HYDRAULIC STRUCTURES
Unit-I

Learning Material

When a barrier is constructed across some river in the form of a dam, water gets stored on the upstream side of the barrier, forming a pool of water, generally called a dam reservoir or an impounding reservoir or a river reservoir.

Storage works are constructed to serve many purposes, which include
1. Storage and control of water for irrigation
2. Storage and diversion of water for domestic uses
3 Water supplies for industrial uses
4. Development of hydroelectric power
5. Increasing water depths for navigation
6. Storage space for flood control
7. Reclamation of low-lying lands
8. Debris control
9. Preservation and cultivation of useful aquatic life
10. Recreation.

Classification of Reservoirs

Depending upon the purpose served by a given reservoir, the reservoirs may be broadly divided into the following three types.

(1) Storage or Conservation reservoir
(2) Flood Control reservoir
(3) Multipurpose reservoir
(4) Distribution reservoir

Storage or Conservation Reservoir

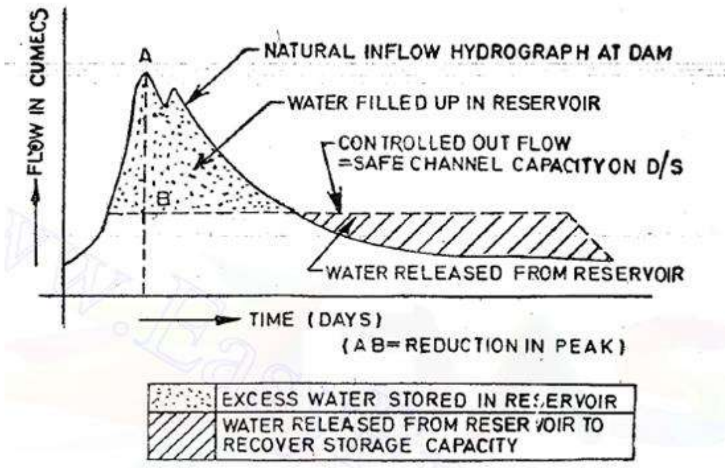
• Storage reservoirs are primarily used to supply water for irrigation, hydroelectric development, domestic and industrial supplies.

• A river does not carry the same quantity of water throughout the year, and may carry large quantities in the other part of the year.

• A storage reservoir is constructed to store the excess water during the it gradually as and when it is needed. Period of large supplies, and release gradually as and when it is needed.

Flood Control Reservoirs

Flood Control Reservoirs Flood control or flood protection reservoirs are those which store water during flood and release it gradually the flood reduces. By the provision of artificial storage during the floods flood damage on downstream is reduced.



Distribution Reservoir:

A distribution reservoir is a small storage reservoir used for water supply in a city . A distribution reservoir accounts for the varying rate of water during the day. Such distribution reservoir permits the pumping plants and water-treatment works etc., to operate at a constant rate. This varying demand rate, exceeding to constant pumping rate, is met from the distribution reservoir.

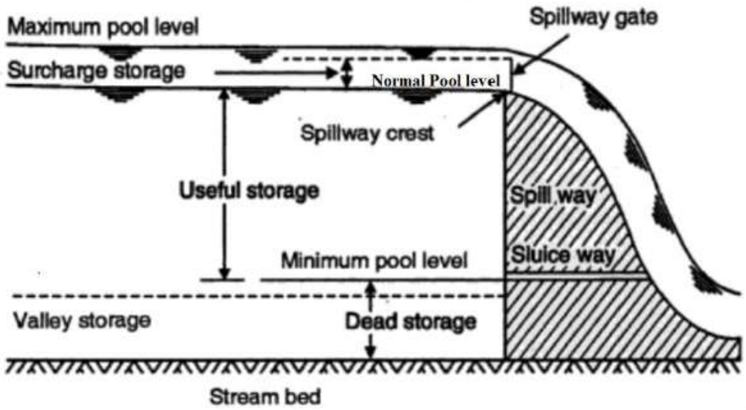
Multipurpose Reservoir:

A multipurpose reservoir is that which serves more than one purpose. For example a reservoir designed to protect the downstream area from floods, and to store water for irrigation and hydroelectric purposes is a multipurpose reservoir.

Selection of site for a reservoir

1. The geological condition of the catchment area should be such that percolation losses are minimum and maximum run-off is obtained.
2. The reservoir site should be such that quantity of leakage through it is a minimum. Reservoir site having the presence of highly permeable rocks reduce the water tightness of the reservoir. Rocks which are not likely to allow passage of water include shales and slates. schists, gneisses, and crystalline igneous rocks such as granite.
3. Suitable dam site must exist. The dam should be founded on sound watertight rock base, and percolation below the dam should be minimum. The cost of the dam is often a controlling factor in selection of a site.
4. The reservoir basin should have narrow opening in the valley so that the length of the dam is less.
5. The cost of real estate for the reservoir, including road, rail road, dwelling re-location etc. must be as less as possible.
6. The topography of the reservoir site should be such that it has adequate capacity without submerging excessive land and other properties.
7. The site should be such that a deep reservoir is formed. A deep reservoir is preferable to a shallow one because of (i) lower cost of land submerged per unit of capacity, less evaporation losses because of reduction in the water spread area, and (less likelihood of weed growth)
8. The reservoir site should be such that it avoids or excludes water form those tributaries which carry a high percentage of silt in water.
9. The reservoir site should be such that the water stored in it is suitable for the purpose for which the project is undertaken. The soil and rock mass at the reservoir site must not contain any objectionable minerals and salts.

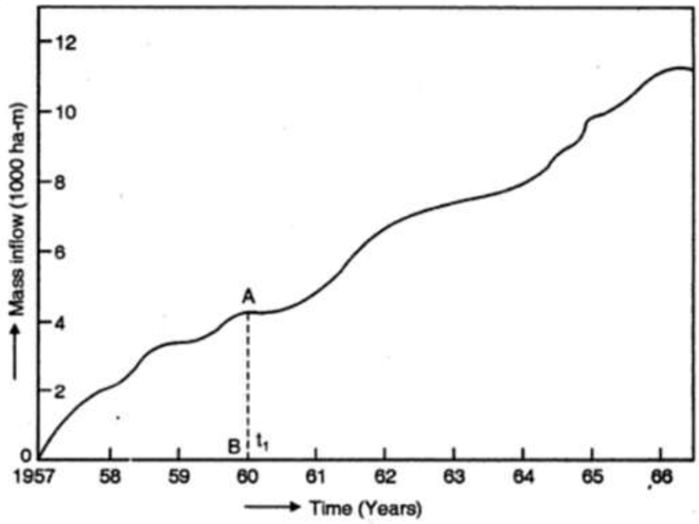
Zones of Storage in a reservoir



The following are various zones of storage in reservoir :
(1) Useful storage.
(2) Surcharge storage.
(3) Dead storage.
(4) Bank storage.
(5) Valley storage.

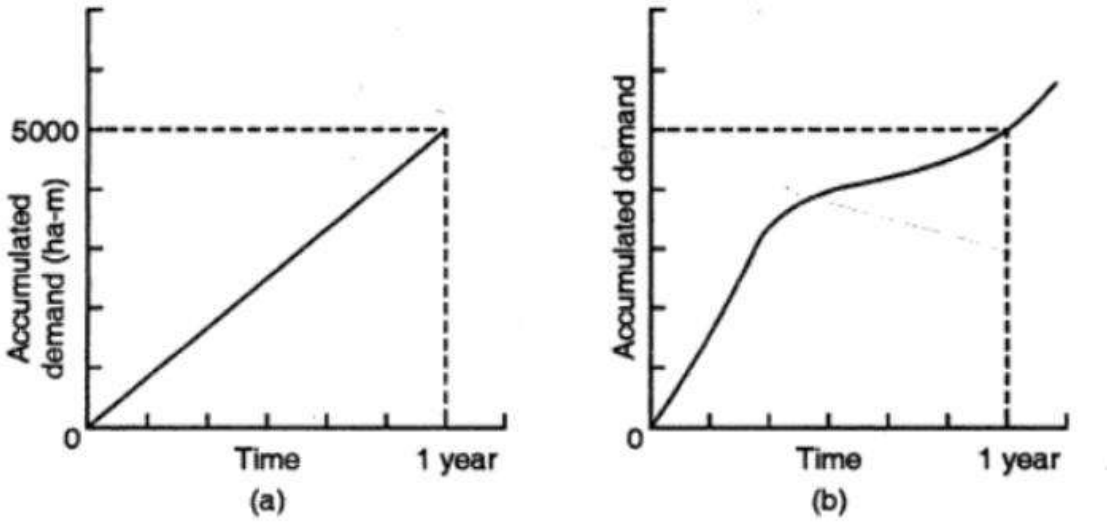
• The maximum level to which the water will rise in the reservoir during ordinary operation condition is called normal pool level.
• The normal pool level is corresponding to either the level of the spillway crest, or to the top level of the spillway gates.
• The level to which water rises during the design flood is known as the maximum pool level.
• The lowest elevation to which the water in the reservoir is to be drawn under ordinary operating conditions is known as the minimum pool level.
• The volume of water stored between the normal pool level and pool level is known as the useful storage.
• The volume of water below pool level is known as the dead storage and is not useful under ordinary operating conditions.
• The volume of water stored between the normal pool level and the maximum level corresponding to a flood is called surcharge storage, and is usually uncontrolled.
• The terms Minimum pool level bank storage and valley storage are referred to the volume of water stored in the pervious formations of the river banks and the soil above it.
Yield is the amount of water that can be supplied from the reservoir in a specified interval of time.
• The interval of time chosen for the design varies from a day for small distribution reservoirs to a year for large conservation reservoirs.
• For example, if 25,000 cubic meters of water is supplied from a reservoir in one year, its yield is 25,000 cubic meters/year or 2.5 hectare-meters per year.
Safe yield or firm yield: The maximum quantity of water that can be guaranteed during a critical dry period is known as the safe yield or firm yield.
Secondary yield: Secondary yield is the quantity of water available in excess of safe yield during periods of high flood.
Average yield: The arithmetic average of the firm and the secondary yield over a long period of time is called average yield.

Mass inflow curve: The reservoir capacity corresponding to a specified yield is determined with the help of mass inflow curve and the demand curve. A mass inflow curve is a plot between the cumulative inflow in the reservoir with time.

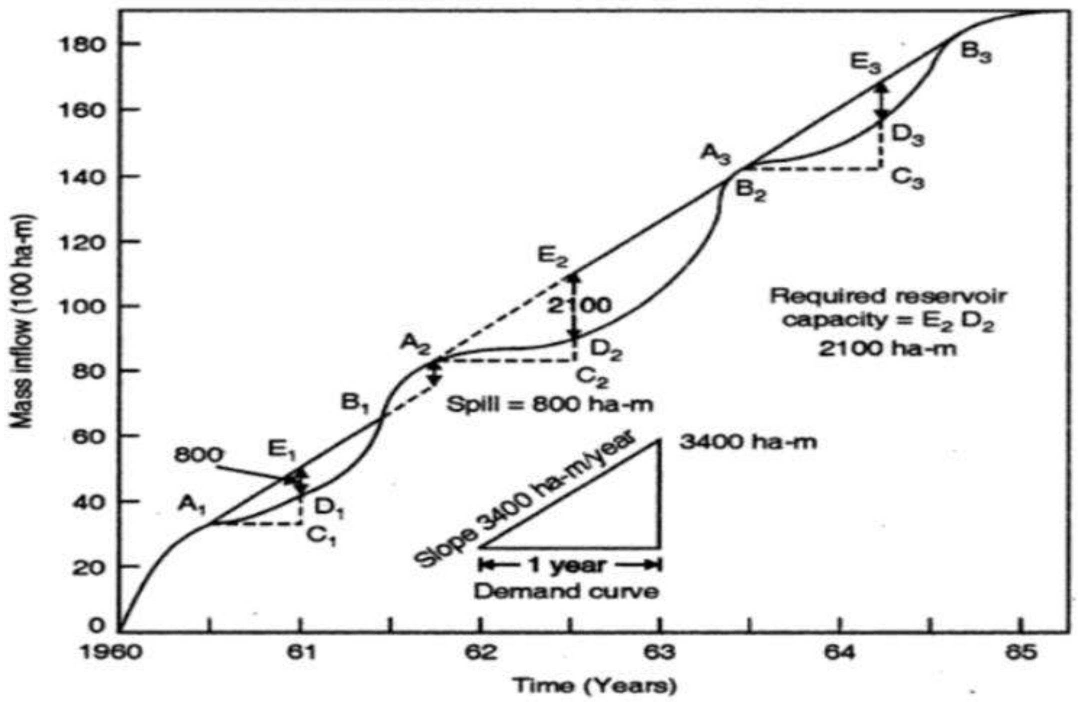


MASS INFLOW CURVE.

Demand curve: A demand curve is a plot between accumulated demand with time. The demand curve representing a uniform rate of demand is a straight line having the slope equal to the demand rate. A demand curve may be curved also, indicating variable rate of demand.



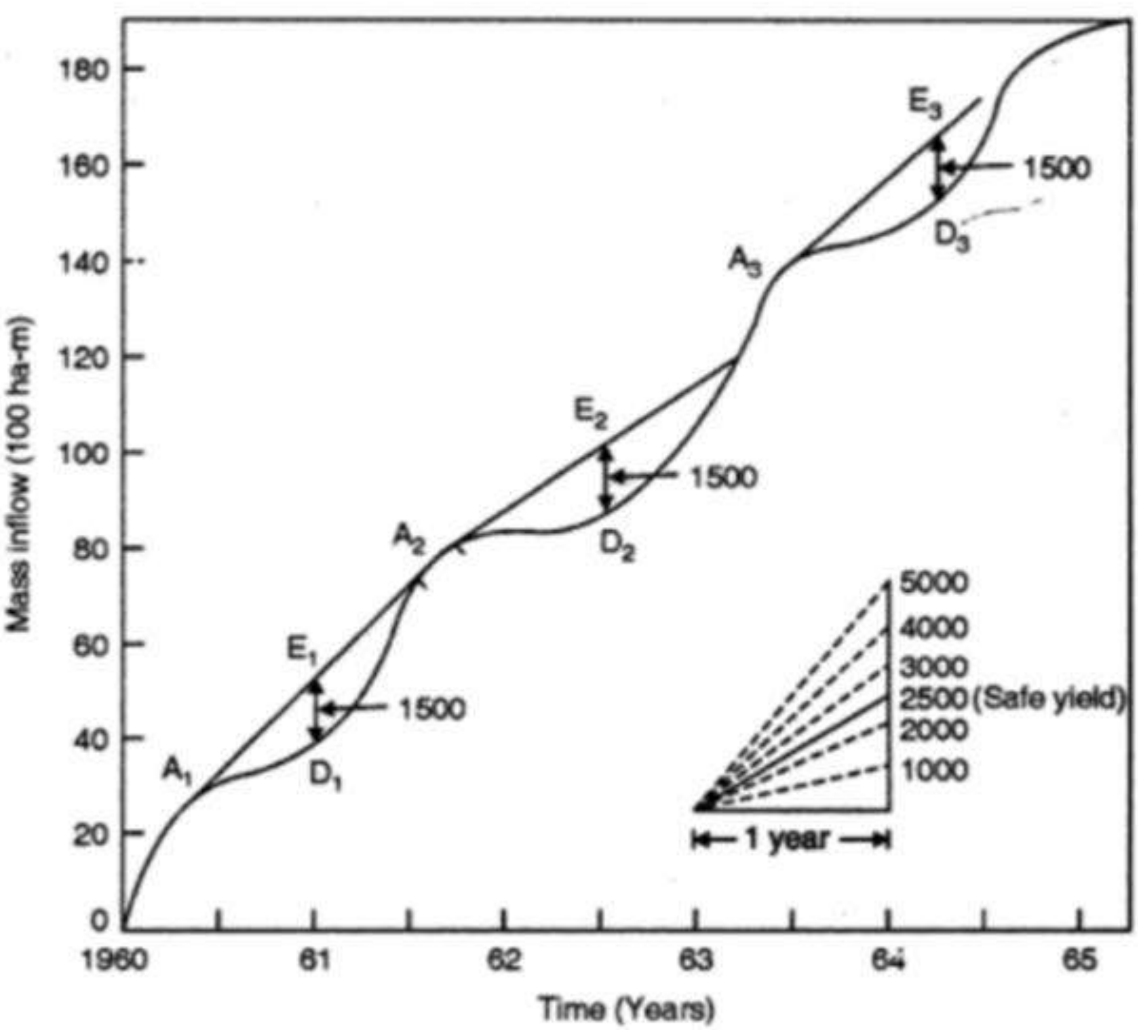
CALCULATION OF RESERVOIR CAPACITY FOR A SPECIFIED YIELD, FROM THE MASS INFLOW CURVE



DETERMINATION OF RESERVOIR CAPACITY.

1. From the flood hydrograph of inflow for several years, prepare the mass inflow curve. Also prepare the mass curve of demand on the same scale.
2. From the apices A_1, A_2 , A_3 etc. of the mass inflow curve, draw tangents parallel to the demand curve.
3. Measure the maximum vertical intercepts E_1D_1 , E_2D_2 , E_3D_3 , etc., between the tangent and the mass inflow curve. The vertical intercepts indicate the volume by which the inflow falls short of demand. For example, for a period corresponding to points A_1 and C_1, C_1D_1 represents the net inflow while C_1E_1 represents the demand. Hence, the volume E_1D_1 has to be provided from the reservoir storage.
4. The biggest of the vertical ordinates amongst E_1D_1 , E_2D_2 , E_3D_3 etc. represent the required reservoir capacity.

Determination of safe yield from a reservoir of a given capacity



The following is the procedure of determining the safe yield from a reservoir of a given storage capacity, with the help of a mass inflow curve

(1)Prepare the mass inflow curve. On the same diagram, draw straight lines, from a common origin, representing demands at various rates, say varying from 0 to 5000 ha-m per year.

(2)From the apices A_1 , A_2 , A_3 etc. of the mass inflow curve, draw tan-gents in such a way that their maximum departure from the mass inflow curve does not exceed the specified reservoir capacity. Thus, in Fig. the ordinates E_1D_1 , E_2D_2 , E_3D_3 etc. are all equal to the reservoir capacity (say 1500 ha-m).

(3)Measure the slopes of each of these tangents. The slopes indicate the yield which can be attained in each year from the reservoir of given capacity. The slope of flattest demand line is the firm yield or safe yield.

DAMS

Dams: Dam is a solid barrier constructed at a suitable location across a river valley to store flowing water. Storage of water is utilized for following objectives:

- ·Hydropower
- ·Irrigation
- ·Water for domestic consumption
- ·Drought and flood control

- ·For navigational facilities

• Other additional utilization is to develop fisheries

Classification of dams

• Based on use, dams are classified as follows
 i) Storage dam,
(ii) Diversion dam,
(iii) Detention dam.

Storage dam:

• This is the most common type of dam normally constructed. Storage dam is constructed to impound water to its upstream side during the periods of excess supply in the river (i.e. during rainy season) and is used in periods of deficient supply.

•Behind such a dam, a reservoir or lake is formed. The storage dams may be constructed for various purposes, such as for irrigation, water power generation or for water supply for public health- purposes, or it may be for a multipurpose project.

• A storage dam may be constructed of wide variety of materials, such as stone, concrete, earth and rockfill etc.

Diversion dam:

•The purpose of a diversion dam is essentially different. While a storage dam stores water at its upstream for future use, a diversion dam simply raises water level slightly in the river and thus provides head for carrying or diverting water into ditches, canals, or other conveyance systems to the place of use.

•A diversion dam is, therefore, of smaller height and no reservoir is formed to store water.

•The common examples of diversion dams are weirs and barrages. During the floods, water passes over or through these diversion dams while during periods of normal flow, the river water, partly or wholly, is diverted to irrigation channel etc. A diversion dam may be constructed for irrigation or municipal or industrial uses.

Detention dam:

A detention dam is constructed to store water during floods and release it gradually at a safe rate, when the flood recedes.

• By the provision of artificial storage during the floods flood damage downstream is reduced.

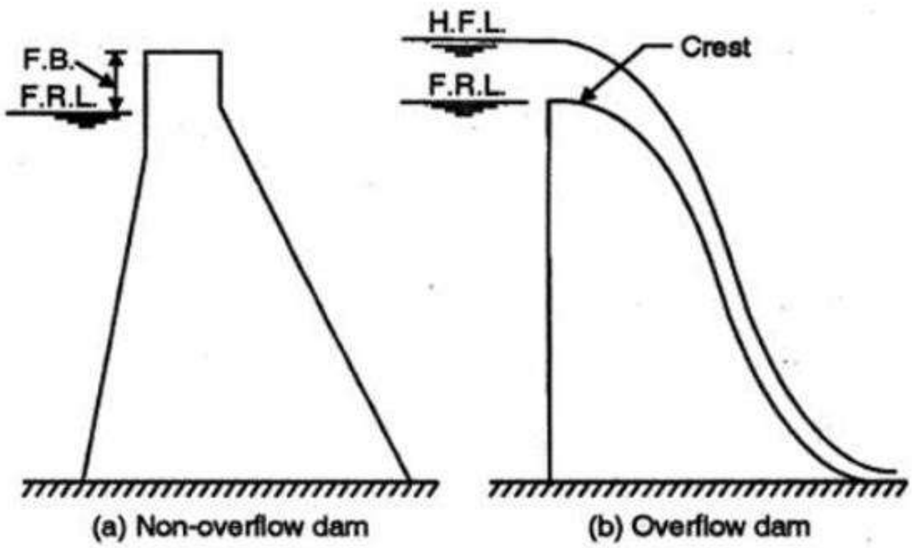
•There are usually two types of detention dams. In the first type discussed above, water is temporarily stored and released through a suitable outlet structure.

•In the other type of detention dam, water is not released and no outlet structure is provided. Instead, water is held in the reservoir as long as is possible. This held water seeps into pervious banks and foundation strata.

•Due to this seepage of water, water-level in wells in the adjoining area is increased and lift 'irrigation may be possible. Also, irrigation may be done in the river bed itself at the downstream side. The seepage water may be sufficient for the growth of the crop and no additional surface watering may be necessary. Such a detention dam is sometimes called water-spreading dam or dike, and is similar to a weir in constructional features.

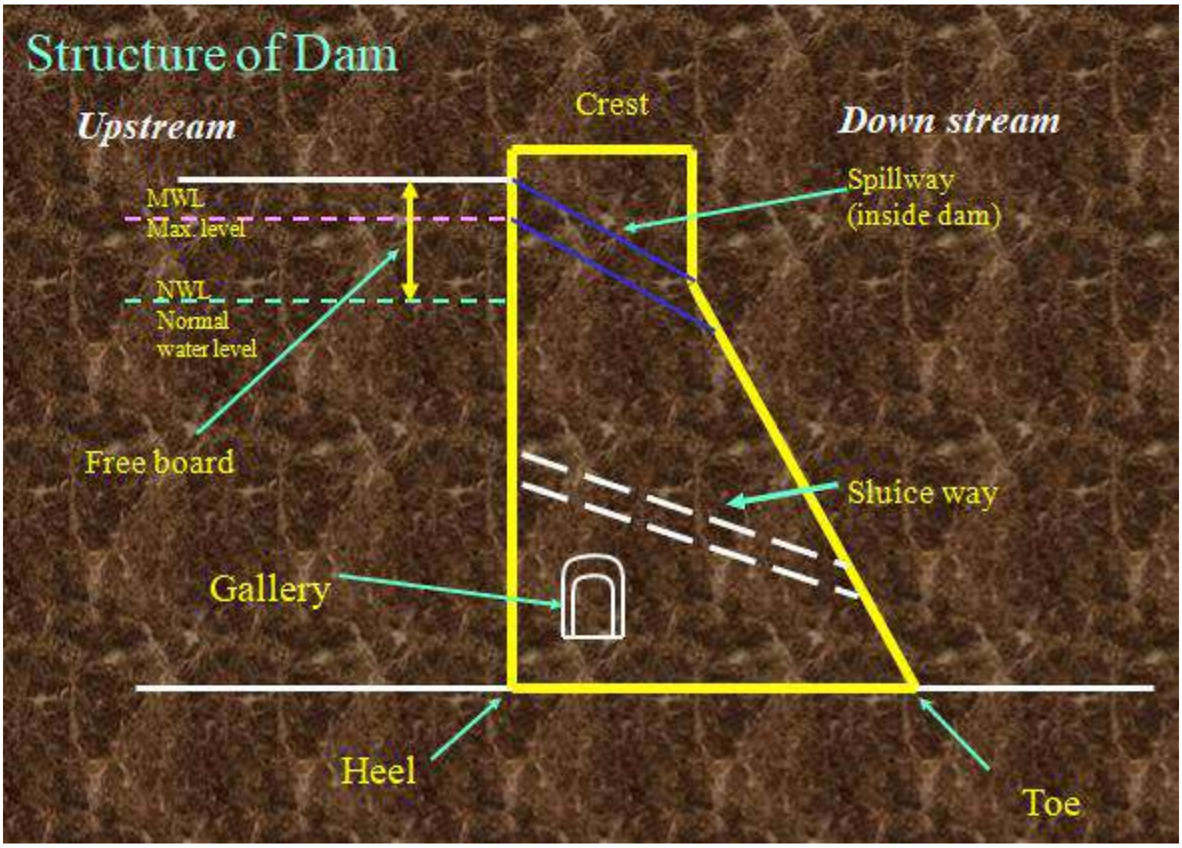
CLASSIFICATION OF DAM ACCORDING TO HYDRAULIC DESIGN

i) Non- over flow dam
ii) Over flow dam

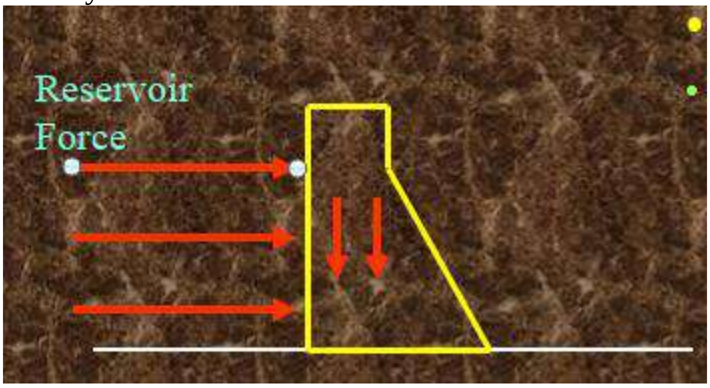


According to this most common classification, the dam may be classified as follows:
1. Rigid dams. 2. Non-rigid dams Rigid dams.
Rigid dams are those which are constructed of rigid materials such as masonry, concrete, steel or timber. Rigid dams may be further classified as follows:
1. Solid masonry or concrete gravity dam.
2. Arched masonry or concrete dam.
3. Concrete buttress dam.
4. Steel dam.
5. Timber dam.

