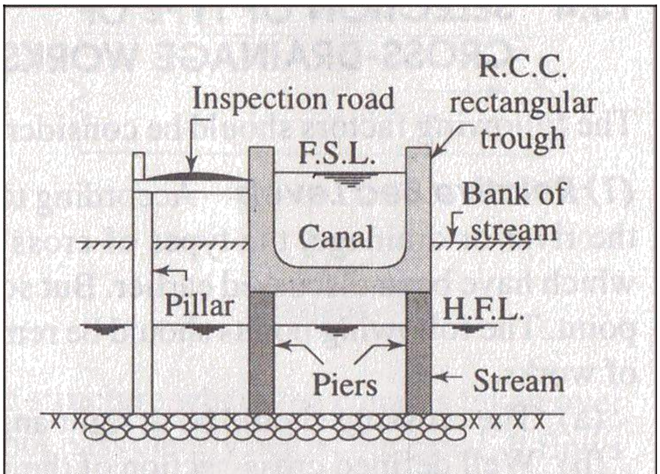


Fig: Aqueduct

Siphon Aqueduct

The siphon aqueduct, the bed of the drainage is depressed below the bottom level of the canal trough by providing sloping apron on both sides of the crossing.

- The sloping apron may be constructed by stone pitching or cement concrete.
- The section of the drainage below the canal trough is constructed with cement concrete in the form of tunnel. This tunnel acts as a siphon.
- Cut off walls are provided on both sides of the apron to prevent scouring.
- Boulder pitching should be provided on the upstream and downstream of the cut-off walls.
- The other components like canal trough, piers, inspection road, etc. should be designed according to the methods adopted in case of aqueduct.

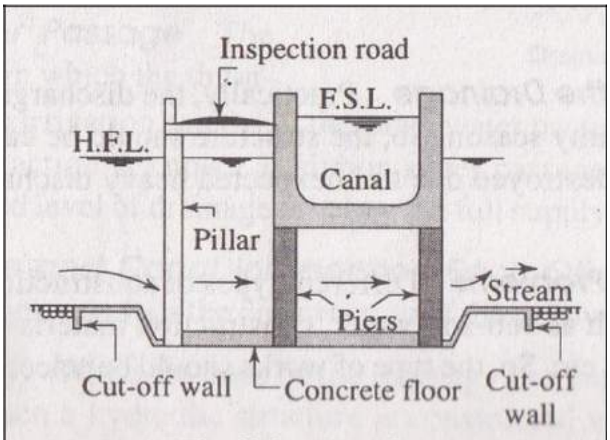


Fig: Siphon Aqueduct

Super Passage

The super passage is just opposite of the aqueduct. In this case, the bed level of the drainage is above the fully supply level of the canal. The drainage is taken through a rectangular or trapezoidal trough of channel which is constructed on the deck supported by piers.

- The section of the drainage trough depends on the high flood discharge.
- A free board of about 1.5 m should be provided for safety.
- The trough should be constructed of reinforced cement concrete.
- The bed and banks of the canal below the drainage trough should be protected by boulder pitching or lining with concrete slabs.
- The foundation of the piers will be same as in the case of aqueduct.

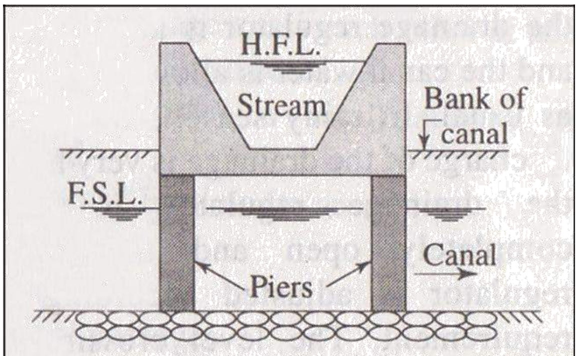


Fig: Super Passage

Siphon Super Passage

It is just opposite siphon aqueduct. In this case, the canal passes below the drainage trough. The section of the trough is designed according to high flood discharge. The bed of the canal is depressed below the bottom level of the drainage trough by providing sloping apron on both sides of the crossing.

- The sloping apron may be constructed with stone pitching or concrete slabs.
- The section of the canal below the trough is constructed with cement concrete in the form of tunnel which acts as siphon.
- Cut-off walls are provided on upstream and downstream side of sloping pron.
- Other components are same as in the case of siphon aqueduct.

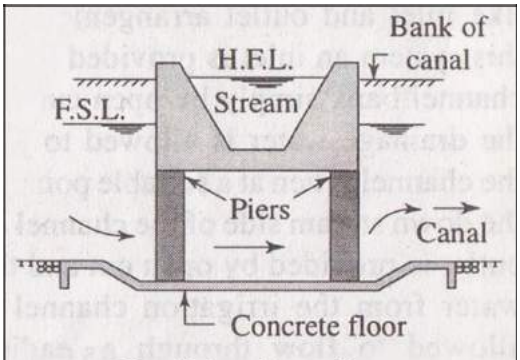


Fig: Siphon Super Passage

Level Crossing

The level crossing is an arrangement provided to regulate the flow of water through the drainage and the canal when they cross each other approximately at the same bed level. The level crossing consists of the following components:

- **Crest Wall:** It is provided across the drainage just at the upstream side of the crossing point. The top level of the crest wall is kept at the full supply level of the canal.
- **Drainage Regulator:** It is provided across the drainage just at the downstream side of the crossing point. The regulator consists of adjustable shutters at different tiers.
- **Canal Regulator:** It is provided across the canal just at the downstream side of the crossing point. This regulator also consists of adjustable shutters at different tiers.

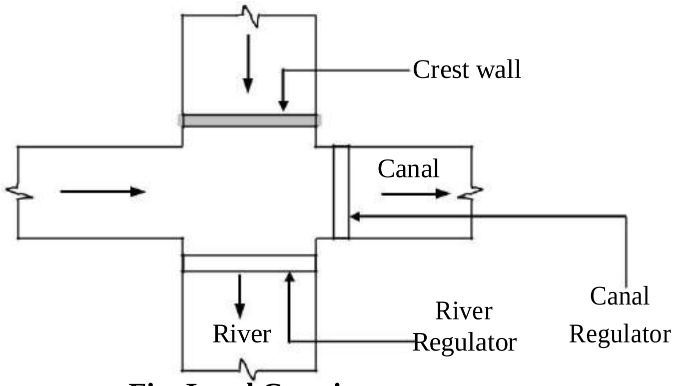


Fig: Level Crossing

Inlet and outlet

In the crossing of small drainage with small channel no hydraulic structure is constructed. Simple openings are provided for the flow of water in their respective directions. This arrangement is known as inlet and outlet.

- In this system, an inlet is provided in the channel bank simply by open cut and the drainage water is allowed to join the channel
- At the points of inlet and outlet, the bed and banks of the drainage are protected by stone pitching.

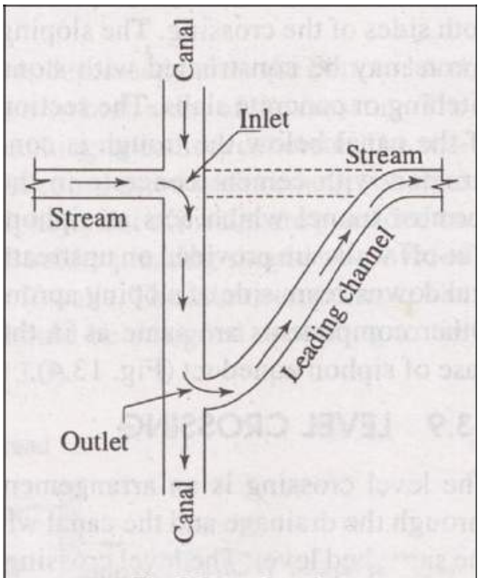


Fig: Inlet and outlet

Design considerations for C.D works

The following steps may be involved in the design of an aqueduct or a syphon-aqueduct.

1. Determination of Maximum Flood Discharge.

The high flood discharge for smaller drains may be worked out by using empirical formulas and for large drains, other reliable methods such as Hydrograph analysis, rational formula, etc. may be used.

