DIVERSION HEAD WORKS

13.1. General

The construction work done at the river or canal for diversion of water to the off-taking canal is known as *head-work*. Depending upon their purposes, the head-works may be divided into following two classes :

(*i*) Storage head-work

(*ii*) Diversion head-work.

The storage head works are constructed across the river to store the water during the period of excess supplies or rains and use it in dry weather flow for meeting the demand.

The main purpose of the diversion head-works is to divert the required quantity of water into the off-taking canal for irrigation purposes. A diversion head work also serves the following purposes :

(*i*) To raise the water level in the river for increasing its command area.

(*ii*) To regulate the intake of water into the canal.

(*iii*) To control the silt entry into the canal.

(*iv*) To store water for meeting emergency requirements.

(*iv*) To prevent fluctuations in the level of supply of the river.

13.2. Objects of Diversion Head Works

The following are the objects of constructing diversion head-works :

(1) To provide an obstruction across a river, so that the water level is raised and water is diverted to the channel at required level. The increased water level helps the flow of water by gravity and results in increasing the commanded area and reducing the water fluctuations in the river.

(2) To regulate the gravity of water entering the channel. This is achieved by the construction of a canal head regulator.

(3) To regulate the quantity of silt entering the channel. Due to the obstruction, the velocity of river water gets decreased and the silt starts depositing at the bed. Clear water with permissible percentage of silt can be allowed to flow through the regulator into the channel.

(4) To prevent the direct transfer of flood water into the channel.

(5) To store the excess of water, when available in the river during rains and supply it, when the demand is more than the usual supply or in dry weather.

13.3. Selection of Site for Diversion Head Works

The selection of site depends upon the condition of the river or the stage of the river flow.

There are four stages of the river flow, which are :

1. Rocky or mountainous stage.
2. Boulder stage or the sub-mountainous stage.
3. Trough stage.
4. Delta stage or Tidal stage.

13.4. Rocky or Mountanious Stage

This is the stage where the river is flowing in hilly or mountanious area. The velocity of the river in this stage is at its maximum. The bed of the river consists of rocks and big boulders. The bed slopes are very steep.

13.5. Boulder Stage or the Sub-mountanious Stage

In this stage, the slope of the river bed is decreased causing the velocity of the river to become less than that during the rocky stage. The bed and sides of the river consist of gravel or boulders and the cross-section of the river is well formed.

13.6. Trough Stage

The river in this stage flows on alluvial plains. The cross-section is made up of alluvium or silt. It is flowing gently in this stage.

13.7. Delta Stage or Tidal Stage

This is the last stage of the river. The velocity of the river when it is approaching the sea or the ocean is at its minimum. The river is like a tired old man. It drops down its sediments and branches out into many channels forming deltas.

For locating the head-works, the rocky stage is satisfactory, from the point of view of constructing the structures. But the cost of construction of channels in rocky areas will be high. Hence, this stage is not preferred.

The delta stage is also not suitable because the commanded area will be very small.

Hence, the stage between the sub-mountanious region and the trough region is selected for the site for the construction of head-works.

13.8. Sub-mountanious Stage

Following are the *advantages* of providing diversion head-works in sub-mountanious stage :

(*i*) Excellent commercial crops like tobacco, transplanted rice, sugarcane etc., can be grown in the areas where there was no irrigation done previously.

(*ii*) Initial cost of head-works will be less than that in the trough region, due to easy availability of stones and firmness of the banks.

(*iii*) There is no necessity of river training works as in trough region.

(*iv*) In this region, a temporary boulder wall can be constructed to divert the river water to the channel. This wall may be constructed every time after it is washed away by the floods, due to its minimum cost.

(*v*) In the boulder stage, falls can be constructed on the canals for the generation of hydroelectric power.

Following are *disadvantages* of providing diversion-works in sub-mountanious stage :

(*i*) Due to heavy sub-soil flow in the bed of the river, there will be sufficient loss of water during periods of short supply.

(*ii*) The canal may have to run in a sub hilly tract made of sand and boulders, for a long length, where it will suffer heavy seepage losses. Considerable length of channel of boulder reach cannot be utilized, as the soil on both sides cannot be used for agricultural purposes.

(*iii*) Large number of cross-drainage works are to be constructed. This will increase the construction cost of the project.

(*iv*) The irrigable area near the boulder region may not be much, as the hilly, damp areas do not require irrigation.

13.9. Trough Stage

Following are the *advantages* of this stage :

1. Sub-soil flow or seepage is less than that in the boulder region.
2. The number of cross-drainage works across the canals are less.
3. Commanded area secured in this region will be more than in the boulder region. No portion of canal in this region will be useless.
4. The requirement of water will not be as emergent as in the hilly region.
5. Water in this region contains more silt and hence will be good for the growth of crops due to its fertile value.

Following are the *disadvantages* of this stage :

1. River in this stage requires to be trained. Hence, river training works are to be constructed.
2. Due to poor foundation conditions in this stage, the cost of head-works will increase. This will involve heavy recurring expenditure for maintenance also.
3. Heavy silt load in the river will create problems. Regulation of the quantity of silt entering the channel will be necessary.

In view of the above considerations, the choice between the boulder region and the trough region must be made very carefully.

After selecting the reach, the exact location of the site for the construction of the head-works should be fixed.

13.10. Head-Work Site Location

The following points must be kept in view while locating the exact site for the head-works :

1. There should be possibility of constructing a narrow straight, well defined off-take channel. The banks of the channel should not be submerged by the river even during highest flood condition.
2. Materials required for the construction should be available in sufficient quantities, within a reasonable distance.
3. The site should be such that the canal commands maximum irrigable areas, with moderate earth-works.
4. It should be possible to connect the site easily to an existing road or railway. This will facilitate the easy transportation of labour and materials.
5. The locality should not pose any health hazards like Malaria, as otherwise, it will seriously affect the labourers and staff residing in the area.

13.11. Classification of Diversion Head-Works

The diversion head-works may be of temporary or permanent nature depending upon the requirements.

Temporary diversion works are called spurs or bunds. These may have temporary boulder walls constructed across a river to raise the level of water and lead it on to a channel. It may be required to construct them every year, as they may be damaged by the floods.

Permanent diversion works are either weirs or barrages.

(*a*) **Weirs.** These are solid walls constructed across a river. A weir helps in raising the water level in the river to the required point so that it can flow under gravitational force into the canal. Weirs are also constructed to store water to supply it in dry period or lean period.

Such weirs are known as 'Storage Weirs'. A regular dam stores water to a greater depth and for a longer duration than a storage weir.

(*b*) **Barrage.** The barrage is also a structure constructed to store water. In this case, no wall is constructed across the river, but there is an arrangement of gates which can be used to store water upto the required level. These gates can be lifted up to allow flood waters to flow downstream. Again, the gates can be lowered to store water upto the required level.

Barrages are costlier than weirs. A diversion head-work is an aggregate of many smaller constructions.

13.12. Component Parts of a Diversion Head-Works

They are as shown in Fig. 13.1.

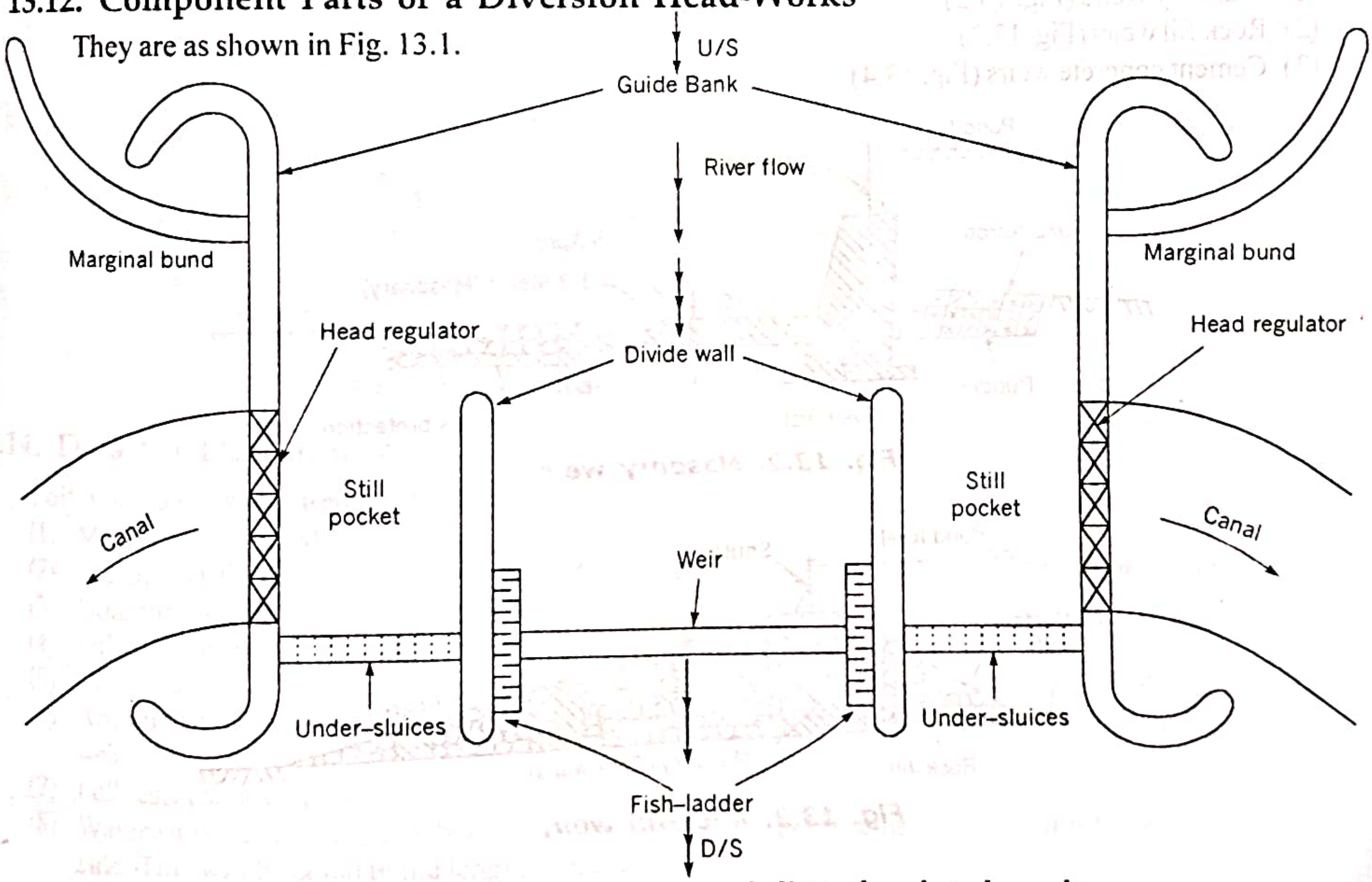


Fig. 13.1. Component parts of diversion head-work.

The different component of diversion head-works are as under :

(1) A Weir or a Barrage.
(2) A Divide Wall.
(3) Approach Channel or still pocket.
(4) Scouring sluices.
(5) Fish ladder.
(6) Silt controlling devices.
(7) Canal head regulator.
(8) Marginal bunds.
(9) Guide banks.

(1) Weir. This is constructed to raise the water level and divert it to the channel. Normally, it is aligned perpendicular to the direction of flow of water of the main river.

(A) There are various types of weirs. They may be classified into the following types, *based on their functions :*

(a) Storage Weirs. The main function of these are to store water.

(b) Diversion Weirs (Intake Weirs). These raise the water level and divert the water to the channel.

(c) Pick-up Weirs. These are constructed across rivers where the canals take off. The water in the dam or storage reservoir is discharged through supply sluices. Across the discharged water is constructed a series of pick-up weirs at different positions, so as to pick up the released water.

(d) Waste Weirs. These are meant for disposing of surplus water. They act as spillways for reservoirs. They are constructed at the outer margin of the reservoir basin. The crest of the waste-weir will be at maximum water level, sot that when the water level increases due to floods, it automatically starts spilling over the weir, thus protecting the main dam or reservoir from damage due to the flood water.

Weirs may also be called (*i*) *gravity weirs,* which remain stable only due to their own weight, and (*ii*) *non-gravity weirs.* Non-gravity weirs are stable against all forces, when the whole weir structure is considered, but its individual components may be or may not be stable against the loads acting over it.

(B) Classification of Weirs based on Materials. Weirs may also be classified as based on the materials used for their construction, as under :

(1) Masonry weirs (Fig. 13.2)
(2) Rock fill weirs (Fig. 13.3)
(3) Cement concrete weirs (Fig. 13.4).

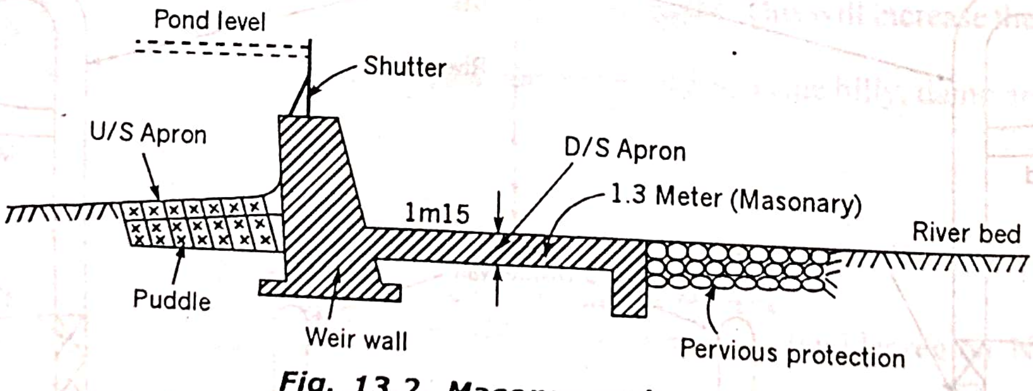


Fig. 13.2. Masonry weir.

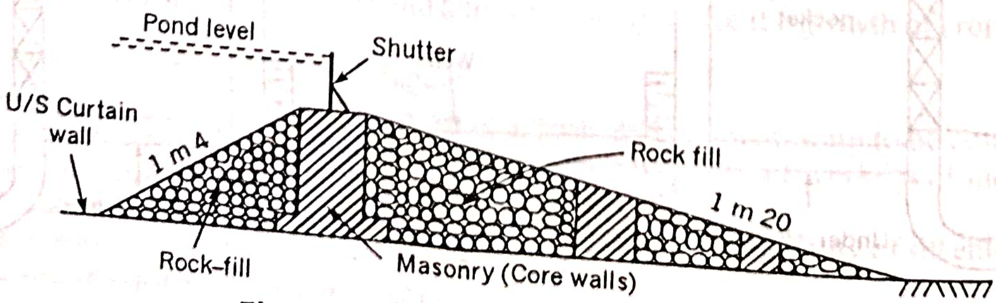


Fig. 13.3. Rock-fill weir.

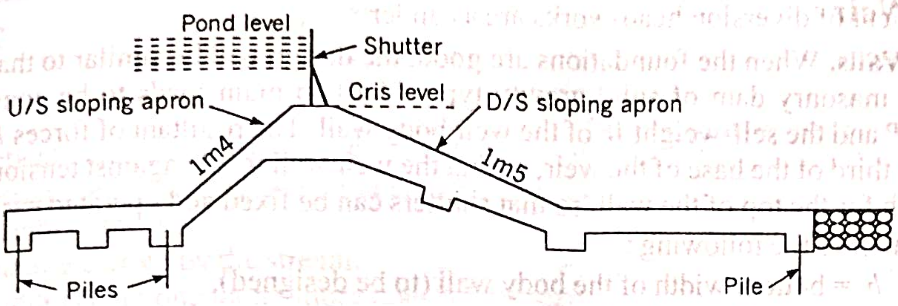


Fig. 13.4. ***Cement concrete weir.***

In all the above type weirs, crest shutters are provided above the top or crest of the weir. These shutters are hinged. When they are vertical, they can store water upto their top level. When excess of water is to be discharged, the crest shutters may be dropped, when they assume a horizontal position, the water overflows on the crest till the level is brought down to the level of the crest. The shutters can be raised to their vertical position, if more water is required to be stored.

13.13. Weir Components

All types of weirs normally have the following components :

(1) Body wall.
(2) Upstream rough stone apron or boulder pitching (pervious).
(3) Upstream curtain wall.
(4) Upstream impervious apron.
(5) Crest shutters.
(6) Downstream impervious apron.
(7) Donwstream curtain wall.
(8) Downstream apron for channel bed.

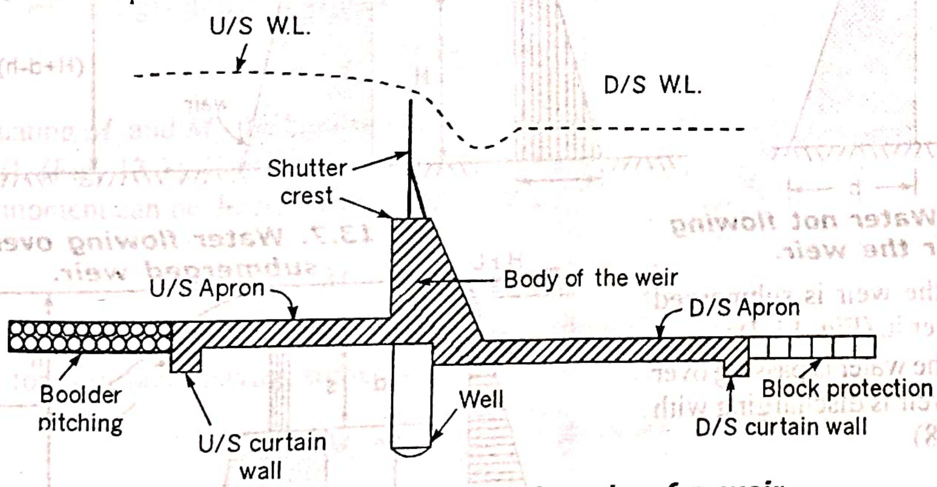


Fig. 13.5. ***Component parts of a weir.***

13.14. Data for Design of a Weir

Following data is required for the design of a weir :

(1) Maximum expected flood discharge (Q).
(2) A graph of flood levels plotted against flood discharge, known as storage discharge curve.
(3) Quantity of silt carried by the river during maximum flood.
(4) Full supply level (F.S.L.) and full supply discharge in the off-take channel.
(5) Cross-section of the river at the weir site and upstream and downstream portions of the weir site.
(6) Amount of efflux or the heading up of water due to the obstruction caused by the body wall of the weir.
(7) Full reservoir level (F.R.L.).
(8) Waterway or clear space available for the flow of the water from the upstream side to the downstream side. This will be equal to the length of the weir.

12

Diversion Headworks

12.1. INTRODUCTION

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

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 raises the water level in the river so that the commanded area can be increased.
2. It regulates the intake of water into the canal.
3. It controls the silt entry into the canal.
4. It reduces fluctuations in the level of supply in the river.
5. It stores water for tiding over small periods of short supplies.

A diversion headwork can further be sub-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. However, for important works, weirs or barrages are constructed since they are of permanent nature if designed properly.

Weir. The weir is a solid obstruction put across the river to raise its water level and divert the water into the canal (Fig. 12.1). If a weir also stores water for tiding over small periods of short supplies, it is called a *storage weir*. The main difference between a storage weir and a dam is only in height and the duration for which the supply is stored. A dam stores the supply for a comparatively longer duration.

Barrage. The function of a barrage is similar to that of weir, but the heading up of water is effected by the gates alone (Fig. 12.2). No solid obstruction is put across the river. The crest level in the barrage is kept at a low level. During the floods,

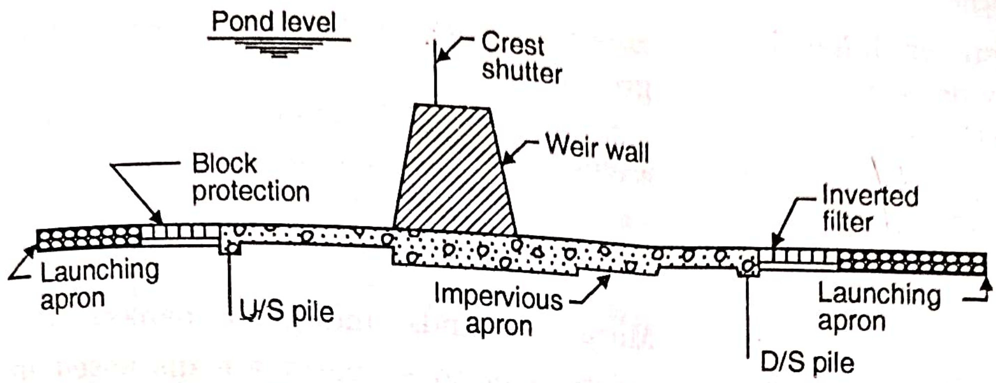


FIG. 12.1. VERTICAL DROP WEIR.

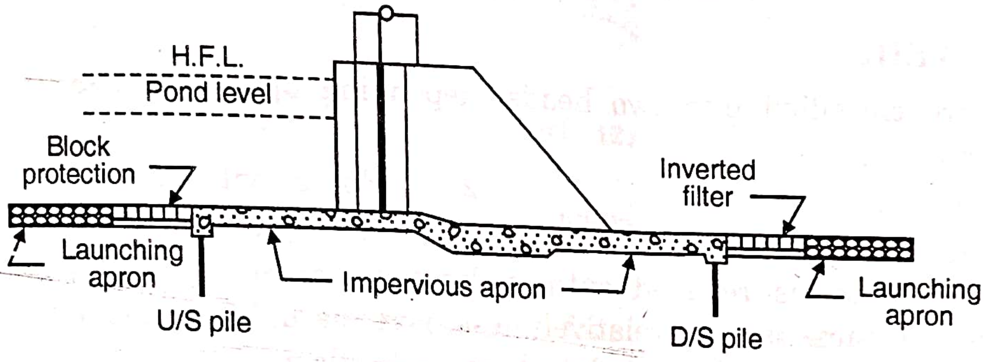


FIG. 12.2. BARRAGE.

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 costlier than the weirs.