Non-rigid dams: Non-rigid dams are those which are constructed of non-rigid materials such as earth and/or rockfill. The most common types of non-rigid dams are 1. Earth dam. 2. Rockfill dam. 3. Combined earth and rockfill dam



Gravity dams



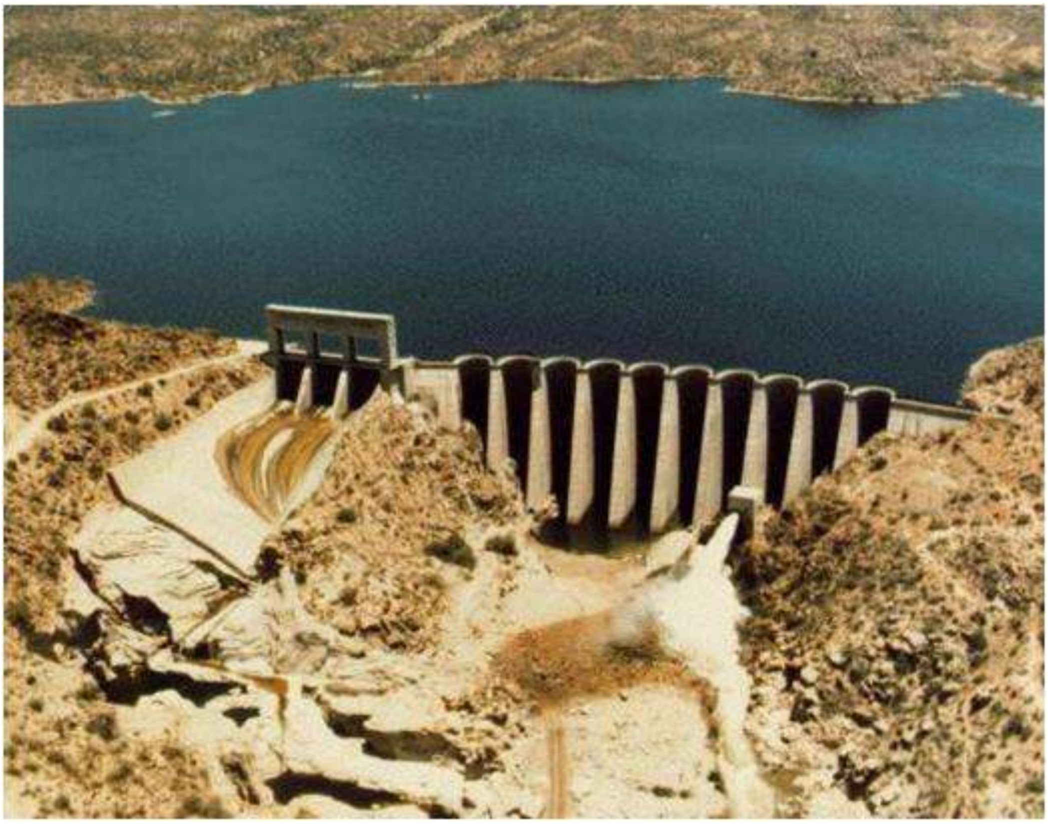
Gravity Dams:

• These dams are heavy and massive wall-like structures of concrete in which the whole weight acts vertically downwards

As the entire load is transmitted on the small area of foundation, such dams are constructed where rocks are competent and stable.

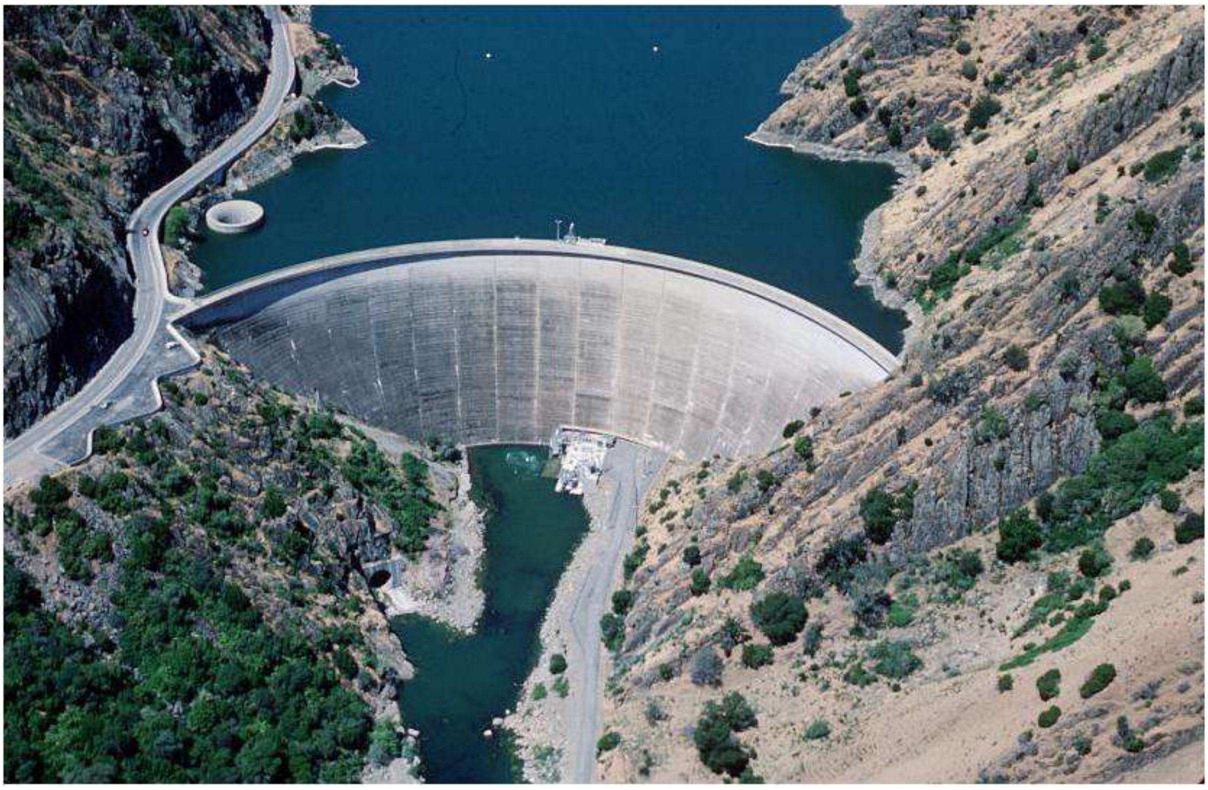
•Bhakra Dam is the highest Concrete Gravity dam in Asia and Second Highest in the world.
•Bhakra Dam is across river Sutlej in Himachal Pradesh
The construction of this project was started in the year 1948 and was completed in 1963

Butress dam:



•Buttress Dam – Is a gravity dam reinforced by structural supports
•Buttress - a support that transmits a force from a roof or wall to another supporting structure

Arch dam



•These type of dams are concrete or masonry dams which are curved or convex upstream in plan

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•This shape helps to transmit the major part of the water load to the abutments

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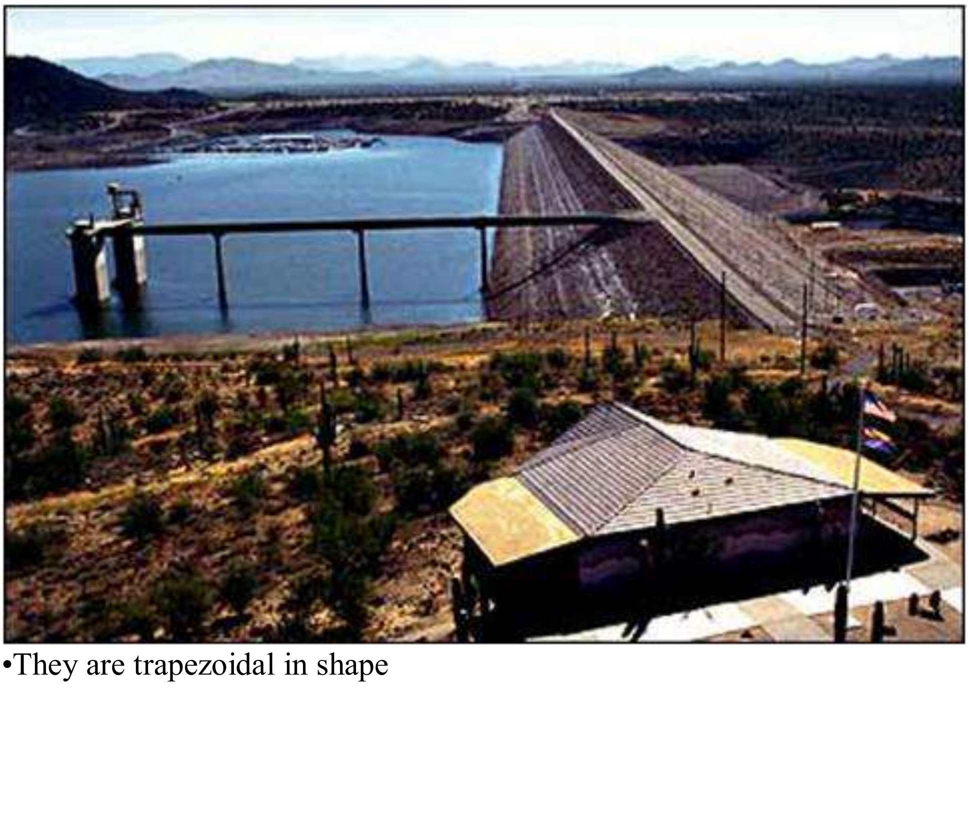
•Arch dams are built across *narrow, deep river gorges, but now in recent years they have been considered even for little wider valleys.*

Earth Dams:

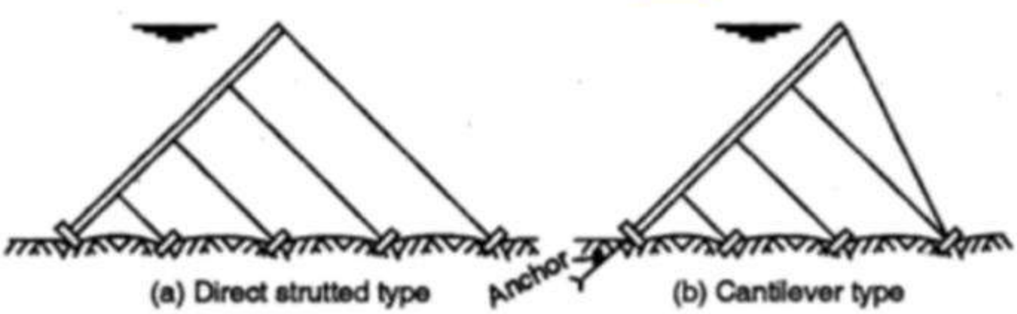


•They are trapezoidal in shape

•Earth dams are constructed where the foundation or the underlying material or rocks are weak to support the masonry dam or where the suitable competent rocks are at greater depth.
•Earthen dams are relatively smaller in height and broad at the base
•They are mainly built with clay, sand and gravel, hence they are also known as Earth fill dam or Rock fill dam

Steel dams

•Steel dams are constructed with a framework of steel with a thin skin plate as deck slab on the upstream side.
• In India. no such darns has been constructed. However, in United States three such dams have been constructed: Ash Fork Darn in Arizona (1898), Redridge Dam in Michigan (1905) and Hauser Lake Darn. in Montana (1901).
• Out of these the first two dams gave satisfactory results while the third dam failed only after one year of service. The failure was mainly due to undermining of the foundation by leakage through or under the steel sheet pile.



Timber dam

•A timber dam is constructed of framework of timber struts and beams, with timber plank facing to resist water pressure.
• A timber dam is an ideal temporary dam , though a well designed, constructed and maintained timber dam may last 30-40 years. They are suitable to places where timber can be available in plenty.

Advantages of Gravity dams

1. Gravity dams are relatively more strong and stable than earth dams. They are particularly suited across gorges having very steep side slopes where earth dam, if constructed, might slip.
2. Gravity darns are well adapted for use as an overflow spillway crest. Earth dams cannot be used as overflow dams.
3. Gravity dams can be constructed of any height, provided suitable foundations are available to bear the stresses. The height of an earth dam is usually limited by the stability of its slopes requiring a very wide base width.
4. Gravity darn is specially suited to such areas where there is likelihood of very heavy downpour. The slopes of earth dam might get washed away in such a situation.
5. A gravity dam requires the least maintenance.
6. The failure of a gravity dam, if any, is not sudden. It gives enough warning time before the area to downstream side is flooded due to the damage to the gravity darns. On the contrary, an earth dam generally fails suddenly.
7. A gravity dam is cheaper in the long run since it is more permanent than any other type. Thus the benefit-cost ratio of such a dam is always higher.

Disadvantages of Gravity dams

The disadvantages of gravity dam, as compared to an earth dam are as follows :
1. Gravity dams can be constructed only on sound rock foundations. They are unsuitable on weak foundations or on permeable foundations on Which earth dams can be constructed with suitable foundation treatment.
2. The initial cost of a gravity dam is always higher than an earth dam. Hence, where funds are limited and where suitable materials are available for the construction of an earth dam, the earth dam may be preferred.
3. If mechanized plants, such as manufacturing and transporting mass concrete, curing of concrete etc. are not available, a gravity dam may take more time to construct.
4. Gravity dame requires skilled labour or mechanized plants for its construction.
5. It is very difficult to allow subsequent rise in the height of a gravity dam, unless specific provisions have been made in the initial design.

Advantages of Arch Dams
1. Arch dams are particularly adapted to the gorges where the length is small in proportion to the height.
2. For a given height, the section of an arch dam is much lesser than a corresponding gravity dam. Hence, an arch dam requires less material and is, therefore, cheaper.
3. Because of much less base width, the problems of uplift pressure are minor.
4. Since only a small part of water load is transferred to the foundation by cantilever action, an arch dam can be constructed in moderate foundations where gravity dam requiring sound foundation rock may be unsuitable.
Disadvantages of arch dams
1. It requires skilled labour and sophisticated form work. The design of an arch dam is also quite specialized.
2. The speed of construction is normally slow.
3. It requires very strong abutments of solid rock capable of resisting arch thrust. Hence, it is not suitable in the locations where strong abutments are not available. Unfortunately, only few sites are suitable for this type of dam.
Advantages of Buttress Dams
1. A buttress dam is less massive than a gravity dam. Hence, the foundation pressures are less in the case of a buttress dam, and it can be constructed even on weak foundations on which the gravity dam cannot be supported.
2. The amount of concrete used in buttress dam is about 1/2 to 1/3 of the concrete used in gravity dam of the same height.
3. In the case of gravity dam, the height of the dam can raised only by the provision of crest shutter at overflow section. However in the case of a buttress darn, further raising of the height is possible and convenient by extending buttress and slab.
4. Power houses and water treatment plants can be housed in between, buttresses, thus saving some cost of construction
Disadvantages of Buttress Dams
1. Skilled labour requirements
2. Deterioration of upstream concrete surface has serious effects on buttress dams with very thin concrete face.
3. Buttress dam is more susceptible to willful damage.
Advantages of Steel Dams
1. With the modern methods of steel fabrication, greater speed can be achieved in construction.
2. They have been found to be cheaper than other rigid dams.
3. Stresses in steel dams are more determinate. Hence, section can be designed economically and with confidence.
4. They have greater flexibility to resist unequal settlement without excessive leakage.
5. They are not affected by frost action.
6. Leaky joints, if any, can be repaired more easily by modern welding processes, than in hollow concrete dams.
Disadvantages
1. Steel dams are lighter, hence are not as adaptable to absorb the shock from vibrations of spilling water.
2. The life of steel darn is known to be shorter than a concrete dam.
3. Steel dam requires greater and more constant maintenance than concrete.
4. They have to be anchored at foundation which is difficult and precarious

Timber dam

Advantages

1. Low initial cost.
2. Suitable for any type of foundation.
3. Where only temporary dams are to be constructed timber dams are more suitable.
4. Greater speed in construction can be achieved

Disadvantages

1. High maintenance cost.
2. Life is short.
3. Suitable only for small heights.
4. Greater seepage loss through the body of the dam.

Unit-II

Learning Material

General:

- A gravity dam is a solid concrete or masonry structure which ensures stability against all applied loads by its weight alone without depending on arch or beam action.
- Such dams are usually straight in plan and approximately triangular in cross-section.
- Gravity dams are usually classified with reference to their structural height which is the difference in elevation between the top of the dam (i.e., the crown of the roadway, or the level of the walkway if there is no roadway) and the lowest point in the excavated foundation area, exclusive of such features as narrow fault zones

1. Gravity dams up to 100 ft (30.48 m) in height are generally considered as low dams.
2. Dams of height between 100 ft (30.48 m) and 300 ft (91.44m) are designated as medium-height dams.
3. Dams higher than 300 ft (91.44 m) are considered as high dams

Forces on a Gravity Dam

The forces commonly included in the design of a gravity dam are shown in Fig.3.1

(*i*) *Dead Load*

The dead load (W_c) includes the weight of concrete and the weight of appurtenances such as piers, gates, and bridges. All the dead load is assumed to be transmitted vertically to the foundation without transfer by shear between adjacent blocks.

(*ii*) *Reservoir and Tail-water Loads* (W_w , W_w', W_1, *and* W_1')

These are obtained from tail-water curves and range of water surface elevations in reservoir obtained from reservoir operation studies. These studies are based on operating and hydrologic data such as reservoir capacity, storage allocations, stream flow records, flood hydrographs, and reservoir releases for all purposes. In case of low overflow dams, the dynamic effect of the velocity of approach may be significant and should, therefore, be considered. If gates or other control features are used on the crest, they are treated as part of the dam so far as the application of water pressure is concerned. In case of non-overflow gravity dams, the tail-water should be adjusted for any retrogression. Any increase in tail-water pressure due to curvature of flow in the downstream bucket of an overflow type gravity dam should also be considered in the design of gravity dams.

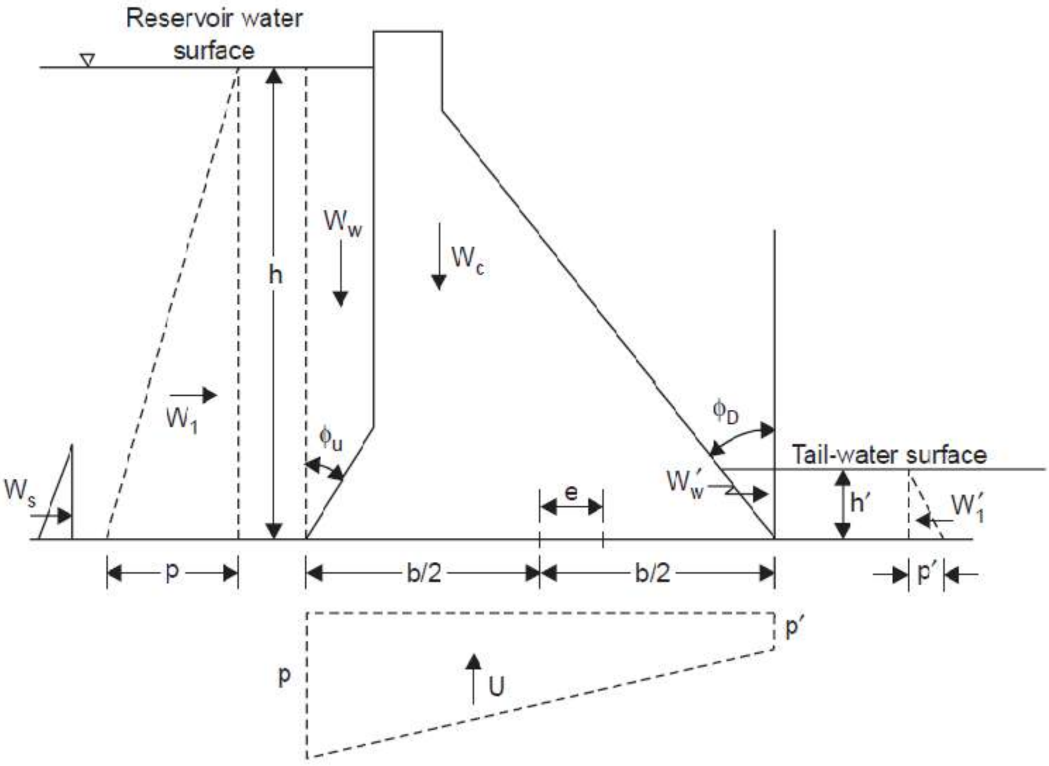


Figure: Usual loading combination for a gravity dam

(iii) Uplift Forces

Uplift forces (U) occur due to internal hydraulic pressures in pores, cracks, and seams within the body of a dam, at the contact between the dam and its foundation, and within the foundation. The distribution of internal hydrostatic pressure along a horizontal section through a gravity dam is assumed to vary linearly from full reservoir pressure at the upstream face to zero or tail-water pressure at the downstream face, and to act over the entire area of the section. The pressure distribution is also adjusted depending upon the size, location, and spacing of internal drains. Experimental and analytical studies indicate that the drains set in from the upstream face at 5 per cent of the maximum reservoir depth and spaced laterally twice that distance will reduce the average pressure at the drains to approximately tail-water pressure plus one-third the difference between reservoir water and tail-water pressures. It is assumed that uplift forces are not affected by earthquakes.

(*iv*) ***Silt Load***

The construction of a dam across river carrying sediment invariably results in reservoir sedimentation which causes an additional force (W_s) on the upstream face of the dam. The horizontal silt pressure is assumed equivalent to a hydrostatic load exerted by a fluid with mass density of 1360 kg/m^3. The vertical silt pressure is assumed equivalent to that exerted by a soil with a wet density of 1925 kg/m^3

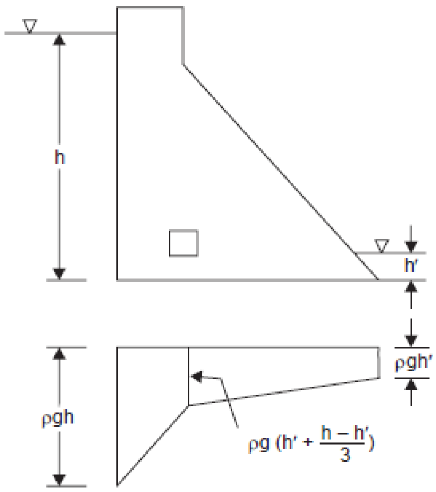


Figure: Modification in uplift force due to drain

(*v*) *Ice Pressure*

If the designer anticipates the formation of an ice sheet of appreciable thickness and its remaining on the reservoir water surface for a long duration, the ice pressures must be computed using a suitable method of their estimation. In the absence of such a method, ice pressure may be taken as 250 kPa (250 kN/m^2) applied over the anticipated area of contact of ice with the face of the dam.