2. Fixing the Waterway Requirements for Aqueducts and Syphon. Aqueducts.

An approximate value of required waterway for the drain may be obtained by using the Lacey's equation, given by

$P=4.75*\sqrt{Q}$

Where P= is the wetted perimeter in metres

Q = Total discharge in cumecs.

3. Afflux and Head Loss through Syphon Barrels.

It was stated earlier that the velocity through syphon barrels is limited to a scouring value of about 2 to 3 m/sec. A higher velocity may cause quick abrasion of the barrel surfaces by rolling grit, etc. and shall definitely result in higher amount of afflux on the upstream side of the siphon or syphon-aqueduct, and thus, requiring higher anJ longer marginal banks. The head loss *(h)* through syphon barrels and the velocity *(V)* through them are

generally related by Unwin's formula*, given as :

$$h = \left[1 + f_1 + f_2 \frac{L}{R}\right] \frac{V^2}{2g} - \frac{V_a^2}{2g} \quad \text{...(14.1)}$$

where L = Length of the barrel.

R = Hydraulic mean radius of the barrel.

V = Velocity of flow through the barrel.

V_a = Velocity of approach and is often neglected.

f_1 = Coefficient of head loss at entry

= 0.505 for unshaped mouth

= 0.08 for bell mouth.

f_2 = is a coefficient such that the loss of head through the barrel due to surface friction is given by

$f_2 \cdot \frac{L}{R} \cdot \frac{V^2}{2g}$; where f_2 is given as :

$$f_2 = a\left(1 + \frac{b}{R}\right) \quad \text{...(14.2)}$$

where the values of a and b for different materials may be taken as given in Table 14.1.

Table 14.1

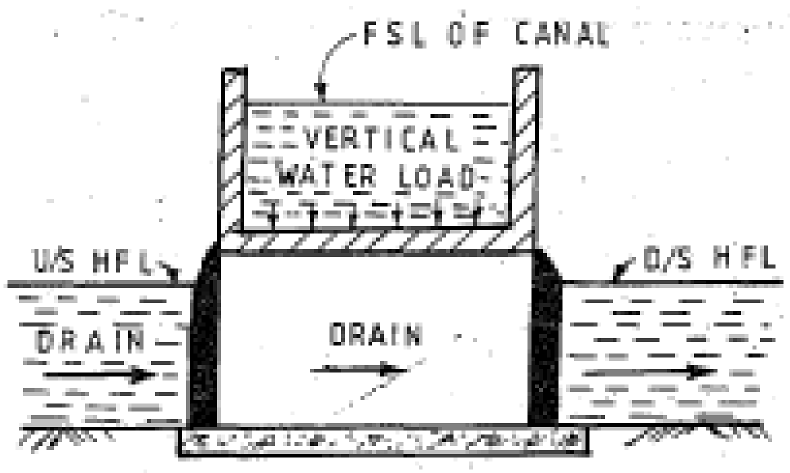
Material of the surface of barrel	*a*	*b*
Smooth iron pipe	0.00497	0.025
Encrusted pipe	0.00996	0.025
Smooth cement plaster	0.00316	0.030
Ashlar or brick work	0.00401	0.070
Rubble masonry or stone pitching	0.00507	0.250

4. Fluming of the Canal.

The contraction in the waterway of the canal *(i.e.* fluming of the canal) will reduce the length of barrels or the width of the aqueduct. This is likely to produce economy in many cases. The fluming of the canal is generally not done when the canal section is in earthen banks. Hence, the canal is generally riot flumed in works of Type I and Type II. However, fluming is generally done in all the works of Type III.

5. Design of Pucca Canal Trough.

For an Aqueduct, In case of an aqueduct, the bottom of the canal *i.e.* the roof of the culvert is subjected to the dead weight and the vertical load of water from the top, as shown in Fig.

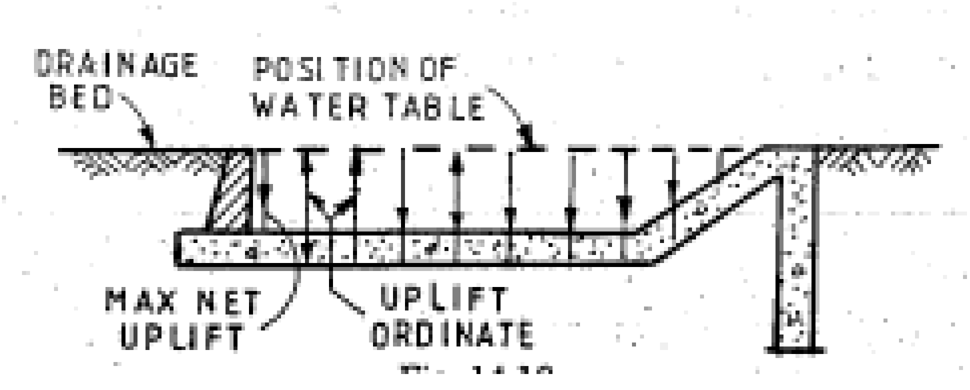


6. Design of Bottom Floor of Aqueduct and Syphon Aqueduct.

The floor of the aqueduct or syphon-aqueduct is subjected to uplift due to two causes:

(a) *Uplift due to water-table.* This force acts where the bottom floor is depressed below the drainage bed, especially in syphon aqueducts.

(b) *Uplift due to seepage of water from the canal to the drainage*. The maximum uplift due to this seepage occurs when the canal is running full and there is no water in the drain.



7. Design of Bank Connections.

Two set of wings are required in aqueducts and syphon-aqueducts. These are :

(i) Canal Wings or Land Wings.

(ii) Drainage Wings or Water Wings.

(i) Canal wings or Land wings. These wings provide a strong connection between the masonry or concrete sides of a canal trough and earthen canal banks. These wings are generally warped in plan so as to change the canal section from trapezoidal to rectangular.

(ii) Drainage wings or Water wings or River wings. These wing walls retain and protect the earthen slopes of the canal, guide the drainage water entering and leaving · the work, and join it to guide banks and also provide a vertical cut-off for the water seeping from the canal into the drainage bed.

Transition formula design of protection works

The following methods may be used for designing the channel transitions:

(i) Mitra's method of design of transitions· (~hen water depth remains constant).

(ii) chaturvedi's method of design of transitions (when water depth remains constant).

(iii) Rind's method of design of transitions (when water depth may or may not vary).

(i) Mitra's Hyperbolic Transition when water depth remains constant. Shri A.C. Mitra, Chief Engineer, U.P. Irrigation Deptt. (Retd.), has proposed a hyperbolic transition for the design of channel transitions. According to him, the channel width at any section *X-X*, at a distance *x* from the flumed section (Fig.) is given by

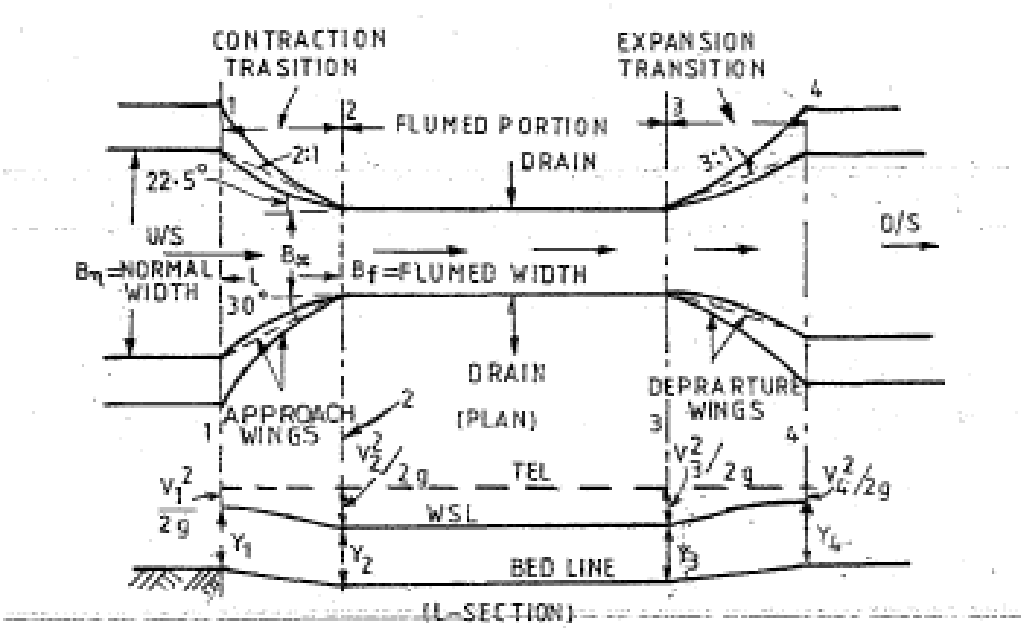
$$B_x = \frac{B_n \cdot B_f \cdot L_f}{L_f B_n - (B_n - B_f)\, x} \qquad \text{...(14.3)}$$

where B_n = Bed width of the normal channel section.