12.2. COMPONENT PARTS OF A DIVERSION HEADWORK

A diversion headwork consists of the following component parts (Fig. 12.3) :

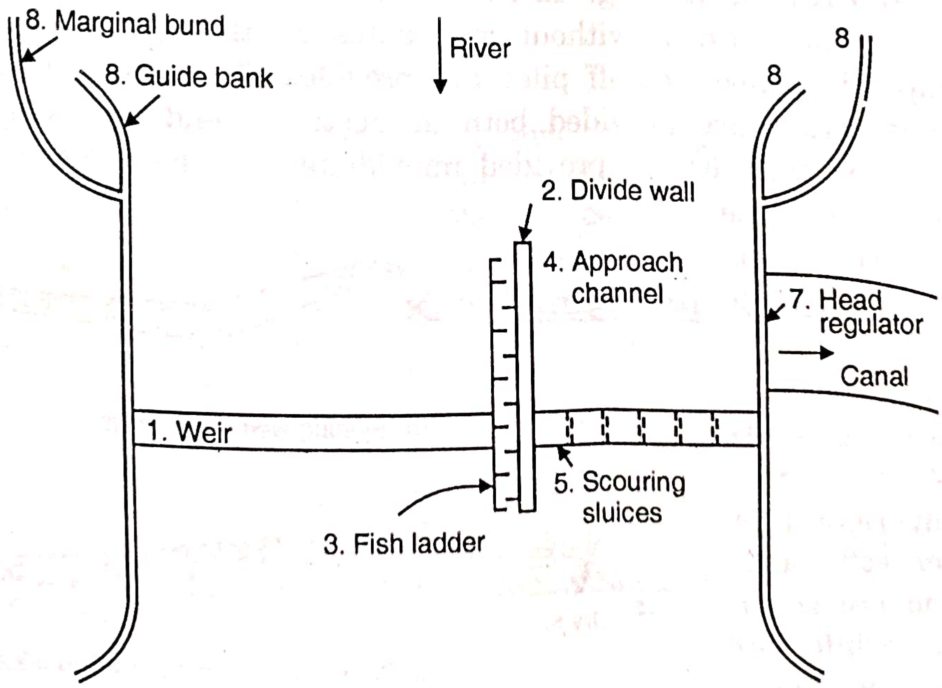


FIG. 12.3. COMPONENT PARTS OF A HEADWORK.

1. Weir or barrage
2. Divide wall or divide groyne
3. Fish ladder
4. Pocket or approach channel
5. Scouring sluices
6. Silt prevention devices
7. Canal head regulator
8. River training works (Marginal bunds and guide banks)

The description and design details of these parts are discussed in the following articles.

12.3. THE WEIR

Weirs are classified into two heads, depending upon the criterion of the design of their floors.

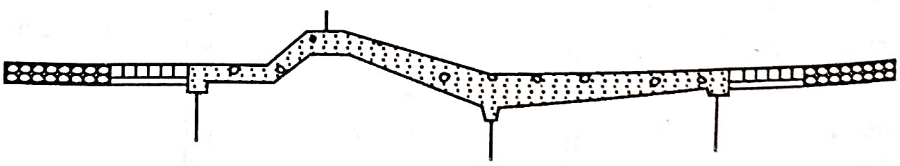
1. Gravity weirs
2. Non-gravity weirs

A gravity weir is the one in which the uplift pressure due to the seepage of water below the floor is resisted entirely by the weight of floor. In the non-gravity type, the floor thickness is kept relatively less, and the uplift pressure is largely resisted by the bending action of the reinforced concrete floor.

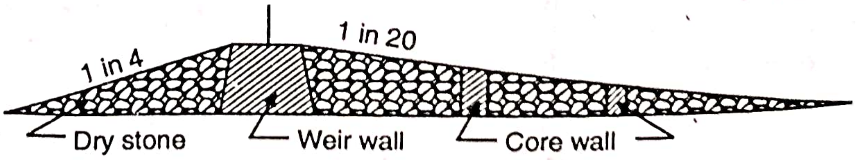
Depending upon the material and certain design features, gravity weir (or simply weirs) can further be sub-divided into the following types :

1. Vertical drop weir.
2. Sloping weir :
   (*a*) Masonry or concrete slope weir
   (*b*) Dry stone slope weir.
3. Parabolic weir.

1. **Vertical drop weir** (Fig. 12.1). A vertical drop weir consists of a vertical drop wall or crest wall, with or without crest gates. At the upstream and downstream ends of the impervious floor, cutoff piles are provided. To safeguard against scouring action, launching aprons are provided both at upstream and downstream end of the floor. A graded inverted filter is provided immediately at the downstream end of the impervious floor to relieve the uplift pressure. Vertical drop weirs are suitable for any type of foundation.



(a) Sloping weir of concrete



(b) Sloping weir of masonry and dry stone

FIG. 12.4. SLOPING WEIRS.

2. **Masonry or Concrete Sloping Weir** [Fig.12.4(*a*)]. Weirs of this type are of recent 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 me-

tres. When water passes over such a weir, hydraulic jump is formed on the sloping glacis.

3. **Dry stone slope weir** [Fig. 12.4(*b*)]. A dry stone weir or a rockfill weir consists of a body wall (or weir wall) and upstream and downstream rockfills laid in the form of glacis, with few intervening core walls. Okhla weir on Yamuna river, near Delhi, is the example of such weir.

4. **Parabolic Weir** (Fig. 12.5). A parabolic weir is similar to the spillway section of a dam. The body wall for such a weir is designed as a low dam. A cistern is provided at the downstream side to dissipate the energy. The upstream and down-stream protection works are similar to that of a vertical drop or sloping glacis weir.

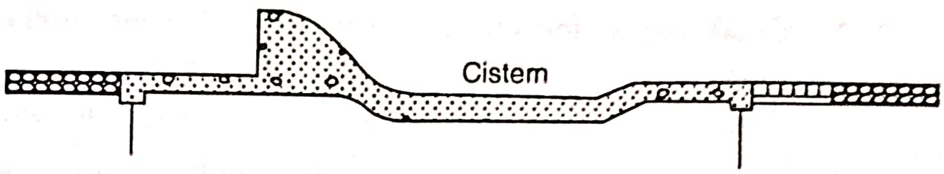


FIG. 12.5. PARABOLIC WEIR.

12.4. LOCATION OF HEADWORKS

A river, in its course, is divided into four distinct regions or stages :

(*i*) *The torrential, rocky or mountaineous stage :* The river in this stage has very steep bed slope and high velocity.

(*ii*) *The sub-mountaineous or boulder stage :* The sides and bed of the river are composed of boulder and gravel. There is strong subsoil flow in this region.

(*iii*) *Trough stage or alluvial plain :* The cross-section of the river is made of alluvial sand and silt. The bed slopes are small and velocities are gentle.

(*iv*) *Delta stage :* The first and last stages are unsuitable for construction of weirs. The choice of site selection lies between the boulder stage and the trough stage. The advantages and disadvantages of the first three stages are given below :

(*a*) **Rocky Stage**

(*i*) ***Disadvantages of Rocky Stage***

1. The soil, suitable for irrigation, must have good depth. Such a soil is not available in the rocky stage. This results in a *long idle length of the canal* from the headworks to the plains where good soil is available.
2. A canal taking off from a headwork situated in rocky stage will have to cross more discharge lines due to uneven mountaineous ground. Hence, *more cross drainage works* are required.
3. The ground in this region has steep gradients. Hence, either the canal bed will have to be lined to permit high velocities or *more falls are necessary* to dissipate the energy.
4. A shingle-excluding regulator is necessary if the headwork is situated on the rocky stage. This requires a *costly head regulator*.
5. The shingle crossing over the weir may damage it. This requires *frequent repairs* of the weir.

6. The high velocity associated with the rocky stage requires *use of better material*, increasing the cost of construction.
7. The water in this region does not contain silt and is thus *devoid of fertilising materials*.
8. The rivers in mountaineous regions are more flashy, resulting in floods appearing and disappearing suddenly. This causes *frequent failures*.

(ii) Advantages of Rocky Stage :

1. Good rocky foundations are available at surface or shallow depths. Hence, cost of construction of weir is less.
2. High heads are available for hydroelectric work.
3. There are no chances of the supply channel getting silted, since the steep slopes give rise to high velocities.
4. Comparative silt free water is fed to the turbines.

(*d*) **Sub-mountaineous or Boulder stage**

(*i*) ***Disadvantages***

1. There is *strong subsoil flow*. This decreases the storage and may cause damage to the floor downstream.
2. As in rocky stage, long *idle length of canal* is required.
3. There will be *more percolation losses* from the canal taking off in boulder tracts. This will increase the cost per cumec per net supply.
4. *More cross-drainage works* are required.
5. In the boulder stage, *rivers are more flashy* than in trough stage.
6. There is less demand of water at head reaches.

(ii) Advantages of Boulder Stage :

1. Cash crops like transplanted rice, tobacco, sugar, tea etc., can be grown in the sub-mountaineous regions, where there was no irrigation earlier.
2. Cost of headworks is less due to availability of local material.
3. Less river training works are required.
4. Falls can be utilized for power generation.

(*c*) **Trough stage**

(i) Disadvantages of trough reach :

1. Cost of headworks more due to poor foundation.
2. More river training works are required.
3. There is problem of silt in the canal.

(ii) Advantages of trough reach :

1. Subsoil flow is comparatively less.

2. The head works constructed in this region serve large area than a weir in boulder region can. There is no idle length of canal.

3. The tracts lying at the foot of hills do not require water with the same urgency as those lower down. Consequentiy headworks in trough regions are more remunerative as the cultivators will be willing to pay higher assessment.

4. The cross-drainage works across the canals are less.

5. The water contains silt and other fertilising material.

12.5. EFFECTS OF CONSTRUCTION OF A WEIR ON THE REGIME OF RIVER

The weir is an obstruction thrown in the path of water. Due to its construction, therefore, the regime of river is affected in the following ways :

1. The silt supporting power of a river or channel mainly depends upon the hydraulic slope. When the weir is constructed, the heading up of water leads to flattening of the surface slope on the upstream side.

2. Due to decrease in the water surface slope, the silt carrying capacity is decreased, and the bulk of silt charge of the river water deposits in the pond, leading to the formation of irregular shoals at u/s of the weir.

3. Also due to silt excluding devices provided at the head regulator, the canal takes less silt. This results in further deposition of silt in the pond.

4. The water passing over the weir and through the scouring sluices now contains a deficient silt charge because much of it has been deposited u/s. In order to maintain a constant silt charge, the flowing water at the d/s scours the bed. This results in a progressive degradation or retrogression of bed levels downstream. The retrogression may undermine the stability of a work by an increase in exit gradient beyond safe limits. During high floods, the retrogression of bed may be from 0.3 to 0.5 m, while at low water levels, it may be as high as 1.2 to 2 metres.

5. As the silting and consequent shoals formation at u/s increases, the resistance to flow of river is increased due to tortuous route the water has to take about shoals. To overcome this resistance, increased head is required. The river starts regaining its original slope, and the afflux is extended more and more to the u/s. A stage is then reached when the upstream section of river cannot take up any more silt, and the normal silt charge is passed on the downstream side. The silt excluding device will also discharge more silt downstream.

6. Due to this, river below the weir will carry an excessive silt charge with a lower discharge. This will result in progressive silting up downstream, an increase in tortuosity and, therefore, a recovery of bed levels downstream. The process of recovery of downstream bed levels after the initial retrogression is slow and steady, and it may take 20 to 30 years to regain the original bed slope. The recovery of levels to the d/s may lead loss of control on the silt regulation. Hence, sufficient margin should be provided between canal F.S.L. and the pond level so that the crest level of the head regulator can be increased in the event of necesssity.

12.6. CAUSES OF FAILURE OF WEIRS AND THEIR REMEDIES

A weir may fail due to the following reasons :

(*i*) Piping

where q = discharge intensity in cumecs/metre.

(4) The length of the impervious apron upstream of the sluice gate may be determined from the Bligh's recommendations as under :

$$l_2 = 3.9\, C \sqrt{\frac{H_S}{13}} \text{ metres.} \qquad ...(12.40)$$

(5) From the consideration of retrogression, the downstream floor should be depressed upto the likely retrogression bed level downstream.

(6) A bridge downstream of the under sluices should always be provided so that the cranes could move on it to lift the gates if the Stoney's arrangement fail.

12.15. THE CANAL HEAD REGULATOR

A head regulator is a structure constructed at the head of a canal taking off from a reservoir behind a weir or a dam. A head regulator may consist of a number of spans separated by piers and operated by gates similar to that provided in a barrage. The modern tendency is to use bigger spans of 6 to 18 m controlled by Stoney gates or sector gates.

Functions of head regulator

(1) To make the regulation of supply in the canal easy.

(2) To control the silt entry in the canal.

(3) To shut out river floods.

Fig. 12.33 shows the section of canal head regulator.

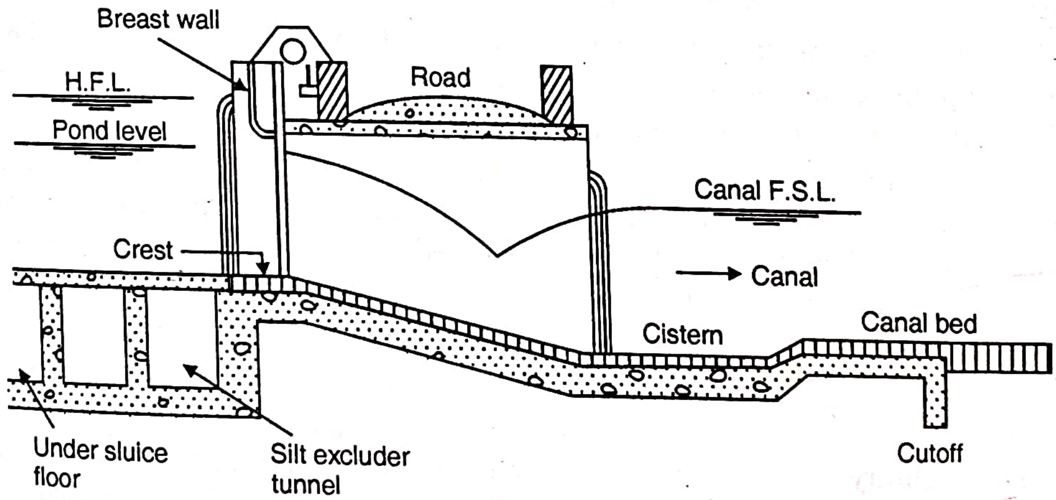


FIG. 12.33. CANAL HEAD REGULATOR.

Design considerations

(1) The water way of the head regulator should be sufficient of pass the full supply discharge of the canal, with ample factor to safety to allow for any silting up of canal. For a head regulator with broad crest and sloping glacis at the downstream, the following discharge equation is applicable :

$$Q = 1.7(L - knH)\, H^{3/2} \qquad ...(12.41)$$

where Q = total discharge in cumecs
L = length of water way in metres
H = head causing flow
n = number of end contractions
k = constant, depending upon the shape of the nose of the pier; varies from 0.01 to 0.03.

(2) The regulators are normally aligned at 90° to the weir, but greater angles, upto 110° are considered preferable for providing smooth entry.

(3) The crest level of the head regulator should be higher than the crest of the under sluices by a minimum of 1 to 1.2 metres if silt excluder is not provided and greater than 1.8 to 2 m if silt excluding device is provided.

(4) The hydraulic jump calculations over the sloping glacis are done in the same way as for sloping glacis weir. The length of cistern below the end of the glacis should at least be equal to $5(D_2 - D_1)$. The level of the cistern bed should be well below the minimum level of the formation of hydraulic jump under various discharge conditions.

(5) The design of the impervious floor should be done on the basis of uplift pressure in the manner similar to that of sloping glacis weir. The worst condition will be during the high floods when the canal is shut and maximum static head acts. In case the floor thickness becomes excessive, a reinforced concrete mat should be provided to resist the uplift pressure by bending action.

(6) A concrete cut-off at the end of the impervious floor should always be provided to keep the exit gradient well within the limits.

(7) The piers separating the various spans of the regulator should be extended beyond the sloping glacis so that they may provide support to the upward bending reinforced concrete mat of the cistern floor.

(8) In order to prevent spilling of water towards the canal during high floods, a reinforced concrete breast wall should be provided from pond level to well above the high flood level. The breast wall is supported between the piers, and is designed for its self weight as well as the water pressure from upstream. They may also be designed for additional weight of gate lifting arrangements, if supported on them.

(9) For the proper operations of the regulator, a bridge is provided, spanning over the piers.

(10) The stability of the piers of the head regulator should be tested to withstand the overturning moment caused by the high pressure head during floods.

(11) There is no empirical formula to determine the length of the *talus* of downstream of head regulator. Generally a *talus* of 4 to 5 times the depth of canal and 0.8 to 1 m thick in concrete blocks or stones are considered to be sufficient.

Types of Regulation

There are two methods of regulation adopted at a head regulator to control the entry of silt into the canal :

(*i*) Still pond regulation.

(*ii*) Open flow regulation.

Still pond regulation. In this method, the pocket sluices are entirely closed and the canal draws water from the still pond in the pocket. The water in excess of the canal requirements is thus not allowed to escape under the sluice gates. The velocity of water in the pocket is very much reduced on account of excessive water-way since only the supply required for the canal enters the pocket. The silt is thus deposited in the pocket and clear water enters the canal. When the silt deposited has a level about $\frac{1}{2}$ to 1 metre below the crest level of the regulator, the supply in the canal is shut off for about 24 hours and the sluice gates are opened to scour the deposited silt and discharge it downstream. The process is repeated.

Open flow regulation. In this system, the under sluices may be kept open so that the river supply in excess of the canal requirements is escaped. Top water passes into the canal while the bottom water maintains a certain velocity in the pocket to keep the silt to remain in suspension. The advantage of this system is that the canal is not to be closed for scouring the silt. However, the method is very treacherous on account of the uncertain approach channel conditions in the river.

12.16. SILT CONTROL AT HEADWORKS

The entry of silt into the canal can be controlled by :

(1) Providing a divide wall in the river at the canal side so as
  (*a*) to create a trap or pocket.
  (*b*) to create the scouring capacity of under sluices by concentrating the current towards them.

(2) Paving the bottom of the approach channel to reduce disturbance.

(3) Installing a silt excluder.

(4) Making entry of clear top water in the canal by
  (*a*) providing raised sill in the canal.
  (*b*) lowering sill level of scouring sluices.

(5) Reducing the velocity of water at the intake by providing wider head regulator.

(6) Avoiding unsteady flow by making the entry smooth.

(7) Handling carefully the regulation of weir.

There are two types of special works constructed to control the silt entering into the canal :

1. Silt excluder. 2. Silt extractor.

1. Silt excluder. Silt excluder is a device by which silt is excluded from water entering the canal. It is constructed in the bed in front of head regulator.

The fundamental principle on which a silt control device acts lies in the fact that in a flowing stream carrying silt in suspension, the concentration of silt charge in the lower layer is greater than in the upper ones. Hence, the device is so designed that the top and bottom layers are separated without any disturbance. The top water is then led towards the canal while the bottom water containing high silt charge is wasted.

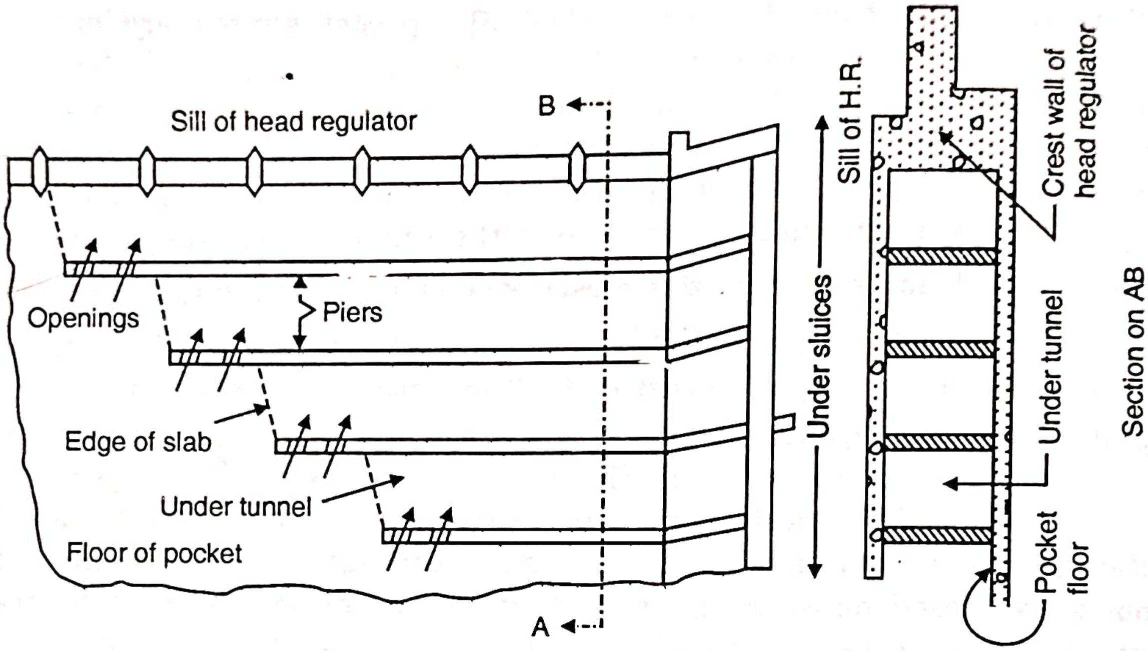


FIG. 12.34. SILT EXCLUDER.