(*vi*) *Wave Pressure*

The upper portion of a dam is also subjected to the impact of waves. Wave pressure against massive dams of large height is usually of little importance. Wave pressure is related to wave height h_w as follows :

(*a*) The maximum wave pressure p_w (in kilopascals) occurs at 0.125 h_w above the still water level and is given by the equation

$$p_w = 2.4\,\rho g\, h_w$$

where, h_w is the height of the wave in metres.

(*b*) The total wave force P_w (in kilonewtons) is given by

$$P_w = 20 h_w^2$$

and acts at 0.375 h_w above the still water level in the downstream direction.

(*c*) The wave height h_w can be calculated using the following relations:

$$h_w = 0.032\sqrt{VF} + 0.76 - 0.27\,F^{\frac{1}{4}} \text{ for } F < 32\ km$$

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

Here, V is the wind velocity in kilometers per hour and F is the fetch in kilometers.

The height of the wave and the wind set-up decide the freeboard which is the vertical distance between the top of the dam and the still water level. The wind set-up S (in metres) is estimated by the Zuider Zee formula

$$s = \frac{V^2 F}{62{,}000\, D}$$

in which, D is the average depth in [redacted]
should be equal to wind set-up plus $_3$ [redacted]
above maximum reservoir level [redacted]
crest elevation for the dam. The [redacted]
mean water level corresponding to the [redacted]

(*vii*) *Earthquake*

Gravity dams are elastic structures [redacted]
Such dams should be designed so [redacted]
earthquake. The design earthquake [redacted]
earthquakes to obtain frequency of [redacted]
and (*iii*) statistical approach to [redacted]
magnitudes during the life of the dam. [redacted]
the maximum credible earthquake [redacted]
larger than any historical recorded [redacted]

Earthquakes impart random [redacted]
pressures acting on the dam and also [redacted]
may take place in any direction. [redacted]
applied in the direction which [redacted]
when the reservoir is full, the [redacted]
upstream (so that the inertial [redacted]
intersecting the base of the dam [redacted]
load and, therefore, the increased [redacted]
earthquake movement as it causes the [redacted]
to weigh less resulting in reduced [redacted]
unfavorable is the downstream ground [redacted]
the resultant may intersect the base of [redacted]
earthquake forces depends on (*i*) their [redacted]
the earthquake, (*ii*) the mass of the [redacted]
on the water load. For estimation of [redacted]
or intensity, usually expressed in [redacted]
of earthquake acceleration to [redacted]
designated as α_h. The value of seismic [redacted]

accelerations for different zones of the country are different and can be obtained from the Codes. Considering a structure of mass M moving with an acceleration $\alpha_h\, g$ in the horizontal direction during an earthquake, the horizontal earthquake force acting on the structure, P_e is given as

$$P_e = 0.555\ \alpha_h\ wh^2$$

where, W is the weight of the structure. The value of α_h has usually been taken as 0.1 in the absence of any other specified value. Similarly, the value of the seismic coefficient in the vertical direction can be taken as 0.05.

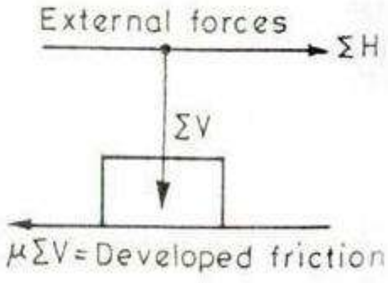
Causes of failure of a Gravity Dam:

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



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
- Frictional resistance and shear strength of the dam

Factor of safety based on frictional resistance:

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

μ = co -efficient of friction between two surfaces

$\sum \boldsymbol{V}$ **= sum of vertical forces acting on dam**

$\sum \boldsymbol{H} =$ **sum of horizontal 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.

3.4 Stability analysis:

Analytical method

Consider unit length of the dam.

i. Work out the magnitude and dimensions of all the vertical forces acting on the dam and their algebraic sum, i.e. ΣV.
ii. Similarly work out all the horizontal forces and their algebraic sum i.e. ΣH.
iii. Determine the lever arm of all these forces above the toe
iv. Determine the moments of these forces about the toe and find the algebraic sum of all those moments, i.e. ΣM.
v. Find out the location of the resulting force by determining its distance from the toe.

$$\frac{\Sigma M}{\Sigma V}$$

vi. Find out the eccentricity (e) of the resultant (R) using e=$\frac{b}{2} - \bar{x}$

viii. Determine the vertical stresses at the toe and heel using

$P_v = \Sigma \frac{V}{b}\left[1 + \frac{6e}{b}\right]$

ix. Determine the maximum normal stresses.

x. Determine the factor of safety against overturning as equal to

$$\frac{\Sigma \text{Stabilising moment}}{\Sigma \text{Disturbing moment}}$$

xi. Determine the factor of safety against sliding using Sliding factor = $\frac{\mu \Sigma V + bq}{\Sigma H}$

Principal and Shear Stresses:

Principal Stress: Consider an elementary triangular section at either the heel or the toe of the dam section such that stress intensities may be assumed to be uniform on its faces. The face of the dam will be a principal plane as water pressure acts on it in the perpendicular direction, with no accompanying shear stress. Since the principal planes are mutually at right angle, the plane AB, considered at right angles to the face AC, well also have only a normal stress on it, and will be the other principal plane. The forces acting on the elementary section are shown in fig.

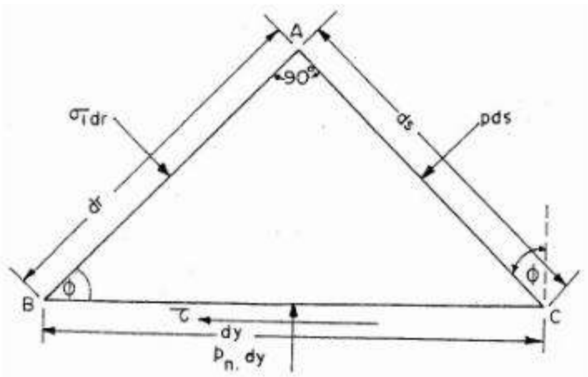


Figure: Principal stresses in a gravity dam

Let ds, dr and dy be the lengths of AC, AB and BC

p= intensity of water pressure;

σ_1 = principal stress on plane AB

τ = shear stress; and

p_n = normal stress.

Considering unit length of the dam, the normal forces on the planes AB, BC and CA are respectively $\sigma_1 dr$, $p_n\, dy$ and $p\, ds$

Resolving all the forces in the vertical direction, we get

$$p_n\, dy = p.\, ds \sin\Phi + \sigma_{1.}\, dr \cos\Phi$$

but $dr = dy \cos\Phi$ and $ds = dy \sin\Phi$

Therefore $p_n\, dy = p.dy \sin^2\Phi + \sigma_{1.}\, dy \cos^2\Phi$

or $p_n = p \sin^2\Phi + \sigma_{1.} \cos^2\Phi$

Hence $\sigma_1 = p_n \sec^2\Phi - m\, p \tan^2\Phi$------------------(i)

Shear stress: Resolving all forces in the horizontal direction, we get

$$\tau.\, dy = \sigma_1\, dr \sin\Phi - p.\, ds \cos\Phi$$

$$\text{Therefore } \tau = \sigma_1 \sin\Phi \frac{dr}{dy} - p\, \frac{ds}{dy} \cos\Phi$$

$$\tau = (\sigma_1 - p) \sin\Phi \cos\Phi$$

or substituting the value of σ_1 from eq-(i) we get

$$\tau = (p_n \sec^2\Phi - m\, p \sec^2\Phi) \sin\Phi \cos\Phi$$

$$\tau = (p_n - p) \tan\Phi$$

Elementary Profile of a Gravity Dam:

The stability conditions required to be met for a gravity dam, subjected only to its self-weight *W*, force due to water pressure *P*, and uplift force *U* can be satisfied by a simple right-angled triangular section with its apex at the reservoir water level, and which is adequately wide at the base where the water pressure is maximum. Such a section is said to be an elementary profile of a gravity dam. For the empty-reservoir condition the only force acting on the dam is its self-weight whose line of action will meet the base at *b*/3 from the heel of the dam and thus satisfy the stability requirement of no tension. The base width of the elementary profile is determined for satisfying no tension and no sliding criteria as given below, and the higher of the two base widths is chosen for the elementary profile.

For the elementary profile shown in Fig. 3.4, if one considers that the resultant *R* of all the three forces W_c (= 0.5 $s\,\rho g b h$), W_1 (= 0.5 $\rho g h^2$), and U (= 0.5 $\rho g h b c'$) (here, s = specific gravity of concrete and c' is a correction factor for uplift force) passes through the downstream middle-third point, one gets

$$(0.5 s\rho g b h)\frac{b}{3} - (0.5 s\rho g h^2)\,\frac{h}{3} - (0.5 s\rho g b h c')\,\frac{b}{3} = 0$$

$$b^2\,(s - c') = h^2$$

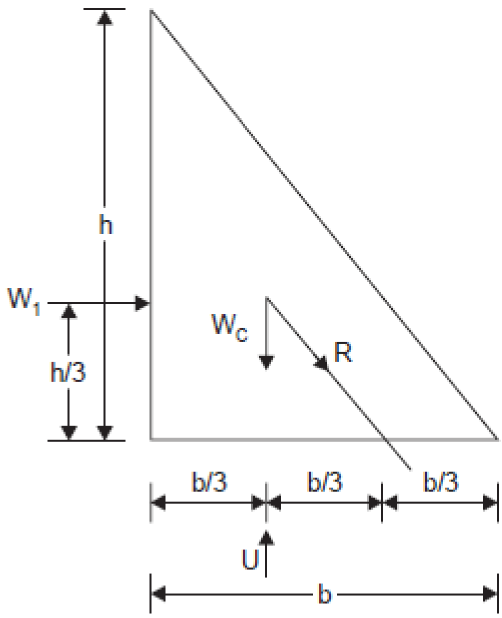


Figure : Elementary profile of a gravity dam

b = $\frac{h}{\sqrt{s-c\prime}}$

For c' = 1, b = $\frac{h}{\sqrt{s-c\prime}}$

and if uplift is ignored, c' = 0

b = $\frac{h}{\sqrt{s}}$

For no-sliding requirement, one obtains $\mu(W_c\text{-}U) = p = W_1$ in which,μ is shear - friction coefficient.

or $\mu(0.5\ s\rho gbh - 0.5\ \rho ghbc') = 0.5\ \rho gh^2$

b =$\frac{h}{\mu(s-c\prime)}$

for c' =1 b = $\frac{h}{\mu(s-1)}$

and for no uplift, c' = 0, and b = $\frac{h}{\mu s}$

it is obvious that for satisfying the requirement of stability, the elementary profile of a gravity dam should have minimum base width equal to the base widths obtained from no-sliding and no- tension criteria.

Again, for an elementary profile, $\sum W = (W_c - U)$

or $\sum W = \frac{1}{2}\rho g h(s - c')$

$$\sigma_{xy} = \frac{\sum W}{b}\left(1 \pm \frac{12cx}{b^2}\right)$$

For no tension in the dam, c = b/6

Therefore, at the toe of the dam (i.e., x= b/2)

$$\sigma_{yD} = \frac{\sum W}{b}$$

$$\sigma_{yD} = \rho g h(s - c')$$

and at the heel of the dam (i.e.,x =-b/2)

$\sigma_{yU} = 0$

Accordingly, the principal stress $\sigma_{1D} = \sigma_{yU} \sec^2\Phi_D$

$$\sigma_{1D} = \rho g h(s - c')\left[1 + \left(\frac{b}{h}\right)^2\right]$$

$$= \rho g h(s - c')\left[1 + \left(\frac{1}{s - c'}\right)^2\right]$$

$$= \rho g h(s - c') + 1$$

similarly, $(\tau_{yx})_D = \sigma_{yD} \tan\emptyset_D = \rho g h(s - c')\frac{b}{h}$

$$= \rho g h(s - c')\frac{1}{\sqrt{s - c'}}$$

$$= \rho g h\sqrt{s - c'}$$

The principal and shear stresses at the heel are, obviously, zero. similarly, when the reservoir is empty, $\sum W = 0.5\ \rho g h s$

$\sigma_{yD} = 0$

$$\sigma_{1U} = \sigma_{yU} = \frac{2\sum W}{b} = \rho g h s$$

Sometimes, depending upon whether or not the compressive stress at the toe σ_{1D} exceeds the maximum permissible stress σ_m for the material of the dam, a gravity dam is called a ‘high’ or ‘low’ dam. On this basis, the limiting height h_l is obtained by equating the expression for σ_{1D} with σ_m. Thus

$$\sigma_m = h_1(s - c' + 1)$$

$$h_1 = \frac{\sigma m}{\rho g(s - c' + 1)}$$

If the height of a gravity dam is less than h_1, it is a lowdam ; otherwise, it is a high dam.

Galleries: A gallery is a formed opening left in a dam. This may run in transverse or longitudinal direction and may run horizontally or on a slope. The shape and size varies from dam to dam and is generally governed by the functions it has to perform. Following are the purposes for which a gallery is formed in the dams.

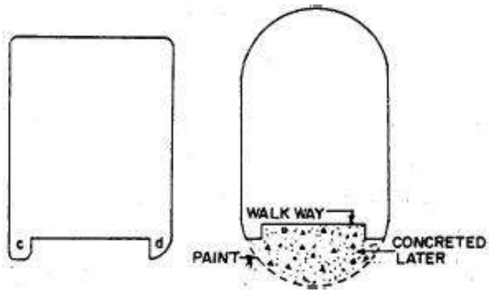
1. To provide drainage' of the dam section. Some amount of water constantly seeps through the upstream face of the dam which is drained off through galleries.

2. To provide facilities for drilling and grouting operations for foundations etc. Drillings for drain is generally resorted to clean them if they are clogged. High pressure grouting and required drilling for it is generally carried out after the completion of dam. This can be best done through galleries.

3. To provide space for header and return pipes for post cooling of concrete and grouting the longitudinal joints after completion of dam.

4. To provide access to observe and measure the behaviour of the structure after its completion by fixing thermo-couples and examining development of cracks etc.

5. To provide an access of mechanical contrivances needed for the operation ofoutlet gates and spillway gates.



Unit-III

Learning Material

DIVERSION HEAD WORKS:

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

Types of diversion head work

1. Storage headwork.
2. Diversion headwork.

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

A **diversion headwork** serves to divert the required supply into the canal from the river. A diversion headwork serves the following purposes.

1. It regulates the intake water into the canal and also it reduces fluctuations in the level of supply in the river.
2. It controls the silt entry into the canal.
3. It stores water for tiding over small periods of short supplies.

A diversion headwork can further be divided into two principal classes:

1. Temporary spurs or bunds.
2. Permanent weirs and barrages.

Temporary spurs or bunds are those which are temporary and are constructed every year after the floods.

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

Barrage: The function of a barrage is similar to that of weir, but the heading up of water is effected by the gates alone. No solid obstruction is put across the river. The crest level in the barrage is kept at a low level. During the floods, the gates are raised to clear off the high flood level, enabling the high flood to pass downstream with minimum afflux. When the flood recedes, the gates are lowered and the flow is obstructed, thus raising the water level to the upstream of the barrage. Due to this, there is less silting and better control over the levels. However, barrages are much more costly than the weirs.

LAYOUT OF DIVERSION HEADWORK:

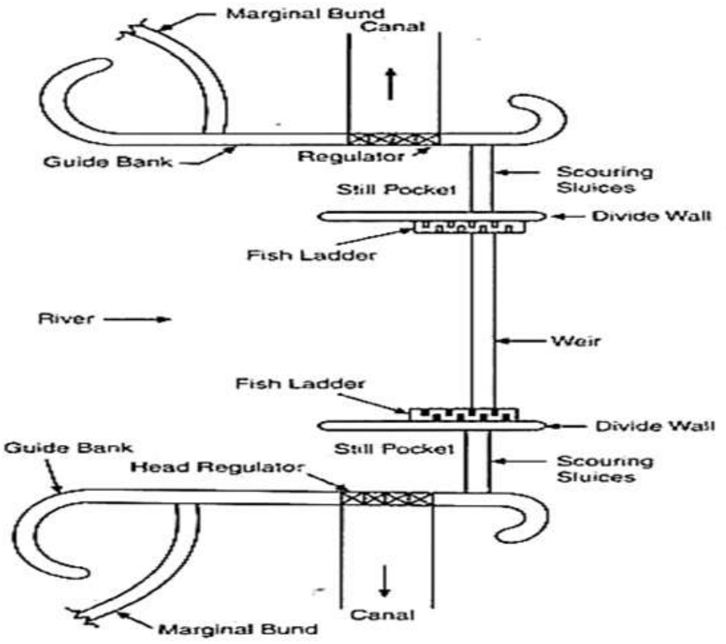


Figure: Layout of Diversion head work

COMPONENTS OF A DIVERSION HEADWORK:

1. Weir or barrage

2. Divide wall

3. Fish ladder

4. Pocket or approach channel

5. Scouring sluices

6. Silt prevention devices

7. Canal head regulator

8. River training works

Types of weirs: Based on the type of construction and design features weirs are further classified into

1) Gravity weir

2) Non-gravity weir

Gravity weir is divided into following

1) Vertical drop weir: A vertical drop weir consists of a vertical drop wall or crest wall with or without crest gates. Vertical drop weirs are suitable for any type of foundation.

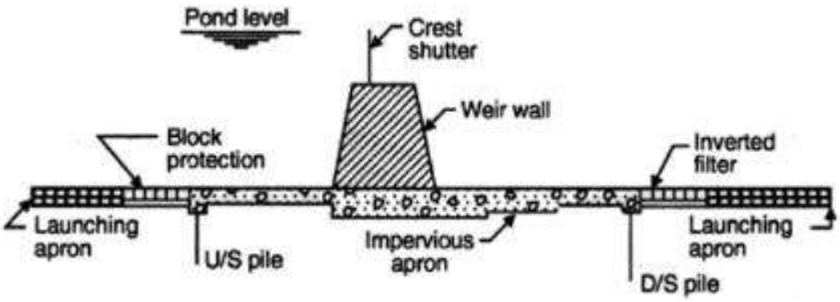


Figure: Vertical drop Weir

Launching aprons are provided on both U/S and D/S to prevent the scouring action, Piles are provided to increase the seepage length and to provide stability to the structure, Inverted filer is provided on the D/S to relieve the uplift pressure

2) Sloping weir:

a) Masonry or concrete slope weir: Weir of this type are of resent origin. They are suitable for soft sandy foundations and are generally used where the difference in weir crest and downstream river bed is limited to 3 meters.

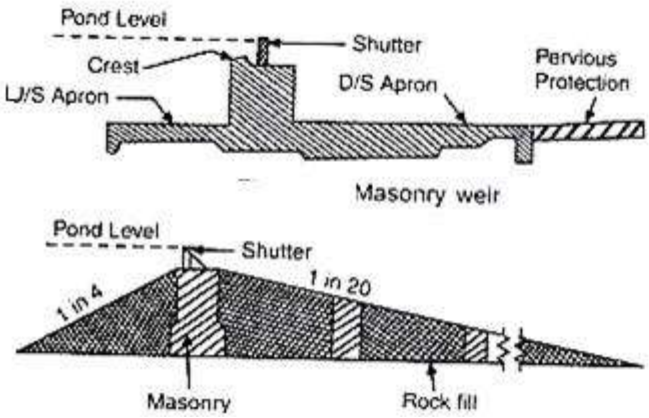


Figure : Masonry or concrete slope weir

b) Dry stone slope weir: A dry stone weir consists of a body wall and upstream and downstream rock fills layer in the form of glacis, with few intervening core walls.

3) Parabolic weir: It is similar to the spillway section of the dam. The upstream and downstream protection works are similar to that of a vertical drop or sloping weir.

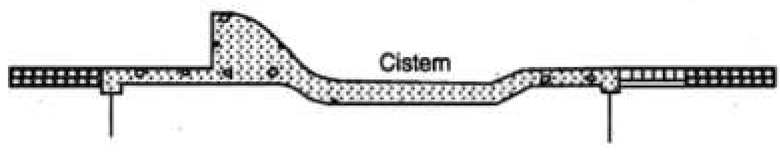


Figure : Parabolic Weir

2) Divide wall:

- The divide wall is a masonry or concrete wall constructed at right angle to the axis of the weir.

- The divide wall extends on the upstream side beyond the beginning of the canal head regulator; and on the downstream side, it extends upto the end of the loose protection of the under-sluices.
- The divide wall is a long wall constructed at right angles in the weir or barrage, it may be constructed with stone masonry or cement concrete. On the upstream side, the wall is extended just to cover the canal head regulator and on the downstream side, it is extended up to the launching apron.

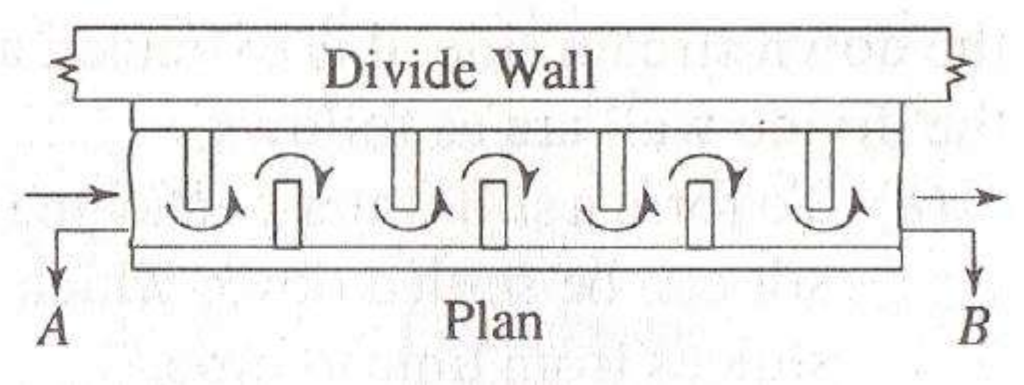


Figure : Divide Wall

The main functions of the divide walls:

- It separates the 'under-sluices' with lower crest level from the 'weir proper' with higher crest level.
- It helps in providing a comparatively less turbulent pocket near the canal head regulator, resulting in deposition of silt in this pocket and, thus, to help in the entry of silt-free water into the canal.
- It helps to keep cross-current, if any, away from the weir.

3. Fish Ladder:

When a weir is constructed across a river with a view to check the flow of water the passage is fully closed. Naturally the fishes, which are always present in river, are obstructed from moving freely. If some provision is not made for them the fish life may be perished. The structure provided for allowing free passage to the fishes is called a fish ladder. The fish ladder is designed in such a way that the velocity of flow is between 3 to 3.7 m/sec. This velocity is such that the fish can travel in or against the direction of flow easily

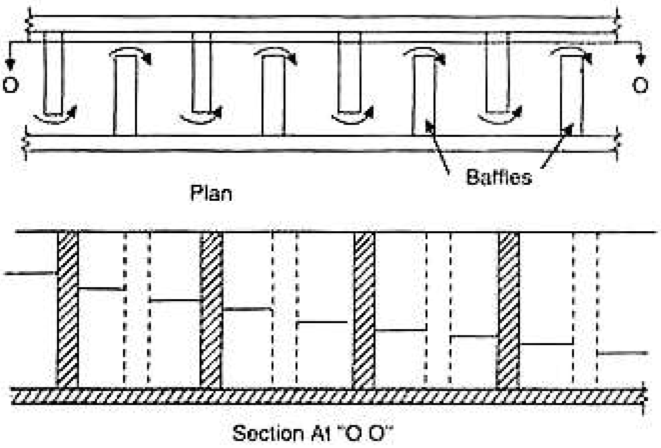


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Figure : Fish Ladder

4. Under Sluices or Scouring Sluices:

They are openings provided in the body of a weir or anicut at low levels. They are located in the smaller compartment in still pond. These sluices are perfectly controlled by means of gates which are operated from top. When a still pond is created in front of the head regulator silting takes place in the pocket. If this silting is allowed in the pocket for a long time the capacity of the pocket gets reduced. The sluices are used to remove or to scour this deposited silt. Naturally the sluices are closed but if the silt deposition is appreciably high then we will lift the gates the muddy water flows through the scouring sluices from U/S to the D/S

<u>Causes of failure of weir or barrage on permeable foundation:</u>

1. Failure due to Subsurface Flow

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

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

2. Failure by Surface Flow

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

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

Bligh's Creep Theory for Seepage Flow:

According to Bligh's Theory, the percolating water follows the outline of the base of the foundation of the hydraulic structure. In other words, water creeps along the bottom contour of the structure. The length of the path thus traversed by water is called the length of the creep. Further, it is assumed in this theory, that the loss of head is proportional to the length of the creep. If 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.