B_f = Bed width of the flumed channel section.

B_x = Bed width at any distance x from the flumed section.

L_f = Length of transition.



(ii} Chaturvedi's Semi Cubical pararabolic transition when water depth remains constant. Pz:of. R.S. Chaturvedi, Head of Civil engineering Deptt. in Roorkee Univer. sity (Retd.), on the basis of his own experiments, had in 1963, proposed the. Following equation for the design of channel transitions when water depth remains constant

$$x = \frac{L \cdot B_n^{3/2}}{B_n^{3/2} - B_f^{3/2}} \left[1 - \left(\frac{B_f}{B_x} \right)^{3/2} \right]$$

Choosing various convenient-values of Bx the corresponding distance *x* can be computed easily from the above equation

(iii) Hind's Method for the design of Transitions when water depth may also vary. This is a general method and is applicable either when the depth in the flumed and unflumed portions is the same, or when these depths are different. · In Fig., the contraction transition *(i.e.* the approach transition) starts at section 1-1 and finishes at section 2-2. The flumed section continues from section 2-2 to section 3-3. The expansion transition starts at section: 3-3 and finishes at section 4-4. From section 4-4 onwards, the channel flows in its normal cross-section and the conditions at this section are completely known. Let V and y with appropriate subscripts refer to velocities and depths at different sections. ·

Design of only aqueduct

Design a suitable cross drainage work, given the following data at the crossing of a canal and a drainage.

Canal

Full supply discharge= 32 cumecs

Full supply level=R.L 213.5

Canal bed level=R.L. 212.0m.

Canal bed width=20.

Trapezoidal canal section with 1 f H: 1 V slopes.

Canal water depth = 1.5 m.

Drainage.

High flood discharge =300 cumecs.

High flood level =210.0m.

High flood depth = 2.5 m.

General ground level = 212.5 m.

Solution Since the drainage is of a large size, work of type. III will be adopted. Further, because the canal bed level (212.0 m) is much above the H.F.L. of drainage *(te.* 210.0 m) an aqueduct will be constructed. The earthen banks of the canal will be discontinued and the canal water taken in a: concrete trough. For effecting economy the canal shall be flumed.

Step 1. Design of Drainage Waterway

Lacey's regime perimeter= $P = 4{:}75 \sqrt{Q}$

where Q = High flood discharge of drain= 300 cumecs (given)

$P = 4.75 \sqrt{1300} = 82.3$ m.

Let the clear span between piers be 9 m and the pier thickness be 1.5 m.

Using 8 bays of 9 m each, clear waterway = 8 x 9 = 72 m.

Using 7 piers of 1.5 each, length occupied by piers= 7 x 1.5 = 10.5 m.

Total length of waterway= 72 + 10.5 = 82.5 m

Step 2. Design of Canal Waterway

Bed width of canal = 20.0 m.

Let the width be flumed to 10.0 m.

Providing a splay of 2: 1 in contraction, the length of contraction transition

$= \frac{20-10}{2} * 2 = 10.0\text{m}$

Providing a splay of 3 : l in expansion, the length of expansion transition

$= \frac{20-10}{2} * 3 = 15.0\text{m}$

Length of the flumed rectangular portion of the canal between abutments = 82.5 m (provided).

In transitions, the side slopes of the canal section' will be warped in plan from the original slope of 1.5: 1 to vertical

Step 3. Head loss and bed levels at different sections. (Fig. 14.20).

At Section 4-4

At section 4-4, where the canal returns to its normal section, we have

Area of trapezoidal canal section

$$= (B + 1.5y)\, y$$

$$= (20 + 1.5 \times 1.5)\, 1.5 = 22.5 \times 1.5 = 33.75 \text{ m}^2$$

$$\text{Velocity} = V_4 = \left(\frac{Q}{A}\right) = \frac{32}{33.75} = \mathbf{0.947\ m/sec}$$

$$\text{Velocity head} = \frac{V_4^2}{2g} = \frac{(0.947)^2}{2 \times 9.81} = 0.046 \text{ m}$$

R.L. of bed at 4-4 = **212.0 m** (given)

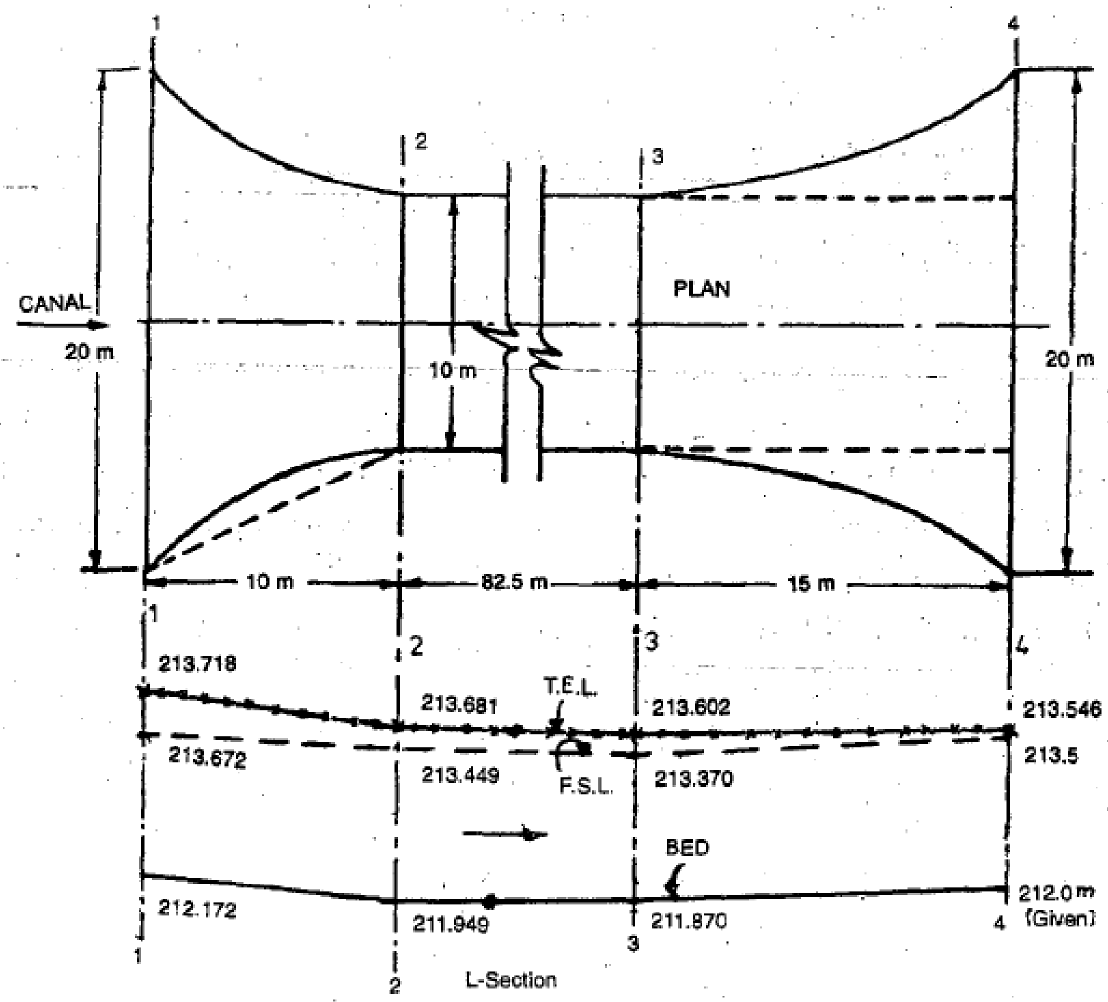


Fig. 14.20. Plan and Section of Canal Trough in Example 14.1.