Fig. 12.34 shows a silt excluder — a type used at Khanki weir. The excluder consists of a number of under tunnels resting on the floor of the pocket. The top level of the R.C. roof of the tunnels is kept the same as the sill level of the head regulator. The various tunnels are made of different lenths — the one near the head regulator being of same length as that of the width of the head regulator and the successive tunnels being of decreasing lengths as shown. This arrangement separates the water into two clear layers. The top layer (above the roof of the under tunnels) enters the head regulator, while the bottom layer, containing relatively heavier silt charge goes to the under tunnels and discharges to the d/s of the river through under sluices. The capacity of these tunnels is kept about 10 % of the canal discharge, and the tunnels are so designed that a minimum velocity of 2 to

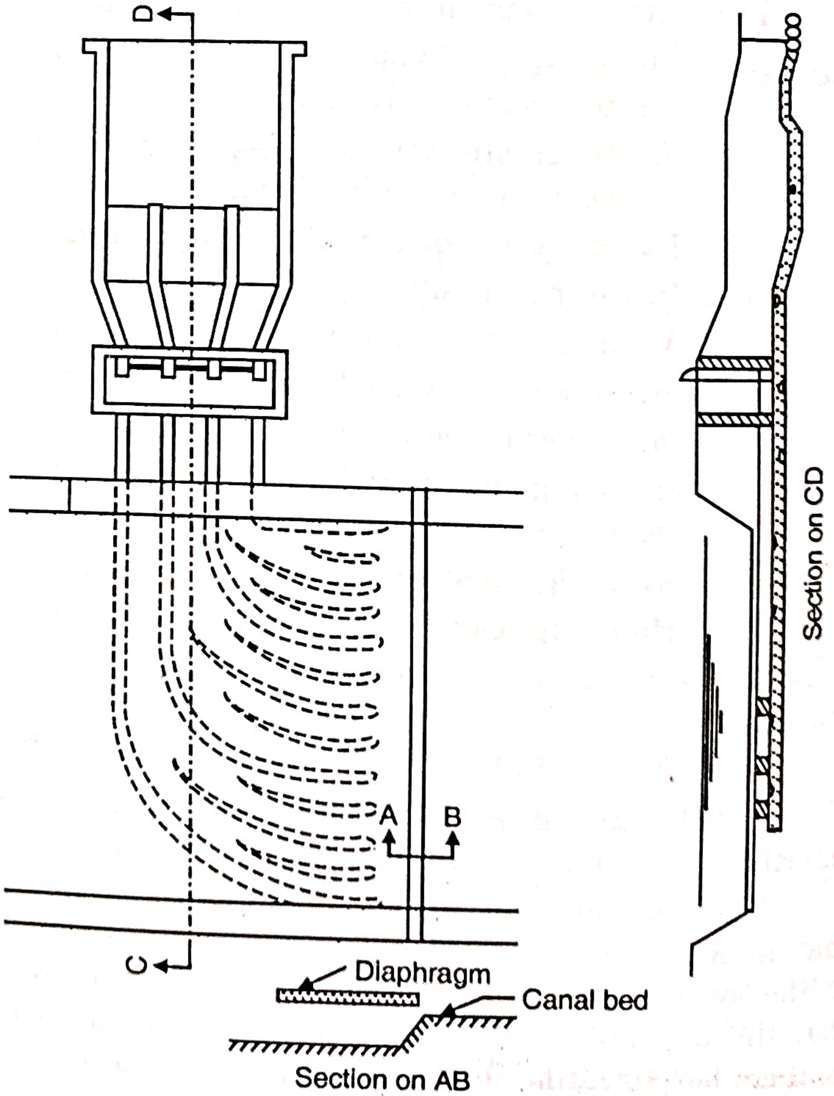


FIG. 12.35. SILT EXTRACTOR.

13

Flow Irrigation

13.1. CANALS : CLASSIFICATION

A canal is an artificial channel, generally trapezoidal in shape constructed on the ground to carry water to the fields either from the river or from a tank or reservoir. Canals can be classified in following ways :

(*a*) **Classification based on the nature of source of supply**

(1) Permanent canal. (2) Inundation canal.

A canal is said to be *permanent* when it is fed by a permanent source of supply. The canal is a well made up regular graded channel. It has also permanent masonry works for regulation and distribution of supplies. A permanent canal is also sometimes known as *perennial canal* when the sources from which canal takes is an ice fed perennial river.

Inundation Canals usually draw their supplies from rivers whenever there is a high stage in the river. They are not provided with any headworks for diversion of river water to the canal. They are, however, provided with a canal head regulator. The head of the canal has to be changed sometimes to suit the changing pattern of river course.

(*b*) **Classification based on financial output**

(1) Productive canal. (2) Protective canal.

Productive Canals are those which yield a net revenue to the nation after full development of irrigation in the area. *Protective canal* is a sort of relief work constructed with the idea of protecting a particular area from famine.

(*c*) **Classification based on the function of the canal**

(1) Irrigation canal. (2) Carrier canal.

(3) Feeder canal. (4) Navigation canal.

(5) Power canal.

An *irrigation canal* carries water to the agricultural fields. A *carrier canal*, besides doing irrigation, carries water for another canal. Upper Chenab canal in West Punjab (Pakistan) is the example of one such canal. A *feeder canal* is constructed with the idea of feeding two or more canals. Examples of such canals are : Rajasthan feeder canal and Sirhind feeder.

(d) Classification based on boundary surface of the canal

Based on the type of *boundary surface,* canals may be of the following types :

1. Alluvial canals
2. Non-alluvial canals

and 3. Rigid boundary canals.

An *alluvial canal* is the one which is excavated in alluvial soils, such as silt. A *non-alluvial canal* is the one which is excavated in non-alluvial soils, such as loam, clay, hard soil (murram), rock etc. *Rigid boundary canals* are those which have rigid sides and rigid base, such as *lined canals.*

(e) Classification based on the discharge and its relative importance in a given network of canals

(1) Main canal. (2) Branch canal.
(3) Major distributary. (4) Minor distributary.
(5) Water course.

Main canal generally carries water directely from the river or reservior. Such a canal carries heavy supplies and is not used for direct irrigation except in exceptional circumstances. Main canals act as water carriers to feed supplies to branch canals and major distributaries.

Branch canals are the branches of the main canal in either direction taking off at regular intervals. In general, branch canals also do not carry out any direct irrigation, but at times direct outlets may be provided. Branch canals are usually feeder channels for major and minor distributaries. They usually carry a discharge of over 5 cumecs.

Major distributaries usually called *Rajbha,* take off from a branch canal. They may also sometimes take off from the main canal, but their discharge is generally lesser than branch canals. They are real irrigation channels in the sense that they supply water for irrigation to the field through outlets provided along them. Their discharge varies from 1/4 to 5 cumecs.

Minor Distributaries or *minors* take off from branch canals or from distributaries. Their discharge is usually less than 1/4 cumecs. They supply water to the water courses through outlets provided along them.

A water course or *field channel* is a small channel which ultimately feeds the water to irrigation fields. Depending upon the size and extent of the irrigation scheme, a field channel may take off from a distributary or minor. Sometimes, it may even take off from the branch canal for the field situated very near to the branch canal.

(f) Classification based on canal alignment

According to the alignment, a canal may be classified as under (Fig. 13.1):

(1) Contour canal.
(2) Watershed canal.
(3) Side slope canal.

The characteristic features of these canals are discussed in the next article.

13.2. CANAL ALIGNMENT

A canal has to be aligned in such a way that it covers the entire area proposed to be irrigated, with shortest possible length and at the same time its cost including the cost of cross drainage works is a minimum. A shorter length of canal ensures less loss of head due to friction and smaller loss of discharge due to seepage and evaporation, so that additional areas can be brought under cultivation.

According to alignment, the canals may be of the following types (Fig. 13.1)

(1) Ridge canal. (2) Contour canal.

(3) Side slope canal.

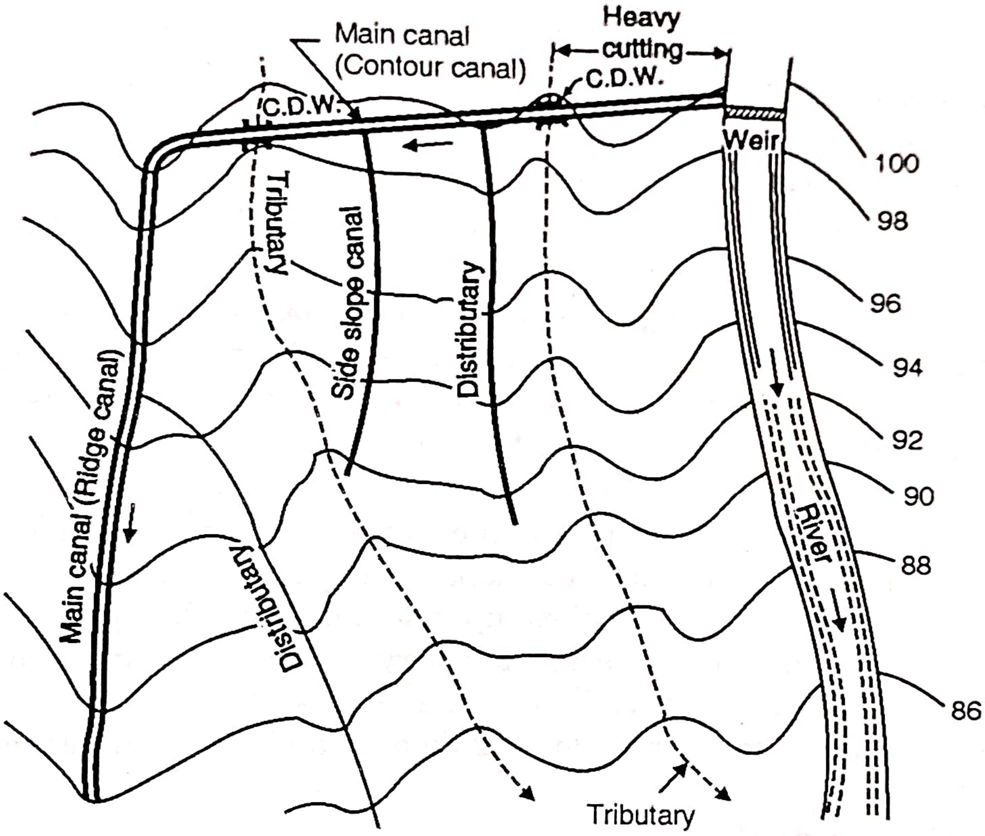


FIG. 13.1. CANAL ALIGNMENT.

1. Ridge Canal

A ridge canal or a watershed canal is aligned along a watershed and runs for most of its length on a watershed. When a channel is on the watershed, it can command areas on both banks and hence a large area can be brought under cultivation. Also, no drainage can intersect a watershed and, hence, the necessity of constructing cross drainage works are obviated.

When the watershed takes a very sharp loop, the canal should be aligned straight to save considerable idle length, as shown in Fig. 13.2. The area of the loop between the watershed and the canal cannot be irrigated by this canal as there will be higher ground on this side and some other arrangement has to be made if the irrigation is important in that area. The canal checks the drainage of this part and a cross-drainage work has, therefore, to be provided. A cost analysis should, therefore, be made before aligning the canal straight to reduce its length. The canal has also to leave watershed to by-pass towns and villages situated at the watershed.

2. Contour Canal

A channel aligned nearly parallel to the contours of the area is called a contour canal.

When the canal takes off from a river in a hilly area, it is not possible to align the canal on the watershed as the watershed on the top of the hill may be very high and the areas which need irrigation are concentrated in the valley. The canal is then aligned roughly parallel to the contours of the area. The contour chosen for the alignment should be so placed as to include all culturable area of the valley on one side of the canal.

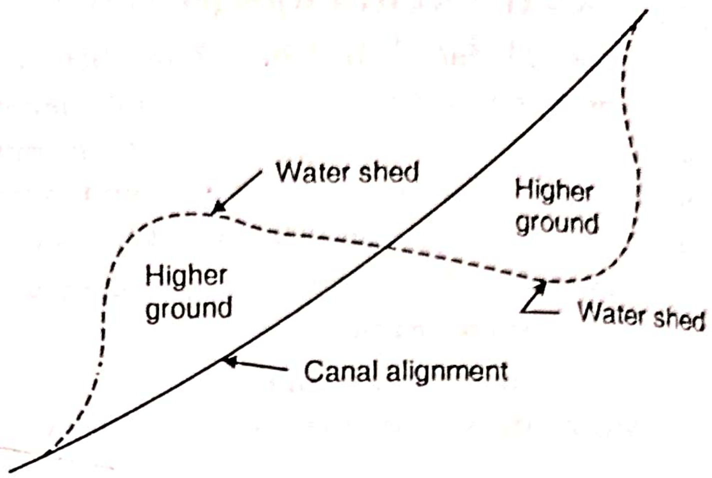


FIG. 13.2. CANAL ALIGNMENT FOR SHARP TURNS IN WATERSHED.

The contour canal can irrigate only on one side. As the ground level on the other side is quite high, there is no necessity of a bank on this side. Hence, a contour canal is sometimes constructed with one bank only, and is known as a *single bank canal.* However, when both the banks are provided, it is known as a *double bank canal.*

The contour canal does not follow the same contour all along. To enable the water to flow by gravity, some surface slope is given. The rate at which the canal alignment leaves one contour and takes up another depends upon its surface slope. It is usual, in highly undulated tracks, to carry the channel in deep cutting across the ridges or spurs and in high embankments across the valleys in order to reduce the unnecessary length of the channel in long detours and sharp curves in the alignment, as shown in Fig. 13.1.

3. Side Slope Canal

It is a channel aligned roughly at right angles to the contours of the country and is neither on the watershed nor in the valley. Such a channel would be roughly parallel to the natural drainage of the country and, hence, it does not intercept any cross-drainage. However, it has very steep bed slope, since the direction of the steepest slope of the ground is at right angles to the contours of the country.

General considerations for alignment

1. The alignment of the canal should be such as to ensure (*i*) the most economical way of distributing the water to the land, (*ii*) as high a command as possible, and (*iii*) minimum number of cross drainage works.

2. The alignment of a canal on a watershed, being the most economical, is preferred. As a general rule, all the watershed lying in a command should be occupied by distributaries.

3. The length of the main canal from the point where it takes off from a river to a point where it mounts on a watershed should be minimum.

4. The contour alignment should be changed this way or that way in order to reduce the number of cross-drainage works to a minimum.
5. The alignment should avoid villages, roads, cart tracks, cremation places, places of worship and other valuable properties.
6. The alignment should pass through the balanced depth of cutting. If not, it should involve minimum depth of cutting or minimum height of filling.
7. The number of kinks and acute curves should be minimum.
8. Idle length of canal should be minimum and branches etc. should be economically planned.
9. The alignment should not be made in a rocky, brakish or cracked strata.

Alignment of a field channel or water course :

Though the maintenance of a field channel is the responsibility of the farmers, its alignment should have the following features :

1. They should be laid along field boundaries.
2. They should be capable of supplying sufficient water to the tail end.
3. Separate field channels should be provided for high and low lands.
4. The field channels should not pass through rocky, brakish, or cracked strata.

13.3. CURVES

Curves should not be provided along the length of a canal except where necessary. Introduction of a curve in a channel disturbs the regime of channel. The concave side is always under erosion and the convex side has a tendency to silt.

If the provision of curve is a must, minimum radius should be provided as shown in Table 13.1.

TABLE 13.1

Capacity of channel (cumecs)	*Minimum radius of the curve (metres)*
Less than 0.3	100
0.3 to 3	159
3 to 15	300
15 to 30	600
30 to 85	900
over 85	1500

13.4. INUNDATION CANALS

Introduction of inundation canal in a particualr area is resorted under the following situations :

1. When the flood in the river is of such a duration that the water level in the canal remains high and fairly constant for quite a long time.

2. When there is an early flood in the river so that Kharif crops can be irrigated.

3. When the river has a late last flood at the end of rainy season so that Rabi crops can be irrigated.

4. When the area to be irrigated is in close proximity of the canal system so that a large area is not submerged in case there is a worst flood in the river.

5. When the soil has a sufficient self-stabilising power so that bed and banks of the canal remain stable.

In general an inundation canal is similar to a permanent canal. However, the major difference between them is that while in the case of a permanent canal a headwork (such as a weir, barrage or a dam) is constructed, no such works are needed for an inundation canal. But instead, an open cut in the river bank is provided for the inundation canal to take off from the river. The bed level of the cut is kept much above the bed level of the river.

Site selection for the offtake of inundation canals

The selection for the offtake point is very important because the success of inundation canal mostly depends upon it. Following are the main points to be considered for the site selection :

1. At the side of offtake, river course should be straight, and the banks should be stable and high. The river should also flow with an average velocity and should have normal width. The variation of water level should also be low. In case the offtake has to be located on a curve it should be located on the outer side of the curve (*i.e.* the concave side) so that there is no problem of silting in head reaches.

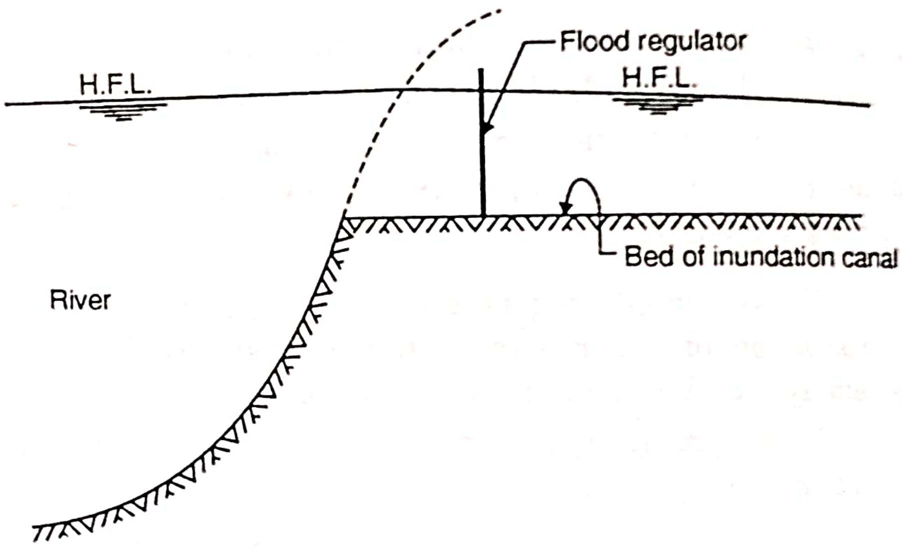


FIG. 13.3. INUNDATION CANAL.

2. The offtake should be located as near the area to be irrigated as possible.

3. If a bypass of a river is available the offtake should be located at the downstream end of the bypass. There may be less of silt trouble in this case since the velocity in the bypass will be almost equal to that required in the inundation canal.

Presence of a shoal creates a still pond in the pocket and thus the silt entry is restricted and reduced.

Design considerations for inundation canals.

The trouble in the inundation canal arises due to lack of control of water levels, causing deposits of sand and formation of bar at mouth with a sudden fall in the river and sanding the canal. Many a times, the flood water finds its way through the canal and submerges vast stretch of land.

Design considerations for inundation canal are as follows :

1. A head regulator, generally known as *flood regulator*, may be provided a few miles downstream of the offtake point. If the flood regulator is provided at the offtake point, there is a danger of the same being washed away during floods.

15.3. LOSSES IN CANALS

When water continuously flows through a canal, losses take place due to seepage, deep percolation and evaporation. These losses are some times known as *transmission losses.* These losses should be properly accounted for, otherwise lesser quantity of water will be available for cultivation at the tail end. Water losses in canals can be broadly classified under three heads :

(*i*) Evaporation losses, (*ii*) Transpiration losses., (*iii*) Seepage losses.

(*i*) *Evaporation losses.* The loss due to evaporation is generally a small percentage of the total loss in unlined canal. It hardly exceeds 1 to 2 percent of the total water entering into the canal. The evaporation losses depend upon *(i) Climatic factors* such as temperature, humidity and wind velocity, and *(ii) Canal factors* such as water surface area, water depth and velocity of flow. Maximum loss is there in summer months when temperatures are high and wind velocities are also high. Similarly, losses are maximum in unlined canals due to wider water surface area, shallower water depth and low velocity. The average evaporation loss per day may vary between 4 mm to 10 mm.

(*ii*) *Transpiration losses.* The transpiration loss takes place through lot of vegetation and weeds growth along the bank of canal. However, this forms a extremely small part of total loss.

(*iii*) *Seepage losses.* Seepage losses constitute major portion of loss in an unlined canal. The seepage losses are due to (*i*) absorption of water in the upper layers of soil below the canal bed, and due to (*ii*) *percolation* of water into the water table, thus raising the water table. If, however water table is much lower, seepage losses are only due to absorption. Percolation losses are always much more than the absorption losses.

Rate of water loss

Canal losses may be expressed in any one of the following methods :

(*i*) as cumecs per million square metre of the wetted perimeter.
(*ii*) as depth of water lost per day over the wetted perimeter.
(*iii*) as percentage of the canal discharge.
(*iv*) as percentage per kilometer length of the canal.

Out of these four methods, the first method is the simplest, and is quite popular. In absence of any other data, the transmission losses may be taken as 2.5 cumecs per million square meter of wetted perimeter for unlined channels and as 0.60 cumecs per million sq. metre of wetted perimeter for lined canals.

In U.P., the loss Q_L in cumeces per km length of unlined canal is given by

$$Q_L = \frac{1}{200}(B + D)^{2/3} \quad ...(15.3)$$

where B and D are in metres.

In Punjab and Haryana, the losses in unlined canals in cumecs per million square metre of wetted perimeter is given by the expression

$$Q_L = 1.9\, Q^{0.0625} \quad ...(15.4)$$

The Central Water Power Commission (CWPC) recommends the following values of losses in unlined canals (Table 15.1).

TABLE 15.1. CWPC RECOMMENDATION FOR LOSSES IN UNLINED CANALS

Soil Type	*Transmission loss (cumec/million sq meter of wetted perimeter)*
1. Rock	0.91
2. Black cotton soil	1.83
3. Alluvial soil	2.74
4. Decayed rock or gravel	3.0

Losses are sometimes expressed as percentage of total discharge, as under.

Main canal and branches : 15% to 20 %

Distributaries and minors : 6% to 7%

Water courses : 17% to 22%

Losses in lined canals : In Punjab, the following formula is used for the determination of total losses in cumces per Mm^2 (million square meters) of wetted perimeter

$$K = 0.349\, Q^{0.056} \qquad ...(15.5)$$

where Q is the discharge in cumecs.

However as per recommendations of C.W.P.C. and IS: 1745-1968, the transmission losses in lined canals can be taken as 0.6 cumecs/Mm^2 of the wetted perimeter.