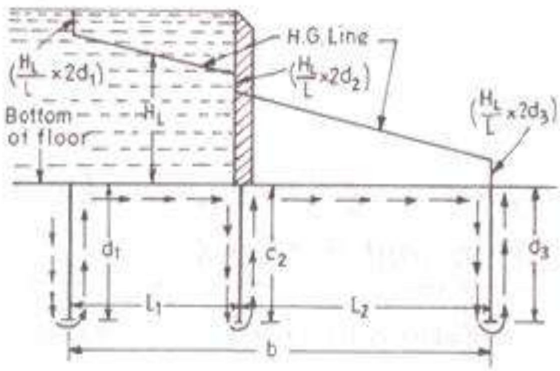


Figure : Bligh's Creep

Consider a section a shown in Fig above. Let HL be the difference of water levels between upstream anddownstream 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 = $\frac{H}{b+2(d_1+d_2+d_3)} = \frac{H_L}{L}$

Head losses equal to $\frac{H_L}{L} \times 2d_1, \frac{H_L}{L} \times 2d_2, \frac{H_L}{L} \times 2d_3$

(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:

SLNo.	Type of Soil	Value of C	Safe hydraulic gradiend 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 I/C. hence, it may be started 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 ordinates 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 counter balanced by the weight of the floor of thickness say 't'.

Uplift pressure = γ_w x h' [where γ_w is unit weight of water]

Downward pressure = (γ_w xG).t [where G is the specific gravity of the floor material]

For equilibrium,

$$\gamma_w \text{ x } h' = \gamma_w \text{ x } G.t$$

$$h' = G \text{ x } t$$

Subtracting t on both sides, we get

$$(h'-t) = (Gxt-t) = t(G-t)$$

$$t = (h'-t)/(G-1) = (h / G-1)$$

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

G – 1 = Submearged 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:

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

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

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

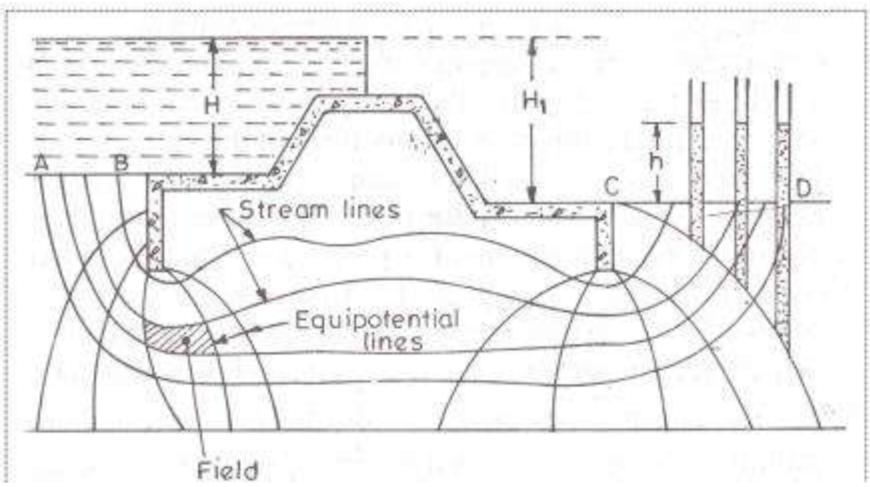
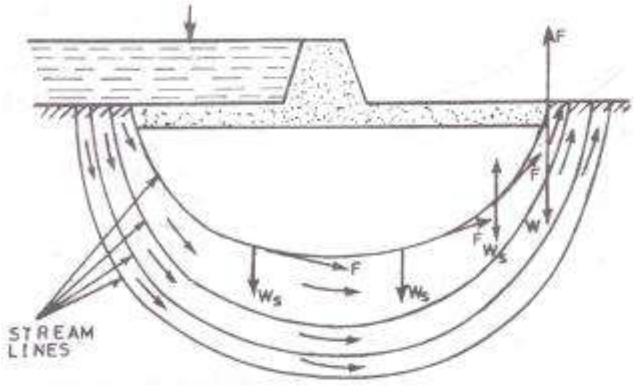


Figure : Khoslas flow net

Equipotential Lines: Treating the downstream bed as datum and assuming no water on the downstream side, it can be easily started that every streamline possesses a head equal to h1 while entering the soil; and when it emerges at the down-stream end into the atmosphere, its head is zero. Thus, the head h1 is entirely lost during the passage of water along the streamlines. 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. 6 Every water particle on line AB is having a residual head h = h1, and on CD is having a residual head h = 0, and hence, AB and CD are equipotential lines. Since an equipotential line represent the joining of points of equal residual head, hence if piezometers were installed on an equipotential line, the water will rise in all of them up to the same level as shown in figure below



(b) The seepage water exerts a force at each point in the direction of flow and tangential to the streamlines as shown in figure above. This force (F) has an upward component from the point where the streamlines turns upward. For soil grains to remain stable, the upward component of this force should be counterbalanced by the submerged weight of the soil grain. This force has the maximum disturbing tendency at the exit end, because the direction of this force at the exit point is vertically upward, and hence full force acts as its upward component. For the soil grain to remain stable, the submerged weight of soil grain should be more than this upward disturbing force. The disturbing force at any point is proportional to the gradient of pressure of water at that point (i.e. dp/dt). This gradient of pressure of water at the exit end is called the exit gradient. In order that the soil particles at exit remain stable, the upward pressure at exit should be safe. In other words, the exit gradient should be safe.

critical exit gradient:

This exit gradient is said to be critical, when the upward distributing 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 1/4 to 1/5 of the critical exit gradient is ensured, so as to kept the structure safe against piping .

the submerged weight (W_s) of a unit volume of soil is given as:

$\gamma_w (1-n)(S_s-1)$ where γ_w =unit weight of water.

S_s=specific gravity of soil particles

n = porosity of the soil material

For critical condition to occur at the exit point

$F = W_s$

where F is the upward distribution force on the grain

force F = pressure gradient at that point = dp/ dl = γ_w * dp/ 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 and curves given in Plate, which can be used for determining the percentage pressures at the various key points.

The simple profiles which hare most useful are: (i) A straight horizontal floor of negligible thickness with a sheet pile line on the u/s end and d/s end. (ii) A straight horizontal floor depressed below the bed but without any vertical cut-offs. (iii)A straight horizontal floor of negligible thickness with a sheet pile line at some intermediate point. The key points are the junctions of the floor and the pole lines on either side, and the bottom point of the pile line, and the bottom corners in the case of a depressed floor. The percentage pressures at these key points for the simple forms into which the complex profile has been broken is valid for the complex profile itself, if corrected for (a) Correction for the Mutual interference of Piles (b) Correction for the thickness of floor (c) Correction for the slope of the floor.

(a) Correction for mutual interference of piles:

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

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

Where

b' = the distance between two pile lines

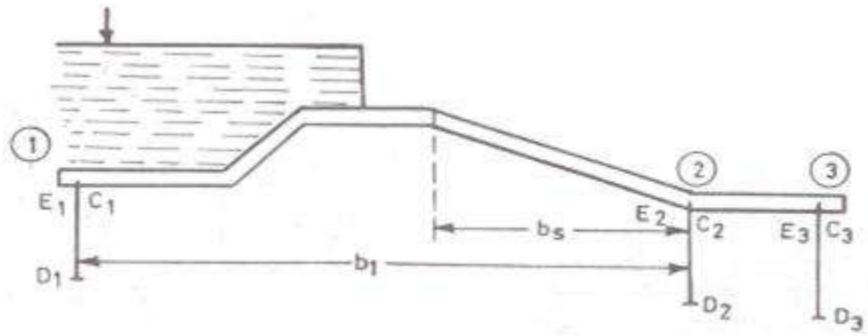
D = the depth of the pile line, the influence of which has to be determined on the neigh boring pile of depth

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

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

b = total floor length

The correction is positive for the points in the rare of back water , and subtractive for the points forward in the direction of flow. the 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 E2 due to pile no (1) shall be negative, since the point E2 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 below table

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 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 end of the slope.

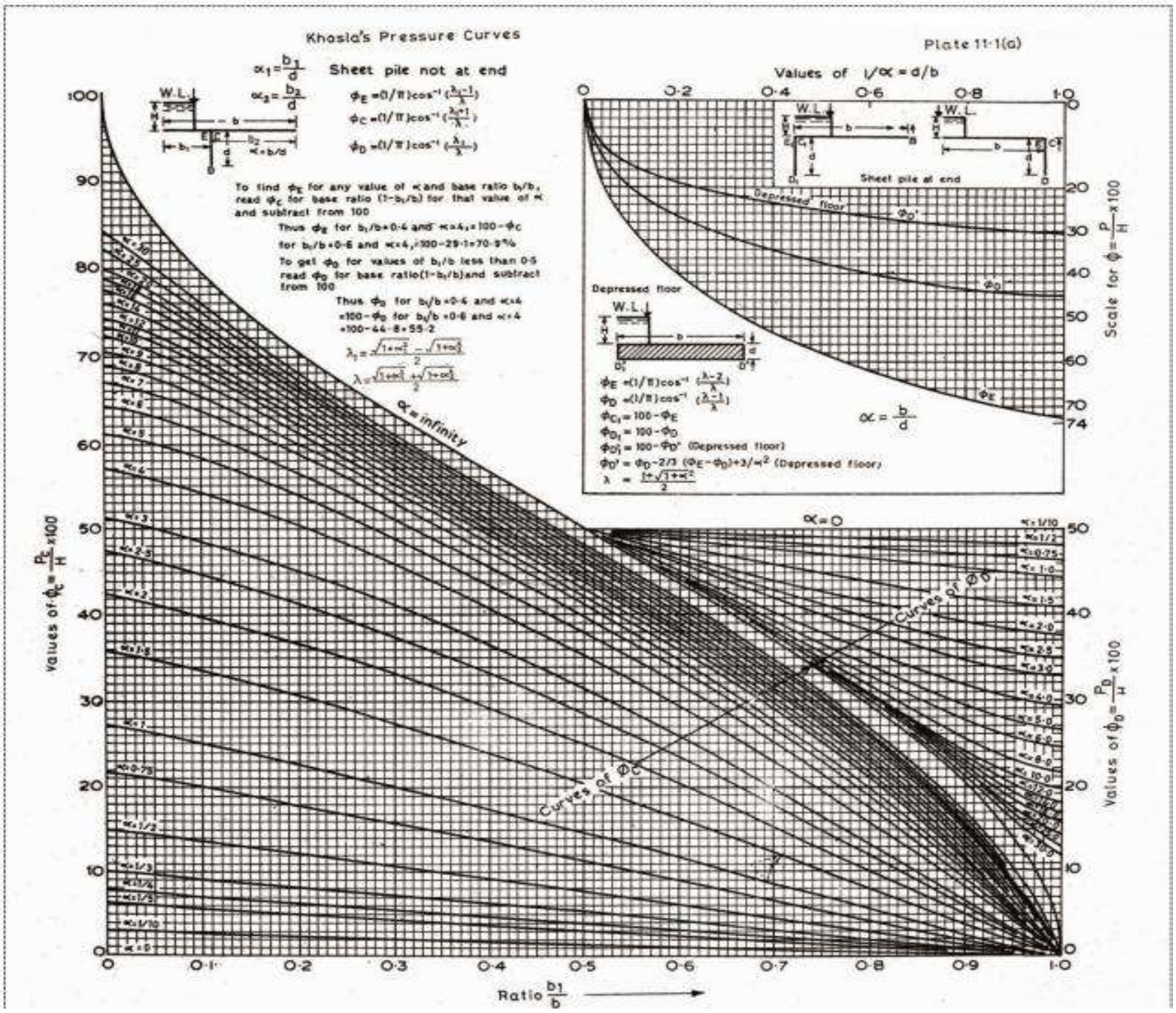
Exit gradient (G_E)

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

$$G_E = \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}} \quad \text{Where , } \lambda = \frac{1+\sqrt{1+\alpha^2}}{2} \quad \alpha = \frac{b}{d}$$

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)

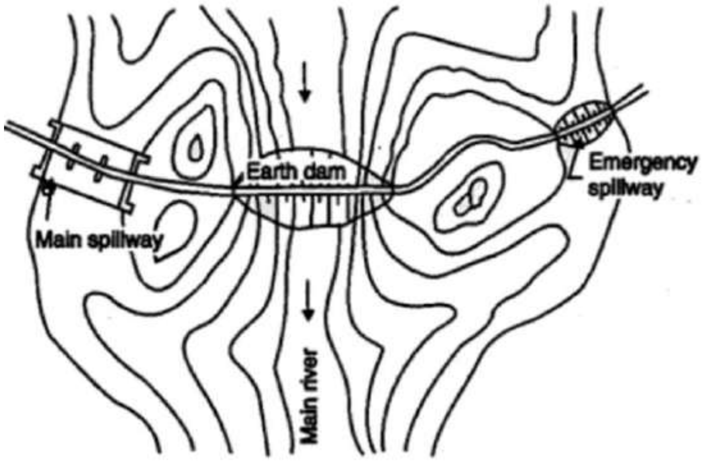


Unit-IV

Learning Material

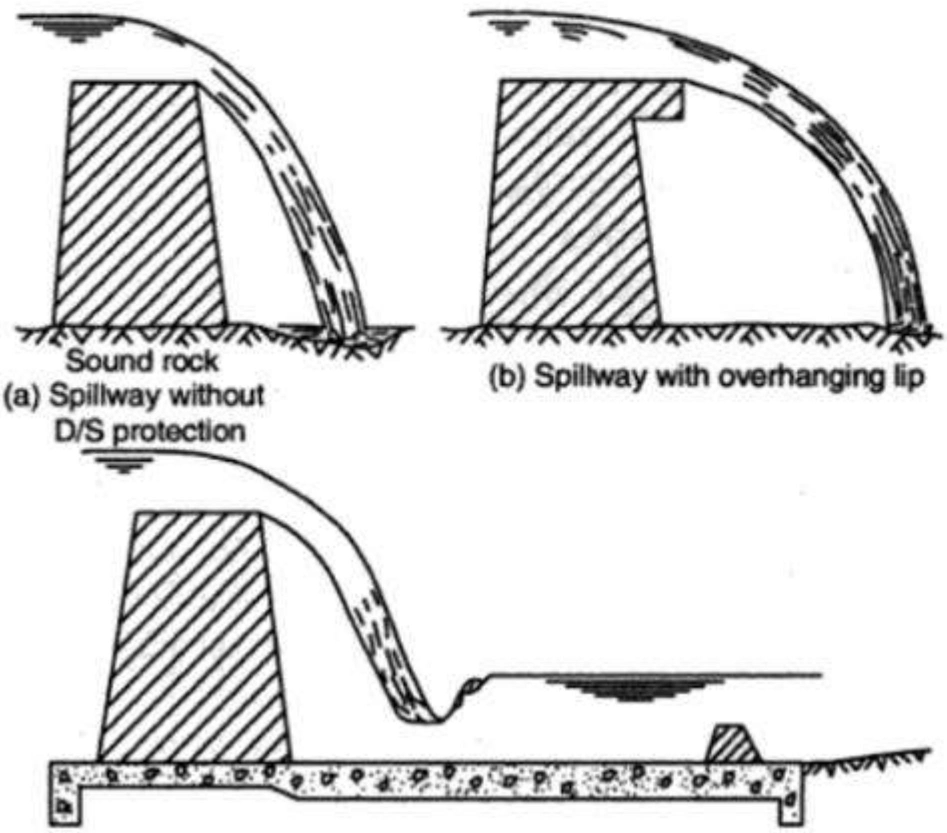
Spillway

•A spillway is the overflow portion of dam, over which surplus discharge flows from the reservoir to the downstream.
•A spillway is, therefore, called a surplussing work, designed to carry this flood water not required to be stored in the reservoir, safely to the river lower down.
•Spillways are very important structures ; many failures of the dams have been caused by improperly designed spillways or by spillways of insufficient capacity.
•A spillway is thus the safety valve for a dam.
•Sufficient capacity of spillway is of paramount importance, specially in earth and rockfill dams where overtopping may be very dangerous.
• A spillway must have the capacity to discharge major floods without damage to the dam or any appurtenant structures, at the same time keeping the reservoir level below some predetermined maximum level.
• In addition to providing sufficient capacity, the spillway must be hydraulically and structurally adequate and must be located so that spillway does not erode or undermine the downstream of the dam.
•Types of Spillways
•Based on their utility, spillways can be of two types :
(i)Main spillway (ii) Subsidiary or emergency spillway.
The main spillway is the one which is called upon to work under the design floods.
A subsidiary or emergency spillway is provided for additional safety during emergency.
Under normal reservoir operation, emergency spillways are never required to function.
Their functioning is required under the situations caused by an enforced shutdown of the outlet works or malfunctioning of spillway gates or when a flood greater than the design flood occurs.
The crest of an emergency spillway is therefore, placed at or above the designed maximum reservoir water surface. Emergency spillways are provided for earth and rockfill dams only, to avoid the overtopping of the main dam embankment because of an emergency condition.



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

Straight drop spillway

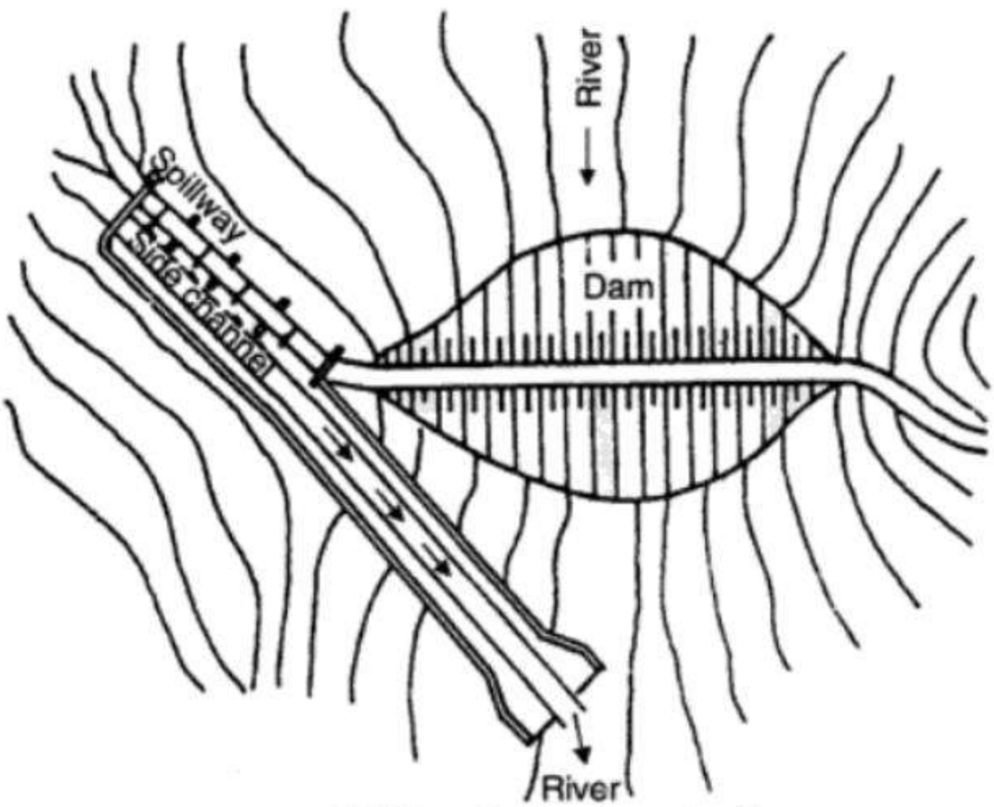


(a) Spillway without D/S protection

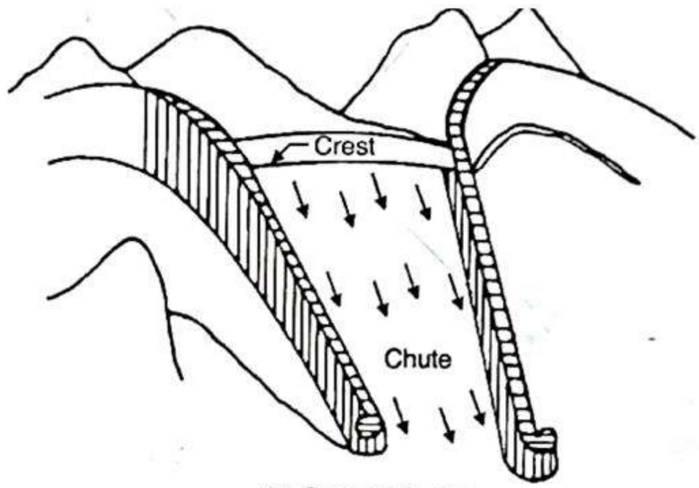
(b) Spillway with overhanging lip

(c) Spillway with D/S protection work

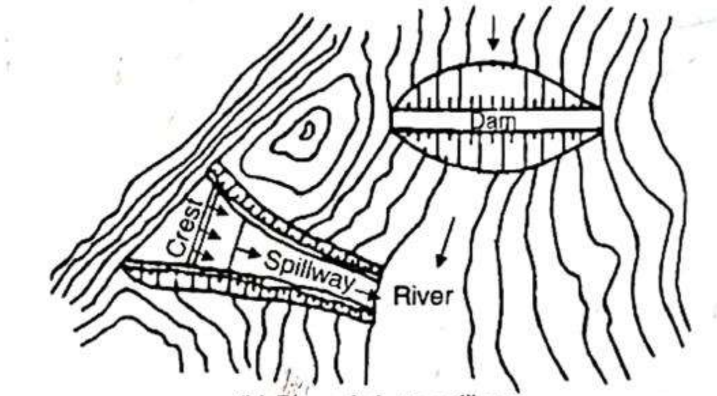
•This is the simplest type of spillway which is constructed in the form of a low height weir having downstream face either vertical or nearly vertical.
•Water drops freely from the crest, and the underside of the falling nappe is ventilated sufficiently to prevent a pulsating, fluctuating, jet. Occasionally, the crest is extended in the form of an overhanging lip to direct the small discharge away from the face of the overfall section
•Fig.(a) shows a straight drop spillway constructed on sound rock
•Fig. (c) shows an impact block basin useful for low heads. The block gives reasonably good dissipation of energy for a wide range of tailwater depth. The energy is dissipated by means of turbulence caused by the impingement of the flow upon the impact block.
Side channel spillway
•A side channel spillway is the one in which the flow, after passing over a weir or ogee crest, is carried away by channel running essentially parallel to the crest.
•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.
•However, this flow may differ from that of over-flow spillway in that the flow in the trough may partly sub-merge the flow over the crest.
•Side channel spillway is suitable for earth or rockfill dams in narrow canyons and for other situations where direct over-flow is not permissible.
•Side channel spillway is also the best choice where a long overflow crest is desired in order to limit the sur-charge head and the abutments are steep.
•This type of spillway is also desirable where the spill-way discharge is to be connected to narrow discharge channel or tunnel



(a) Plan of side channel spillway



(a) Schematic view



(b) Plan of chute spillway

Chute or trough spillway

•A chute spillway is the one which passes the surplus discharge through a sloped open channel, called a chute or trough, placed either along a dam abutment or through a saddle.

•Generally, this type of spillway is provided on earth or rockfill dam, and is isolated from the main dam. Its crest is kept normal to its center line.

• It consist of a discharge channel to the river in an excavated trench which is usually paved with concrete in whole or in part. The crest or the spillway proper is usually of insignificant height or actually flat

•The chute is sometimes of constant width, but usually narrowed for economy and then widened near the end to reduce discharge velocity. Factors influencing the selection of chute spill-ways are the simplicity of their design and construction, their adaptability to almost any foundation condition and the overall economy often obtained by the use of large amounts of spillway excavation in the dam embankment.

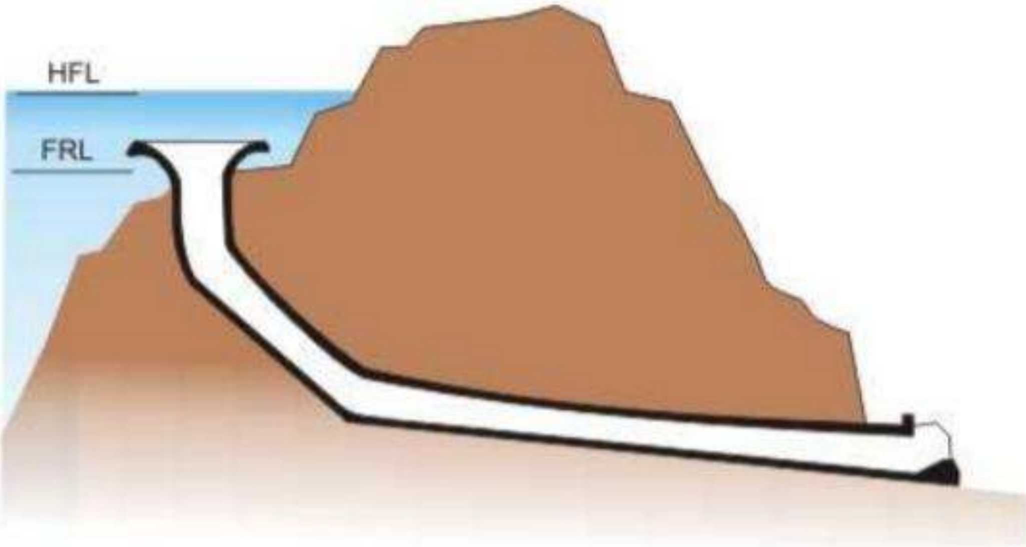
Conduit spillway or tunnel spillway

•Conduit spillway or tunnel spillway is the one in which a closed channel is used to convey the discharge around or under a dam.