R.L. of water surface at 4-4 $= 212.0 + 1.5 = \mathbf{213.5\ m}$

R.L. of T.E.L. at 4.4 $= 213.5 + 0.046 = 213.546$ m

The known condition of 4-4 shall now be utilised for finding the bed levels etc. at

At Section 3-3

Keeping the same depth of 1.5 m throughout the channel, we have at section 3.3 :

Bed width $= 10$ m

Area of channel $= 10 \times 1.5 = 15$ sq m

$$\therefore \text{Velocity} = V_3 = \frac{32}{15} = 2.13 \text{ m/sec}$$

$$\text{Velocity head} = \frac{V_3^2}{2g} = \frac{(2.13)^2}{2 \times 9.81} = 0.232 \text{ m}$$

Assuming that the loss of head in expansion from section 3-3 to section 4-4 is taken

$$= 0.3 \left[\frac{V_3^2 - V_4^2}{2g}\right]$$

$$= 0.3\,[0.232 - 0.046]$$

$$= 0.3 \times 0.186 = 0.0558 \text{ m ; say } \mathbf{0.056\ m}$$

R.L. of T.E.L. at section 3-3 = R.L. of T.E.L. at 4-4 + Loss in expansion

$= 213.546 + 0.056 = 213.602$ m

$\therefore$ R.L. of water surface at 3-3 = R.L. of T.E.L. at 3-3 – Velocity Head

$= 213.602 - 0.232 = \mathbf{213.370}$ **m**

R.L. of bed at 3.3

$= 213.370 - 1.5 = \mathbf{211.87}$ **m**

At Section 2-2

From section 2-2 to 3-3, the trough section is constant. Therefore, area and velocity at 2-2 are the same as at 3-3. But from 2-2 to 3-3, there is a friction loss between 2-2 and 3-3 which may be computed by Manning's formula as equal to

$$H_L = \frac{n^2 \cdot V^2 \cdot L}{R^{4/3}}.$$

where n is rugosity coefficient whose value in concrete trough may be taken as 0.016; and L is the length of trough = 82.5 m.

Area of trough section $(A) = 10 \times 1.5 = 15$ sq m

Wetted perimeter $(P) = 10 + 2 \times 1.5 = 13$ m

Hydraulic mean depth $(R) = \frac{A}{P} = \frac{15}{13} = 1.16$ m

Velocity in trough $= \frac{Q}{A} = \frac{32}{15} = 2.13$ m/sec

$$\therefore \quad H_L = \frac{(0.016)^2 \times (2.13)^2 \times 82.5}{(1.16)^{4/3}}$$

$= 0.0787$ m; say **0.079 m**

R.L. of T.E.L. at 2-2 = R.L. of T.E.L. at 3-3 + Friction loss in trough

$= 213.602 + 0.079 = 213.681$ m

R.L. of water surface at 2-2

$= 213.681 - 0.232 = \mathbf{213.449}$ **m**

R.L. of bed at 2-2

$= 213.449 - 1.5 = \mathbf{211.949}$ **m**

At Section 1-1

Loss of head in contraction transition from 1-1 to 2-2

$$= 0.2\left(\frac{V_2^2 - V_1^2}{2g}\right)$$

$$= 0.2\left[\frac{(2.13)^2 - (0.947)^2}{2 \times 9.81}\right]$$

$= 0.2\,[0.232 - 0.046] = \mathbf{0.037}$ **m**

R.L. of T.E.L. at 1-1 = R.L. of T.E.L. at 2-2 + Loss in contraction

$= 213.681 + 0.037 = \mathbf{213.718}$ **m**

R.L. of water surface at 1-1

$$= 213.718 - 0.046 = \mathbf{213.672\,m}$$

R.L. of bed at 1-1

$$= 213.672 - 1.5 = \mathbf{212.172\,m}$$

All the bed levels, F.S.L. and T.E.L. are plotted in Fig. 14.20.

Step 4. Design of Transitions

(*a*) *Contraction Transition.* Since the depth is kept constant, the transition can be designed on the basis of Mitra's Hyperbolic transition equation (14.2) given as :

$$B_x = \frac{B_n \cdot B_f L_f}{L_f B_n - x\,(B_n - B_f)}$$

where $B_f = 10$ m

$B_n = 20$ m

$L_f = 10$ m

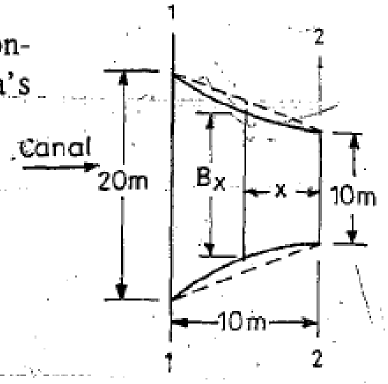


Fig. 14.21

Substituting we get

$$B_x = \frac{20 \times 10 \times 10}{10 \times 20 - x\,(20 - 10)} = \frac{2{,}000}{200 - 10x}$$

For various values of x lying between 0 to 10 m, various values of B_x are worked out, as shown below in Table 14.2. The distance x is measured from flumed section *i.e.* 2-2, as shown in Fig. 14.21.

Table 14.2

x in metres	0	2	4	6	8	10
$B_x = \frac{2{,}000}{200 - 10x}$ *in metres*	10.0	11.11	12.5	14.29	16.67	20.0

The contraction transition can be plotted with these values.

Expansion Transition. In this case $B_n = 20$ m, $B_f = 10$ m, and $L_f = 15$ m.

Using Eqn. (14.2), we get

$$B_x = \frac{B_n \cdot B_f \cdot L_f}{L_f \cdot B_n - x\,(B_n - B_f)}$$

$$= \frac{20 \times 10 \times 15}{15 \times 20 - x\,(20 - 10)} = \frac{3{,}000}{300 - 10x}.$$

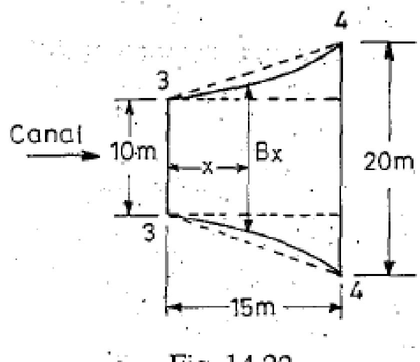


Fig. 14.22

For various values of x lying between 0 to 15 m, various values of B_x are worked out by using the above equation, as shown in Table 14.3.

Table 14.3

x *in metres*	0	2	4	6	8	10	12	14	15
$B_x = \frac{3{,}000}{300 - 10x}$ *in metres*	10.0	10.71	11.54	12.5	13.64	15.0	16.67	18.75	20.0

The expansion transition can be easily plotted with these values.

Step 5. Design of Trough

The trough shall be divided into two equal compartments of 5 m each and separated by an intermediate wall of 0.3 m thickness. The inspection road shall be carried on the top of left compartment as shown in Fig. 14.23.