15.4. SCHEDULE OF AREA STATISTICS AND CHANNEL DIMENSIONS

The design of channel cross-section from km to km is carried out in a Tabular form called the *Schedule of Area Statistics and Channel Dimensions.*

The schedule of area statistics and channel dimensions is shown in Tables 15.2 and 15.7. Working of the Table is explained below and has later been illustrated by Example 15.3.

TABLE 15.2. SCHEDULE OF AREA STATISTICS AND CHANNEL DIMENSIONS

Below km	*Gross commended area (G.C.A.)*	*Culturable commended area (C.C.A.)*	*Area to be irrigated*			*Outlet discharge factor*	*Outlet discharge*	*Losses in reach*	*Total loss*	*Total discharge*	*Channel Dimensions*						*Velocity (V) of flow*	*V / V$_0$ Ratio or silt factor f*
			Rabi (%)	*Perennial (%)*	*Kharif (%)*						*Bed slope*	*Bed width*	*Water depth*	*Design discharge*	*Height of Bank*	*Width of Bank*		
	ha	*ha*	*ha*	*ha*	*ha*	*ha/cumec*	*cumec*	*cumec*	*cumec*	*cumec*	*cu/km*	*m*	*m*	*cumec*	*m*	*m*	*m/s*	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
0																		
1																		
2																		
:																		
:																		
n																		

Col. 1. The actual design of channel is carried from km to km. Sometimes an off-taking channel may take off in between the kilometre interval. The channel dimensions are then also found out at the downstream of the cross regulator.

Col. 2. Column 2 indicates the gross commanded area, i.e. the entire area under the command of the channel below the particular kilometre at which it is being designed.

Col. 3. In this column, the area actually under cultivation below the particular km under the command of the channel is indicated

Col. 4 to 6 indicate the percentage of area under Rabi, perennial and Kharif crops. Mostly Rabi crops are controlling crops for finding the discharge of the channel. Channels either designed for Rabi or Kharif crops suffice the purpose for sugar cane irrigation. However, under some local conditions the discharge needed for sugar-cane irrigation is found out.

Col. 7. Indicates the outlet discharge factor for the controlling crop, calculated by the methods indicated in Chapter 3.

Col. 8. Indicates the outlet discharge required and can be found out by multiplying area to be irrigated to the outlet discharge factor.

Col. 9. Indicates the losses in the reach from kilometre to kilometre. Generally losses per million square metre of the wetted surface area is known and so the channel losses in different reaches have to be calculated on the basis of tentative design for channels.

Col. 10. Indicates the total losses in the channel below any particular kilometre of the channel.

Col. 11. Indicates the total discharge for which the channel has to be designed. This discharge includes outlet discharge and losses.

Col. 12 to 18. These columns pertain to channel dimensions and are filled up after designing the channel in each reach based on Kennedy theory using Garret's diagrams or on Lacey's theory using Lacey's diagrams.

15.5. USE OF GARRET'S DIAGRAMS FOR CHANNEL DESIGN ON KENNEDY'S THEORY

Garret's diagrams give the graphical method of designing the channel dimensions based on Kennedy's theory. The original diagrams were prepared in F.P.S.units. Fig. 15.3 (*a*) and (*b*) show two such diagrams converted into metric units. The diagram has the discharge plotted on the abscissa. The ordinates on left indicate the slope and that on right the water depth in the channel and critical velocity V_0. The discharge lines are curved and bed width lines are shown dotted.

The proccdure in designing the channel consists of the following steps.

1. Find out the discharge for which the channel is to be designed. This can be found from Schedule of Area Statistics.

2. Find out the slope of the channel from its longitudinal section.

3. In Fig. 15.3, follow the discharge line and find out its intersection with the horizontal lines from slope. Interpolation may be done if needed to locate the discharge line. Mark the intersection point *P*.

however, the canal has to be carried through deep cutting or filling. A channel section may, therefor, be either :

1. In cutting 2. In filling, or 3. In partial cutting and filling.

The channel section in these three conditions are shown in Fig. 15.6. When the ground level is above the top of the bank, the canal is said to be in cutting. Similarly, when the ground level is below the bed level of canal, it is said to be in filling. A canal is in partial cutting or filling when the ground level is in between bed level and top of bank.

A canal can have a dowla section on one side or a bank section on both sides. The section may also be designed to have a bank section on one side and dowla section on the other side. Usually the left bank of canal has a dowla section and service road and right bank of the canal has a bank section.

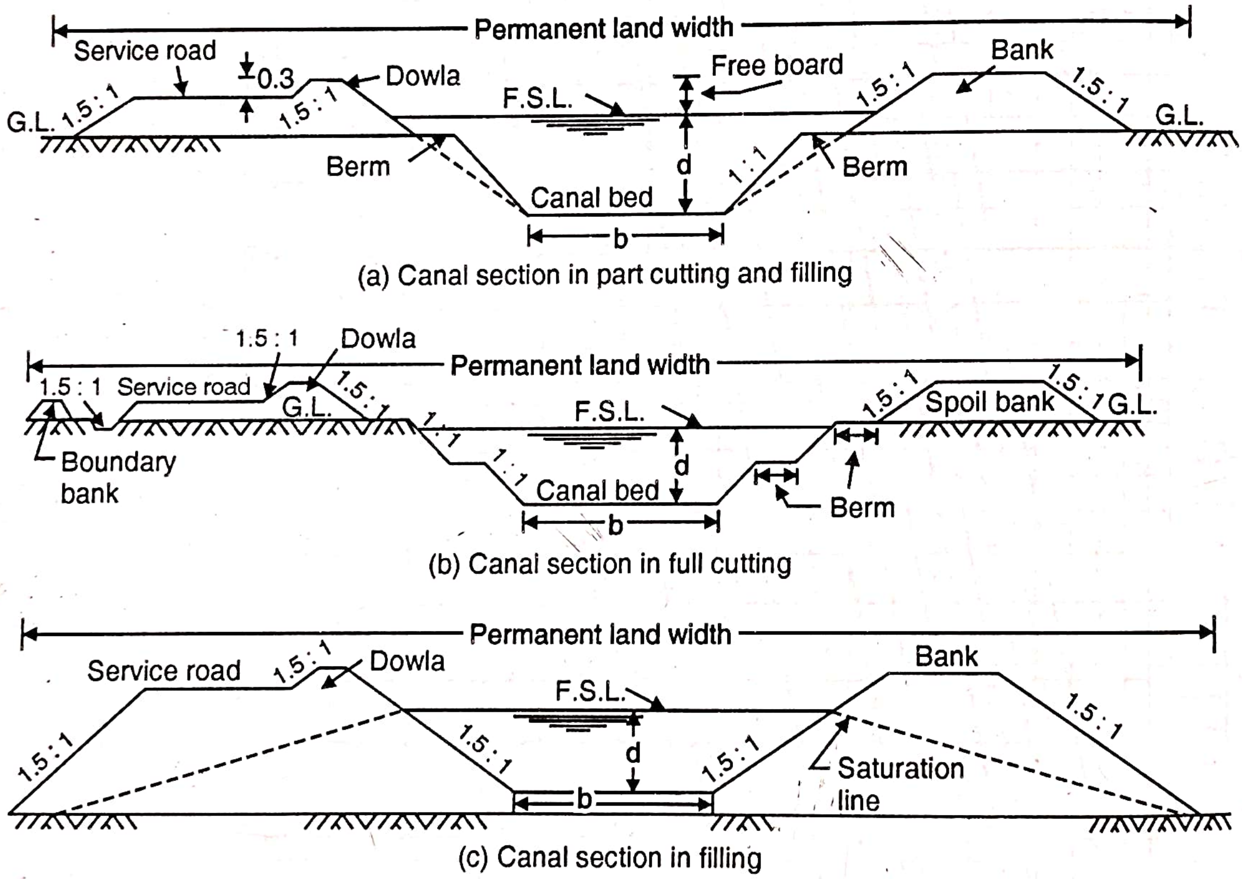


FIG. 15.6. TYPICAL CANAL SECTIONS.

Side Slopes

For the computation of the values of hydraulic mean depth R, the area and wetted perimeter are worked out with $\frac{1}{2}:1$ side slopes for design purpose even though in execution actually flatter slopes depending upon stability and type of soils are adopted. $\frac{1}{2}:1$ side slope is assumed with the presumption that the sides of the channel get silted up to $\frac{1}{2}:1$ slope and channel capacities would be reduced accordingly in due course. However, in case a channel takes off from such storage reservoirs where silt

15.11. BACK BERM, COUNTER BERM

Even after providing the usual embankment section of a bank, the saturation gradient may cut the downstream side of the bank. In such a case the saturation line should always be covered by 0.3 to 0.6 m of earth. This is best done by the help of counter berm as shown in Fig. 15.11.

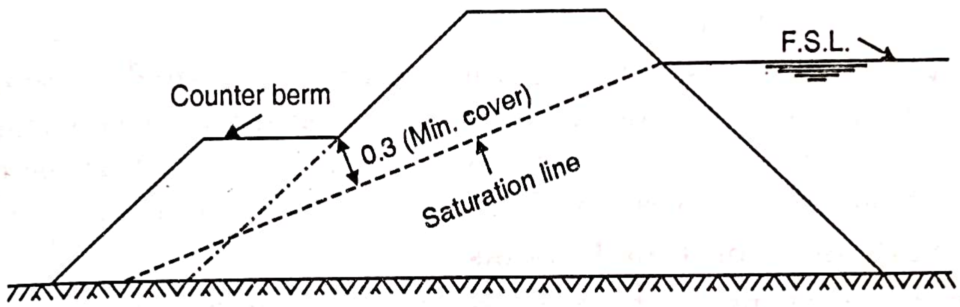


FIG. 15.11. COUNTER BERM.

15.12. MAINTENANCE OF IRRIGATION CHANNELS

After the construction of irrigation system in an area is completed, it becomes essential to maintain it for its proper and efficent functioning. There are various reasons due to which a canal may cease to function effciently. These are :

(*i*) Silting of canal
(*ii*) Breaching of canal due to weak banks
(*iii*) Weed growth
(*iv*) Overflow of canal banks

1. Silt Removal :

When the silt is deposited on the bed and sides, the capacity of the canal reduces. It is better to exclude silt by providing silt excluder and ejector as explained in Chapter 12. Curved wing-cum silt vanes and silt tunnels-cum-curved wings can be fitted near the head of an off-taking canal for exclusion of silt. However, none of these methods can put a complete check over the entry of silt in the channel and hence silt is deposited in the channel bed even after the best care.

The following measures are adopted to remove the silt :

(*a*) ***Flushing.*** Flushing of the canal with clear water will lift up deposited silt. Absolutely clear water should be used for flushing but if this is not available then the water which contains minimum quantity of silt should be used for flushing. Flushing should generally be over done to cause some scour. This will create room for further silting and will thus reduce the frequency of flushing.

(*b*) ***Silt scouring fleet.*** The method consists of having three lower barges connected to the upper barge by a cable operated by winch. The lower barges have movable shutters. The silt is kept agitated by manoeuvering the barge up and down. The method was used in Punjab but was unsuccessful.

(*c*) ***Bundle of thorny bushes*** tied together and pressed down by weight of stones are pulled inside the channel by animals. They are quite useful in dislodging the fine muddy silt.

(*d*) ***Iron rakes*** are also dragged in the channel to dislodge silt.

(*e*) ***Reduction of area of flow.*** Loaded boats are put across the section to reduce the area of flow and increase the velocity of flow.

(*f*) ***Stirring of silt by water jet.*** A pump fitted with a pipe and nozzle is placed on a barge. The high velocity jet is directed to the bed to stir the silt and prevent silting.

(*g*) ***Dredging.*** A dredger is very rarely employed for removing silt from canal as it is very costly method.

(*h*) ***Excavation.*** The silt deposited in a channel is cleared off by manual labour. The method is quite costly as it requires recurring expenditure. This method is generally adopted for silt clearance in distributaries and minors. The silt must be deposited clear off the channel so that it does not find its way back to the channel.

2. Strengthening of Canal Banks

To prevent breaching of the canal bank, it should be strengthened properly so that valuable loss of irrigation and property is prevented due to breaching a canal section. There are four methods of strengthening a canal bank :

(*a*) External silting system. (*b*) Internal silting system.

(*c*) Formation of berm by internal silting.; (*d*) Formation of back berm.

(*a*) ***External silting system.*** In this method subsidiary banks are constructed which run parallel to the main banks. Cross bunds are constructed within the subsidiary bank and main bank at a distance of 150 to 1500 m to form compartment of silting or *silting tanks.* Water is allowed to get into the compartment from upstream side and is held there for some time before discharging it back to the canal from outlet end. When the cross bunds are separated at a distance of 150 to 300 m, the capacity of the bank is small and only a portion of full supply discharge is taken into the compartment. It is then known as *into out system.* But when the length of the compartment is large say 1200 to 1500 m, full discharge of the canal can be taken inside the compartment and it is then known as *long reach system.* The method is practised only when no water is required for irrigation downstream of the reach.

(*b*) ***Internal silting system.*** In this system the canal banks are set back away from their original positions. The section of the canal provided is large than required and, therefore, its velocity is low. The section, therfore, gets silted up very quickly. To induce silting and accelerate the process, low submersible spurs are constructed. The silted berm is shown in Fig. 15.12.

Hanging groyne or suspended groyne is also very useful for inducing silting.

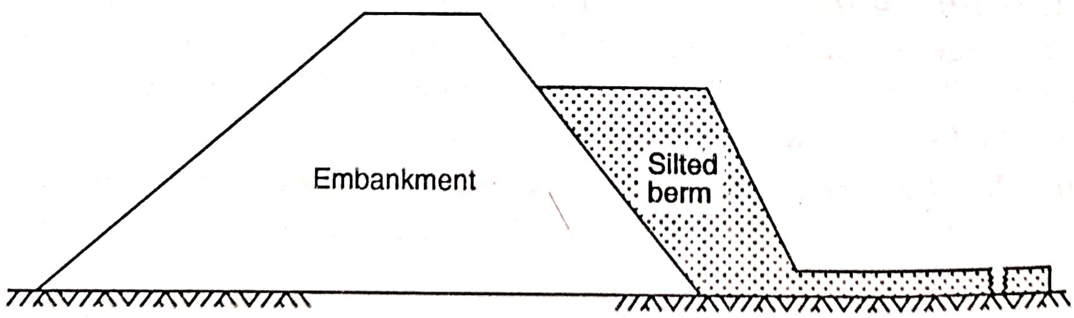


FIG. 15.12. SILTED BERM.

(c) ***Formation of berms by internal silting.*** Banks can also be strengthened by formation of berm by internal silting. The silting of the berm can be accelerated by constructing permeable spurs from the side of the channel section.

(d) ***Formation of back berm.*** A back berm may be formed if the saturation line crosses the downstream slope of the bank as explained in § 15.11.

3. Weed control

Water weeds are unwanted plants that grow profusely in water under certain favourable conditions. They tend to reduce the discharging capacity of channel by reducing the area of the channel section and velocity of flow. The problem of weed growth is more marked in Deccan where the heavy weed growth may reduce the channel discharge to even less than 15%. The nuisance has, therefore, to be checked to permit the channel to function efficiently.

There are a variety of weeds growing on canal bed, water surface and water marks. They tend to thrive better in a range of 20° to 30° C. Weed growth is not possible in channels having high velocity of flow but when the channel has velocity less than 0.6 m per second weed growth is generally possible. The deposition of silt has no direct effect on weed growth, yet profuse weed growth is known to take place where silt is deposited. Light has a considerable effect on weed growth. Weed growth is accelerated in presence of light.

The *weed growth* can be checked by passing higher velocity than regime velocity in the channel. This will keep the silt in suspension and will make water turbid. Thus the light rays are cut off and silt is not deposited on channel bed.

Yet another way of weed control is *rush rotation*. In the process of rush rotation, the channel is run with full supply discharge for some time and then it is left completely dry for some time. This helps in excluding more light when higher depth is flowing, thus reducing the weed growth. During closure weed is unable to resist scorching rays of sun. Long duration closure has killing effect on the weed growth.

Weed removal may done by plucking them by hand and burning them when canal is dry.

15.13. MAINTENANCE OF SERVICE ROADS

The canal service road is unmetalled and, therefore, in monsoon season grass and small bushes grow on the road surface. The surface of road also wears and tears off due to some traffic over it. Sometimes unauthorised traffic also pass over canal roads which make the conditions still bad. The maintenance of service roads, therefore, consists of :

(*i*) Removal of grass and small bushes. (*ii*) Levelling of road surface.

(*iii*) Ramming and watering of top wearing surface.

Maintenance is usually carried out after the monsoon season. To check unauthorised traffic of cart, sometimes a check barrier is constructed at every crossing. The check barrier consists of a small earth mound with a slope of 1 : 4 on upstream and down-stream side with some top width. Jeeps can easily be crossed over these mounds but for trucks and bullock–carts it becomes a real barrier.

7. *Prevention of seepage from water reservoir.* Adequate and suitably designed toe filters are provided so that seepage ultimately finds its way into the natural stream.

8. *Depletion of ground water storage by pumping.* The surplus groundwater which causes undesirable rise in the water-table can be pumped out by :

(*a*) *Shallow well pumping.* Water is pumped from top aquifers to depress the water-table. This water may be utilized for irrigation in some other area.

(*b*) *Deep well pumping.* The water is pumped out from several water bearing strata by a series of wells scattered over large areas and discharge is used for further irrigation.

In area where the danger of waterlogging has become imminent, further canal irrigation should not be introduced. Instead, tube wells should be sunk and the area should be irrigated by tube wells.

Irrigation from masonary wells also reduces waterlogging.

9. *Changes in crop pattern.* A change in crop pattern may minimise the damage to plant line.

10. *Adoption of sprinkler method for irrigation.* This reduces the percolation losses from watercourses as only predetermined amount of water is applied to the land.

16.5. LOSSES IN CANAL

The losses in canal comprise evaporation from the surface and seepage through the bed and sides of the drains.

Loss due to evaporation from a canal system depends upon the climatic conditions of the region and hence it can never be prevented. However, losses by evaporation forms a minor part, hardly 1 to 5% of seepage loss and hence, in most of the cases evaporation loss is not significant.

Loss due to seepage is the most significant as this forms the major portion of the loss of the canal water. The seepage loss depends mainly upon the following factors :

1. Position of subsoil water-table.
2. Porosity of soil and subsoil.
3. Extent of absorbing medium.
4. Design of canal cross-section.
   (*a*) Depth of water in canal : greater the depth, greater is the loss of water.
   (*b*) Velocity of water in the canal : the loss decreases with increase in velocity.
5. Physical properties of canal water :
   (*a*) Temperature of water. The loss increases with increase in temperature of water.
   (*b*) Amount of silt carried in suspension. The loss decreases with an increase in amount of silt carried on suspension.
6. Conditions of canal system. The loss decreases with the age of canal and increases with the extent of absorbing medium.

The losses in canal are usually *measured* by a simple method known as *inflow and outflow method.* In the method a long reach of the canal is selected. Discharge

$$L = \frac{4\,k\,(b^2 - a^2)}{P_{av}\,L/8.64 \times 10^6}$$

or

$$L = \left[\frac{(8.64 \times 10^6)\,4\,k\,(b^2 - a^2)}{P_{av}}\right]^{1/2}$$

See Example 16.4 for illustration.

Drainage coefficient

Drainage coefficient is the rate at which water is removed by the drain. Its value depends upon the rainfall, but also varies with type of soil, type of crop, degree of surface drainage etc. It is expressed as the depth of water in cm (or m) to be removed in 24 hours from the drainage area. The common values of the drainage coefficient, suggested for humid regions by the U.S. Soil Conservation Service are : 1 to 2.5 cm/day for mineral soils and 1.25 to 10 cm per day for organic soils for different crops. Its value is so fixed that 1% of the average annual rainfall in that area is removed per day.

16.11. LINING OF IRRIGATION CHANNELS

1. Necessity

Lining of canal is necessary :

(*i*) to minimise the seepage losses in canal.

(*ii*) to increase the discharge in canal section by increasing the velocity.

(*iii*) to prevent erosion of bed and side due to high velocities.

(*iv*) to retard the growth of weeds, and

(*v*) to reduce maintenance of canal.

2. Advantages of Lining.

The main advantages derived by canal lining are mentioned below :

1. The lining of canals prevents seepage loss and thus more area can be irrigated by the water so saved. The cost of irrigation is, therefore, reduced.
2. The lining of canal is an important anti-water logging measure as it reduces seepage to the adjoining land.
3. The lining provides a smooth surface. The rugosity coefficient, therefore, decreases. The resistance to flow also decreases and hence the velocity of flow in the lined canal increases.
4. The increased velocity minimises the losses due to evaporation.
5. The increased velocity helps to provide a narrow cross-section for lined channels.
6. Higher velocity helps in providing a flatter hydraulic gradient or bed slope. Thus better command can be obtained.
7. Higher velocity prevents silting of channel.
8. Lining makes the banks more stable in light textured soil.
9. Lining reduces maintenance costs and possibility of breaching due to increased stability of section.
10. Lining of a canal prevents or reduces weed growth.

11. Lining of a canal increases available head for power generation as a flatter gradient can be provided.
12. Canal lining assures economical water distribution.
13. Canal lining prevents water to come in contact with harmful salts during transit.

3. **Disadvantages of Canal Lining**

The canal lining has certain disadvantages although the advantages far out-weigh the meagre disadvantages. The disadvantages of canal lining are mentioned below:

1. Canal lining requires a heavy initial investment.
2. Lining being permanent, it is difficult to shift the outlets very often.
3. It is very difficult to repair the damaged lining.
4. A lined channel section is without a berm. The additional safety provided by the berm for vehicular and pedestrian traffic is, therefore, absent in a lined channel.

4. **Suitability of Canal Lining Material**

A canal lining material, to be suitable, should have the following properties :

1. The material used for lining should provide complete *water tightness.*
2. The material used should have low coefficient of rugosity so as to make the section *hydraulically more efficient.*
3. The material chosen for canal lining should be *strong and durable.*
4. The lining should not have a very *high initial cost.* Subsequent maintenance cost of canal lining should be *very low.*
5. The material used should be able to *resist growth of weeds and attack of burrowing animals.*
6. The material used should be unaffected by *tramping of cattles.*
7. The material should withstand high velocity.
8. The material should permit construction of required slope easily.