• The closed channel may be in the form of a vertical or inclined shaft, a horizontal tunnel through earth dam or a conduit constructed with open cut and backfilled with earth materials.

•These spillways ate designed to flow partly full.

•Full flow is not allowed in the tunnel or conduit,/ the siphonic action may develop and negative Pressures may be created in the conduit. To ensure free flow in the tunnel, the ratio of flow area to the total tunnel area is often limited to 75%.



Shaft spillway

•Shaft spillway, drop inlet spillway or morning glory spillway is the one which has horizontally positioned lip through which water enters and then drops through a vertical or sloping shaft, and then to a horizontal conduit which convey the water past the dam.

• A shaft spillway can often be used where there is inadequate space for other types of spillways. It is generally not desirable to use a spillway over or through an earth dam.

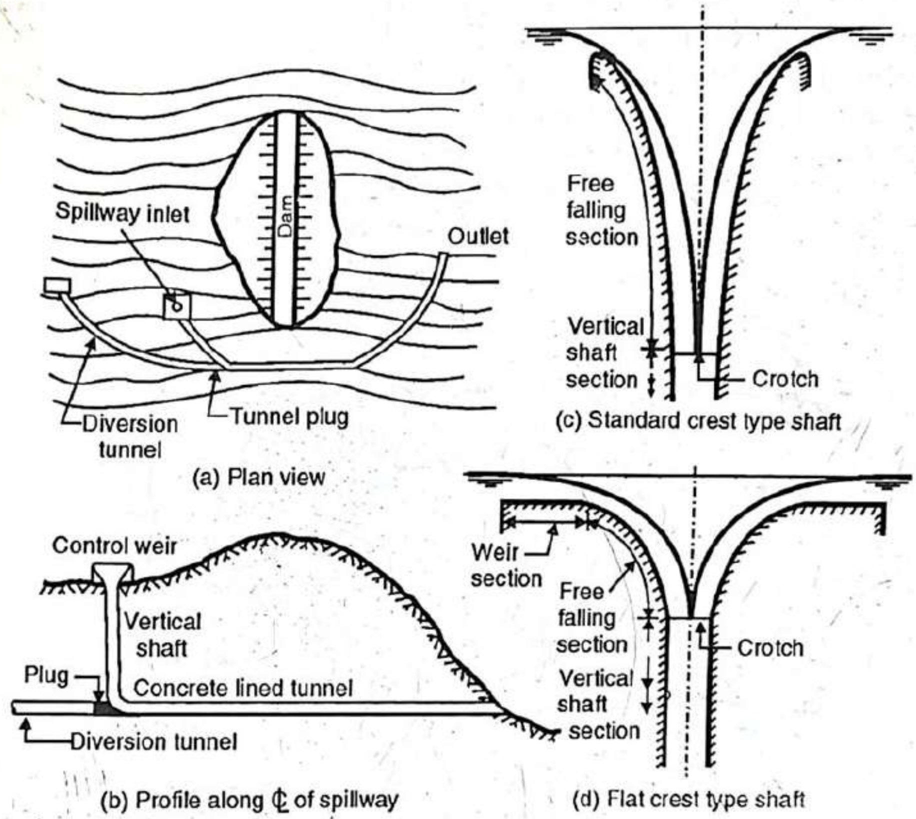
•Thus, on an earth dam location, if there is no enough space or if the topography prevents the use of a chute or side channel spillway., the best alternative would be to use shaft spillway.

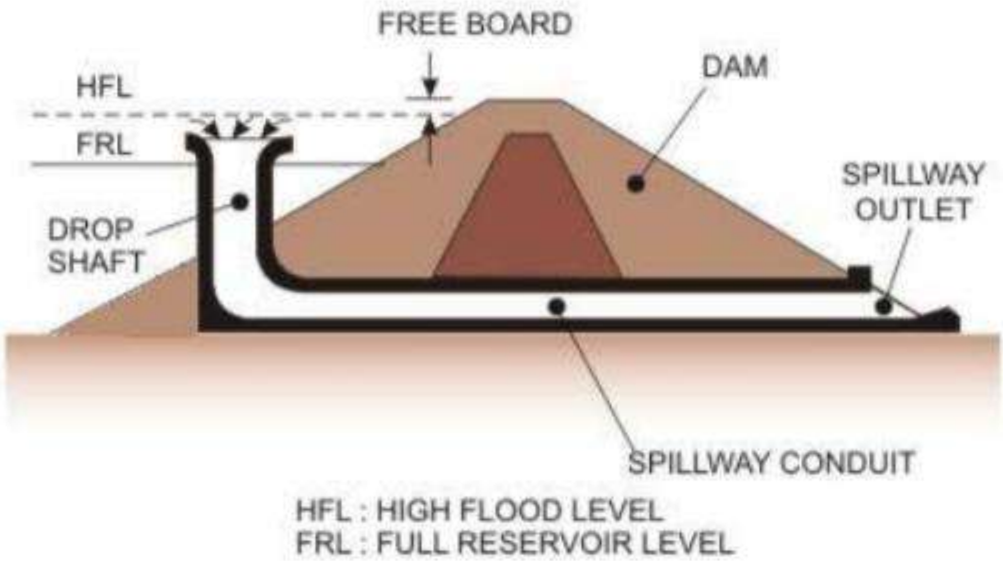
•A shaft spillway normally consists of three elements : (i) overflow control weir (called a flared inlet or morning glory, (ii) vertical control, and (iii) closed. discharge channel.

•Small shaft spillway may be constructed entirely of metal or concrete pipe or clay tile. The vertical shaft of large structures is usually made of reinforced concrete and the horizontal conduit is tunnelled in rock.

•Sometimes the diversion tunnel, used for diverting river water-during the construction may be plugged at the upstream end, and , then connected to the vertical shaft of shaft spillway

Shaft spillway





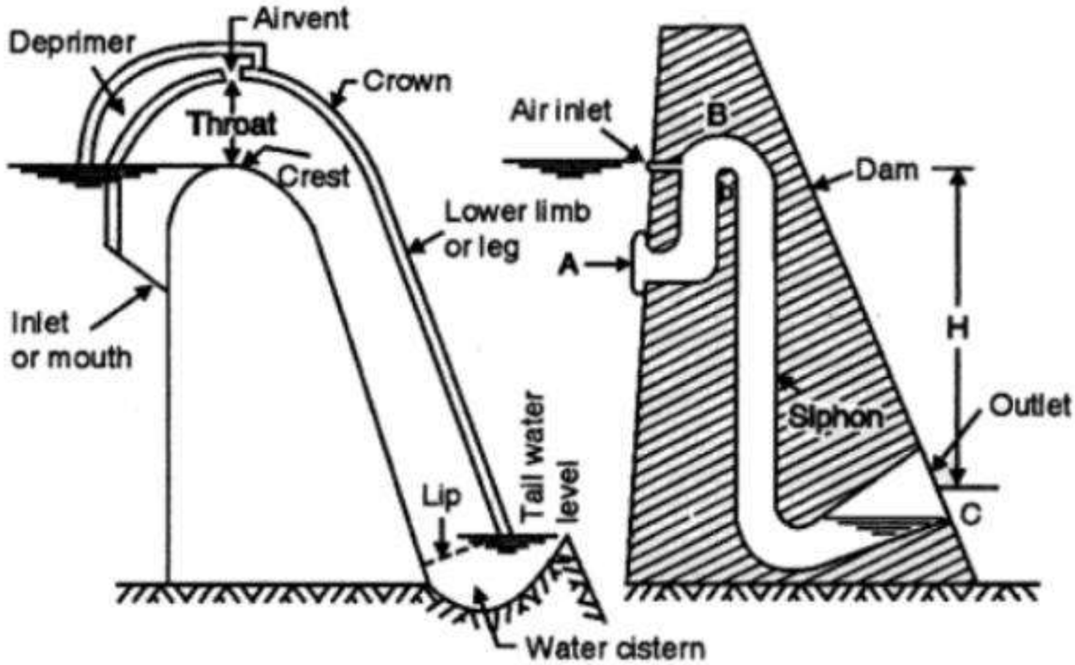
Siphon spillways

•Instead of allowing water to spill over the crest of a dam or weir, the surplus water may be discharged downstream through a siphon spillway consisting of one or more siphon units.

•A siphon spillway is the one which utilizes the siphonic action to discharge the surplus water.

• Generally, a siphon spillway consists of 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 level.

•When the water level in the reservoir rises above its normal level, water flows over the crest, and then siphonic action takes place. Continuous flow is then maintained by suction effect due to the gravity pull of the water in the lower leg of the siphon. There are generally two types of siphon spillways : (i) Saddle siphon spillway (ii) Volute siphon spillway.



(a) Hooded type (b) Tilted outlet type

•Fig. shows two types of saddle siphons, the former being more common. It essentially consists of reinforced concrete hood constructed over an overflow section of gravity dam. The inlet or mouth of the main hood is kept submerged in water so that floating debris etc. do not enter the siphon. A small deprimer hood is kept above the main hood, and both these hoods are connected through an air vent.

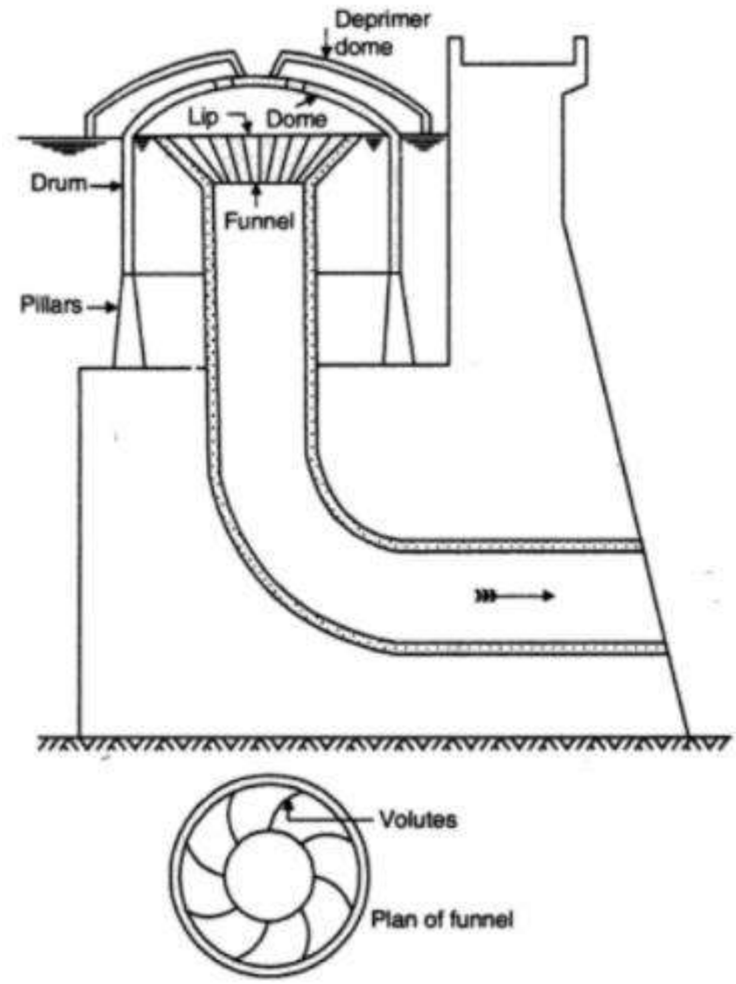
•The inlet of the deprimer hood is kept just at the reservoir level. The lower limb or leg of the main hood is generally kept submerged in the tailwater so that there is no air entry from the down stream end.

• When the water level in the reservoir increases, it seals the air entry though the mouth of the deprimer dome. Water spills over the crest of the spillway.

• Since air entry is sealed both from u/s as well as dls end, the spilling water sucks all the remaining air from the hood within minutes. Siphonic action is established after the air in the bend over the crest has been exhausted.

•This action is known as priming. The siphon runs full and water is discharged downstream under a greater effective (siphonic) head.

•During receding flood, when water level has gone down just to the reservoir level, air enters through the mouth of deprimer dome and the siphonic action is broken. This action is called depriming of the siphon and is achieved through the deprimer dome.



•Volute siphon spillway is a special type of siphon spillway designed in India by Ganesh Iyer.

• It consists of a vertical barrel or shaft bent at the discharge end and opened out in the form of a funnel at the top.

•The top or lip of the funnel is kept at the reservoir level and a number of volutes (like blades of a centrifugal pump or turbine) are fixed in the funnel to induce a spiral motion to the water passing along them.

•A dome, supported on number of pillars, is placed over the funnel. Over the main dome is attached a deprimer dome.

•The entrance end of the deprimer dome is kept just at reservoir level. When water level in the reservoir rises, it seals the air entry and sheet of water enters to the funnel from all sides. The spiral flow of water through the volute causes suction which sucks all the remaining air and priming is thus achieved very quickly. The siphon then runs full. When the reservoir level falls, air enters through the deprimer dome thus breaking siphonic action.

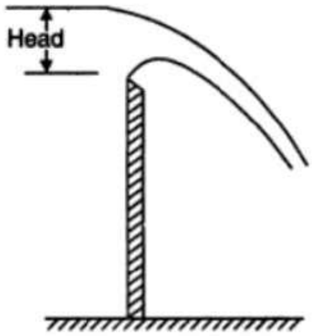
The discharge through the volute siphon is given by

$$Q = A\sqrt{2g(H - H_L)} = CA.\sqrt{2gH}$$

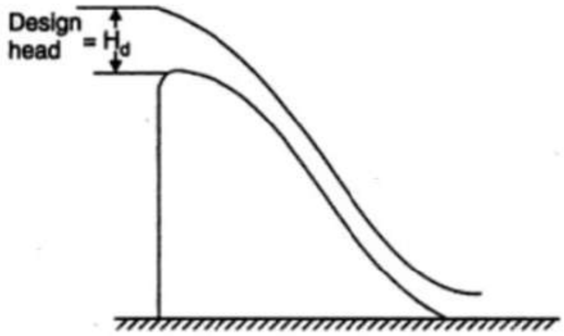
where A = area of cross-section of pipe.
C = coefficient of discharge.
H = maximum operating head.
H_L = head loss through the siphon.

Ogee Spillway

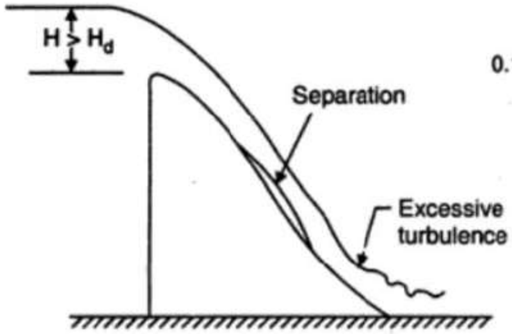
•This is the most common type of spillway provided on gravity dams. The profile of the spillway is Ogee or 'S' shaped.
•The overflowing water is guided smoothly over the crest and profile of the spillway so that the overflow water does not break contact with the spillway surface. If this is not assured, a vacuum may form at the point of separation and cavitation may occur.
•In addition to cavitation, vibration from the alternate making and breaking of contact between the water and face of the dam may result in serious structural damage. Hence the upper profile of the ogee is made to conform with the lower nappe of a freely falling jet of water over a sharp crested weir, when the flow rate corresponds to the maximum designed capacity of the spillway.
•Here, the essential difference between the straight drop spillway and the ogee or overflow spillway should be clearly noted. In the former type, the jet falls clearly away from the face of the spillway and the gap between the jet and face is kept ventilated.
•In the ogee or overflow spillway, the falling water is made to glide over the curved profile of the spillway.



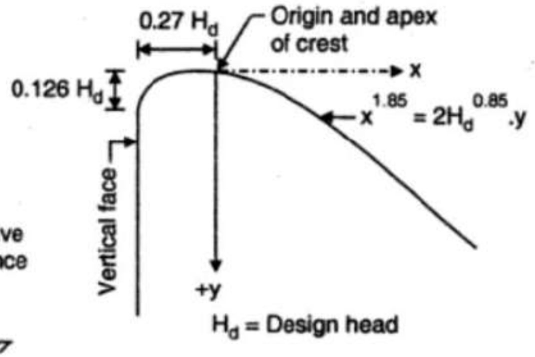
(a) Sharp crested weir



(b) Ogee spillway at design flow



(c) Ogee spillway with greater flow



(d) Elements of nappe-shaped crest profile

Head : The head (H) is the distance measured vertically from the water surface (upstream of the commencement of drawdown) to the crest axis.

Design head (H_d) : The design head is that value of head for which the ogee profile is designed.

Head due to velocity of approach (H_a) : It is the velocity head given by = $V_a^2/2g$, where V. is the velocity of approach.

Total energy head (H_e) : It is equal to the actual head plus the head due velocity of approach. Thus. H_e=H +H_a

•Ogee profile : The ogee profile to be acceptable should provide maximum possible hydraulic efficiency, structural stability, and economy and also avoid the formation of objectionable sub-atmospheric pressures at the surface.

•Fig.(a) shows the flow over a sharp crested weir.

•The ideal spillway (b) should have the form of the underside nappe of a sharp crested weir

•Fig (a)when the flow rate corresponds to the maximum design capacity of spillway. However, the head during an unanticipated flood may rise higher than the design head Fig(c) and the lower nappe of the falling jet may leave the ogee profile, thereby causing negative pressure and cavitation.

•On the contrary, when the head on the spillway is less than the design head, the falling jet would adhere to the profile of ogee, causing a positive hydrostatic pressure and reducing the discharging capacity.

(a) Crest profile for vertical u/s Face

(i) Design criteria of downstram profile

The details of the dowstream crest profile for vertical u/s face are shown in [Fig.11.3(*d*)]. The downstream curve of the ogee has the following equation:

$$x^{1.85} = 2\,H_d^{0.85}\,.\,y$$

•Where x and y are the coordinates of the crest profile measured from the apex of the crest and H_d is the design head excluding head due to velocity of approach.

Design criteria of upstream crest profile

•The u/s profile should be tangential to the vertical face and should have zero slope at the crest axis to ensure that there is no discontinuity along surface of flow

According to the latest analytical studies of U.S. Army, the upstream curve of the ogee shape has the following equation.

$$y = \frac{0.724(x + 0.270\,H_d)^{1.85}}{H_d^{0.85}} + 0.126\,H_d - 0.4315\,H_d^{0.375}(x + 0.27\,H_d)^{0.625}$$

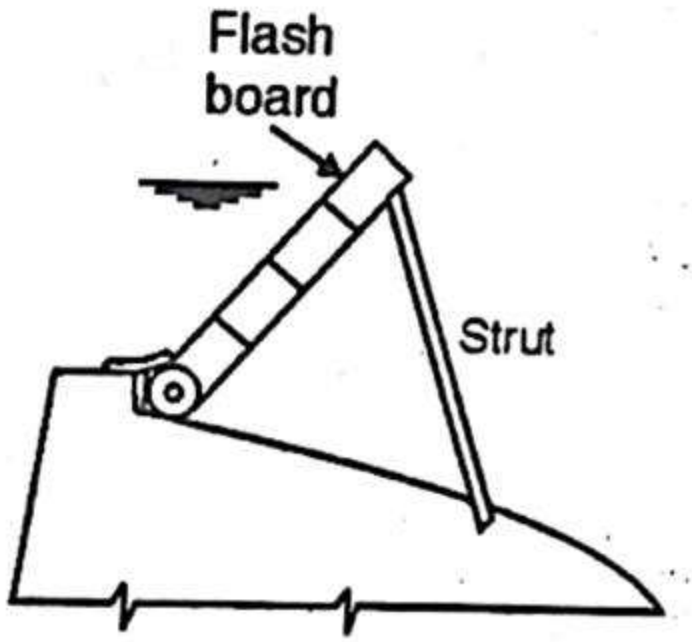
Crest profile for inclined u/s face

$X^n = KH_d^{n-1}\,.y$

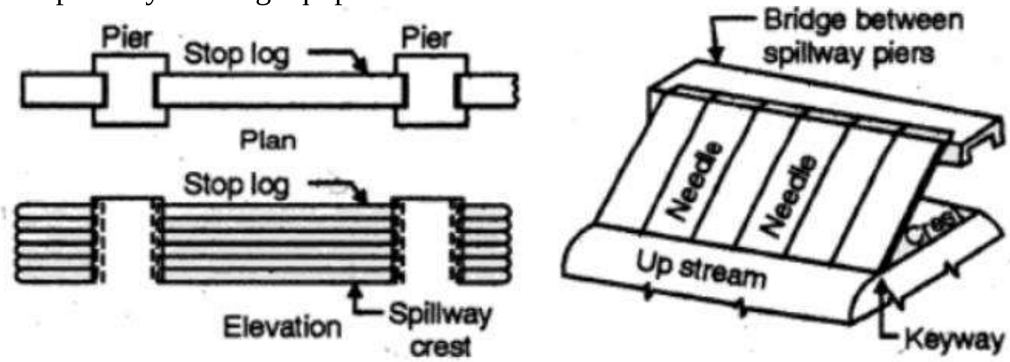
Where x and y are the coordinates of the crest of the spillway with origin at the highest point of the crest.

The following are some of the common gates over spillways

1. Flash boards, stop logs and needles
2. Radial gates.
3. Drum gates.
4. Vertical lift gate
5. Bear trap gates
6. Rolling gates

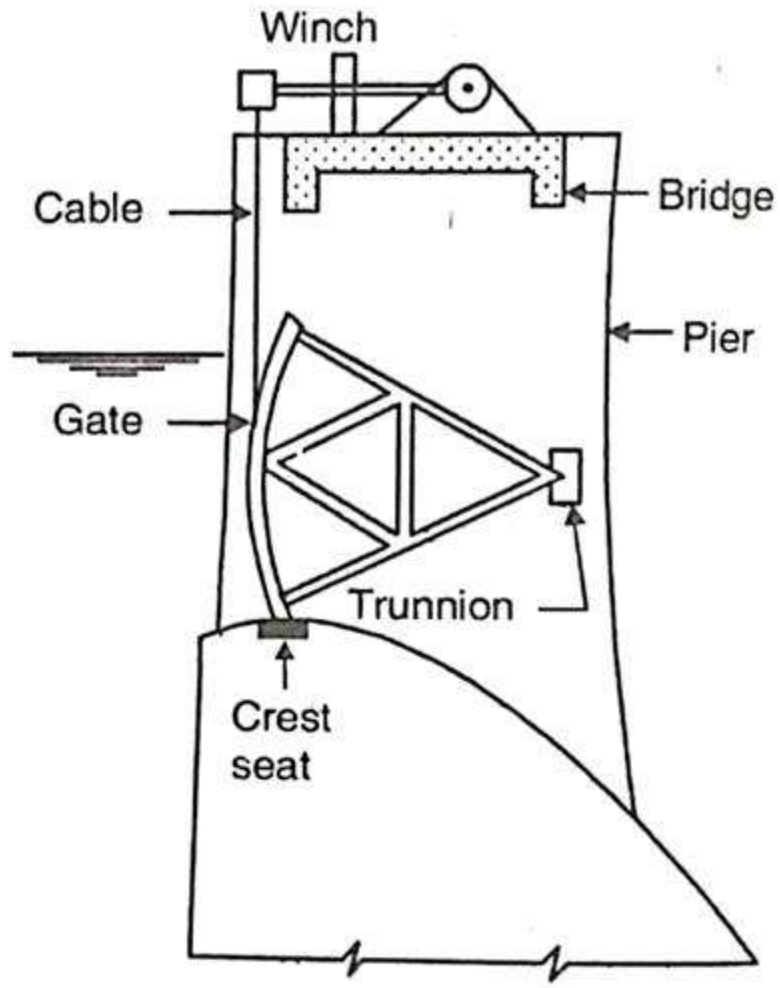


•Flash boards : These are the temporary gates used only for small spillway of minor importance.
•Fig. shows two types of flash boards. They consist of wooden panels supported by pins on the edges. The pins are supported on pipe sockets along the crest of the dam. Flash board may be either braced or hinged or automatic. Automatic flash boards are designed in such a way that they fall automatically at a certain head.
•Stop logs . They consist of horizontal timber planks spanning across piers having grooves. These planks are kept over one another as shown in Fig.and can be removed either by hand or with the help of any hoisting equipment.