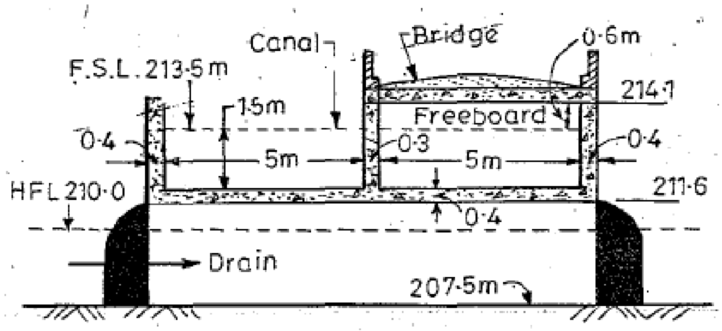


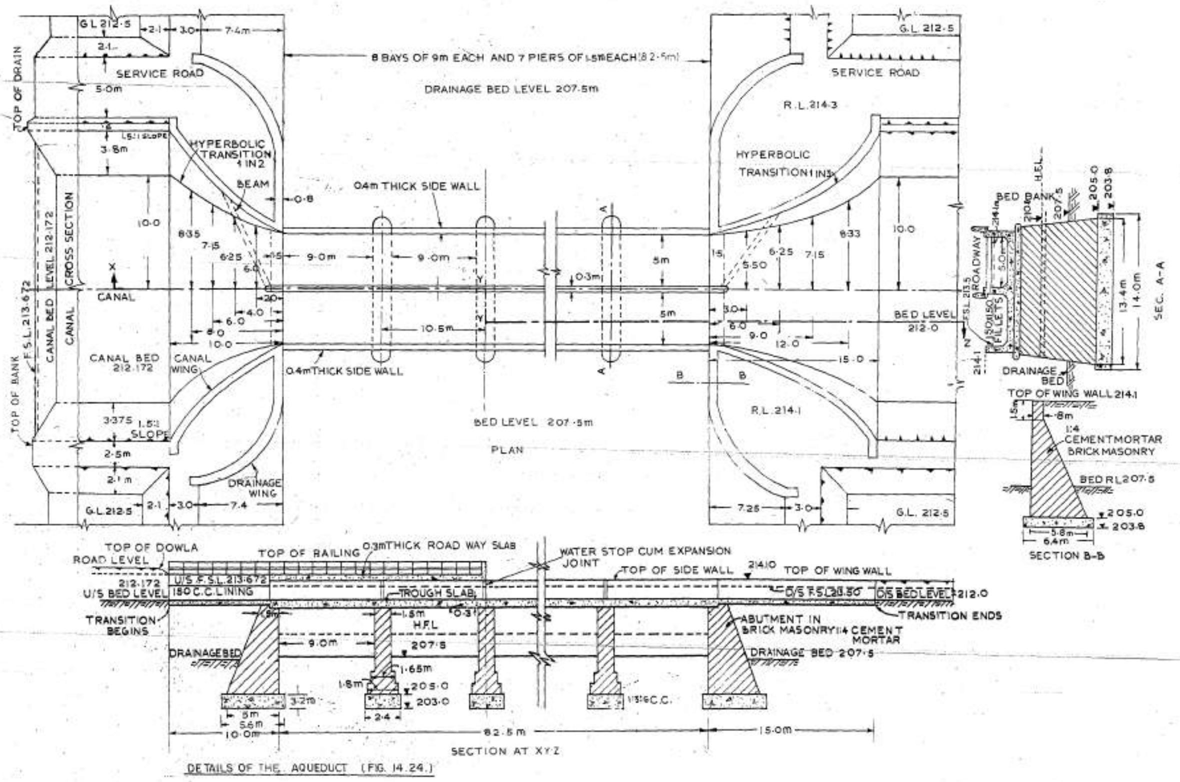
Fig. 14.23

A freeboard of 0.6 m above the normal water depth of 1.5 m is sufficient, and hence, the bottom level of bridge slab over the left compartment can be kept at $1.5+0.6=2.1$ m above the bed level of the trough. The height of the trough will, therefore, be kept equal to 2.1 m. The entire trough section will be constructed in monolithic reinforced concrete and can be designed by usual structural methods. The tentative thicknesses may be used as follows :

Outer walls = 0.4 m thick

Bottom slab of trough = 0.4 m thick

The intermediate partition wall is to be extended in the transitions so as to provide the necessary clear width of 10 m. The detailed drawing of the aqueduct is illustrated in attached chart Fig. 14.24.



Module 5

Canal Regulation Works

Introduction

The works which are constructed in order to control and regulate discharges, depths, velocities etc. in canals, are known as canal regulation works. These structures ensure the efficient functioning of a canal irrigation system, by giving full control upon the canals. The important of these structures are:

(i) Canal Falls.

(ii) Canal Regulators (Head Regulator and Cross Regulator).

(iii) Canal Escapes.

(iv) Metering Flumes, etc.

(v) Canal Outlets and Modules.

CANAL REGULATORS

A head regulator provided at the head of the off-taking channel, controls the flow of water entering the new channel.

While a cross regulator may be required in the main channel downstream of the off-taking channel, and is operated when necessary so as to head up water on its upstream side, thus to ensure the required supply in the off-taking channel even during the periods of low flow in the main channel.

Main functions of a head regulator:

- To regulate or control the supplies entering the off-taking canal
- To control the entry of silt into the off-taking canal
- To serve as a meter for measuring discharge.

Main functions of a cross regulator:

- To control the entire Canal Irrigation System.
- To help in heading up water on the upstream side and to fed the off-taking canals to their full demand.
- To help in absorbing fluctuations in various sections of the canal system, and in preventing the possibilities of breaches in the tail reaches.
- Cross regulator is often combined with bridges and falls, if required.

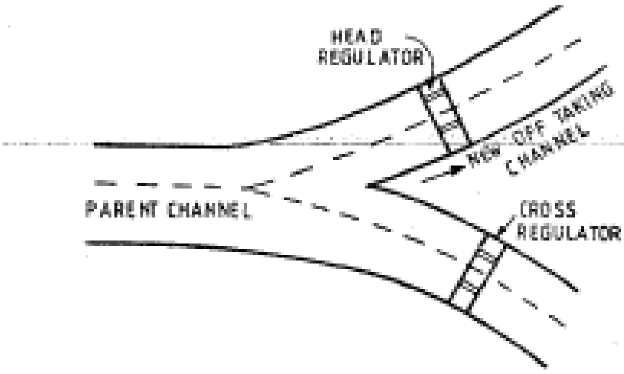


Fig. 13.1

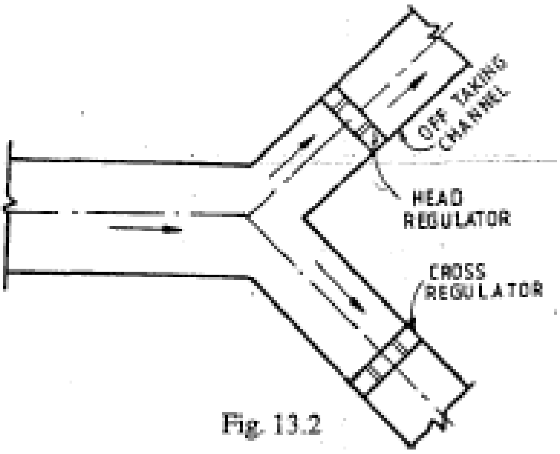


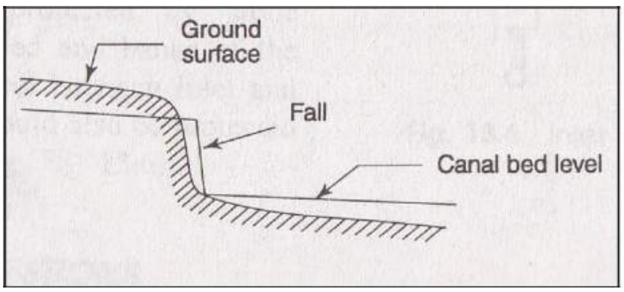
Fig. 13.2

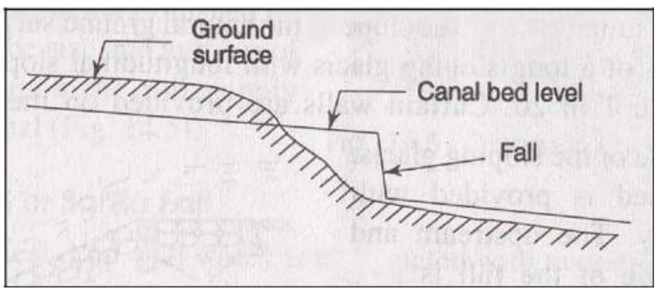
Canal falls

Irrigation canals are constructed with some permissible bed slopes so that there is no silting or scouring in the canal bed. But it is not always possible to run the canal at the desired bed slope throughout the alignment due to the fluctuating nature of the country slope. Generally, the slope of the natural ground surface is not uniform throughout the alignment. Sometimes, the ground surface may be steep and sometimes it ma be very irregular with abrupt change of grade. In such cases, a vertical drop is provided to step down the canal bed and then it is continued with permissible slope until another step down is necessary. This is done to avoid unnecessary huge earth work in filling. Such vertical drops are known as ***canal falls or simply falls***.