16.12. TYPES OF LINING

The following are the important types of concrete lining used in India:

(*a*) ***Hard surface type lining*** :

1. Cement concrete lining.
2. Shotcrete lining.
3. Precast concrete lining.
4. Cement mortar lining.
5. Brick lining.
6. Stone blocks, or undressed stone lining.
7. Asphaltic lining.

(*b*) ***Earth type lining*** :

8. Soil cement lining.
9. Clay puddle lining.
10. Sodium carbonate lining.

(*c*) ***Buried and protected membrane type lining***:

11. Prefabricated light membrane lining.
12. Bentonite soil and clay membrane lining.
13. Road oil lining.

17

Canal Outlets

17.1. INTRODUCTION

An outlet is a small structure which admits water from the distributing channel to a water course or field channel. Thus, an outlet is a sort of head regulator for the field channel delivering water to the irrigation fields. The responsibility of maintenance of the distributing channel and the whole canal network lies with Government, while that of the field channel lies with the farmer. The outlet is the connecting medium for the two.

Since the efficiency of a canal network mostly depends upon functioning of the outlets and other regulation works, a canal *outlet should fulfill the following requirements*:

1. The outlet should be strong, with no moving parts liable to be damaged or requiring periodic attention and maintenance.
2. It should be so designed that the farmer cannot temper with its functioning and any interference from him should be easily detectable.
3. Since a large number of outlets are fixed on a distributing channel, the most essential requirement is that it should be cheap.
4. The design should be simple so that it can be constructed or fabricated by local masons or technicians.
5. It should be possible for the outlet to work efficiently with a small working head.
6. The outlet should draw its fair share of silt carried by the distributing channel.
7. From the farmer's point of view, the outlet should give a fairly constant discharge. However, from the canal regulation point of view, the outlet should draw proportionately more or less discharge with the varying supply in the distribution channel.

17.2. TYPES OF OUTLETS

Outlets may be classified under the following there heads :

1. Non-modular Outlet
2. Semi-module or Flexible Module
3. Rigid Module.

Non-modular Outlet : A non-modular outlet is the one in which the discharge depends upon the difference in level between the water levels in the distributing channel and the water course. The discharge through such an outlet varies in wide limits with the fluctuations of the water levels in the distributing and the field channels. The common examples under this category are : submerged pipe outlet, masonry sluice and orifices, and wooden shoots.

Semi-module or Flexible outlet. A flexible outlet or semi-module is the one in which the discharge is affected by the fluctuations in the water level of the distributing channel while the fluctuations in the water levels of the field channel do not have any effect on its discharge. The various outlets in common use that fall under this category are pipe outlet, Kennedy's gauge outlet, Crump's open flume outlet and pipe-cum open flume outlet.

Rigid Module. A rigid module is the one which maintains constant discharge, within limits, irrespective of the fluctuations in water levels in the distributing channel and/or field channel. The most common outlet that falls under this category is the Gibb's rigid module.

17.3. IMPORTANT DEFINITIONS

To understand the criteria for judging the behaviour and function of outlets, the following definitions are useful :

1. Flexibility : It is ratio of rate of change of discharge of an outlet to the rate of change of the discharge of the distributing channel.

Thus, $$F = \frac{dq}{q} \Big/ \frac{dQ}{Q} \qquad \text{...(17.1)}$$

where F = Flexibility
q = Discharge through the outlet
Q = Discharge of the distributing channel.

Now, for the field channel.

$$q = kH^m$$

where k = constant, m = outlet index
and H = head acting on the outlet.

$$\therefore \quad dq = mk\,H^{m-1}\,dH$$

$$\therefore \quad \frac{dq}{q} = \frac{mk\,H^{m-1}\,dH}{k\,H^m} = m\frac{dH}{H} \qquad \text{...(1)}$$

Similarly, for the parent channel.

$$Q = CD^n$$

where C = constant and n = canal index
D = depth of water in the canal.

$$\therefore \quad dQ = n\,CD^{n-1}\,dD$$

$$\therefore \quad \frac{dQ}{Q} = \frac{n\,CD^{n-1}\,dD}{CD^n} = n\frac{dD}{D} \qquad \text{...(2)}$$

$$\therefore \quad F = \frac{dq}{q} \Big/ n\frac{dD}{D} \qquad ...(2)$$

Comparing (1) and (2), we get

$$S = nF \qquad ...(17.7)$$

It is evident from Eq. 17.7 that the sensitivity of a rigid module is zero.

7. Efficiency

Efficiency is the measure of a conservation of head by an outlet. It is defined as the ratio of the head recovered to the head put in. Less is the working head required for the functioning of the outlet, more will be its efficiency.

8. Drowning ratio

It is ratio between the depths of water level over crest on the downstream and upstream of the module.

9. Minimum modular head

The minimum modular head or the *minimum modular loss* is the minimum loss of head or the difference between the upstream and downstream water levels which is absolutely necessary to be maintained to enable the module to pass its design discharge.

10. Modular limits and modular range

The *modular limits* of an outlet are the upper and the lower limits of any one or more factors beyond which an outlet is incapable of acting as a module or semi-module. *Modular range* is the range between the modular limits. It is the range of various factors which a module or semi-module works as designed.

17.4. NON-MODULAR OUTLETS : PIPE OUTLET

The most common types of non-modular outlets are : submerged pipe outlet, masonry sluice and orifices, and wooden shoots. Fig. 17.1 shows a pipe outlet. The pipes vary from 10 to 30 cm in diameter, and are frequently laid on a light concrete foundation to prevent uneven settlement and consequent leakage. They are generally fixed horizontally at right angles to the direction of flow.

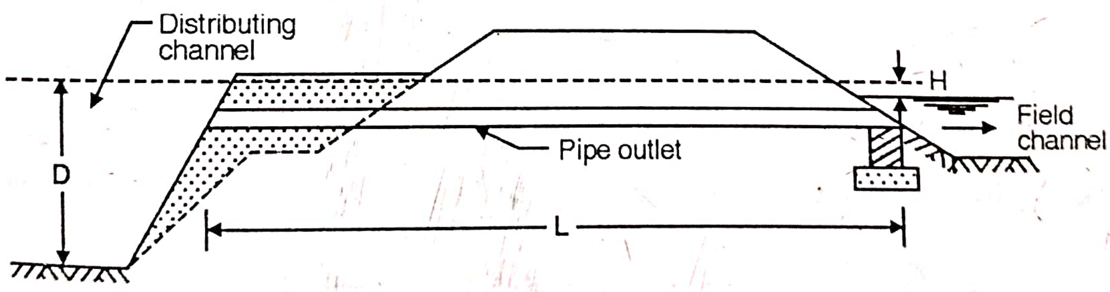


FIG. 17.1. PIPE OUTLET

The loss of head, H through the outlet is given by

$$H = (\text{Entry loss}) + (\text{Frictional loss}) + (\text{Velocity head at the exit})$$

$$H = 0.5\frac{v^2}{2g} + \frac{4fLv^2}{2gd} + \frac{v^2}{2g} \qquad ...(17.8\ a)$$

$$\left(\frac{1.5\,d}{400\,f}+L\right)f \Big] \qquad 20\left[\left(\frac{1.5\times 22.8}{400\times 0.01}+9\right)0.0\ldots\right]$$

[N]ext trial, taking $C = 0.57$, we get

$$0.04 = 0.57\,A\sqrt{2\times 9.81\times 0.1}, \text{ from which } A = 0.05 \text{ m}^2$$

$$d = \sqrt{\frac{4\times 0.05}{\pi}} \simeq 0.25 \text{ m}$$

Using this diameter and substituting in Eq. 17.9 (b), we get $C = 0.583$

$$\therefore \quad 0.04 = 0.583\,A\sqrt{2\times 9.81\times 0.1} \text{ from which } A = 0.049 \text{ m}^2$$

$$\therefore \quad d = \sqrt{\frac{4\times 0.049}{\pi}} \simeq 0.25 \text{ m} = 25 \text{ cm}$$

Hence provide a pipe of **25 cm** dia.

17.5. SEMI-MODULE OR FLEXIBLE OUTLETS

In the case of a flexible outlet, the discharge is affected by the change in the water level of the distributing channel but not with the change in the water level of the field channel. The following are the common types of flexible outlets :

(*i*) Pipe outlet discharging freely in the atmosphere,
(*ii*) Kennedy's gauge outlet.
(*iii*) Open flume outlet
(*iv*) Orifice semi-modules.

(1) Pipe outlet discharging freely

If the pipe outlet is so set that it discharges freely in the atmosphere, the discharge through it becomes independent of the water level in the field channel, and hence it acts as semi-module. The discharge cannot be increased by the cultivator by digging the water course. Such outlets, therefore, worked well. Later, however, the farmers invented a method of increasing the discharge by constructing a ramp in the water-course thereby heading up the water to the top level of the pipe and drowning it.

For the outlet to be proportional, $F = 1$. Hence $\frac{H}{D} = \frac{m}{n}$. Taking $n = \frac{5}{3}$ and $m = \frac{1}{2}$, the setting is equal to 0.3. Usually a pipe outlet is set lower than this and is sub-proportional. The efficiency of the pipe outlet is high and its silt conduction is good.

(2) Kennedy's gauge outlet :

This was the earliest type of semi module, invented by R.G. Kennedy, Chief Engineer, Irrigation, Punjab, sometime in 1906. This outlet is made of cast iron and consists of three main parts (Fig. 17.4):

1. An orifice with bellmouth entry.
2. A long expanding delivery pipe.
3. An intervening vertical air column above the throat.

Now $$q = C\,B_t\,H^{3/2}$$

or $$0.06 = 1.6 \times 0.07\,H^{3/2}$$

From which $$H = 0.66 \text{ m}$$

Now minimum modular head required for open flume outlets

$$= 0.2\,H = 0.2 \times 0.66 = 0.132 \text{ m} = 13.2 \text{ cm}$$

This is less than the available working head of 15 cm. Hence the design is satisfactory.

$$\text{Setting of outlet} = \frac{H}{D} = \frac{0.66}{0.85} = 0.776 < 0.9$$

Hence the out let will work as hyper-proportional outlet.

17.6. RIGID MODULE

The three common types of rigid modules, having no moving parts are :

1. Gibb's rigid module.
2. Khanna's rigid module.
3. Foote module.

Gibb's module is described below.

Gibb's module

The outlet was designed by A.S. Gibb's, formerly Executive Engineer, Punjab Irrigation. Fig. 17.9 shows the plan and the section of the module. The essential feature of the outlet is an eddy chamber, semi-circular in plan, round which water flows giving rise to a free vortex flow. Water enters through an inlet pipe having bell mouth entry and is directed to the eddy chamber through a 180° rising pipe in which free vortex flow is developed. The characteristic feature of the free vortex flow is that the product of the velocity and radius is constant for all filaments (i.e. $V \times r =$ constant). Thus the water at the outer circumference of the chamber has greater radius and hence lesser velocity, resulting in rise of water level there. Thus the water surface in the eddy chamber slopes down towards the inner circumference. A series of baffles are suspended from the roof of the eddy chamber, with their lower edges sloping at the required height above the sill of the module. If the head causing flow increases, water banks up at the outer circumference of the eddy chamber and impinges against the baffles imparting an upward, rotational, direction of flow to the water, which spins round in the compartment between two successive baffles and finally drops on the oncoming stream of water, thus dissipating excess energy. This keeps discharge constant for a wide range of variation in the head. The number of baffles coming into action depends upon the variations in the head causing flow. The angle of eddy chamber varies from a semi-circle to $1\frac{1}{2}$ complete turns depending upon the discharge and the range of working required for the module.

Gibb give the following formula for the discharge through the outlet :

$$q = r_0\sqrt{2g}\,(d_i + h_0)^{1.5}\left\{\frac{m^2-1}{m^3}\log_e m + \frac{1}{m}\log_e m - \frac{m^2-1}{2\,m^2}\right\} \qquad \ldots(17.14)$$

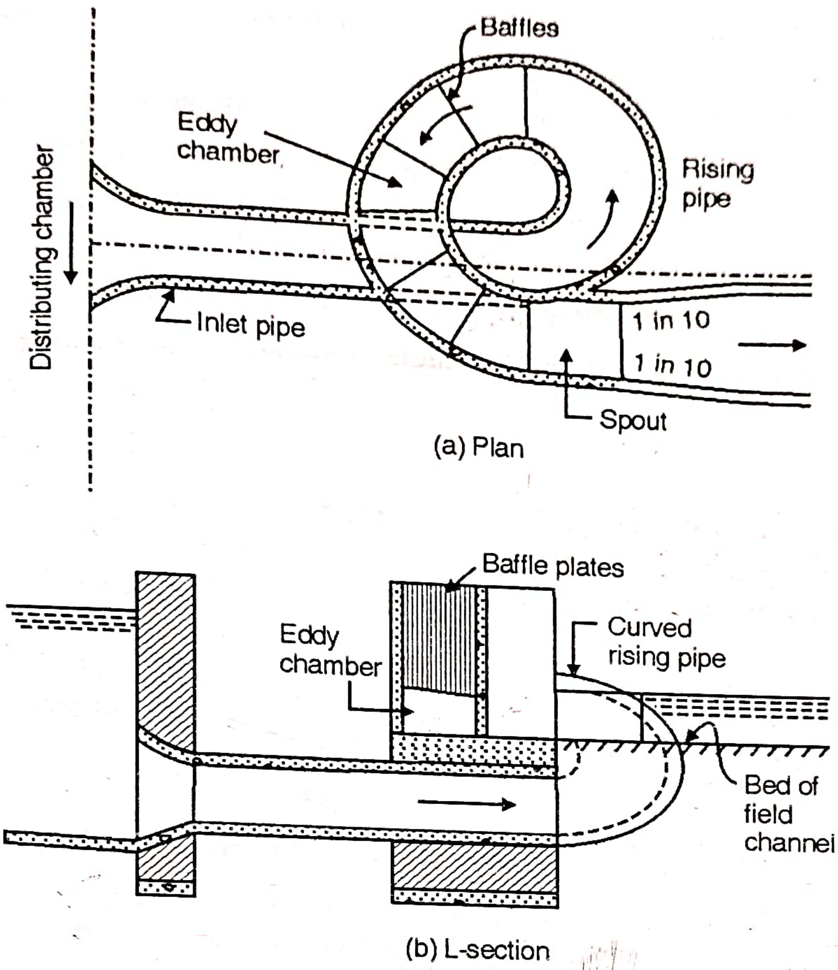


FIG. 17.9. GIBB'S MODULE

where r_0 = radius of outer semi-circle

r_i = radius of inner semi-circle.

$$m = \frac{r_0}{r_i}$$

d_i = depth of water at inner circumference.

h_0 = head at outer circumference.

Gibb's formula is valid only for his standard design in which $m = 2$ and $\frac{h_0}{D} = \frac{1}{7}$.

PROBLEMS

1. What is an outlet ? Write down the requirements that an outlet should fulfill.
2. What do you understand by flexibility of an outlet ? Derive an expression for the same.
3. Define proportionality of an outlet. Distinguish between a hyper-proportional outlet and a sub-proportional outlet. Find out expressions for the setting of both the types.

18

Canal Regulation Works

18.1. INTRODUCTION

Any structure constructed to regulate the discharge, full supply level or velocity in 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 categorised as under :

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

(A) CANAL FALLS

18.2. NECESSITY AND LOCATION OF FALLS

A fall is an irrigation structure constructed across a canal to lower down its water level and destroy the surplus energy liberated from the falling water which may otherwise scour the bed and banks of the canal.

We have seen in Chapter 14 that the canal requires a certain slope, depending upon the discharge, to overcome the frictional losses. This slope may vary from 1 in 4000 for a discharge of about 1.5 cumecs to about 1 in 8000 for a discharge of 3000 cumecs. This slope is, therefore, quite flat in comparison to the available ground slope of an average value of 5 to 20 cm per kilometre length (*i.e.*, 1 in 200 to 1 in 50). Thus the ground slope in nature is always very much steeper than the design bed slope of irrigation canal, based on the silt theories. If an irrigation canal, taking off from its head, is in cutting, it will soon meet with condition when it will be entirely in embankment.

It has been stated earlier in Chapter 15 that if the canal is in embankment, the cost of construction and maintenance is *very high* and at the same time the percolation and seepage losses are excessive. Also, there is always a danger of the adjacent area being flooded if some cut or breach takes place in the canal banks. Hence, the canal should never be in high embankment. However, the divergence between the gentle bed slope of canal and the steep ground slope throws the canal in embankment after a certain distance though it started in cutting at its head. To overcome this difficulty, *falls* are introduced at appropriate places, and the water surface of the canal is lowered. Arrangements are made to dissipate the excess energy liberated from the falling water.

The location of a fall is decided from the following considerations :

1. For the canal which does not irrigate the area directly, the fall should be located from the considerations of economy in cost of excavation of the channel with regard to balancing depth and the cost of the falls itself.

2. For a canal irrigating the area directly, a fall may be provided at a location where the F.S.L. outstrips the ground level,. but before the bed of the canal comes into filling. After the drop, the F.S.L of the canal may be below the ground level for $\frac{1}{2}$ to $\frac{1}{4}$ kilometer.

3. The location of the fall may also be decided from the consideration of the possibility of combining it with a regulator or a bridge or any other masonry work.

4. A relative economy of providing large number of small falls v/s small number of big falls should be worked out. The provision of small number of big falls results in unbalanced earth-work, but there is always some saving in the cost of the fall structure.

18.3. DEVELOPMENT OF FALLS

The ancient people always tried to avoid falls by aligning canals along zig-zag route in order to increase the length of the canal and thus dissipate the excess energy head in friction. The Eastern Yamuna Canal constructed by Mughal Emperors had no falls, and the canal, followed a sinuous path. The falls were first constructed by the British in India in the nineteenth century. The development of falls, since then, took place gradually. Among the earlier type of falls are : Ogee falls, rapids and stepped falls. Later, notch falls, vertical falls and glacis type falls were developed.

1. Ogee fall

The Ogee fall was first constructed by Sir Proby Cautley on the Ganga Canal (Fig. 18.1). This type of fall has gradual convex and concave curves, with an aim to provide a smooth transition and to reduce disturbance and impact. This preserved the energy (without dissipating it). Due to this, the Ogee fall had the following defects :

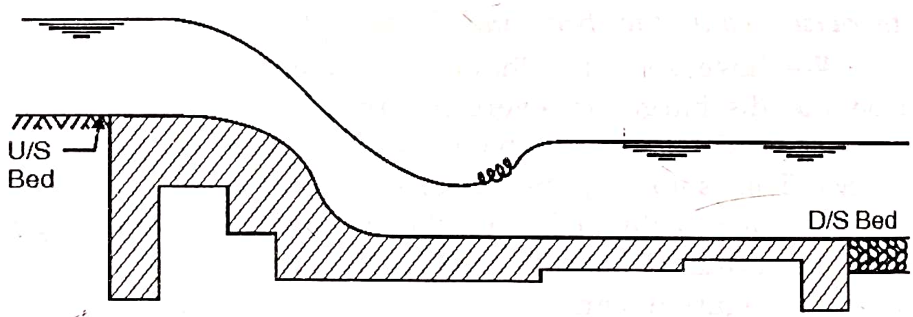


FIG. 18.1. OGEE FALL.

(i) There was considerable draw down effect on the u/s resulting is bed erosion.

(ii) Due to smooth transition, the kinetic energy was preserved till sufficient depth was scoured out below the fall to ensure the formation of the hydraulic jump.

2. Rapid fall

Fig. 18.2 shows a *rapid fall.* Such falls were provided on Western Yamuna Canal and were designed by Lieut. R.F. Croften. Such a fall consists of a glacis sloping at 1 vertical to 10 to 20 horizontal. The long glacis assured the formation of hydraulic jump. The gentle slope admitted timber traffic. Hence, the fall worked admirably. However, there was very high cost of construction.

glacis is checked for the worst condition of the jump trough at different discharge intensities, as explained in Chapter 12 (design of sloping glacis weir).