(b) Stop logs (c) Needles

•Needles : Needles consists of wooden planks kept in inclined position, with lower ends resting in a keyway on the spillway crest and upper ends at the top of a bridge girder, as shown in Fig.
Radial Gates

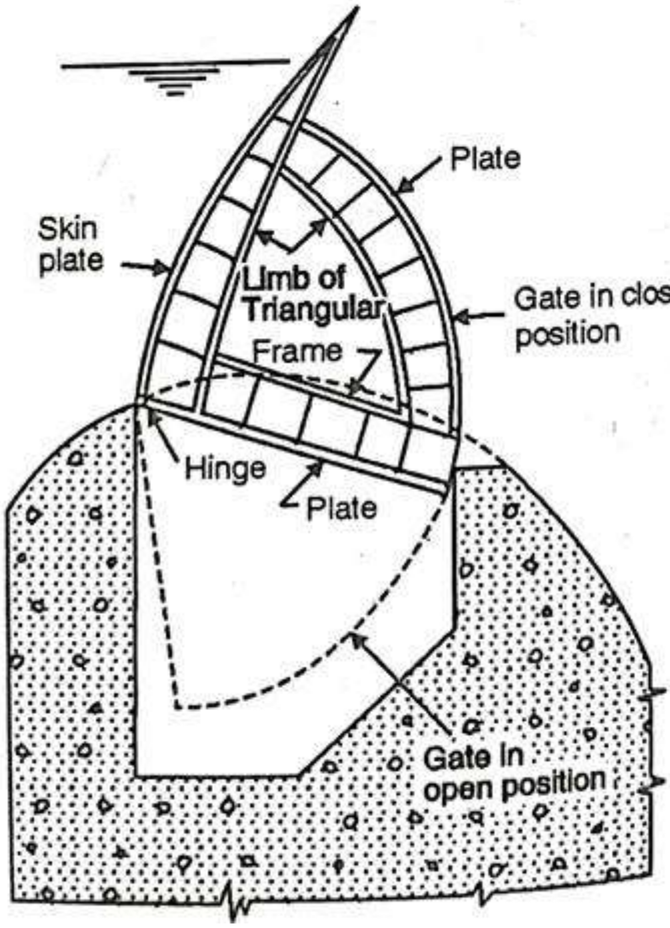


Radial Gates A radial gate, also known as a tainter gate has its water supporting face, made of steel plates, in the shape of sector of a circle, properly braced and hinged at the pivot, as shown in Fig.

•The gate can thus be male to rotate about fixed horizontal axis.

•The load of the gate and water etc. is carried on bearings, mounted on piers. The gate can be lifted by means of ropes and chains acting simultaneously at both ends or with the help of power driven winches.

Drum Gates



•Drum' Gates Drum gates are normally used for long span. Fig. shows the U.S.B.R. type drum gate which consists of a circular sector in cross-section formed by skin plates attached to internal bearings.

• It is hinged at the centre of curvature in such a way that the entire sector may be raised above the crest or may be lowered so that upper surface becomes coincident with the crest line.

•The buoyant forces due to head water pressure underneath aid in its lifting. It is enclosed on all the three faces and at ends to form a water-tight vessel.

Vertical Lift Gates or Rectangular Gates

•These are rectangular gates spanning horizontally between the grooves made in the supporting spillway piers

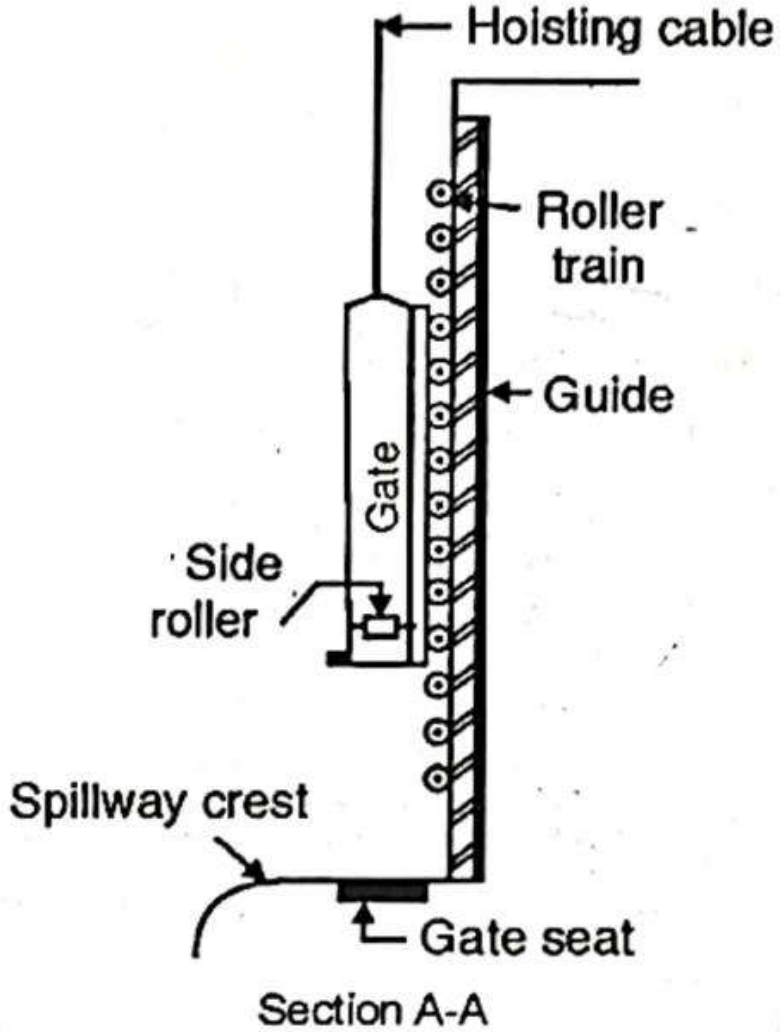
• The grooves are generally lined with rolled steel channel sections of appropriate size, so as to provide a smooth bearing surface having sufficient bearing strength and are known as grove guides.

•These rectangular gates move between the groove guides, and can be raised or lowered by a hoisting mechanism at the top.

•The gates are often made of steel, although they may be made of concrete or wood. They are generally placed vertical, although they may be kept slightly inclined downstream

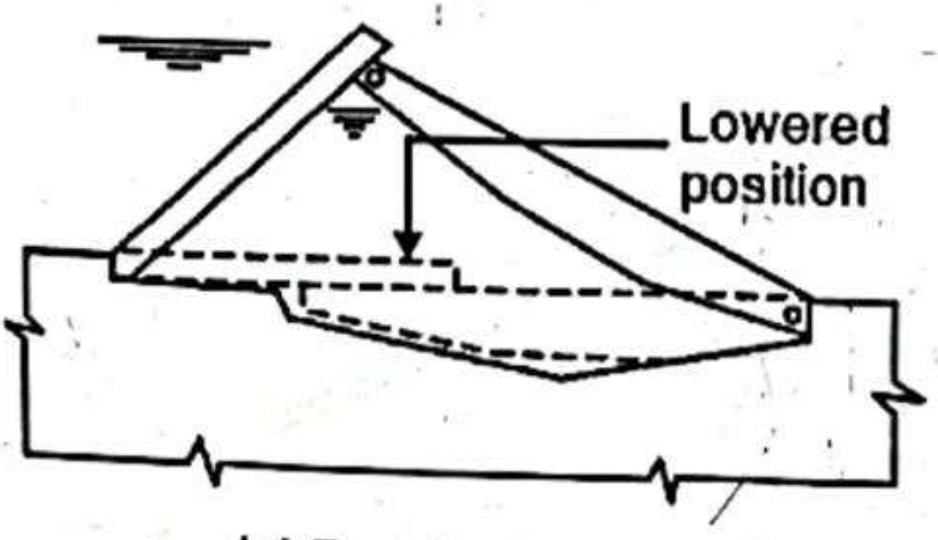
•Because of the hydrostatic force caused by the upstream water standing against the gate, large friction is developed between the gate and the downstream groove guides. Hence, if the gate is in direct contact with the guides, as is there in a sliding gate, large friction, will be developed, and it will be very difficult to move the gate.

• Hence, in a sliding gate relatively larger hoisting capacity is required to operate the gate because of the sliding friction that has to be overcome. The sliding gates are, therefore, seldom used.



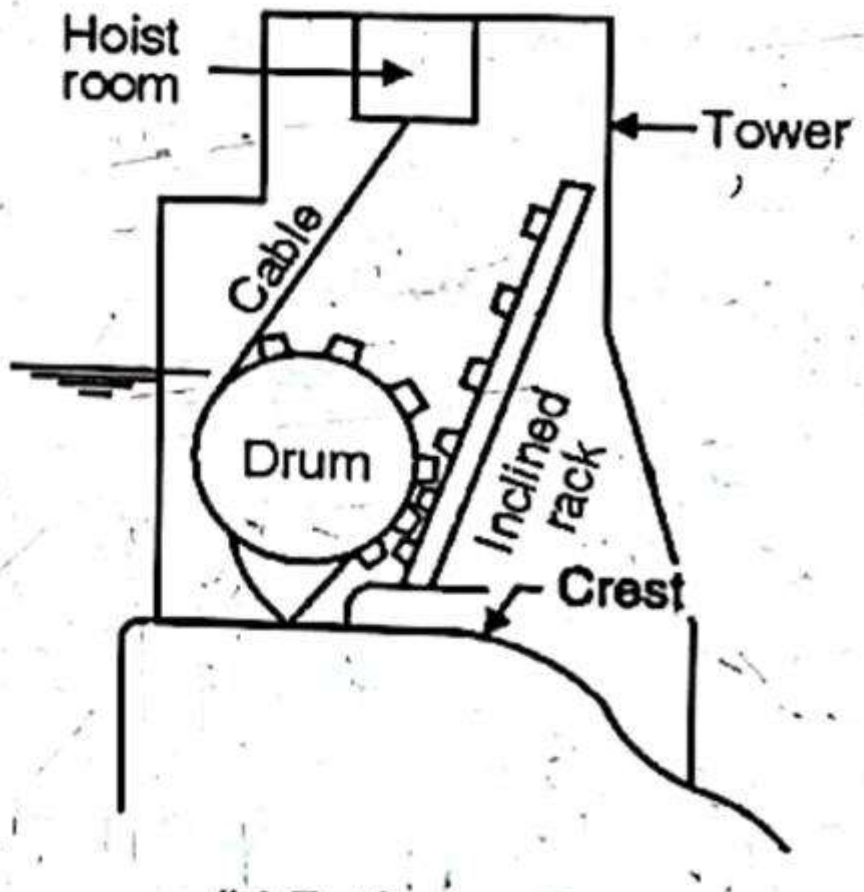
Bear Trap Gates

Bear trap gate consists of two leaves of either timber or steel hinged to the dam.
• These gates are lifted up by admitting water to the space under the leaves. The downward or downstream leaf is often made hollow so that its buoyancy aids in the lifting operation.
• These gates are often used for low navigation dams.



(a) Bear trap gate

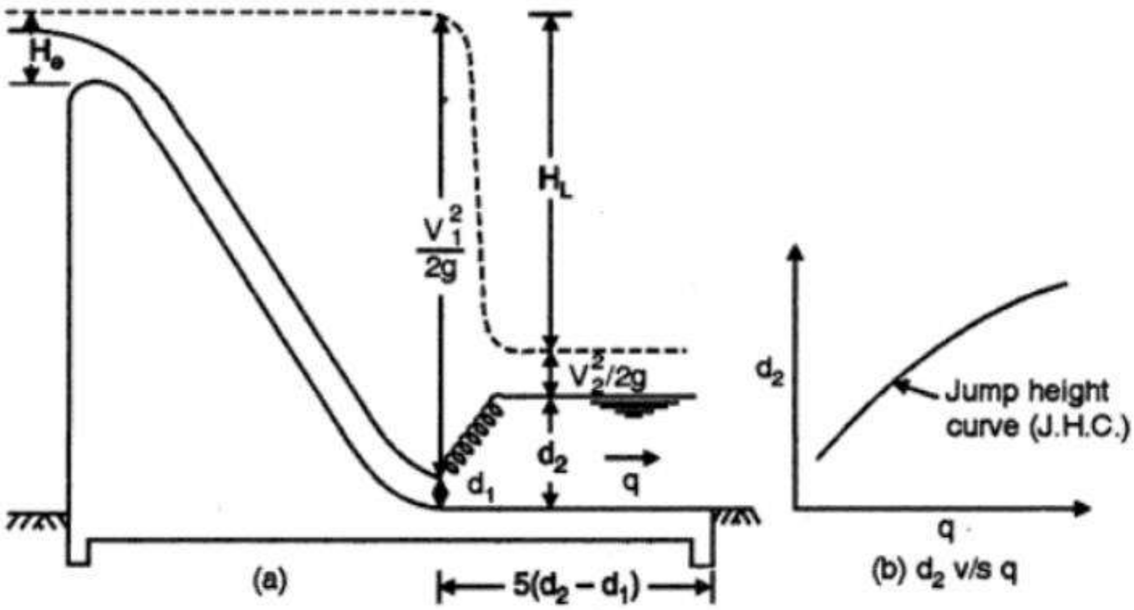
Rolling Gate



(b) Rolling gate

•A rolling gate essentially consists of steel cylinder as large in diameter as the height of opening and spanning between piers.
•A heavy annular rim having gear teeth at its periphery encircles each end of the cylinder.

•Each pier has an inclined rack which engages the gear teeth. The gate is rolled up the inclined rack by means of pull from the hoisting cable operated from the hoist room.
•A cylindrical segment, attached to the lower portion of the gate, makes contact with spillway.
•When flood discharge passes over the spillway crest, it has high potential energy which gets converted into kinetic energy as it glides along it. This high energy has to be dissipated otherwise it would cause erosion at the downstream toe. There is a varied practice regarding the extent of protection against erosion below a spillway to be provided. In general, the protection can be rendered by two ways : (i) By dissipating the energy by means of hydraulic jump.
(ii) By directing the jet of water so as to fall away from the structure by a deflector bucket or lip, and dissipating the energy by impact.
Hydraulic jump computations



1. **For the given discharge q per metre length of the spillway, calculate the head H_e over the crest to the total energy level :**

$$H_e = \left(\frac{q}{C}\right)^{2/3}$$

2. **Find total energy level at u/s :**

$$\text{u/s T.E.L.} = \text{crest level} + H_e$$

3. Assuming no losses, the specific energy E_1 at the toe of the spillway will be equal to the T.E.L. u/s.

$$E_1 = \text{T.E.L. u/s.}$$

4. Knowing E_1 and q, find the prejump depth d_1 by trial and error from the relation:

$$E_1 = d_1 + \frac{V_1^2}{2g} = d_1 + \frac{q^2}{2g\,d_1^2}$$

5. Calculate Froude Number F_1 :

$$F_1 = \frac{V_1}{\sqrt{g\,d_1}} = \frac{q}{\sqrt{g\,d_1^3}}$$

6. Compute the post jump depth d_2 from the relations :

$$d_2 = \frac{d_1}{2}\left[\sqrt{1+8F_1^2} - 1\right]$$

or by

$$d_2 = \frac{d_1}{2}\left[\sqrt{1+\frac{8q^2}{g d_1^3}} - 1\right]$$

Plotting jump height curve

•The above equations can be used for calculating post jump depths d_2 for various values of flow discharge q.

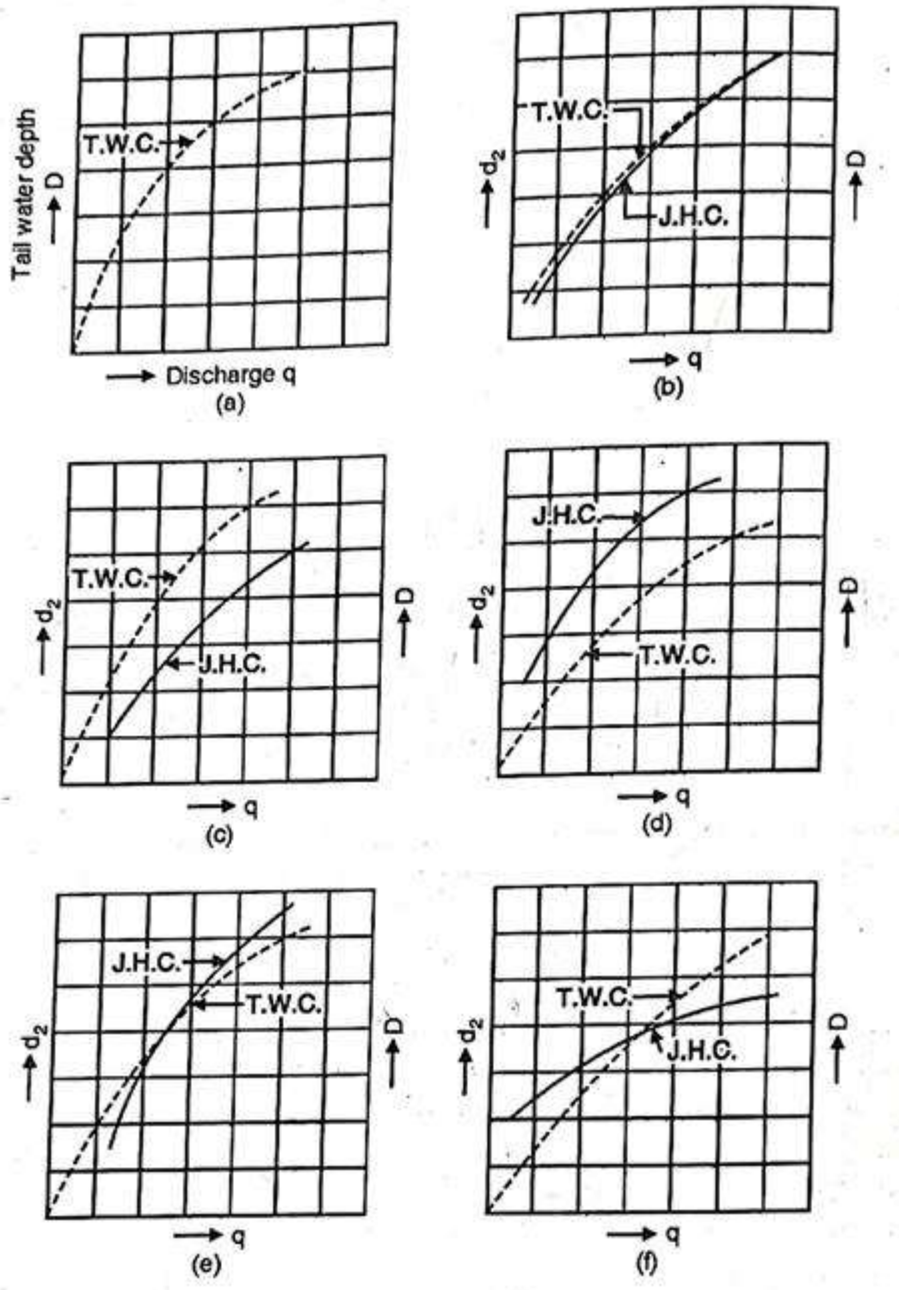
•Fig(b) shows such a curve, known as jump height curve (J.H.C.) showing the variation of post jump depth d_2 with the discharge q.

•**Plotting tails water curve (T.W.C.)** The efficiency of the hydraulic jump in dissipating the energy, and the corresponding protection works in the stilling basin will depend upon the position or height of tail water in relation to the post jump depth d_2 for that discharge.

•Tail water curve gives the relation between the discharge q and the tail water depth D.

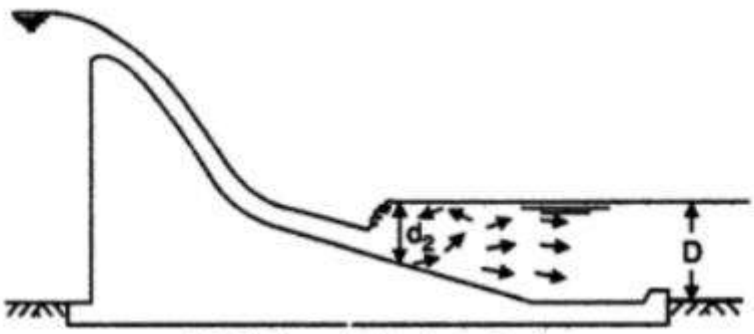
•Relative position of jump height curve and tail water curve. There may be five conditions that govern the relationship between the jump height curve (J.H.C.) and tail water curve (T.W.C.) :

1. Both the curves coincide
2. J.H.C. lies lower than T.W.C. at all discharges
3. J.H.C. lies above T.W.C. at all discharges
4. J.H.C. lies lower than T.W.C. at small discharges and higher than T.W.C. at large discharges
5. J.H.C. lies above T.W.C. at small discharges and lower at higher discharges

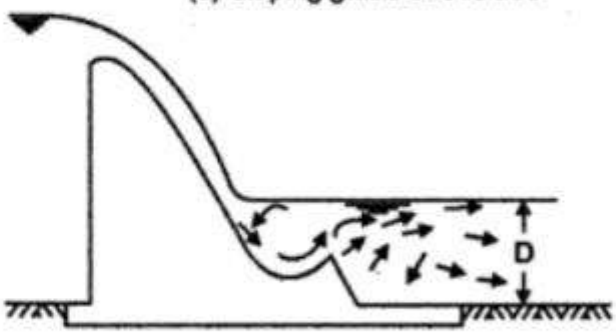


•1. Protection works for condition I. In this condition, both the curves coincide, as shown in Fig. (b). This is an ideal condition, since the post jump depth required for the formation of hydraulic jump is just available at the channel. The jump formation will thus be perfect at all discharges and hydraulic jump will be formed at the toe of the spillway. Protective measures. In such a case a simple horizontal apron of a length 5 (d2 – d1) will be sufficient,

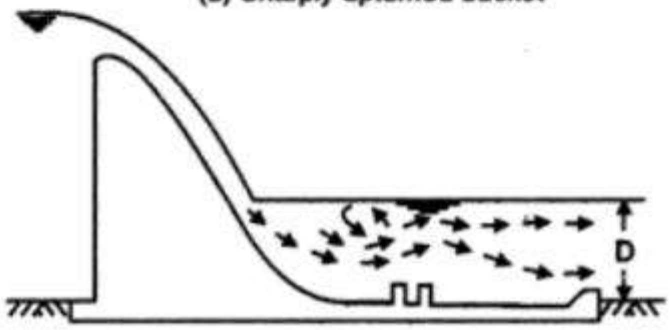
•2. Protection works for condition II. In this condition, shown in Fig. C the jump height curve lies lower than the tail water curve at all discharges. In other words, the available tail water depth is greater than required for the formation of hydraulic jump. The jump in such a case will be completely submerged and no visible standing wave would occur. Thus, very little energy will be dissipated unless arrangements are made to reduce the tail water depth at the point of formation of hydraulic lump.



(a) Sloping glacis above bed

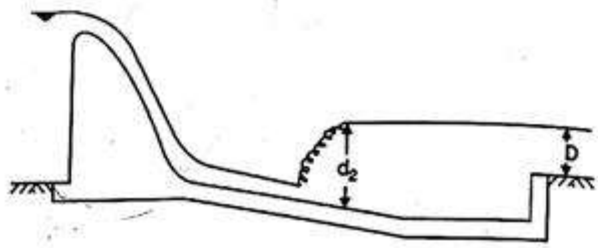


(b) Sharply upturned bucket

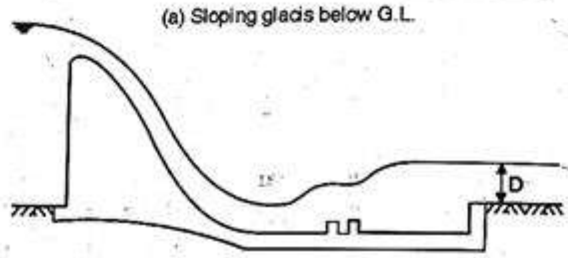


(c) End sill with baffles

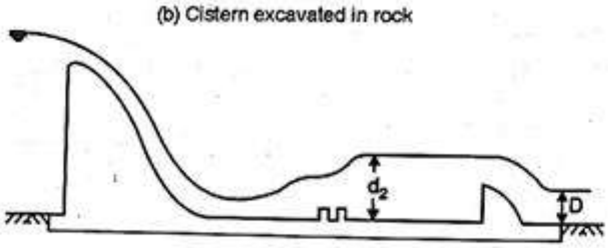
•3. Protection works for condition III. In this condition the jump height curve is higher than the tail water curve at all discharges, as shown in Fig.(d). Thus the available tail water depth is lesser than the depth required for the formation of hydraulic jump.