<u>Necessity of Canal Falls</u>:

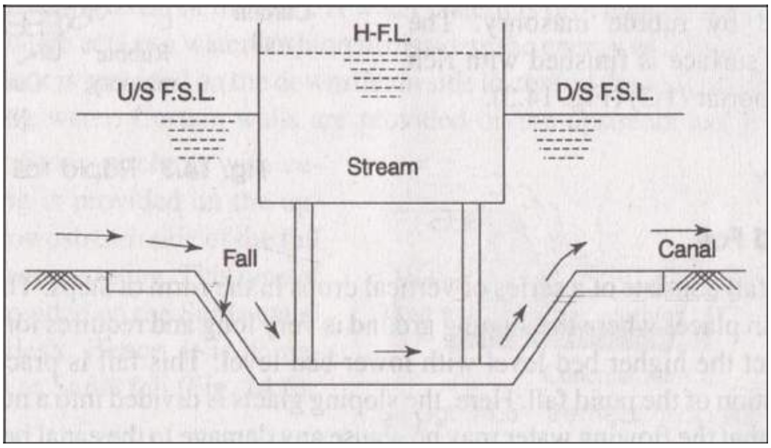
- When the slope of the ground suddenly changes to steeper slope, the permissible bed slope can not be maintained. It requires excessive earthwork in filling to maintain the slope. In such a case falls are provided to avoid excessive earth work in filling





- When the slope of the ground is more or less uniform and the slope is greater than the permissible bed slope of canal. In that case also the canal falls are necessary.

- In cross-drainage works, when the difference between bed level of canal and that of drainage is small or when the F.S.L of the canal is above the bed level of drainage then the canal fall is necessary to carry the canal water below the stream or drainage.



Types of Canal Falls

The following are the different types of canal falls that may be adopted according to the site condition:

Ogee Fall

In this type of fall, an ogee curve (a combination of convex curve and concave curve) is provided for carrying the canal water from higher level to lower level. This fall is recommended when the natural ground surface suddenly changes to a steeper slope along the alignment of the canal.

- The fall consists of a concrete vertical wall and concrete bed.
- Over the concrete bed the rubble masonry is provided in the shape of ogee curve.
- The surface of the masonry is finished with rich cement mortar (1:3).
- The upstream and downstream side of the fall is protected by stone pitching with cement grouting.
- The design consideration of the ogee fall depends on the site condition.

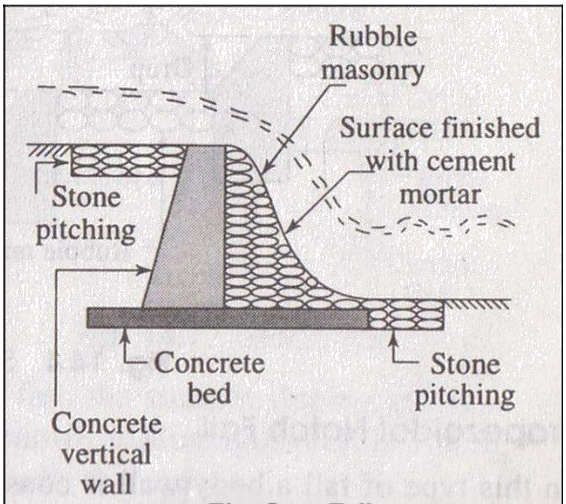


Fig: Ogee Fall

Rapid Fall

The rapid fall is suitable when the slope of the natural ground surface is even and long. It consists of a long sloping glacis with longitudinal slope which varies from 1 in 10 to 1 in 20.

- Curtain walls are provided on the upstream and downstream side of the sloping glacis.
- The sloping bed is provided with rubble masonry.
- The upstream and downstream side of the fall is also protected by rubble masonry.
- The masonry surface is finished with rich cement mortar (1: 3).

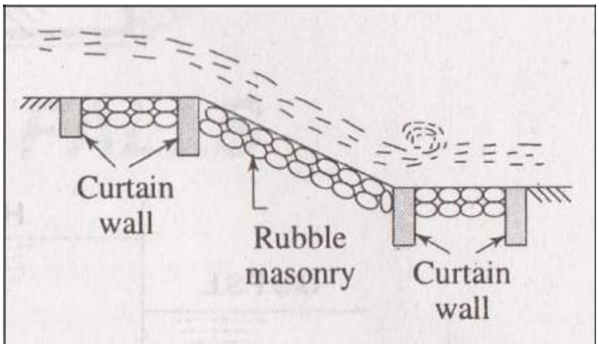


Fig: Rapid Fall

Stepped Fall

Stepped fall consists of a series of vertical drops in the form of steps. This fall is suitable in places where the sloping ground is very long and requires long glacis to connect the higher bed level with lower bed level.

- This fall is practically a modification of the rapid fall.
- The sloping glacis is divided into a number of drops so that the flowing water may not cause any damage to the canal bed. Brick walls are provided at each of the drops.
- The bed of the canal within the fall is protected by rubble masonry with surface finishing by rich cement mortar (1:3).

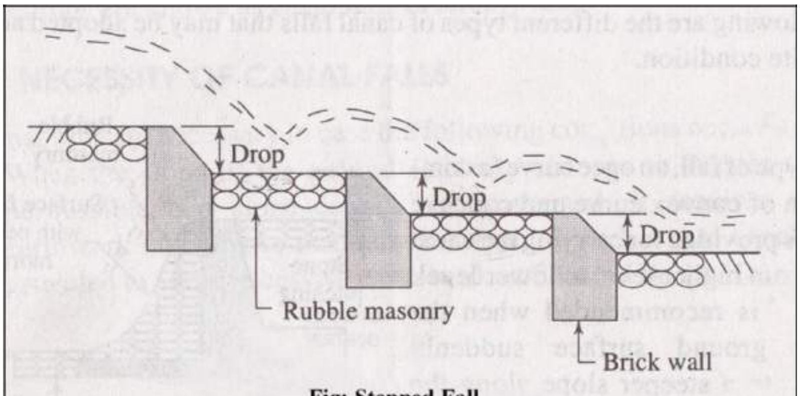


Fig: Stepped Fall

Trapezoidal Notch Fall

In this type of fall a body wall is constructed across the canal. The body wall consists of several trapezoidal notches between the side piers and the intermediate pier or piers. The sills of the notches are kept at the upstream bed level of the canal.

- The body wall is constructed with masonry or concrete.
- An impervious floor is provided to resist the scoring effect of the falling water.
- The upstream and downstream side of the fall is protected by stone pitching finished by cement grouting.
- The size and number of notches depends upon the full supply discharge of the canal.

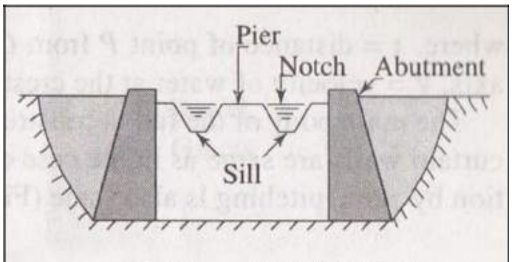


Fig: Trapezoidal Notch Fall

Vertical Drop Fall

It consists of a vertical drop walls which is constructed with masonry work. The water flows over the crest of the wall. A water eastern is provided on the downstream side which acts as a water cushion to dissipate the energy of falling water.

- A concrete floor is provided on the downstream side to control the scouring effect of the flowing water.
- Curtain walls are provided on the upstream and downstream side.
- Stone pitching with cement grouting is provided on the upstream and downstream side of the fall to protect it from scouring.