18.9. INGLIS TYPE FALL

Designed by C.C. Inglis, the fall makes use of horizontal impact for energy dissipation. The design consists of a standard long throated weir flume followed by a glacis slope and a pavement on which a baffle is fixed to dissipate the energy. The baffle holds the jump stable on a horizontal platform. A cistern downstream of the baffle with a deflector at the d/s end of the cistern is provided. Pucca or impervious floor is provided only upto the end of the deflector. To the d/s of the deflector a second cistern in provided, which is only pitched with bricks or stones. (Fig. 18.15)

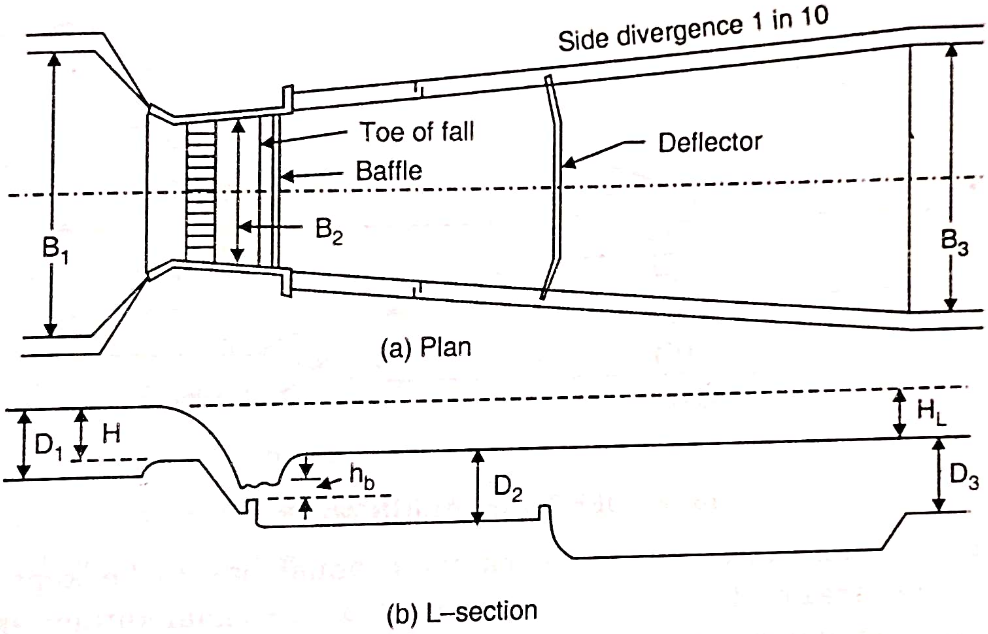


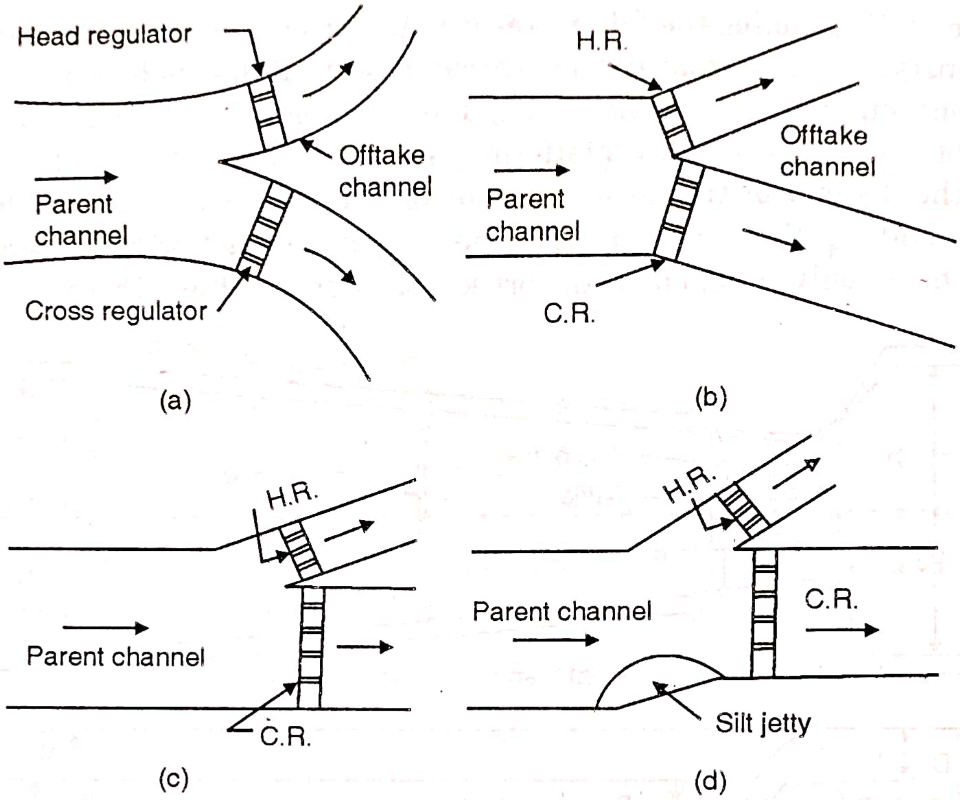
FIG. 18.15. INGLIS TYPE FALL.

The maximum dissipation of energy by a hydraulic jump occurs when the jump forms at the toe of the glacis. However, the jump may also form either on the glacis, or at the d/s of the toe. If the jump forms on the sloping glacis, the baffle should be fixed on a platform at a higher level than the canal bed level so that the natural wave will form at the toe of the platform. If the jump would form at the toe of the fall, the baffle should be fixed at pavement level. If, however, the jump would form at d/s of toe, the glacis should be extended and a cistern provided of such depth as to bring the wave to the toe and the baffle fixed on the bed of the cistern. Even though the energy is effectively dissipated by baffle near the toe of a fall, the distribution of velocities is normal ; hence, a cistern and deflector are provided. The cistern and deflector are meant to restore the normal distribution of velocities in the channel.

(B) CANAL REGULATORS

18.10. OFF-TAKE ALIGNMENT

When a distributing channel takes off from the parent channel, their off-take alignment should be very carefully designed. The best alignment of an off-take is when it makes zero angle with the parent channel, as shown in Fig 18.16(a). A transition will have to be designed. If the transition curves are not given, both channels (*i.e.* parent channel and the distributing channel) should make an angle with the alignment of the parent channel upstream of the off-take Fig.18.16(*b*).



18.16. OFF-TAKE ALIGNMENT

Under the circumstances when the parent channel has to be kept straight both upstream and downstream of the off-take, the edge of the canal rather than the centre line should be considered in deciding the angle of off-take [Fig.18.16(c)]. In that circumstance, the section should not be narrowed down equally on both sides. An unbalanced off-take results in the formation of a jetty [Fig.18.16(*d*)]. The reduction in sectional area caused by the jetty would be made up by the scouring of the bed along the line of the deviated current.

18.11. 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 d/s 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 u/s and feeds the off-take channel in rotation.

3. It helps in closing the supply to the d/s 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. Incidently, bridges and other communication works can be combined with it.

18.12. DESIGN OF CROSS-REGULATOR AND DISTRIBUTARY HEAD REGULATOR

1. Design of crest

The discharge is determined by the drowned weir formula :

$$Q = \frac{2}{3} C_1 L \sqrt{2g}\ [(h + h_a)^{3/2} - h_a^{3/2}] + C_2 L d \sqrt{2g\,(h + h_a)} \qquad \text{...(18.24)}$$

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.18.24. 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 coditions, 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.

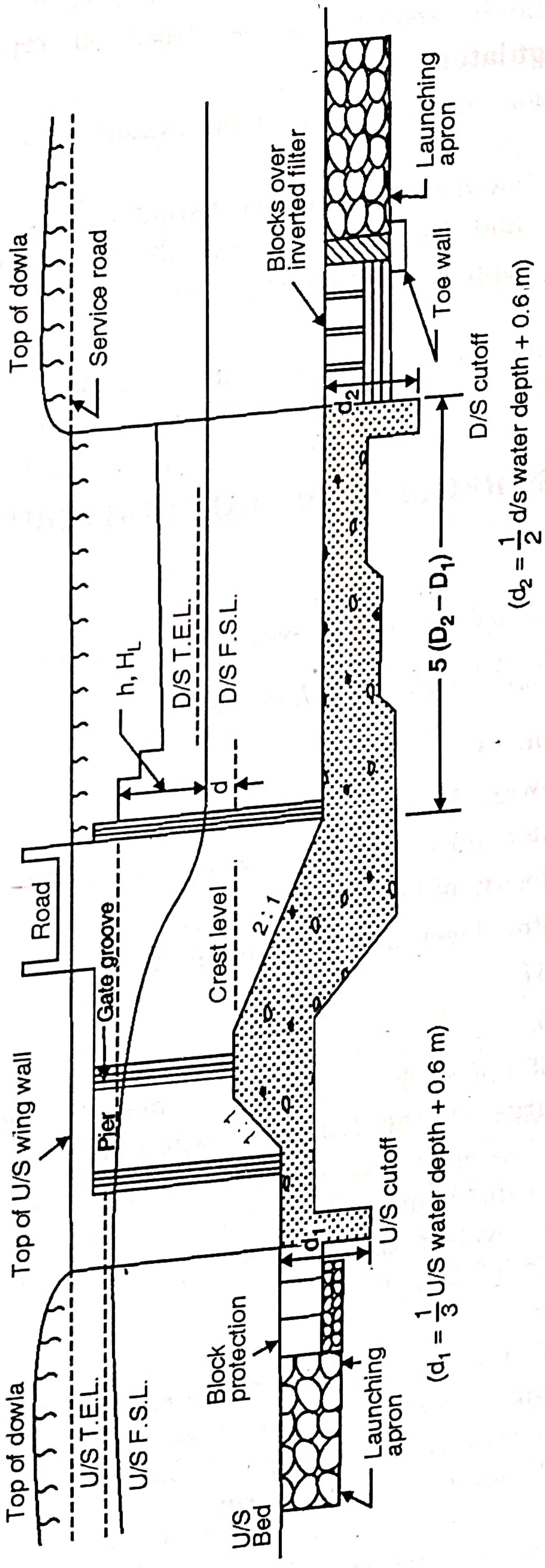


FIG. 18.17. DISTRIBUTARY HEAD REGULATOR.

(*v*) Thickness of floor at 6 m from the toe of the glacis :

$$\% \text{ pressure } = 28.3 + \frac{75-28.3}{16} \times 4.5 = 41.4\ \%$$

$$\therefore \quad \text{Thickness} = \frac{0.414 \times 2.6}{2.24-1} = 0.84 \text{ m}$$

∴ Provide a thickness of 0.90 m for the next 2.5 m.

(*vi*) *Thickness at 8.5 m from the toe*

$$\% \text{ pressure } = 28.3 + \frac{75-28.3}{16} \times 2 = 34.1\ \%$$

$$\therefore \quad \text{Thickness} = \frac{0.341 \times 2.6}{2.24-1} = 0.71 \text{ m.}$$

Hence, provide 0.7 m thick floor for its 2 m length.

Step 4. Design of u/s protection

$$\text{U/s scour depth} = \frac{1}{3} \times 2.5 + 0.6 \triangleq 1.5 \text{ m}$$

Hence, provide the same protection as in the case of cross regulator.

Step 5. Design of d/s protection

$$\text{D/s scour depth} = \frac{1}{2} \text{ d/s water depth} + 0.6 \text{ m}$$

$$d_2 = \frac{1}{2} \times 1.5 + 0.6 = 1.35 \text{ m}$$

(*i*) *Inverted filter*

$$\text{Volume} = d_2 = 1.35 \text{ m}^3/\text{m}$$

Provide 0.5 m thick cement concrete blocks over 0.5 m thick graded filter.

$$\text{Length required} = \frac{1.35}{1} = 1.35 \text{ m.}$$

However, provide two rows of 0.8 m × 0.8 m × 0.5 m thick concrete blocks over 0.5 thick graded filter.

(*ii*) *Launching apron*

$$\text{Volume} = 2.25\, d_2 = 2.25 \times 1.35 = 3.04 \text{ m}^3/\text{m.}$$

Provide 1 m thick launching apron for a length of 3.5 m. Provide a masonry toe wall 0.4 m wide and 1.20 m deep between the filter and the launching apron.

The design details have been shown in Fig. 18.18(*b*).

18.13. CANAL ESCAPES

A canal escape is a structure constructed on an irrigation canal for the purpose of wasting some of its water. Depending upon the purpose, there can be three types of escapes :

(1) Canal scouring escape,

(2) Surplus escape,

and (3) Tail escape.

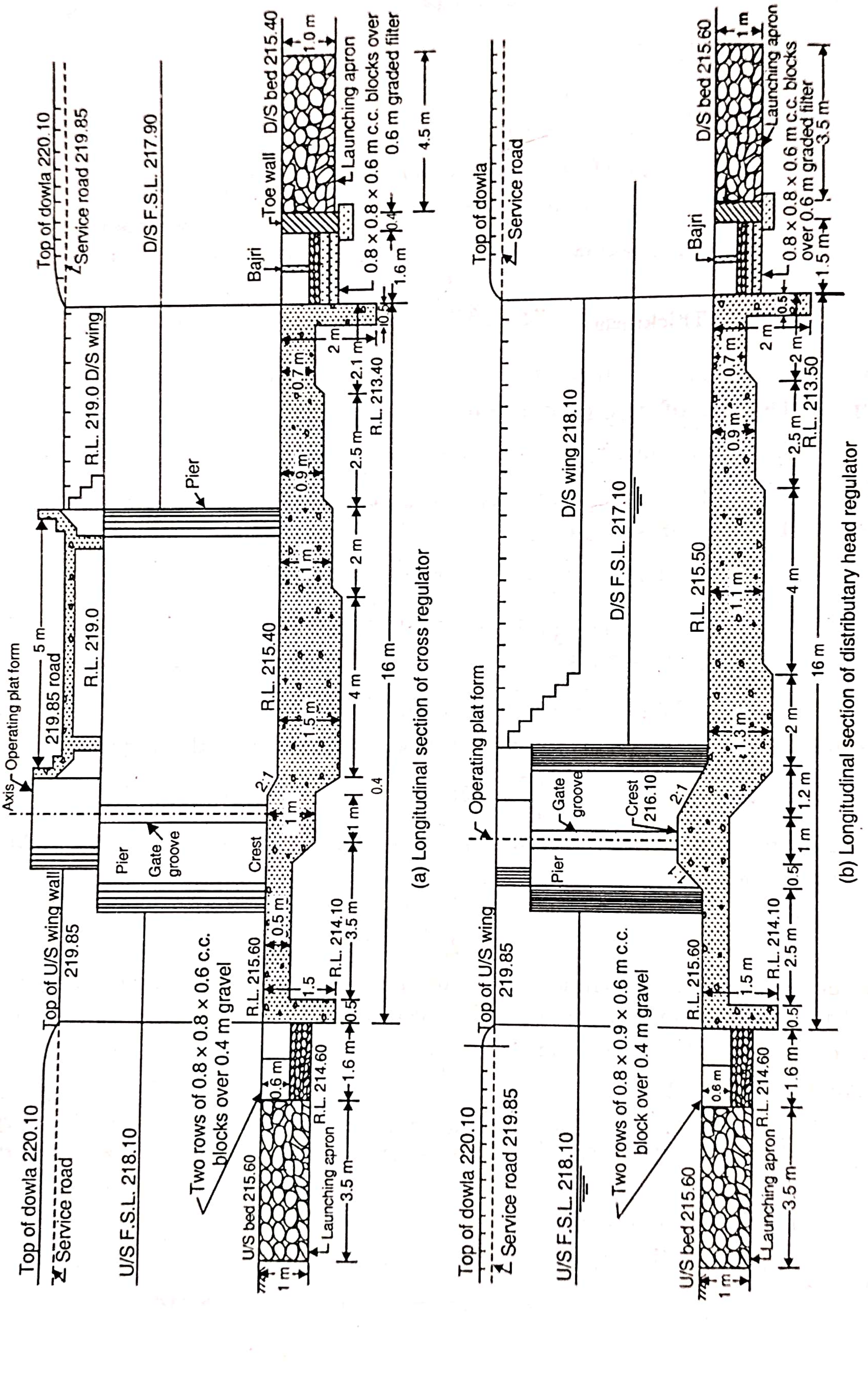
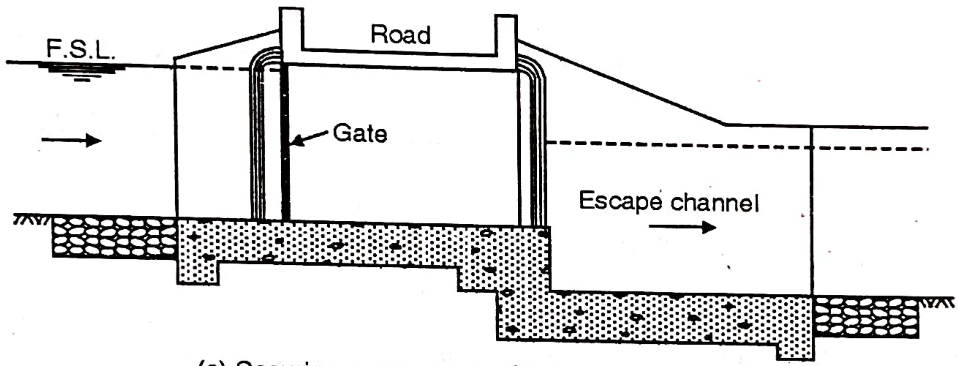


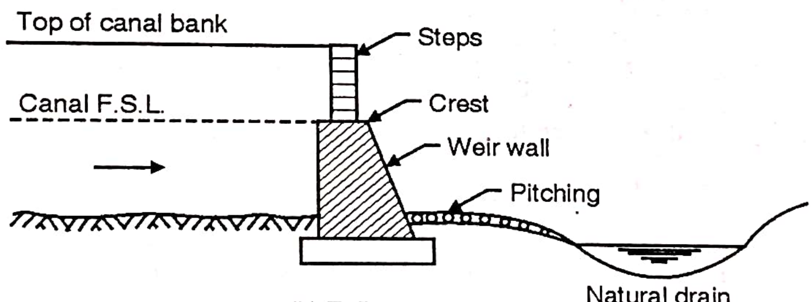
FIG. 18.18. CROSS-REGULATOR AND DISTRIBUTARY HEAD REGULATOR

The scouring escape is constructed for the purpose of scouring off excess silt from time to time. Escapes are also constructed to dispose off excess supplies of the parent channel. Excess supplies in the canal take place either during heavy rains or due to the closure of canal outlet by the farmers. In that case, the escapes save the d/s section of the canal from overflow of banks.

Escapes are thus essential safety valves for the canals, and should be constructed at intervals. The canal leading the surplus water to a natural drain is known as *escape channel.* The capacity of escape channel should not be less than 50% of capacity of the parent channel at that point. A canal surplus escape may be weir type, with the crest of weir wall at F.S.L. of parent canal bed level [Fig.18.19(*b*)]. A tail escape [Fig.18.19(*b*)] is provided at the tail end of the canal, and is useful in maintaining the required F.S.L. at the tail end. The structure is weir type with its crest level at the required F.S.L. of canal at its tail end.



(a) Scouring escape or surplus water escape



(b) Tail escape

FIG. 18.19. CANAL ESCAPE.

PROBLEMS

1. What do you understand by a fall in a canal ? Why it is necessary? How do you select its location ?
2. Write a note on Notch type fall.
3. What is 'cistern element' in fall ? Give various expressions for its dimensions.
4. Explain the procedure of designing Sarda type fall.

19

Cross Drainage Works

19.1. INTRODUCTION

A cross drainage work is a structure carrying the discharge of a natural stream across a canal intercepting the stream. When a canal is to be taken to the watershed, it crosses a number of natural streams in the distance between the reservoir to the watershed. Once the canal is taken to the watershed, no cross-drainage works are normally necessary except when the canal leaves the watershed for some distance forming a loop. However, when the canal is aligned as a contour canal, a number of cross-drainage works are necessary.

A cross-drainage work is generally a very costly item, and should be avoided as far as possible by (*i*) diverting one stream into another, or (*ii*) changing the alignment of the canal so that it crosses below the junction of two streams.

19.2. TYPES OF CROSS-DRAINAGE WORKS

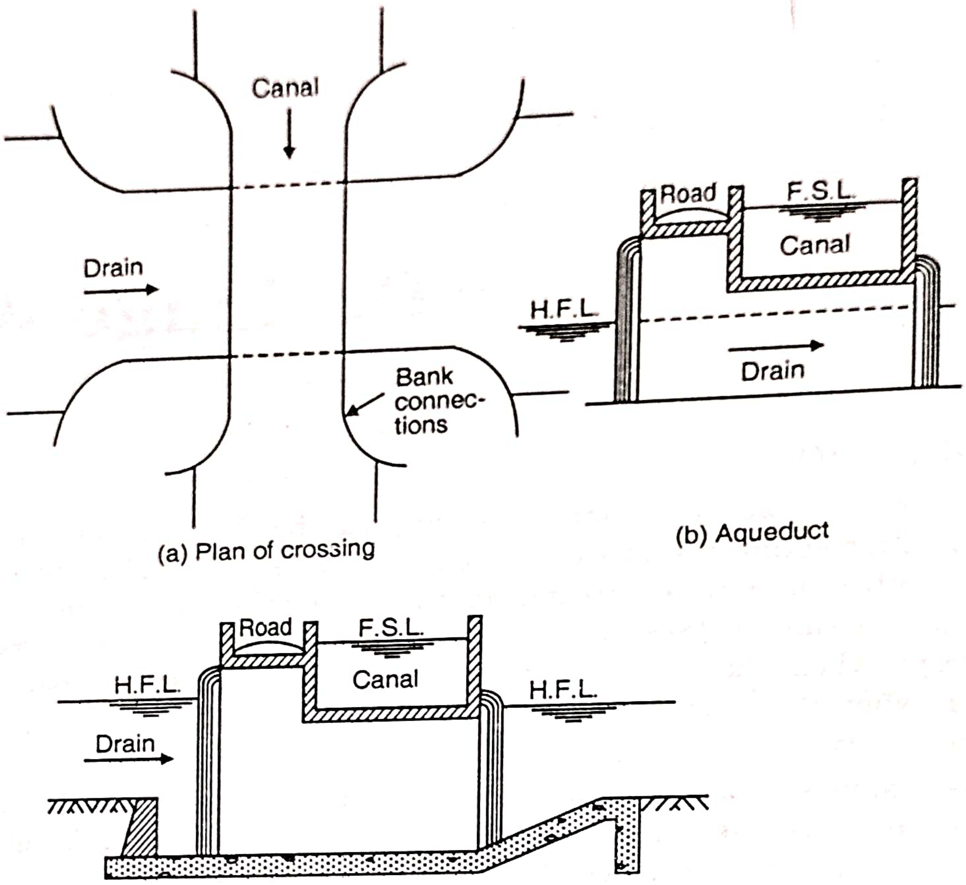
Depending upon the relative levels and discharges, cross-drainage works may be of the following types :

(I) **C.D. Works carrying canal over the drainage**

In this type of C.D.work, the canal is carried over the natural drain (Fig. 19.1). The advantage of such arrangement is that the canal, running perennially, is above the ground and is open to inspection. Also, the damage done by floods is rare. However, sometimes during heavy floods the foundation can be scoured, or the waterway of the drain may be chocked with trees etc. This is the usual type of work constructed when the drain is very big in comparison to the section of the canal. The structures that fall under this type are :

1. Aqueduct
2. Syphon aqueduct.

Fig. 19.1(*b*) and (*c*) show the aqueduct and syphon aqueduct respectively. As is clear from Fig. 19.1(*b*), the H.F.L. of the drain is much below the bottom of the canal trough in the case of aqueduct so that drainage water flows freely under gravity. However, in the case of a syphon aqueduct, the H.F.L. of the drain is much higher above the canal bed, and the water runs under syphonic action through the aqueduct barrels. In this case, the water surface level of the flood is depressed when it passes under the canal trough. The bed of the drainage is also lowered.



(a) Plan of crossing (b) Aqueduct

(c) Syphon aqueduct

FIG. 19.1. AQUEDUCT AND SYPHON AQUEDUCT.

(II) C.D. Works carrying drainage over the canal

In this type of C.D. work, drainage is carried over the canal (Fig. 19.2). The advantage of this type is that the C.D. works themselves are less liable to damage than the earth-work of the canal.