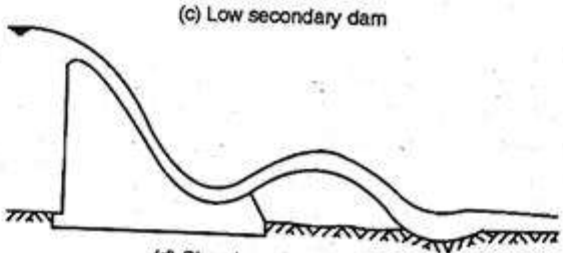
(a) Sloping glacis below G.L.



(b) Cistern excavated in rock

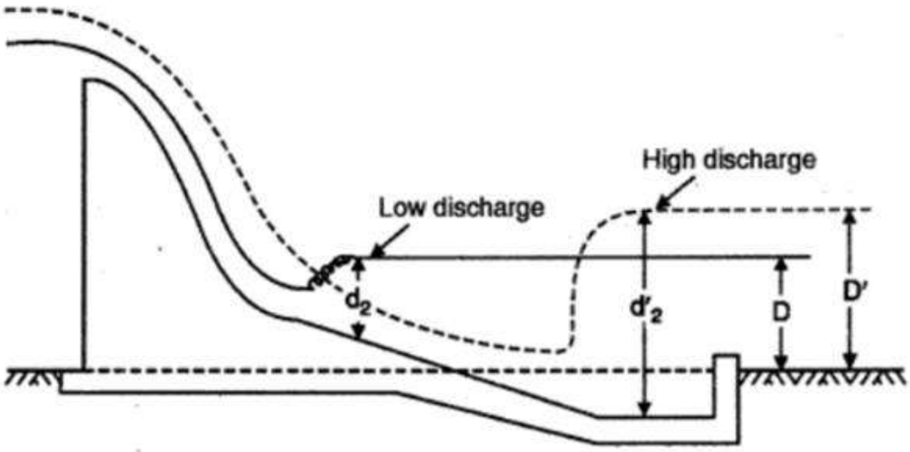


(c) Low secondary dam



(d) Sharply upturned bucket

•4. Protection works for condition IV: In this condition the jump height curve lies lower than the tail water curve at low dis-charges, and higher at high discharges, as shown in Fig. (e). Thus, at the low dis-charges, the jump will be drowned, while at high dis-charges, insufficient tail water depth will be there.



SLOPING APRON PARTLY BELOW AND PARTLY ABOVE THE BED.

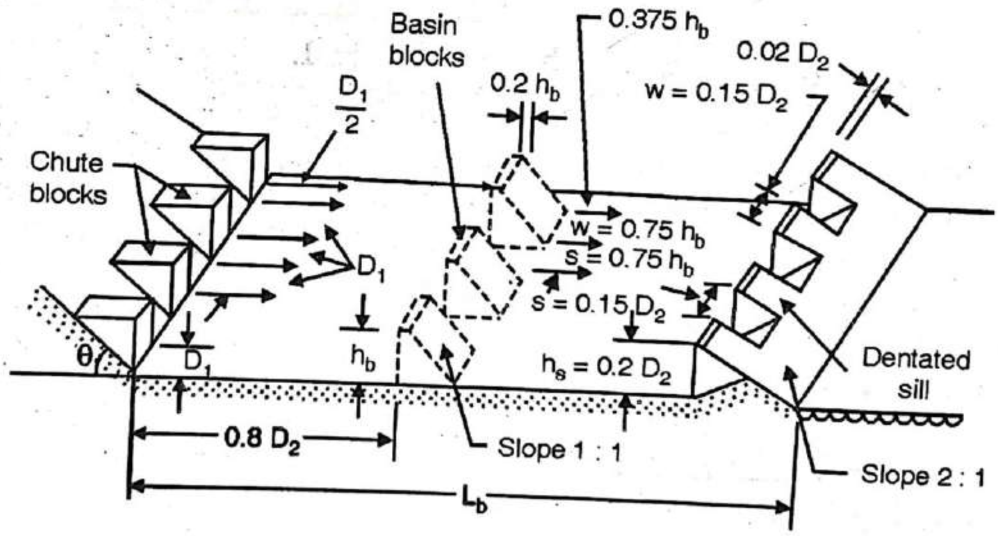
•Protection works for condition V. This is just the reverse of condition IV. as illustrated in Fig.(f), the jump height curve lies above the T.W.C. at low discharge, and lower at high discharges. Thus at low discharges in-sufficient tail water will be there while at higher discharges the jump will be drowned. The protection measure similar to that shown in for Condition IV is provided. At low discharges, the jump will be formed lower down, while at high discharges, the jump will be formed higher up.

•The Indian Standard IS : 4997 — 1968 lays down the criteria for the design of hydraulic jump type stilling basins of rectangular cross section with horizontal and sloping apron utilising various energy dissipators such as chute blocks, basin or floor blocks and end sill.

•A stilling basin is a structure in which all or part of the energy dissipating action is confined.

•In a stilling basin the kinetic energy first causes turbulence and is ultimately lost as heat energy.

•A hydraulic jump type stilling basin is basin in which dissipation of energy is accomplished basically by hydraulic jump which may be stabilised using chute blocks, basin blocks, end sill, etc.



(b) Appurtenances for basin

Unit-V

Learning Material

•Any structure constructed to regulate the discharge, full supply level or velocity inn a canal is known as a regulation work. Such a structure is necessary for the efficient working and safety of an irrigation channel. The various regulation works may be categorized s under

1. Canal fall.
2. Head regulator or head sluice.
3. Cross regulator.
4. Canal escape.
5. Canal outlet

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•HEAD REGULATORS AND CROSS-REGULATORS

•Head regulator and cross-regulator regulate the supplies of the off-taking channel and the parent channel respectively.

•The distributary head regulator is provided at the head of the distributary and controls the supply entering the distributary. It is a necessary link between the parent channel and the distributing channel.

• A distributary head is a regulator, a metre of supply and a silt selective structure. A cross-regulator is provided on the main canal at the die of the off-take to head up the water level and to enable the off-taking channel to draw the required supply.

Functions of distributary head regulator

1.They regulate or control the supplies to the off-taking channel

2. They serve as a meter for measuring the discharge entering into the off-taking canal.

3. They control the silt entry in the off-taking canal.

4. They help in shutting off the supplies when not needed in the off-taking canal. or when the off taking channel is required to be closed for repairs.

•Functions of cross-regulator

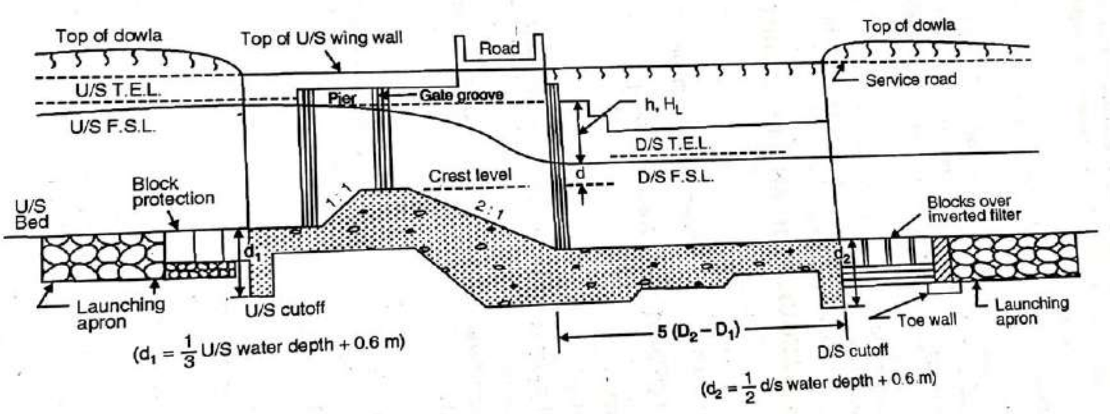
1. The effective regulation of the whole canal system can be done with help of cross-regulator.

2. During the periods of low discharges in the parent channel, the cross-regulator raises water level of the &a and feeds the off-take channel in rotation.

3. It helps in closing the supply to the dis of the parent channels, for the purposes of repairs etc

4. They help in absorbing fluctuation in various sections of the canal system, and in preventing the possibilities of breaches in the tail reaches. 5. Incidentally, bridges and other communication works can be combined with it.

Design of cross regulator and head regulator

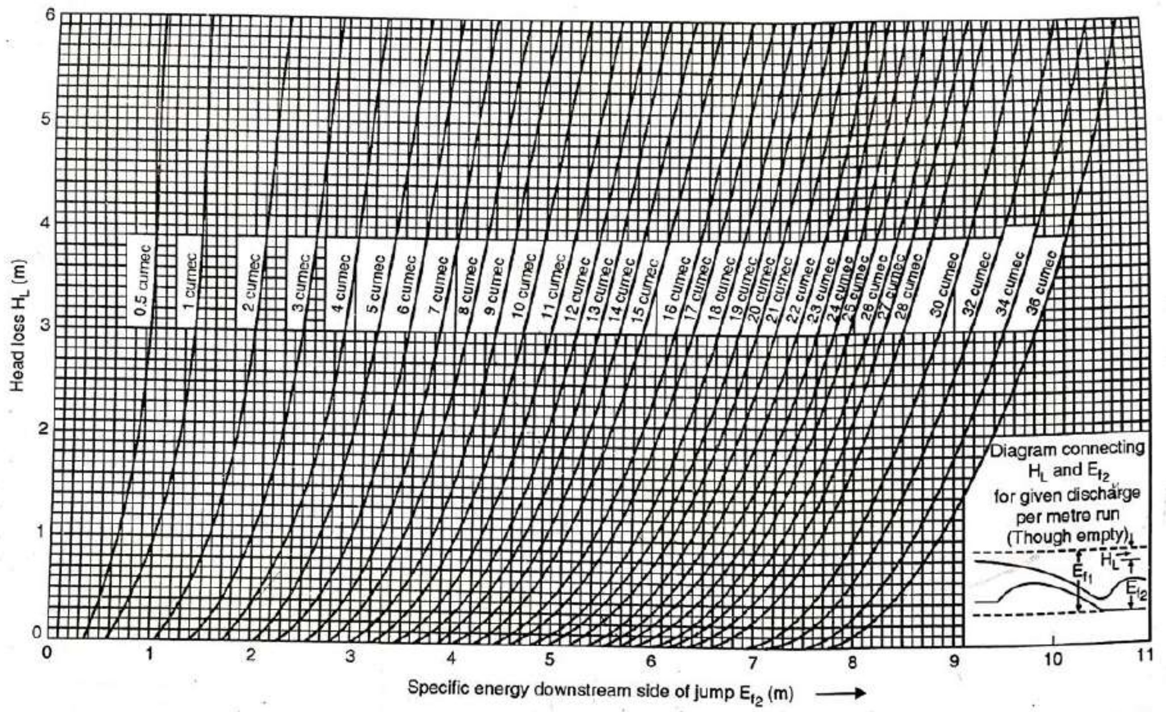


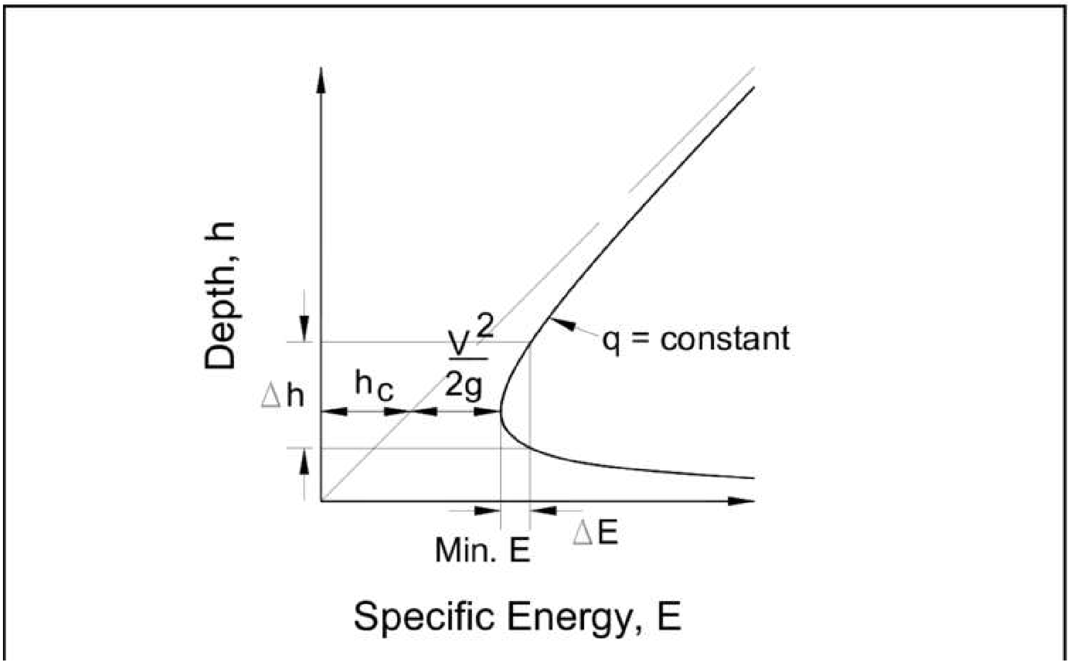
1. Design of crest

The discharge is determined by the drowned weir formula :

$$Q = \frac{2}{3} C_1 L \sqrt{2g}\ [(h + h_a)^{3/2} - h_a^{3/2}] + C_2 L d \sqrt{2g\,(h + h_a)}$$

•where Q = discharge, in cumecs
• L = length of water-way, in metres
•h= difference in water level u/s and d/s of the channel, in metres
•h_a= head due to velocity of approach
• d=depth of d/s water level in the channel, measured above the crest
• C_1= constant = 0.557
•C_2 = constant = 0.80.
•Generally the velocity of approach is small, and may be neglected while using Eq. Knowing the discharge Q, the length of water way L can be calculated.
• For the cross-regulator, the crest level is kept equal to the upstream bed level of the parent channel. For the distributary head regulator, the crest level is kept 0.3 to 1 m higher than the crest level of the cross-regulator. The crest is joined to the d/s floor with a sloping glacis of 2 : 1.
•2. Design of d/s floor: The level and length of the d/s floor is determined under two flow conditions:
(i) full supply discharge passing through both the head regulator and cross-regulator, and
(ii) the discharge in the parent channel being insufficient, the cross regulator gate is partially opened and the off-taking channel is running full. Or, the head regulator gate is fully open.
For both of these conditions, the discharge intensity q and the head loss H_L (= h) are known. Hence, the value of E_{f2} can be found from the Blench curves.





•D/s floor level = d/s T. E. L. - E_{f2} ≅ d/s F. S. L. - E_{f2}

• However, the d/s floor level, calculated from the above relation should never be provided higher than the d/s bed level.

•Now $E_{f1} = E_{f2} + H_L$

•Hence, the depth D_1 and D_2 corresponding to E_{f1} and E_{f2} respectively are found from specific energy curves.

• Then length of d/s floor = $5(D_2 - D_1)$

•However, the d/s floor should be at least 2/3 rd of the total impervious length of the floor.

•Design of impervious floor

Total length of the impervious floor should be found from the consideration of permissible exit gradient.

The depth of u/s cutoff d_1 = 1/3 u/s water depth + 0.6 m

The depth of d/s cutoff d_2 = 1/2 d/s water depth + 0.6 m

Maximum static head H_s= u/s F. S. L. - d/s floor level

The floor thickness is found from the considerations of uplift pressure. A minimum thickness of 0.3 to 0.5 m is provided from the practical considerations.

Design of upstream and down stream protection

•U/s scour depth d1 is taken equal (1/3 u/s water depth + 0.6 m). The d/s scour depth d_2 is taken equal to d/s water depth + 0.6 m). These scour depths are below the corresponding bed levels, and protection works are to be designed corresponding to these. (a) U/s protection. The u/s protection consists of a block protection having cubic contents = d_1 cubic metres/m. The cubic contents of u/s launching apron is kept equal to 2.25 d_1 cubic metre/metre width of regulator. (b) D/s protection. The cubic contents d of d/s inverted filter is kept equal to d_2 cubic metre/metre. The cubic contents of d/s launching apron is kept equal to 2.25 d2 cubic metre/metre width of regulator.

Canal Outlets:

When the canal water has reached near the fields to be irrigated, it has to be transferred to the watercourses. At the junction of the watercourse and the distributary, an outlet is provided. An outlet is a masonry structure through which water is admitted from the distributary into a watercourse. It also acts as a discharge measuring device. The discharge though an outlet is usually less than 0.085 m3/s. Thus, an outlet is like a head regulator for the watercourse.

The main objective of providing an outlet is to provide ample supply of water to the fields, whenever needed. If the total available supply is insufficient, the outlets must be such that equitable distribution can be ensured. The efficiency of an irrigation system depends on the proper functioning of canal outlets which should satisfy the following requirements

(i) The outlets must be strong and simple with no moving parts which would require periodic attention and maintenance.

(ii) The outlets should be tamper-proof and if there is any interference in the function-ing of the outlet, it should be easily detectable.

(iii) The cost of outlets should be less as a large number of these have to be installed in an irrigation network.

(iv) The outlet should be able to draw sediment in proportion to the amount of water withdrawn so that there is no silting or scouring problem in the distributary down-stream of the outlet.

(v) The outlets should be able to function efficiently even at low heads.

Types of Outlet:

Canal outlets are of the following three types: (i) Non-modular outlets, (ii) Semi-modular outlets, and (iii) Modular outlets.

Non-modular outlets are those whose discharge capacity depends on the difference of water levels in the distributary and the watercourse. The discharge through non-modular outlets fluctuates over a wide range with variations in the water levels of either the distributary or the watercourse. Such an outlet is controlled by a shutter at its upstream end. The loss of head in a non-modular outlet is less than that in a modular outlet. Hence, non-modular outlets are very suitable for low head conditions. However, in these outlets, the discharge may vary even when the water level in the distributary remains constant. Hence, it is very difficult to ensure equitable distribution of water at all outlets at times of keen demand of water.

The discharge through a semi-modular outlet (or semi-module or flexible outlet) depends only on the water level in the distributary and is unaffected by the water level in the watercourse provided that a minimum working head required for its working is available. A semi-module is more suitable for achieving equitable distribution of water at all outlets of a distributary. The only disadvantage of a semi-modular outlet is that it involves comparatively greater loss of head.

Modular outlets are those whose discharge is independent of the water levels in the distributary and watercourse, within reasonable working limits. These outlets may or may not have moving parts. In the latter case, these are called rigid modules. Modular outlets with moving parts are not simple to design and construct and are, hence, expensive.

A modular outlet supplies fixed discharge and, therefore, enables the farmer to plan his irrigation accordingly. However, in case of excess or deficient supplies in the distributary, the tail-end reach of the distributary may either get flooded or be deprived of water. This is due to the reason that the modular outlet would not adjust its discharge corresponding to the water level in the distributary. But, if an outlet is to be provided in a branch canal which is likely to run with large fluctuations in discharge, a modular outlet would be an ideal choice. The outlet would be set at a level low enough to permit it to draw its due share when the branch is running with low supplies. When the branch has to carry excess supplies to meet the demands of the distributaries, the discharge through the modular outlet would not be affected and the excess supplies would reach up to the desired distributaries. Similarly, if an outlet is desired to be located upstream of a regulator or a raised crest fall, a modular outlet would be a suitable choice.

Parameters for Studying the Behavior of Outlets:

Flexibility

The ratio of the rate of change of discharge of an outlet (dQ_0/Q_0) to the rate of change of discharge of the distributary channel (dQ/Q) (on account of change in water level) is termed the flexibility which is designated as F. Thus,

$$F = (dQ_0/Q_0)/(dQ/Q)$$

Here, Q and Q_0 are the flow rates in the distributary channel and the watercourse, respectively. Expressing discharge Q in the distributary channel in terms of depth of flow h in the channel as

$$Q = C_1 h^n$$

$$\frac{dQ}{Q} = n\frac{dh}{h}$$

Similarly, the discharge Q_0 through the outlet can be expressed in terms of the head H on the outlet as

$$Q_0 = C_2 H^m$$

For semi-modular outlets, the change in the head dH at an outlet would be equal to the change in the depth of flow dh in the distributary. Therefore,

$$\frac{dQ_0}{Q_0} = m\frac{dH}{H}$$

Here, m and n are suitable indices and C_1 and C_2 are constants. Thus,

$$F = \frac{m}{n} \times \frac{h}{H} \times \frac{dH}{dh}$$

If the value of F is unity, the rate of change of outlet discharge equals that of the distributary discharge. For a modular outlet, the flexibility is equal to zero. Depending upon the value of F, the outlets can be classified as: (*i*) proportional outlets ($F = 1$), (*ii*) hyper-proportional outlets ($F > 1$), and (*iii*) sub-proportional outlets ($F < 1$). When a certain change in the distributary discharge causes a proportionate change in the outlet discharge, the outlet (or semi-module) is said to be proportional. A proportional semi-module ensures proportionate distribution of water when the distributary discharge cannot be kept constant. For a proportional semi-modular outlet ($F = 1$),

$$\frac{H}{h} = \frac{m}{n}$$

The ratio (H/h) is a measure of the location of the outlet and is termed *setting*. Every semi-module can work as a proportional semi-module if its sill is fixed at a particular level with respect to the bed level of the distributary. A semi-module set to behave as a proportional outlet may not remain proportional at all distributary discharges. Due to silting in the head reach of a distributary, the water level in the distributary would rise and the outlet located in the head reach would draw more discharge although the distributary discharge has not changed. Semi-modules of low flexibility are least affected by channel discharge and channel regime and should, therefore, be used whenever the modular outlet is unsuitable for given site conditions.

The setting for a proportional outlet is equal to the ratio of the outlet and the channel indices. For hyper-proportional and sub-proportional outlets the setting must be, respectively, less and more than m/n. For a wide trapezoidal (or rectangular) channel, n can be approximately taken as 5/3 and for an orifice type outlet, m can be taken as 1/2. Thus, an orifice-type module will be proportional if the setting (H/h) is equal to (1/2)/(5/3), *i.e.*, 0.3. The module will be hyper-proportional if the setting is less than 0.3 and sub-proportional if the setting is greater than 0.3. Similarly, a free flow weir type outlet ($m = 3/2$) would be proportional when the setting equals 0.9 which means that the outlet is fixed at 0.9 h below the water surface in the distributary.

Sensitivity:

The ratio of the rate of change of discharge (dQ_0/Q_0) of an outlet to the rate of change in the water surface level of the distributary channel with respect to the depth of flow in the channel is called the '*sensitivity*' of the outlet. Thus

$$s = \frac{\left({dQ_0}/{Q_0}\right)}{\frac{dG}{h}}$$

Here, S is the sensitivity and G is the gauge reading of a gauge which is so set that $G = 0$ corresponds to the condition of no discharge through the outlet (*i.e.* , $Q_0 = 0$). Obviously, $dG = dh$. Thus, sensitivity can also be defined as the ratio of the rate of change of discharge of an outlet to the rate of change of depth of flow in the distributary channel. Therefore,

$$S = \left(\frac{dQ_0}{Q_0}\right)(dh/h)$$

$$S = \left(\frac{dQ_0}{Q_0}\right)(dQ/Q)$$
$$= \left(\frac{dQ_0}{Q_0}\right)/n\,(dh/h)$$
$$= \frac{1}{n}S$$
$$S = nF$$

Non-Modular Outlets:

The non-modular outlet is usually in the form of a submerged pipe outlet or a masonry sluice which is fixed in the canal bank at right angles to the direction of flow in the distributary. The diameter of the pipe varies from 10 to 30 cm. The pipe is laid on a light concrete foundation to avoid uneven settlement of the pipe and consequent leakage problems. The pipe inlet is generally kept about 25 cm below the water level in the distributary. When considerable fluctuation in the distributary water level is anticipated, the inlet is so fixed that it is below the minimum water level in the distributary. Figure 5.4 shows a pipe. If H is the difference in water levels of the distributary and the watercourse then the discharge Q through the outlet can be obtained from the equation,

$$H = \frac{V^2}{2g}\left[0.5 + \frac{fL}{d} + 1\right]$$
$$H = \frac{V^2}{2g}\left[1.5 + \frac{fL}{d}\right]$$
$$V = \frac{Q}{(\pi/4)d^2} = \sqrt{2gh}\left(\frac{d}{1.5d+fL}\right)^{1/2}$$

d =diameter of pipe outlet

L = length of pipe outlet

f = friction factor for pipe.

alternatively, the discharge Q can be expressed as

Q = AV

$$= \left(\frac{\pi}{4}d^2\right)\sqrt{2gh}\left(\frac{d}{1.5d+fL}\right)^{1/2}$$

or Q = CA $\sqrt{2gh}$

in which C = $\left(\frac{d}{1.5d+fL}\right)^{1/2}$

Semi-Modular Outlets (Semi-Modules or Flexible Outlets)

The simplest type of semi-modular outlet is a pipe outlet discharging freely into the atmosphere. The pipe outlet, described as the non-modular outlet, works as semi-module when it discharges freely into the watercourse. The exit end of the pipe is placed higher than the water level in the watercourse. In this case, the working head H is the difference between the water level in the distributary and the centre of the pipe outlet. The efficiency of the pipe outlet is high and its sediment conduction is also good. The discharge through the pipe outlet cannot be increased by the cultivator by digging the watercourse and thus lowering the water level of the watercourse. Usually, a pipe outlet is set so that it behaves as sub-proportional outlet, *i.e.*, its setting is kept less than 0.3. Other types of flexible outlets include Kennedy's gauge outlet, open flume outlet, and orifice semi-modules.