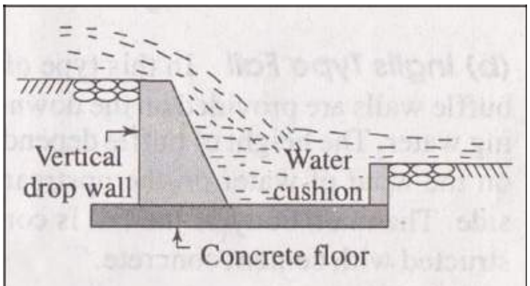


Fig: Vertical Drop Fall

Glacis Fall

It consists of a straight sloping glacis provided with a crest. A water cushion is provided on the downstream side to dissipate the energy of flowing water.

- The sloping glacis is constructed with cement concrete.
- Curtain walls and toe walls are provided on the upstream and downstream side.
- The space between the toe walls and curtain walls is protected by stone pitching.
- This type of fall is suitable for drops up to 1.5 m.

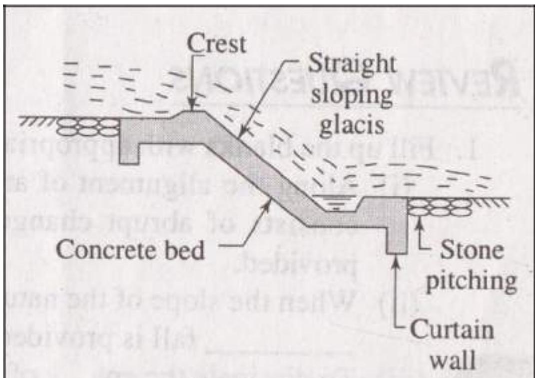


Fig: Glacis Fall

For the improvement in energy dissipation, the glacis falls have been modified as follows:

(a) Montague Type Fall

In this type of fall, the straight sloping glacis is modified by giving parabolic shape which is known as Montague profile. Taking "0" as the origin, the Montague profile is given by the equation,

$$X = \upsilon \sqrt{\frac{4y}{g}} + Y$$

Where, x = distance of point P from OX axis, Y = distance of point P from OY axis,

υ = velocity of water at the crest,

g = acceleration due to gravity

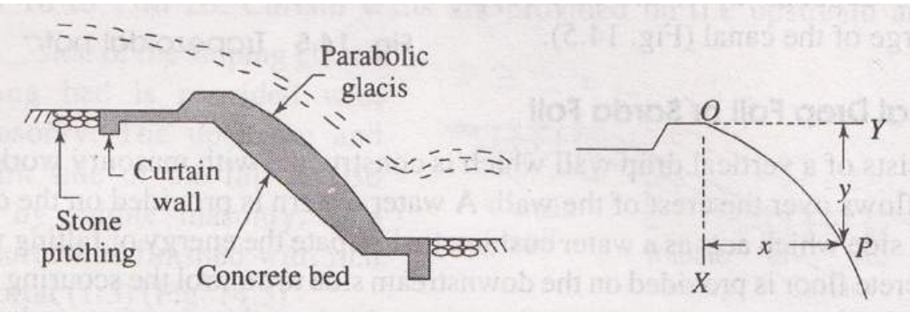


Fig: Montague Type Fall

The main body of the fall is constructed with cement concrete. Toe walls and curtain walls are same as in the case of straight sloping glacis. The bed protection by stone pitching is also same.

(b) Inglis Type Fall

In this type of fall, the gracis is straight and sloping, but buffle walls are provided on the downstream floor to dissipate the energy of flowing water.

- The height of buffle depends on the head of water on the upstream side.
- The main body of the fall is constructed with cement concrete.
- The toe walls and curtain walls are same as straight glacis.
- The protection works with stone pitching are also same. Sometimes, this fall is known as buffle fall.

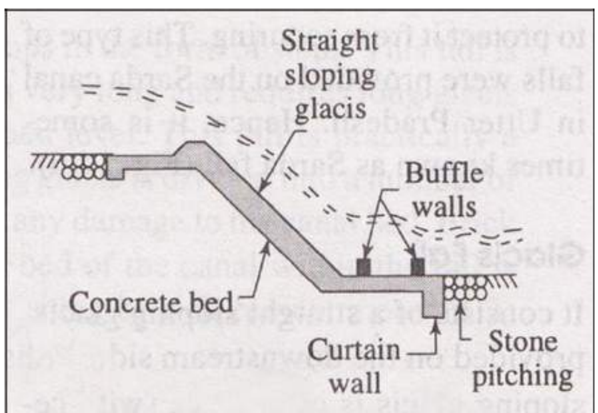


Fig: Inglis Type Fall

CANAL OUTLETS OR MODULES

Canal Outlets/Modules:

A canal outlet or a module is a small structure built at the head of the water course so as to connect it with a minor or a distributary channel.

It acts as a connecting link between the system manager and the farmers.

Requirements of a good module:

- It should fit well to the decided principles of water distribution.
- It should be simple to construct.
- It should work efficiently with a small working head.
- It should be cheaper.

- It should be sufficiently strong with no moving parts, thus avoiding periodic maintenance.
- It should e such as to avoid interference by cultivators.
- It should draw its fair share of silt.

Types of Outlets (Modules)

The various available types of outlets can be classified into three classes:

(i) Non-modular outlets are those through which the discharge depends upon the difference of head between the distributary and the water-course. The discharge through such a module, therefore, varies widely with either a change in the water level of the distributary or that of the water-course. The common examples of this type of outlets are : (z) *open sluice,* and *(ii) drowned pipe outlet.*

(ii) Semi-modules or Flexible modules are those through which the discharge is independent of the water level of the water course but depends only upon the water level of the distributary so long as a minimum working head is available. The discharge through such an outlet will, therefore, increase with a rise in the distributary water surface level and *vice versa.* The common examples of this type of modules are : *pipe outlet, venturi flume, open flume and orifice semi-module.*

(iii) Rigid modules or Modular outlets are those through which the discharge is constant and fixed within limits, irrespective of the fluctuations of the water levels of either the distributary or of the water course or of both. *Gibb's module* is a common example of such a module.

Types of Semi-Modules or Flexible Outlets

The common types of semi-modules are :

(i) *Pipe outlet discharging freely into the air.*

(ii) Venturi-flume outlet or Kennedy's Gauge outlet.

(iii) Open flume outlet.

(iv) Adjustable orifice semi-module.

Free Pipe Outlet. Pipe outlet discharging freely into the atmosphere is the simplest and the oldest type of a flexible outlet. The discharge through such an outlet with depends only upon the water level of the distributary, and will be independent of the water level of the water-course so long as the pipe is discharging freely. Silt conduction for such an outlet is quite good and efficiency is high. But a freely falling jet outlet can be provided only at a few places where sufficient level difference between the distributary and water-course is available. The discharge can be easily computed by using the equation.

$$Q = C_d \cdot A \cdot \sqrt{2g\, H_0}$$

where C_d is coefficient of discharge = 0.62 for average condition of free over fall.

H_0 = Head on u/s side measured from FSL of distributary up to the centre of pipe outlet.

A = Area of cross-section' of pipe

Venturi Flume Outlet or Kennedy's Gauge Outlet. Kennedy's Gauge Outlet is of a Venturi flume type and is shown in Fig. It is made of cast iron and consists of three main parts:

(a) an orifice with a bell mouth entry ;

(b) a long expanding delivery pipe ;

(c) an air-vent connecting the throat of the delivery pipe to the atmosphere.