The major disadvantage of this work is that the perennial canal is not open to inspection. Also, if the silt is deposited in the barrels of the work, it is difficult to clear it out.

The structures that fall under this type are

1. Super-passage
2. Canal syphon.

Fig. 19.2(*b*) shows a super-passage. A super-passage is similar to an aqueduct, except that in this case the drain is over the canal. The F.S.L. of the canal is lower than the underside of the trough carrying drainage water. Thus, the canal water runs under gravity. Fig. 19.2(*c*) shows a canal syphon, or simply *syphon*. In this, case, the levels are such that the F.S.L. of the canal is much above the bed level of the drainage trough, so that the canal runs under syphonic action under the trough. The canal bed is lowered and a ramp is provided at the exit so that the trouble of silting is minimised.

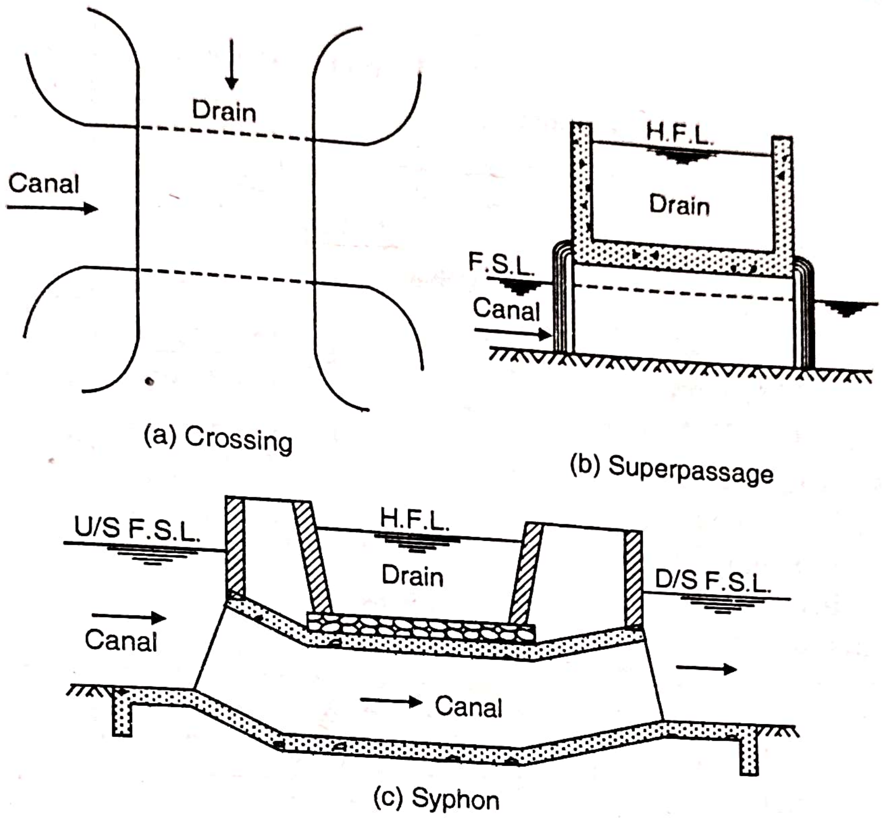


FIG. 19.2. SUPER-PASSAGE AND SYPHON.

(III) C.D. works admitting the drainage water into the canal

In this type of work, the canal water and the drainage water are permitted to intermingle with each other. The only advantage of this type of work is its low initial cost. Such type of works have the following disadvantages :

(*i*) Regulation of such a work is difficult and requires additional staff.

(*ii*) The canal has to be designed to carry the increased flood discharge of the drain.

(*iii*) The faulty regulation of gates may damage the canal.

(*iv*) There is additional expenditure of silt clearance.

Following are the structures under this type C.D. works :

1. Level crossing
2. Inlet and outlets.

Fig. 19.3(*a*) shows the schematic plan of a *level crossing* which is constructed in the circumstance when the beds of the canal and drainage are practically at the same level. In this type of work, the drainage water is passed into the canal and then taken out at the opposite bank. The work consists of (*i*) construction of a crest, with its top at the F.S.L. of the canal, at the u/s junction with the canal, (*ii*) provision of the head regulator across the drainage at its d/s junction with the canal, and (*iii*) a cross-regulator across the canal at its d/s junction with the drainage. When the drainage does not carry any water, its regulator is closed while the cross-regulator of the canal is kept fully open so that the canal flows without any interruption. During the floods, however, the drainage regulator is opened so that the flood discharge, after spilling over the crest and mixing with the canal water, passes through it to the downstream of the drainage. The accurate supplies in the canal are maintained by a cross-regulator. Level crossings are suitable for canals of all sizes. They are, however, specially suited to crossings of very large drains when the cost of other cross-drainage works would be very high.

A *canal inlet* [Fig. 19.3*b*] is constructed when cross-drainage flow is small, and its water may be absorbed into the canal without causing appreciable rise. However, if the canal is small, an *outlet* may be constructed to pass out the additional discharge which has entered the canal. It is not necessary that the number of inlets and outlets should be the same. There may be one outlet for two or three inlets.

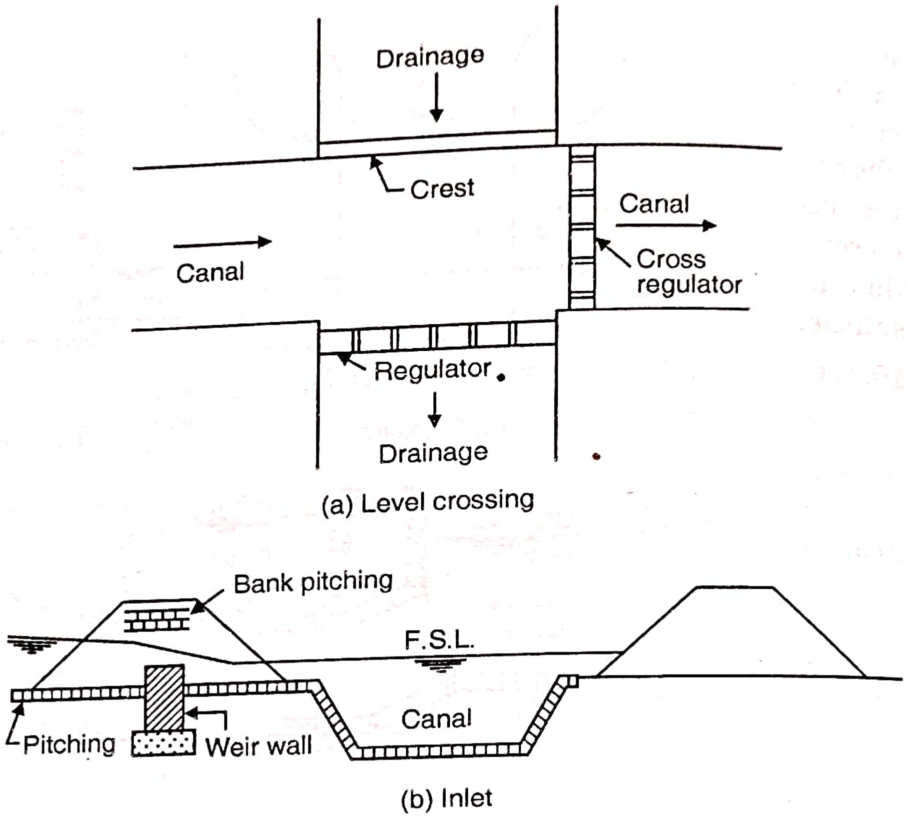


FIG. 19.3. LEVEL CROSSING AND INLET.

19.3. SELECTION OF SUITABLE TYPE OF CROSS-DRAINAGE WORK

The factors which affect the selection of the suitable type of cross-drainage works are (*i*) relative bed levels and water levels of the canal and the drainge, and (*ii*) size of the canal and the drainage. The following considerations are important :

1. When the bed level of the canal is much above the H.F.L. of the drainage, so that sufficient headway is available for floating rubbish etc., and also for the structural elements of the work, an aqueduct is the obvious choice. Similarly, if the bed level of the drain is well above the F.S.L. of the canal, super-passage is provided.

2. The necessary headway between the canal bed level and the drainage H.F.L. can be increased by shifting the crossing to the d/s of the drainage. If, however, it is not possible to change the canal alignment, or if such a shifting does not give sufficient headway between the two levels, a syphon aqueduct may be provided. Thus, in the case of sypon aqueduct, the H.F.L. of the drain is above the bed of the canal.

3. When the canal bed level is much lower, but the F.S.L. of the canal is higher than the bed level of the drainage, a canal syphon is preferred.

4. When the drainage and the canal cross each other practically at the same level, a level crossing may be preferred. This type of work is avoided as far as possible.

The considerations governing the choice between aqueduct and syphon aqueduct (or super-passage and a syphon) are : (*i*) suitable canal alignment, (*ii*) suitable soil available for bank connections and (*iii*) nature of available foundation. As indicated earlier, the relative difference between the bed level of the canal and H.F.L. of the drainage can be suitably altered by changing the canal alignment so that the point of crossing is shifted upstream or downstrem of the drainage. For example, if the canal alignment is such that headway is not available between the H.F.L. of the drain and the bed of the canal, a syphon aqueduct is to be constructed at the crossing. But, however, if other conditions are not favourable for the construction of the syphon aqueduct, the canal alignment may be changed so that the crossing is shifted to the d/s and sufficient headway required for the construction of an aqueduct is available.

19.4. CLASSIFICATION OF AQUEDUCTS AND SYPHON AQUEDUCTS

Depending upon the nature of the sides of the aqueduct (or syphon aqueduct) it may be classified under three heads :

(*i*) Sides of the aqueduct in earthen banks, with complete earthen slopes [Fig. 19.4 (*b*)].

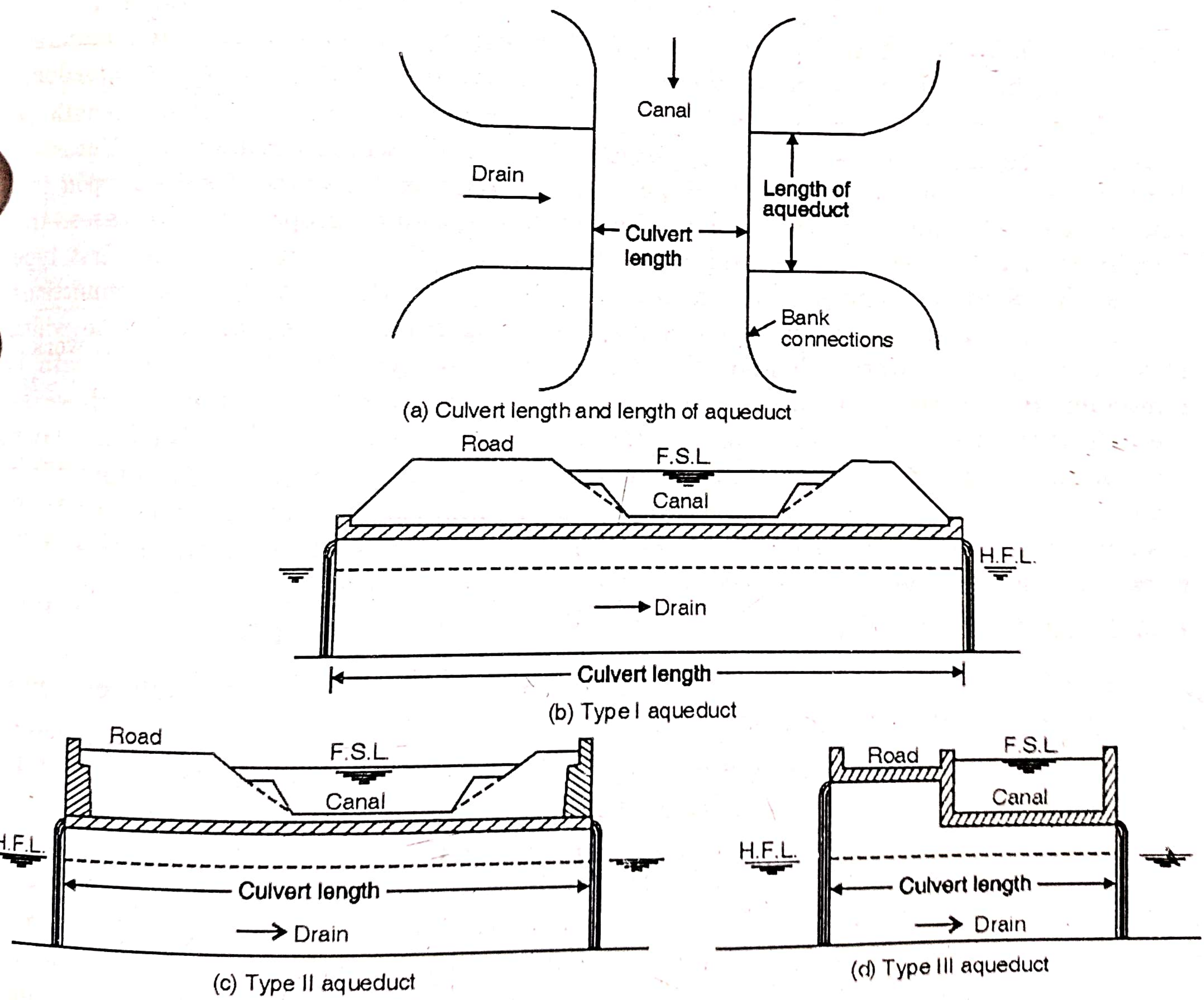


FIG. 19.4. CLASSIFICATION OF AQUEDUCTS

(*ii*) Sides of the aqueduct in earthen banks, with outer slopes supported by masonry walls [Fig. 19.4 (*c*)].

(*iii*) Sides of the aqueduct made of concrete or masonry [Fig. 19.4 (*d*)].

In the first type, the original canal section is retained and no fluming is done. The sides of the aqueduct are kept in earthen banks with complete earthen slopes. In the third type, on the contrary, the canal section is reduced, fluming is done and the sides of the canal are made of concrete or masonry instead of earth slopes. Type two is intermediate between the two in which the sides of the aqueduct are in earthen banks but the outer section is reduced by supporting it by masonry or concrete walls.

Selection of a suitable type.

In order to select a suitable type out of the three, it is necessary to understand three terms : (*i*) culvert length, (*ii*) length of the aqueduct, and (*iii*) bank connections. *Culvert length* is the width of the aqueduct, measured along the drain. The culvert length naturally depends upon the shape and size of the canal section. The *length of the aqueduct* is the length measured perpendicular to the drain. It is evidently equal to the width of the drain. *Bank connections* consist of masonry wings etc., required to connect the regular section of the canal to its modified section over the aqueduct.

In the first type, since canal is fully in earthen section, the culvert length is maximum. Hence, the cost per unit length of aqueduct will be maximum. However, bank connections are not required. Hence, the selection of this type depends upon the relative cost of bank connections and that of the aqueduct proper. In all cases, the cost of bank connection is independent of the length of the aqueduct. Hence, first type is suitable when the length of the aqueduct is small and the cost of bank connections would be large in comparison to the saving resulting from the reduction in the width of work if type III were adopted. On the contrary, in type III the culvert length is minimum. Hence, the cost per unit length of the aqueduct will be minimum. However, the cost of the bank connections will be additional. Therefore, type III is suitable where the length of culvert is large (*i.e.*, a big drain). Type II is suitable for intermediate conditions. To sum up, for a very large drain, type III is more suitable while for a small drain, type I is more economical. The correct way of selecting a type is to work out cost of the three types and see which is cheaper.

19.5. FEACTURES OF DESIGN OF CROSS-DRAINAGE WORKS

Following the some of the important features of design of cross drainage works.

(A) Hydraulic design :

1. Determination of maximum flood discharge and the high flood level (H.F.L.).
2. Fixation of waterway of the drain.
3. Contraction of canal waterway (for type III aqueducts)
4. Clearances and free board
5. Head loss through syphon barrels.
6. Determination of uplift pressure on the roof of trough.
7. Determination of uplift pressure on the floor.
8. Design of bank connections.

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HYDROLOGY

★ HYDROLOGY!-→ Hydrology is the science which deals with the occurrence, distribution, and movement of water.
→ The movement of water includes the movements of water in atmosphere on earth surface and under ground.

★ Hydrology Cycle!- The hydrology cycle is the follows by the water in the three phases:-

a) Evaporation
b) Precipitation
c) Run off

Precipitation!- It is the amount of water Considering on the earth Surface as rainfall and Snowfall.

→ It is measured by Raingauge

Evaporation!- Is the water quantity lost due to evaporation from water and soil surface.

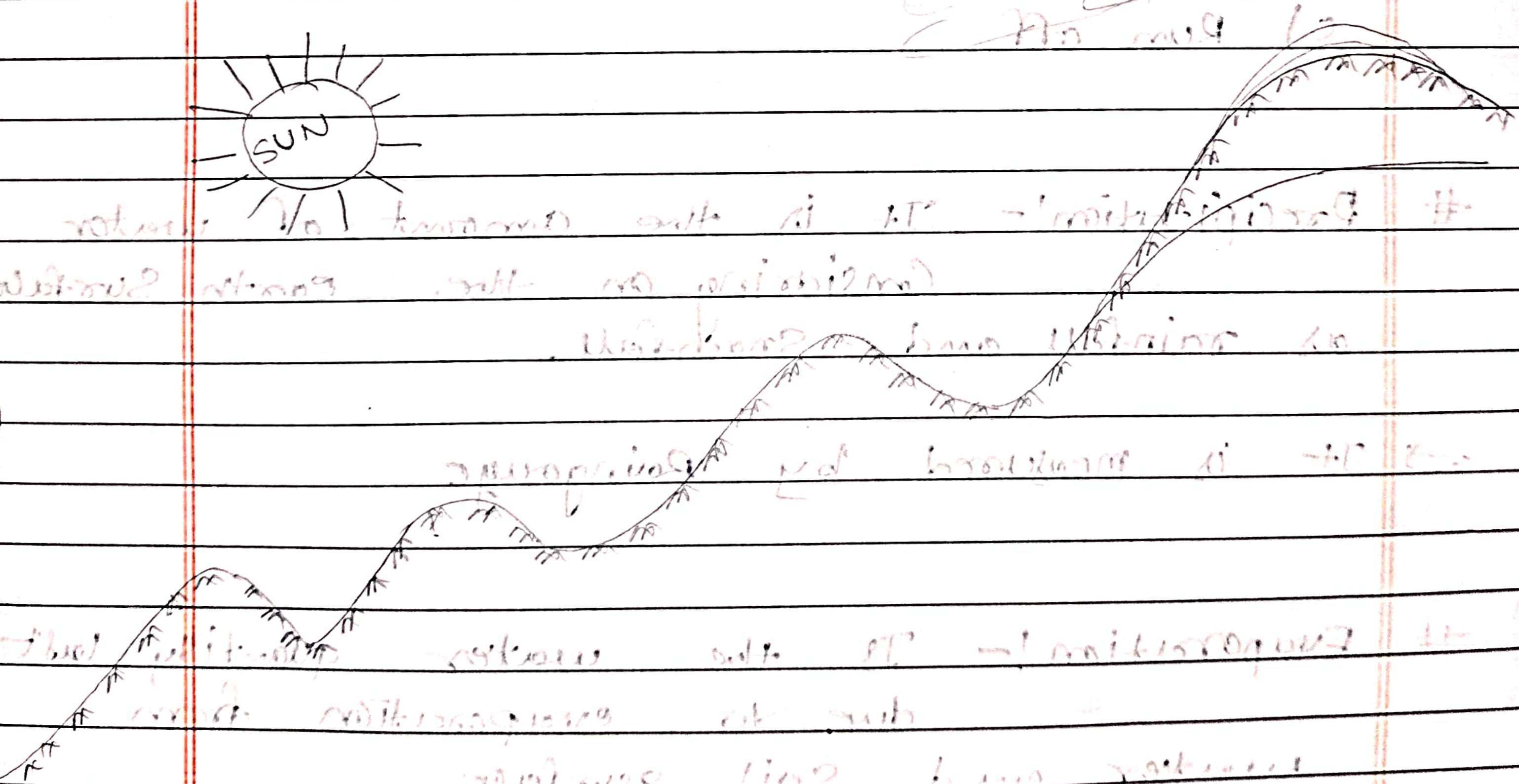
Transpiration!- The water passed off in to atmosphere through leaves of

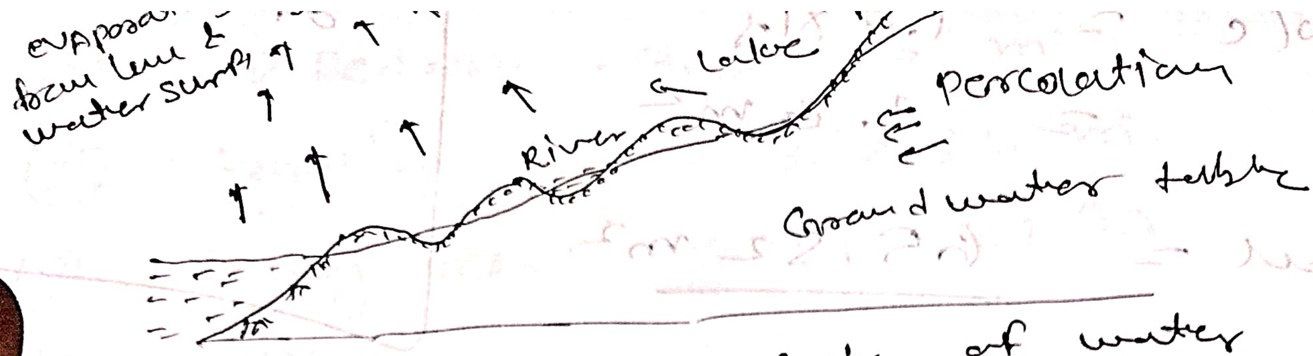
the tree is known as transpiration.

→ The measurment of the transpiration evaporation is done by evaporation pan.

Run off :- The quantity of water running of on the surface of the ground is known as surface runoff.

→ It is measured by actual measurments by runoff impirical formula, by probability methods.





Hydrology cycle of water

→ The water which gose in the atmosphere by evaporation and transpiration again comes back in the from of precipitation under favaourable climatic Conditions.