Unit-VI

Learning Material

Cross drainage work: 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. So a Cross drainage work is a structure carrying the discharge of a natural stream across a canal intercepting the stream. At the meeting point of canals and drainages, bed levels may not be same. Depending on their bed levels, different structures are constructed and accordingly they are designated by different names.

Types of Cross Drainage Works

Type I (Irrigation canal passes over the drainage)

(a) Aqueduct

(b) Siphon Aqueduct

Type II (Drainage passes over the irrigation canal)

(a) Super passage

(b) Siphon super passage

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

(a) Level crossing

(b) Inlet and outlet

Type-I - Irrigation canal passes over the Drainage:

Aqueduct

The hydraulic structure in which the irrigation canal is taken over the drainage (such as river, stream etc.) is known as aqueduct. This structure is suitable when bed level of canal is above the highest flood level of drainage. In this case, the drainage water passes clearly below the canal.

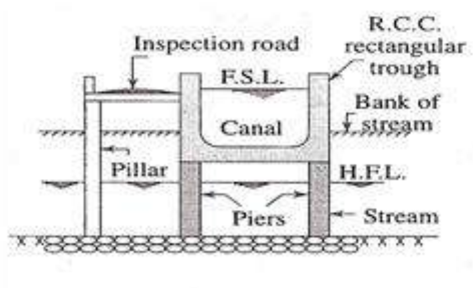


fig: Aqueduct

Siphon Aqueduct

In a hydraulic structure where the canal is taken over the drainage, but the drainage water cannot pass clearly below the canal. It flows under siphonic action. So, it is known as siphon aqueduct. This structure is suitable when the bed level of canal is below the highest flood level.

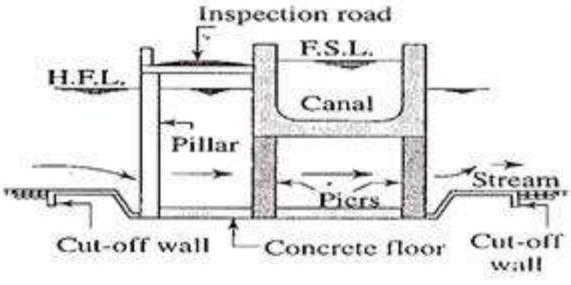


fig: Siphon aqueduct

Type-II Drainage Passes over the irrigation Canal

Super Passage

The hydraulic structure in which the drainage is taken over the irrigation canal is known as super passage. The structure is suitable when the bed level of drainage is above the full supply level of the canal. The water of the canal passes clearly below the drainage.

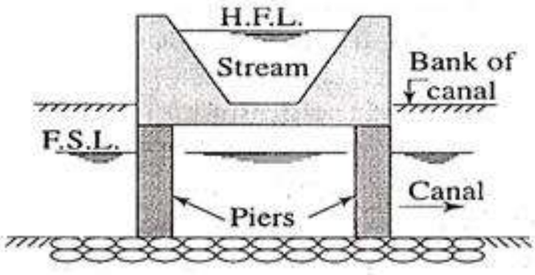


fig: Super passage

Siphon Super Passage

The hydraulic structure in which the drainage is taken over the irrigation canal, but the canal water passes below the drainage under siphonic action is known as siphon super passage. This structure is suitable when the bed level of drainage is below the full supply level of the canal.

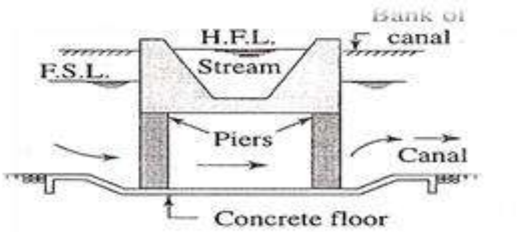


fig: Siphon super passage

Type III Drainage and Canal Intersect each other at the same level.

Level Crossing

When the bed level of canal and the stream are approximately the same and quality of water in canal and stream is not much different, the cross drainage work constructed is called level crossing where water of canal and stream is allowed to mix. With the help of regulators both in canal and stream, water is disposed through canal and stream in required quantity. Level crossing consists of following components (i) crest wall (ii) Stream regulator (iii) Canal regulator

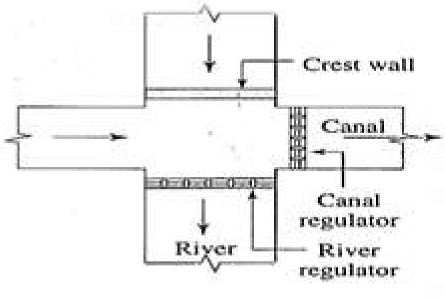


fig : Level crossing

Inlet and Outlet

When irrigation canal meets a small stream or drain at same level, drain is allowed to enter the canal as in inlet. At some distance from this inlet point, a part of water is allowed to drain as outlet which eventually meets the original stream. Stone pitching is required at the inlet and outlet. The bed and banks between inlet and outlet are also protected by stone pitching. This type of CDW is called Inlet and Outlet.

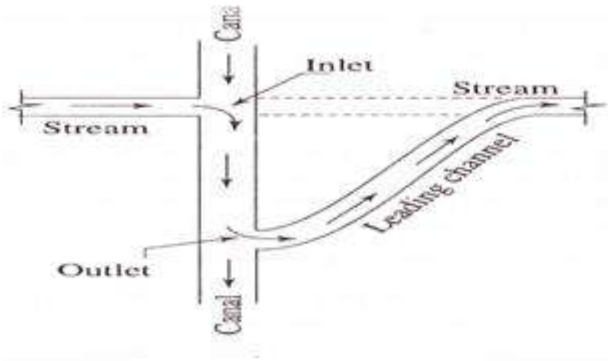


fig : Inlet and outlet

Selection of Type of Cross Drainage Works

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

(i) Relative Bed Level

According to the relative bed levels of the canal and the river or drainage, the type of cross drainage work are generally selected which has been discussed earlier. But some problems may come at the crossing point

The following points should be remembered while recommending the type of work,

(a) The crossing should be at right angle to each other

(b) Well defined cross-section of the river or drainage should be available.

(c) At the crossing point the drainage should be straight for a considerable length.

(d) The width of the drainage should be narrow as far as possible.

Considering the above points The C/s can be shifted to the downstream or upstream.
(ii)Availability of Suitable Foundation
For the construction of cross drainage works suitable foundation is required. By boring test, if suitable foundation is not available, then the type of cross drainage work should be selected to site Condition.
(iii)Economic Consideration
The cost of construction of cross drainage works should be justified with respect to the project cost and overall benefits of the project. So, the type of works should be selected considering the economical point of view.
(iv)Discharge of the drainage
Practically the discharge of the drainage is very uncertain in rainy season. So, the structure should be carefully selected so that it may not be destroyed due to unexpected heavy discharge of the river or drainage.
(v)Constructional of Problems
Different types of constructional problems may arise at the site such as sub soil water, construction materials, communication, availability of land etc. So the type of works should be selected according to the site condition.

Design Principles of Cross-Drainage Works

A) Hydraulic Design

- Determination of maximum flood discharge
- Fixation of waterway of the drain
- Contraction of canal water way
- Head loss through siphon barrels
- Determination of uplift pressure
- Determination of bank connections

B) Structural Design

- Design of piers and abutments
- Design of foundation

Determination of Maximum Flood Discharge

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

In general the methods used in the estimation of the flood flow can be group as:

- Physical Indications of past floods
- Empirical formulae and curves
- Overland flow hydrograph and unit hydrograph

Fixation of waterway of the drain

Lacey's regime perimeter equation gives good basis for calculating the drainage waterway. The equation is

$p = 4.75\sqrt{Q}$

P is the waterway to be provided for drain at the site in m

Q is flood discharge of the drain in m3/sec.

As the piers reduce the actual waterway available, the length between the abutments (P) may be increased by 20%.

Velocity of Flow through Barrel

The velocity of flow through the barrel may range from 1.8 m/sec to 3 m/sec, The reason for selecting this range is that the lower velocities may cause silting in the barrels. Whereas when the velocity is higher than 3 m/sec the bed load may cause abrasion of the barrel floor and subsequently it may be damaged.

Height of Opening:

Once the waterway, discharge and velocity are fixed the depth of flow may be obtained easily. There should be sufficient headway or clearance left between the HFL and the bottom of the canal bed. A clearance of 1 m or half the height of the culvert, whichever is less would be sufficient. Hence, Height of opening = Depth of flow + Clearance or headway.

Number of Spans

The number of spans to be provided may be fixed on the basis of the following two considerations:

i. Structural strength required, and

ii. Economical consideration.

Contraction of canal water way

In type I aqueducts, the canal cross-section is not changed at the crossings. This is not economical. In such cases, type III aqueducts are adopted, where the original width of the canal is reduced. This is termed as Fluming of the canal.

Generally fluming is done in such a way that velocity of flow should not be more than 3 m/sec.

This precaution is taken to avoid the possibility of formation of a hydraulic jump.

The approach transition wings should not be stepper than 30°.(2:1).

The departure wings should not be stepper than 22.5°. (3 : 1).

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

- Mitra’s method of design of transition (when water depth remains constant)
- Chaturvedi’s method of design of transitions(when the depth remains constant)
- Hind’s method of design of transitions (when water depth may or may not vary).

Mitras method of design Transition when water depth remains constant:

Shri A.C. Mitra, Chief Engineer, U.P, Irrigation Department 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 is given by.

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

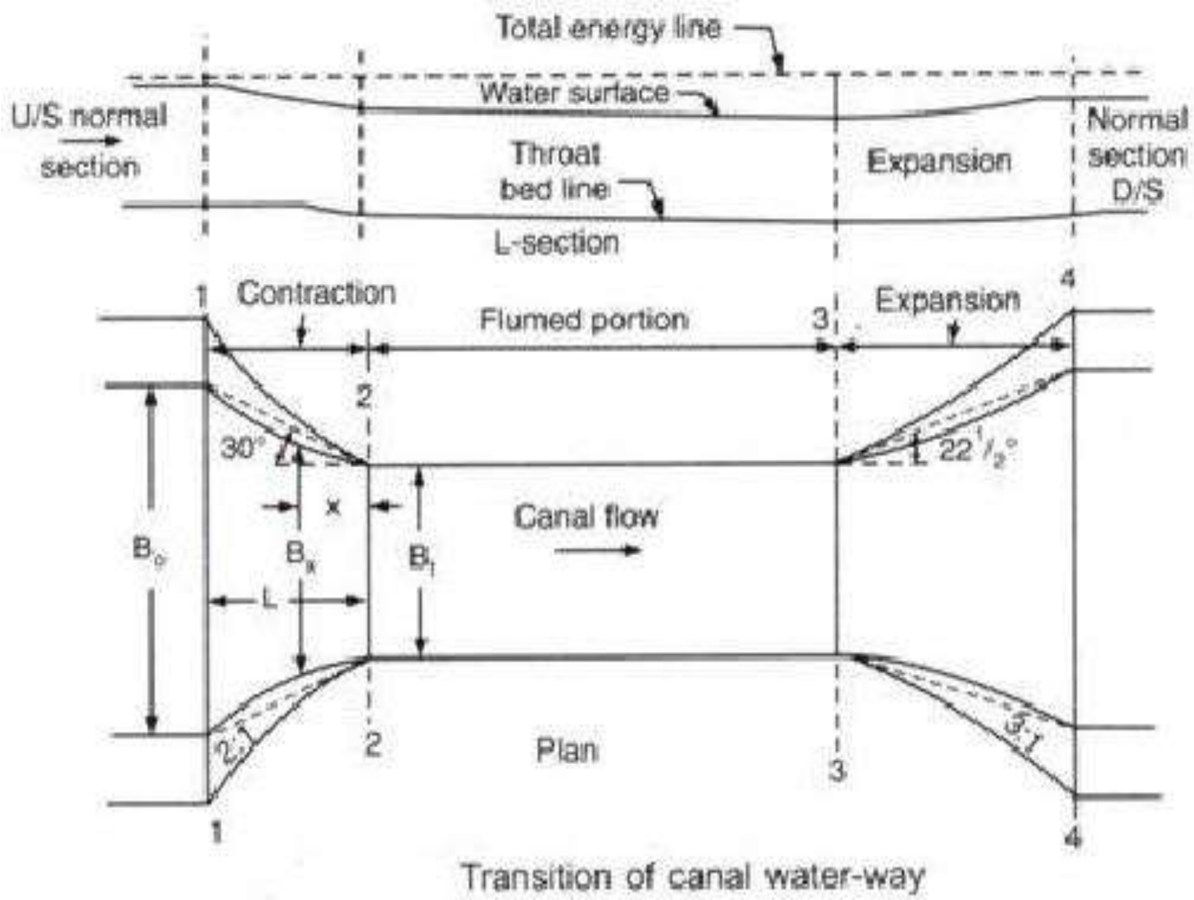
B_n = bed width of the normal channel section

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

Hind's method:



It may be seen that sections 1-1, 2-2, 3-3 and 4-4 indicate start of contraction, end of contraction, start of expansion and end of expansion respectively.

Up to section 1 and beyond section 4 the canal flows under its normal conditions and therefore the canal parameters at these two points are equal and already known. So also conditions of flow and canal parameters are same between sections 2 to 3 which represents throat or trough portion.

Let D, V be the depth and velocity at different sections respectively,

Since canal levels and dimensions are already known at section 4-4:

Step 1: TEL at section 4-4 = Water surface elevation + $V_4^2/2g$

Where, water surface elevation at sec. 4-4 = Bed level + D_4

Step 2: TEL at sec. 3-3 = (TEL at sec 4-4) + (energy loss between sec. 3 and 4)

Energy loss between sections 3-3 and 4-4 takes place due to expansion of streamlines and also due to friction. Neglecting loss due to friction which is small and taking loss due to expansion to be $0.3({V_3}^2 - {V_4}^2)/2g$

TEL at sec. 3-3 = (T.E.L at sec.4-4) + $0.3({V_3}^2 - {V_4}^2)/2g$

Water surface level at sec. 3-3 =(TEL at sec.3-3) – (${V_3}^2/2g$) and

Bed level at sec. 3-3 = Water surface level at sec.3-3) – (D_3)

Step 3: Similarly TEL at sec- 2-2 = (TEL at sec. 3-3) + Head loss between sections 2 and 3

Between sections 2-2 and 3-3 canal flows through section of same dimension and therefore flow is uniform. Loss of head in the trough is only due to friction losses. The loss of head can be calculated using Manning's equation.

$Q = A.\frac{1}{N} R^{2/3}. S^{1/2}$

Water surface level at sec. 2-2= (TEL at sec.2-2) – (${V_2}^2/2g$) and

Bed level at sec. 2-2 = (Water surface level at sec.2-2) – (D_2)

Since depth and velocity are same in the trough from sections 2 to 3 it may be noted that the total energy line water surface line and bed line are parallel to each other.

Step 4: On similar lines

TEL at sec. 1-1 = (T.E.L at sec.2-2) + Head loss between sections 1 and 2

Between sections 2-2 and 1-1 energy loss takes place due to contraction of stream lines and friction may be taken to be $0.2({V_2}^2 - {V_1}^2)/2g$

TEL at Sec-1-1 =TEL at sec. 2-2 +$0.2({V_2}^2 - {V_1}^2)/2g$

Water surface level at sec. 1-1= (TEL at sec.1-1) – (${V_1}^2/2g$) and

Bed level at sec. 1-1 = (Water surface level at sec.1-1) – (D_1)

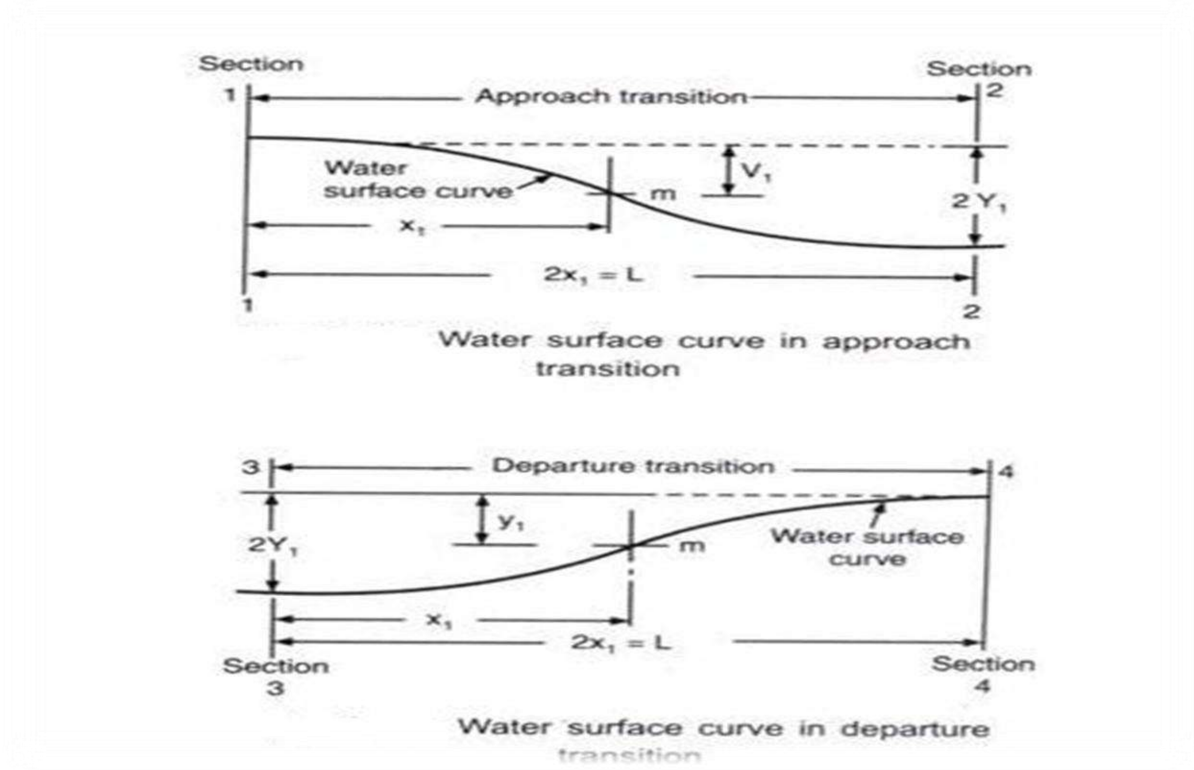
Step-5

Now the TE line, water surface line and the bed line can be drawn as follows:

(a)The total energy line can be drawn by joining these points at four sections by a straight line.

(b) The bed line may also be drawn as straight lines between adjacent sections if the fall or rise of bed level is small. The corners should be rounded off. In case the drop in the bed line is appreciable the bed lines should be joined with a smooth tangential reverse curve.

(c) It is now clear that between any two consecutive sections the drop in water surface level may result due to (i) drop in TE line between the two sections; (ii) increased velocity head in contraction; and (iii) decreased velocity head in expansion.



Water surface curve in approach transition

Water surface curve in departure transition

Head loss through Siphon Barrel:

The head which causes flow (it also represents head loss in barrel) through the inverted siphon barrel may be obtained from Unwin's formula

$$h=(1+f_1+f_2\frac{L}{R})V^2/2g- V_a^2/2g$$

Where h is the head causing flow, it is also the loss of head in the barrel in m.

L is the length of barrel in m.

R is hydraulic mean radius of barrel in m.

V is velocity of flow through barrel in m/sec.

V_a is velocity of approach in m/sec, it is generally neglected.

f1 is a coefficient for loss of head at entry and generally taken as 0.505 for unshaped mouth 0.08 for bell mouth

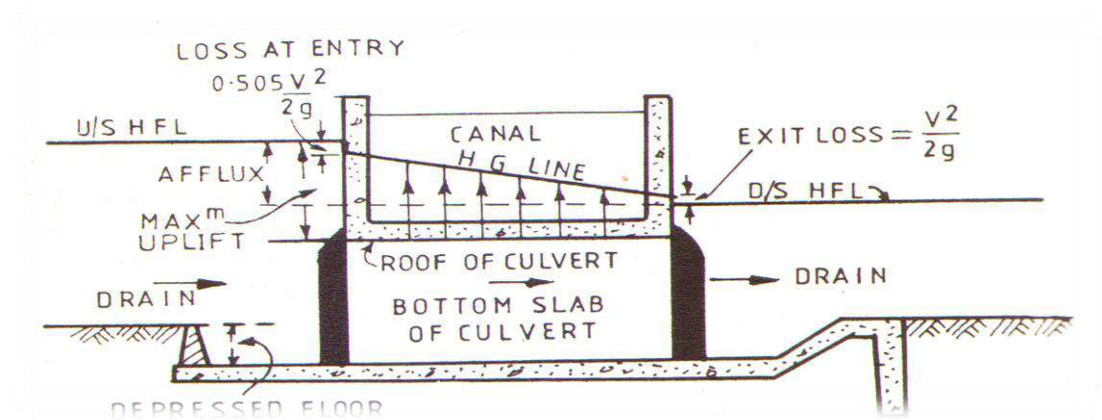
f2 is a coefficient which accounts for friction in the barrel.

$f_2 = a\ (1 + 0.305\frac{b}{R})$

Uplift Pressure on the Barrel Roof

The amount of the uplift pressure exerted by the drain water on the roof of the culvert can be evaluated by drawing the hydraulic Gradient line (H.G).

The uplift pressure at any point under the roof of the culvert will be equal to the vertical ordinate between hydraulic gradient line and the underside of the canal trough at that point from the uplift diagram it is very evident that the maximum uplift occurs at the upstream end point near the entry. The slab thickness should be designed to withstand this maximum uplift.



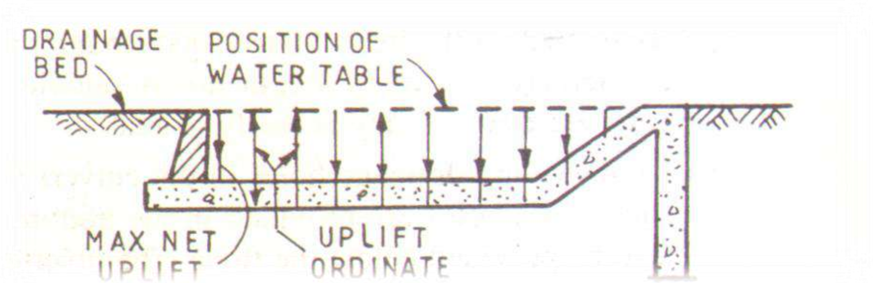
The floor of the aqueduct or siphon is subjected to uplift due to two cases:

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

The maximum uplift under the worst condition would occur when there is no water flowing in the drain and the water table has risen up to the drainage bed. The maximum net uplift in such case would be equal to the difference in level between the drainage bed and the bottom of the floor.

Uplift Pressure 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. The computation of this uplift due to this seepage occurs when the canal is running full and there is no water in the drain. The computation of this uplift, exerted by the water seeping from the canal on the bottom of the floor, is very complex and difficult, due to the fact that the flow takes place in three dimensional flow net. The flow cannot be approximated to a two dimensional flow, as there is no typical place across which the flow is practically two dimensional. Hence, for smaller works, Beligh’s Creep theory may be used for assessing the seepage pressure, But for larger works, the uplift pressure must be checked by model studies.



Design of Bank Connections

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

Canal wings or Land wings

Drainage Wings or Water Wings

Canal Wings: These wings provide a strong connection between masonary 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. They should be extended upto the end of splay. These wings may be designed as retaining walls for maximum differential earth pressure likely to come on them with no water in the canal. The foundations of these wings should not be left on filled earth. They should be taken deep enough to give safe creep length.

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 from the water seeping from the canal into drainage bed. The foundations of these wings wall should be capable of withstanding the maximum differential pressure likely to come on them.

The layouts of these sets of wings depend on the extent of contraction of canal and drainage waterways, and the general arrangement of the work.