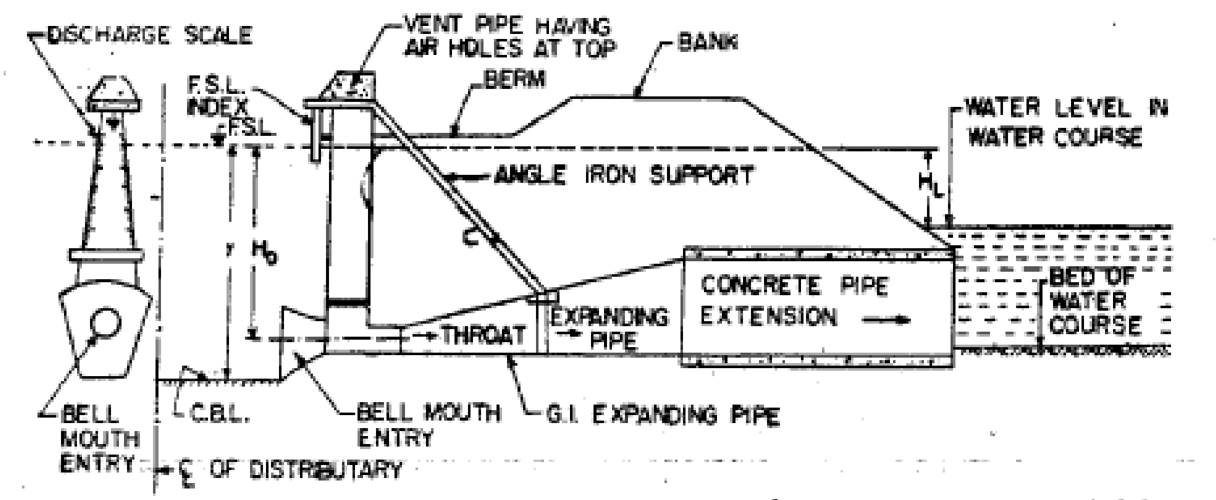
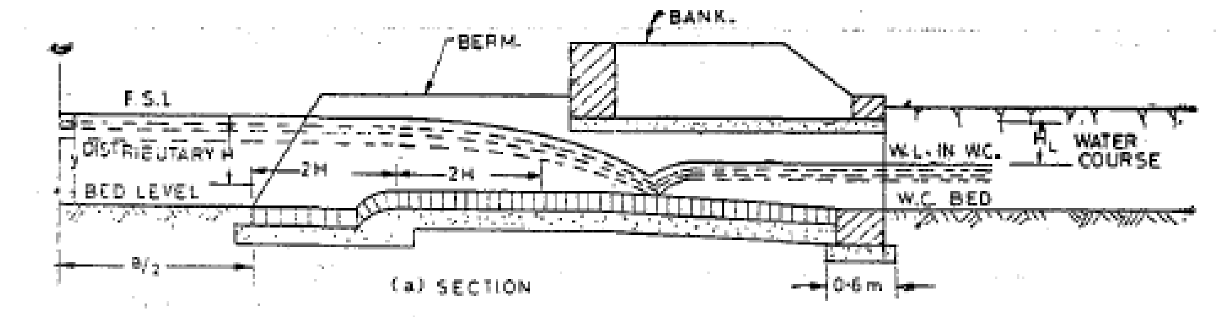


Fig. Kennedy's Gauge Outlet is of a Venturi flume

Open Flume Outlet. It is a weir type outlet with a constricted throat and an expanding flume on the downstream, as shown in Fig. Due to the constriction, a super-critical velocity is ensured in the throat and thereby allowing the formation of a jump in the expanding flume. The formation of hydraulic jump makes the outlet discharge independent of the water level in the water course; thus making it a semimodule.



. Fig. Open Flume Outlet

Adjustable Orifice Semi-Modules. Various types of orifice semi-modules have been designed since olden days. The one which found a lot of popularity is called Crump's adjustable proportionate module (APM).

Further improvements in approaches etc, have since been carried out in crump's APM, and the latest model, which is now used in Punjab and Haryana, is calied an Adjustable orifice semi-module (A.O.S.M.). Typical dimensions of such an outlet are shown in the attached chart Fig.

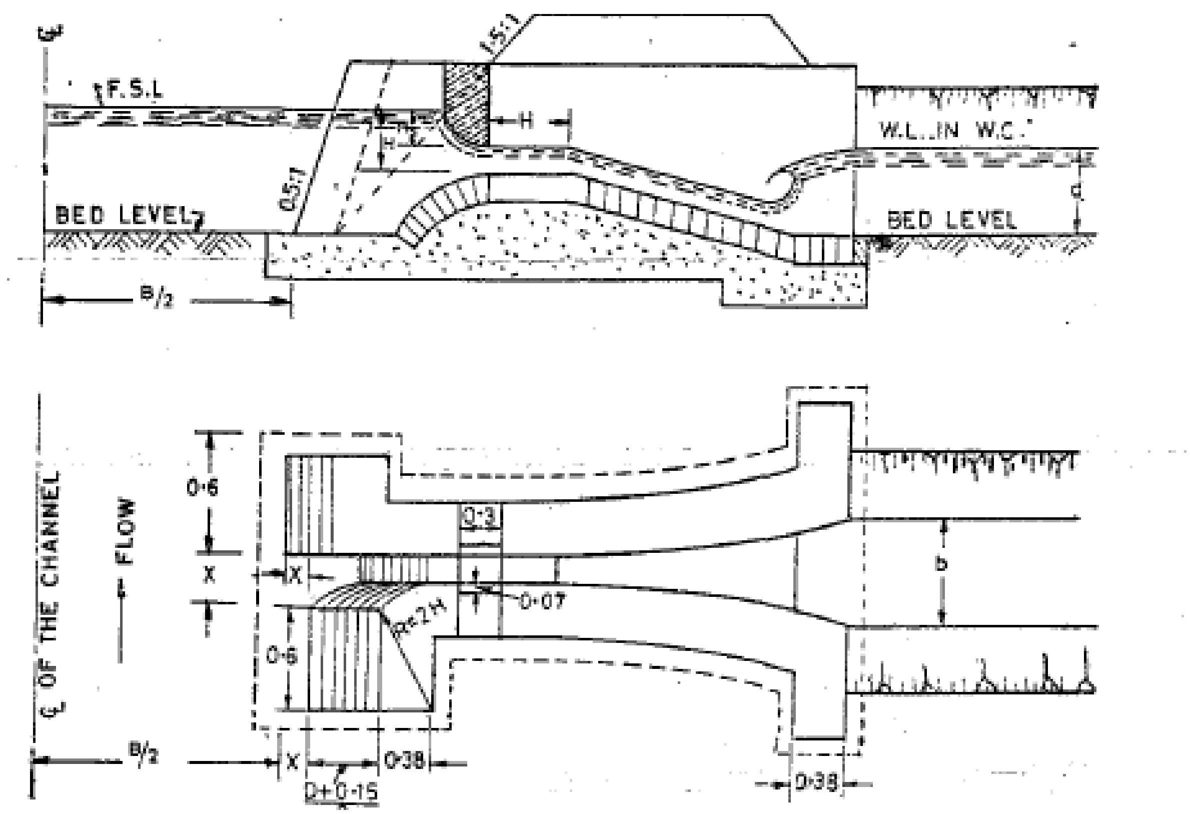


Fig. crump's APM Outlet

Types of Rigid Modules

There are a few types of rigid modules which have no moving parts, such as :

(i) Gibb's module ;

(ii) Khannas rigid module

(iii) Foote module.

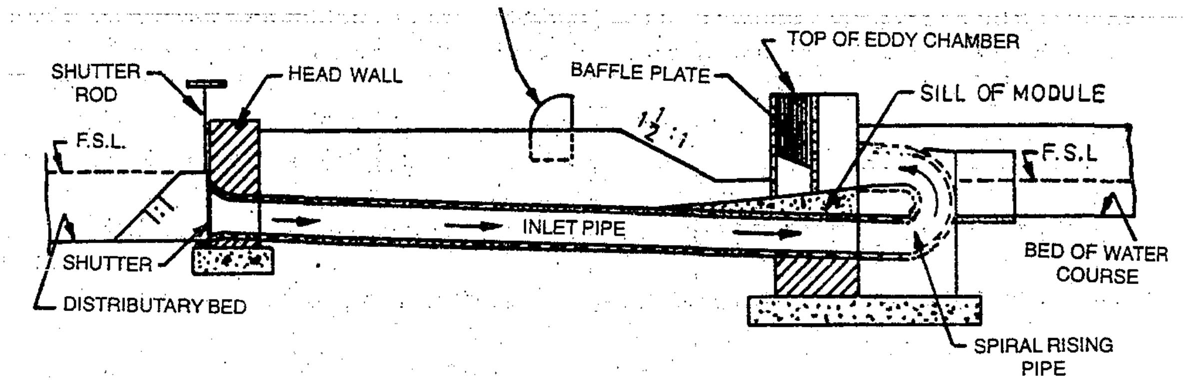


Fig. Gibb's module