→ Due to Sun's heat the water from the earth surface lake, river see etc: evaporate and rises upwards.

☆ Precipitation !–

→ It is the primary sorce of water abilable on the earth and includes rainfall, snow-fall, Hail, and sleet.

→ All water sorces receive their supply of water from the precipitation.

→ The water of precipitation further gose off in the fallowing ways.

i) Runoff! –

ii) percolation ! –

iii) Evaporation ! –

iv) Transpiration ! –

forms of precipition :-

following are the various from of precipitation.

i) Drizzel :-
ii) Glaze :-
iii) Rain :-
iv) Sleet :-
v) Snow :-
vi) Snow flakes :-
vii) Hail :-

i) Drizzel :- In the from of precipitation the size of water droplates is below 0.5 mm and intencity is less then 1 mm/hour.

ii) Glaze :- In the from of precipition the rain freezes as it comes in contact with cold objects.

iii) Rain :- The size of droplate is between 0.5 mm to 6.25 mm the droplate of larger size then 6.25 tend to break up while ~~faith~~ falling.

iv) Sleet :- Light rain along with ~~sa~~ sleet occurs some times. ~~The~~ In the from of precipition frozen rain drops cooled to ice stage fall the rain drops while falling through air at sub-freezing temperature are frozen.

v) Snow :- The water ~~wap~~ vapour directely change to ice the snowfall dose not occur with rain as in case of sleet.

vi) Snow flakes :- These are formed when number of ice christle are futed together.

vii) Hail :- In the from of precipitation the lumps of ice above 5mm diameter.

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Types of precipitation :-

Precipitation are of the following types

i) Cyclonic Precipitation :- It occurs from lifting of air masses converging in to low pressure area known as cyclone. Cyclonic precipitation may be

a) Frontal
b) Non-frontal

a) Frontal :- Front is the border region between two adjacent air masses having different temparature and humidity characteristics.

→ Front may be warm air front or cold front depending upon the conditions.

b) Non-Frontal precipitⁿ :- In this type precipitⁿ is caused by the moist warm air mass is stationary. the cold air masses which is moving, meets it. the moist warm air is light and heavy cold air is cutting it.

→

ii) Orographic precipitation :- In this case the warm moisture air masses are lifted to high altitud due to topographic barrier such as mountains.

→ This warm moisture air at high altitud cools down and causes precipt^n.

→ This precipt^n is hilly areas it is mostly due to the orographic air mass which are rich in moisture content.

iii) Convective Precipitation:- This type of precipt^n is caused by natural rising of warmer air in colder, denser surrounding.

→ The unequal heating at the surface of earth, unequal cooling at the high altitude and the top of air layers cause temperature differences.

→ It is spotty and its intensity may vary from light showers to cloude bursts.

→ precipt^n due to turbulant ascent after its travels over ocean the air mass is forced to rise upward due to grater friction of earth surface.

→ The increased turbulant and friction also caused the moist air mass to rise upward.

→ At the high altitude the moist air mass cools down and causes precipt^n.

Example:-

winter rain of Madras State.

★ measurment of Rain fall :-

Rain fall is measured in mm of water. with the help of standard rain gauge.

The quality of rain fall at any place during even period is calculated on the basis of the depth of water which word accumulate on the level surface. If their is no loss of water in any way such as percolation, evaporation, etc.

i) Standard Rain Gauge :-
ii) Symon's Rain Gauge
iii) Automatic Recording Rain Gauge.
iv) Float Automatic Rain Gauge.

★ Run OFF :-

When the rain fall on the ground a portion of it percolates in the ground a portion evaporates and a portion flows on the ground and fiche reaches river and streams.

The quantity of water which reaches the streams or rivers both from surface flow as well as base flow is known as runoff.

The run off mainly depends on the following factors :-

i) Area of the catchment
ii) Shape and slop of the catchment area.
iii) The degree of porosity of the soil of the catchment area.
iv) ~~Pr~~ Duration of rainfall.
v) Intensity of rainfall
vi) Obstruction in the flow of water due to ~~trees~~ trees, field, garden etc.

★ Run-off losses :-
→ The actual runoff is equal to the total rainfall minus losses due to evaporation, absorption, percolation.
→ It is very difficult to estimate these losses individually or collectively.
→ The runoff losses are broadly classified as follows:-

a) Evaporation losses from water surface.
b) Percolation losses.
c) Absorption losses.

A) Evaporation losses :- During summer ~~season~~ season the atmospheric temperature as well as water temperatures are higher and the humidity in the atmospheric air is side low. This causes the vapour pressure near the water surface to rise there by increasing the rate of evaporation.

B Percolation Losses:-
The water which ~~peni~~ penetrates or percolates in the ground and dose not join the runoff, come under percolation losses. this loss mainly depends on the nature of the strata under laying the surface soils and the sub soil.

c) Absorption losses:-
The quantity of water evaporated from the land surfaces mainly depends upon the moisture presen in the atmosphere and and surfaces characteristics of the ~~th~~ surface soil which effect the infiltration, percolation and absorption.
→ Climatic condition play vital role in it.

Rain Gauge Network
It is absolutely essential to design a proper network of rain ~~ag~~ gauge in a given catchment [water shed] to collect the necessary ~~partici~~ precipitation data. The rain gauge density or network density is defined as the ratio of total area of catchment to the total number of gauges in the catchment.

→ The world meteorological organisation [WMO] has the following norms for minimum network density.

Region	Description	Network Density	
		Minimum	Tolerable
I	Flat Region	1 Gauge for 600-900 km^2	1 Gauge for 900-3000 km^2
II	Mountaneous area	1 Gauge for 100-250 km^2	1 Gauge for 250-1000 km^2
III	Arid and polar Zone	1 Gauge for 1500-10000 km^2	

☆ IS Coad Recommendation

IS:4987 - 1968

has recommendated the following cities density

i) one gauge per 520 km^2 in plain areas

ii) one gauge per 260 km^2 - 390 km^2 in region with average elevation of 1000 m above mean see level.

iii) one gauge per 130 km^2 in hilly region with heavy rainfall.

It is also recommended that 10% of the gauges are of the recording time type

The network arrived at based on the above recommendation's should be so located that all the gauges will have more or less equal.

The optimum number of raingauge station (N) as per IS is given by

$$N = \left[\frac{C_v}{P}\right]^2$$

where

C_v = Coefficient of variation of rainfall values of existing station.

P = Desired degree of error in estimating mean rainfall.

Both C_v & p should be expressed as percentage

☆ Computation of average rainfall over a basin :-

⇒ In order to compute the average rain fall over a basin or catchment area, the rain fall is measured at a number of rain gauge stations ~~such~~ suitably located in the area. The network density of rain gauge depends upon the uses for which the rain fall data is intended.

→ A network should be so planned should be as to have a representative picture of the areal distribution of rain fall.

→ There should be no Concentration of gauges in heavy rain fall areas at the expence of dry areas or vice versa.

→ If a basin or catchment area contains more then 1 rain gauge station the Computation of average precipitation or rain fall may be done by the following methods:-

i) Arithmatic Average Method
ii) Thiessen polygon method
iii) Isohyetal Method
iv) Grid point Method.

i) Arithmatic Average method:-
If the rainfall is uniformly distributed on its real pattern the simplest method of estimating average rain fall is to Compute the Arithmatic Average of the recorded rainfall values at various Stations.

$$P_{average} = \frac{P_1 + P_2 + \cdots P_n}{n}$$

ii) Thiessen polygon method :-
This is a more common method of weighing the raingauge observation according to the area. Thiessen polygon method is also called weighted mean method and its more accurate than the arithmetic average method.

$$P_{average} = \frac{A_1P_1 + A_2P_2 + \dots + A_nP_n}{A_1 + A_2 + \dots + A_n}$$

iii) Isohyetal method :-
→ An Isohyet is a line on a rainfall map of the basin joining places of equal rainfall readings.
→ An Isohyetal map showing contours of equal rainfall presents a more accurate picture of the rainfall distribution over the basin.
→ This method is the most elaborate and accurate.
→ In this method is ~~elaborate~~ that the raingauge station best represent area which is close to it.

$$P_{average} = \frac{\cancel{\Sigma A}\ \Sigma A \times \left[\frac{P_1 + P_2}{2}\right]}{\Sigma A}$$

iv) Grid point method:-
In this method all the grid stations are marked on the map of the basin along with the depth of precipitation on this map drawn to a suitable scale. A uniform rectangular grid at suitable spacing is super emposed.

★ Losses of abstraction from precipitation:-

When precipition takes place on land surface, whole of it is not available as run off because of losses that take place during or after the precipitation. important losses consist of i) interception, (ii) evaporation (iii) transpiration (iv) infiltration (v) depression storage (vi) water shed leakage :- Out of these, evaporation, transpiration and infiltration are the major losses.

Thus, Precipitation - surface runoff = Total loss

where total loss =
Interception + evaporation + Transpiration + infiltration + depression storage + watershed leakage.

★ Interception:- Interception may be defined as the amount of precipitation water which is intercepted by vegetative foliage, buildings and other objects laying over the land surface. Interception does not reach the land surface but is returned by back to the atmosphere by evaporation.

☆ Depression Storage :- A catchment area generally has many depressions of shallow depth and of varying size and shape. When precipitation take place, water runs towards these depressions and fill them before actual over land flow or run off towards take place.

☆ Water shed leakage :- Water shed leakage may be defined as follow as water from one basin to another basin, to the sea through major faults, fissures or other geographical features. Due to this faults / fissures underground hydraulic conduit so formed convey this discharge filling over a part of the catchment.

☆ Estimation of evaporation from surface of water bodies :-

The following are prominent methods of estimation of evaporation from surface of water bodies.

i) measurement using evaporation pan
ii) use of empirical equation
iii) water budget method
iv) Energy budget method.

iii) Water budget method:- This method balance all the incoming, outgoing and stored water in a lake or reservoir over a period of time, using the following equation.

$\sum$ Inflow $-$ $\sum$ Outflow = Change in storage + evaporation loss.

or

$$E = \sum I - \sum O \pm \Delta S$$

The above equation can be generalised as under, taking all the factors of inflow and outflow.

$$E = (P + I_{sf} + I_{gf}) - (O_{sf} + O_{gf} + T) \pm \Delta S$$

Where

P = Precipitation

I_{sf} = Surface water inflow

I_{gf} = ground water inflow

O_{sf} = surface water outflow

O_{gf} = ground water outflow

T = Transpiration loss which may be neglected.

ΔS = Change in Storage.

∴ The above methods dose give accurate results because it is very difficult to measure Igf and Ogf for a lake/reservoir.

★ Factor affecting Run off :-

1. Precipitation charactoristics :-
ii) Shape and size of the catchment
iii) Topography
iv) Geological characterstics
v) Metrological characterstics (Temperature)
vi) Character of the catchment surface
vii) Storage characterstics

1 Precipitation charactorstics :-
This is most important factor on which run off depends. Important precipitations are (a) intancity (b) duration (c) aerial distrubution (d) direction of storm moment (e) form of precipitation and (f) evapo-transpiration. The more the rain fall, more will be the run off depends on the type of the storm causing precipitation and also upon its duration

ii) Shape and size of the catchment :-
More intense rainfalls are generally distributed over a relativity smaller area a stream collecting water from a small catchment area is likely to give greater runoff intensity per unit area. In the case of very big catchment uniform rain

seldom falls over the entire area with the result that only very few tributaries of the stream feed water to main stream during a particular storm. Thus runoff intensity of larger stream per unit catchment area is lesser.

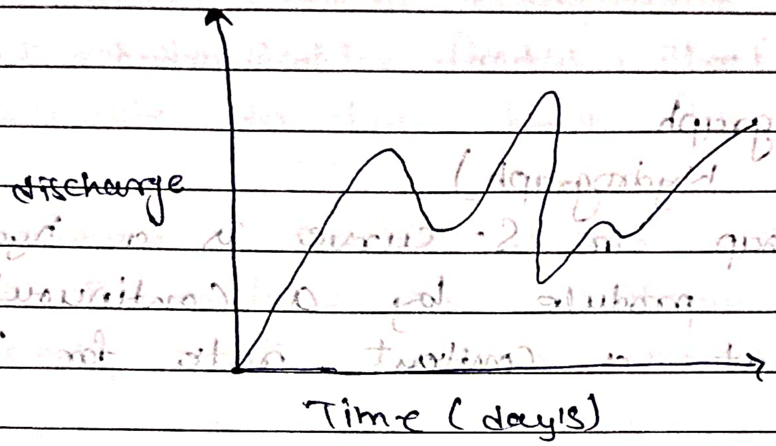
3 Topography of Catchment :-
The runoff depends upon weather the surface of the catchment is smooth or rugged. If the surface slope is steep water will flow quickly and absorption and evaporation losses will be less, resulting in greater runoff. If the catchment is mountainous and is on the windward side of the mountains. The intensity of rainfall will be more and hence runoff will also be more.

5 Geological characteristics of basin :-

☆ Hydrograph :-

→ Hydrograph is a graph showing of discharge with time at a particular point of a stream

→ It show the time distribution of total run-off at the point of measurement.



☆ Storm hydrograph :-

→ A hydrograph is generated from runoff due to precipitation resulting from either and isolated storm or a series of consecutive storms. [known as complex storm] it is also known as storm hydrograph.

→ ~~Con~~

☆ UNIT HYDROGRAPH :- [UH] A unit hydrograph is a hydrograph representing one (1) centimeter or 1 inch of run off from a rainfall of ~~sum~~ sum unit duration and specific ~~and~~ areal distribution. ~~Unit duri~~

Assumption of unit hydrography :-

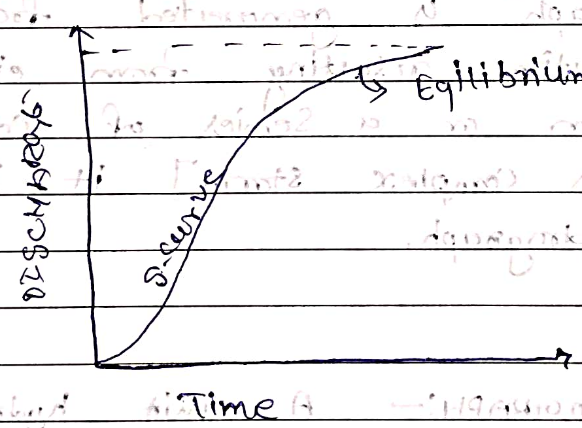
i) The effective rainfall is uniformly distributed within its duration of specified period of time.

ii) The effective rainfall is uniformly distributed through out the whole area of the drainage basin.

iii)

★ S- Hydrograph (Summation Hydrograph)

- S- hydrograp or S- curve is a hydrograph that is produce by a continuous effective rainfall at a constant rate for indefinite period.
- It is a continuous rising curve in the form of letter S till equilibrium is reached.



★ Peak flow determination:-

I. A flood is an unusally high stage of a river due to runoff rainfall and melting of snow in quantities to grate to be confined in the normal water surface elevation of the river or stream. In the peak flow determination reference is made to the following three type of floods

i) Standard project Flood (SPF):
It is the flood that would result from the most severe combination of meteorological and hydrological factors that are reasonably applicable to the basin.

ii) Maximum probable Flood :-
It is defined as extreme flood that is physically possible in a region as a result

UNIT - 02

WATER REQUIREMENTS of CROPS

Page No
Date

★ IRRIGATION:-

→ Irrigation may be define as the process of artificially supplying water to soil for ~~cultivating~~ crops.

→ It is science of planning and designing an efficient, low cost, economic irrigation system.

Necessity:-

→ India is basically an ~~agriculture~~ agricultural country and all its resources depend on the agriculture output.

i) Less Rainfall:-

→ When the total rain fall is less then needed for the crop, artificial supply is necessary.

→ In such a case, irrigation works may be constructed at a place where more water is available, and then to convey the water to the areas where there is deficiency of water.

ii) Non uniform Rainfall:-

→ The rainfall in a particular area may not be uniform over the crop period. During the early periods of the crop rains may be there, but no rain water may be available at the end with the result that either the yield may be less, or the crop may die altogether.

iii) Growing a number of crops during a year:-
The rainfall in an area may be sufficient to raise only one type of crops during the rainy season (i.e. kharif crops) for which no irrigation may be required. However with the provision of irrigation facilities in that area, crop can be raised in other season also.

iv) Growing perennial crops:-
Perennial crops such as sugar cane etc. with need water throughtout the year, can be raised only through the provision of irrigation facilities in the area.

v) Commercial crops with additional water:-
The rain fall in a particular area may be sufficient to raised the usual crops but more water may be necessary for raising commercial and cash crops.

vi) Controlled water supply:-
By the construction of proper distribution system the yield of the crop may be increased because of controlled supply of water.

UNIT-3

Reservoir Planning

3.1Purposes of reservoir

Dams are constructed across the rivers and streams to create an artificial lake or reservoir behind it. Dams and reservoirs are the most important and expensive elements in multi-purpose river basin development. They require very careful planning, design, and operation.

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 Reclamation of low-lying lands

7. Debris control

8. Preservation and cultivation of useful aquatic life

9. Recreation.

Classification of reservoir based on purpose:

Depending upon the purposes served, reservoirs may be classified as under:

(i) Storage or conservation reservoirs
(ii) Flood protection reservoirs
(iii) Distribution reservoirs
(iv) Multipurpose reservoirs.
(I) Storage or Conservation Reservoir: Storage reservoirs are primarily used for water supplies 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 period of large supplies, and release it gradually as and when it is needed.

(II) Flood Control Reservoirs: Flood control or flood protection reservoirs are those which store water during flood and release it gradually at a safe rate when the flood reduces. By the provision of artificial storage during the floods, flood damage downstream is reduced.

(III) 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.

(IV) 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.

INVESTIGATIONS FOR RESERVOIR PLANNING

The following investigations are required for reservoir planning:

- Engineering surveys.
- Geological investigations.
- Hydrological investigations.

1. **Engineering surveys:**

   The area at the dam site is surveyed in detail and a contour plan is prepared.

   From the plan, the following physical characteristics are prepared:

   (a) Area-elevation curve.

   (b) Storage-elevation curve.

   (c) Map of the area to indicate the land property to be surveyed

Area elevation curve and Storage elevation curve:

Area-elevation and Storage-elevation Curves.

Fig. 6.2 shows a typical contour plan at the reservoir site. The hatched area shows the water spread area. The area A_1, A_2, A_3 enclosed by the successive contours can be determined with a planimeter. A close observation of Fig. 6.2 shows that as the value of a contour (*i.e.*, elevation) increases, its area increases. A curve, such as curve AB shown in Fig. 6.3, may be drawn between elevation and area.

The reservoir capacity, or the volume of storage, corresponding to a given water level in the reservoir may be calculated either by trapezoidal formula or by prismoidal formula. Thus, if V is the storage volume and h is the contour interval, the formulae are:

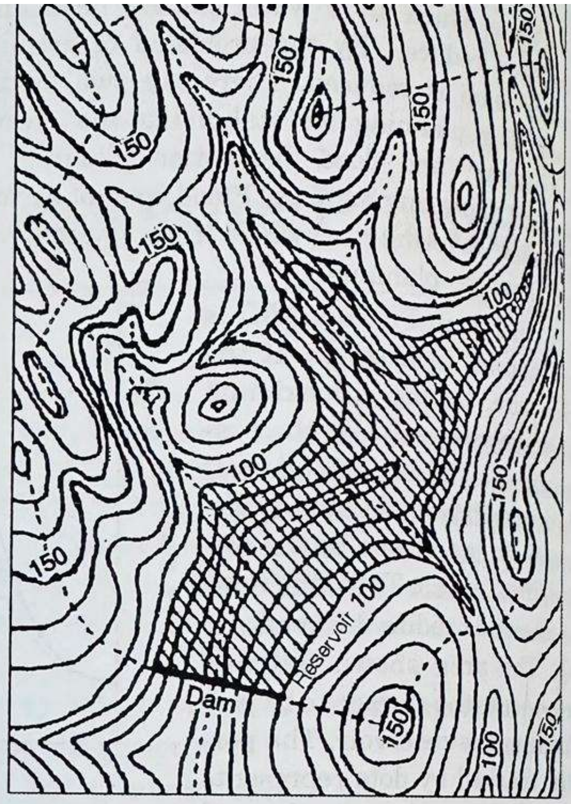


FIG. 6.2. A TYPICAL CONTOUR PLAN.

(1) $V = \Sigma \frac{h}{2}(A_1 + A_2) \ldots$ (Trapezoidal formula)

$$= \left\{\frac{A_1 + A_n}{2} + A_2 + A_3 \ldots\ldots\ldots\ldots A_{n-1}\right\} \quad \ldots(6.1)$$

(2) $V = \Sigma \frac{h}{3}(A_1 + A_2 + \sqrt{A_1 A_2}\ ) \ldots\ldots\ldots$(Cone formula) ...(6.2)

(3) $V = \frac{h}{3}[(A_1 + A_n) + 4(A_2 + A_4 + ..) + 2(A_3 + A_5 + \ldots)]$ ...(Prismoidal formula) ...(6.3)

where A_n is the area of the contour corresponding to the water surface elevation in the proposed reservoir. The volumes of storage corresponding to various water-surface elevation may be calculated and a curve, such as CD in Fig. 6.3, may be plotted between elevation and storage.

The contour plan (Fig. 6.2) also indicates the water spread corresponding to the reservoir elevation and enables us to determine the area under submergence and the compensation to be paid to the owners.

2.Geological investigations:

In almost all major civil engineering projects, geological advice is the most essential. Geological investigations cost very little in comparison to the total cost of the project; in a number of cases in recent years it has amounted to between 1/4 to 1 percent of the whole. This relatively small amount represents a valuable insurance against expensive difficulties, otherwise unforeseen, which might arise during construction but which it may be possible to predict from a study of the geological factors involved.

Geological investigations are required to give detailed information about the following items:

1. Water tightness of reservoir basin.
2. Suitability of foundations for the dam.
3. Geological and structural features, such as folds, faults, fissures etc. of the rock's basin.
4. Type and depth of over burden (superficial deposits).
5. Location of permeable and soluble rocks, if any.
6. Ground water conditions in the region.
7. Location of quarry sites for materials required for the dam construction and quantities available from them.

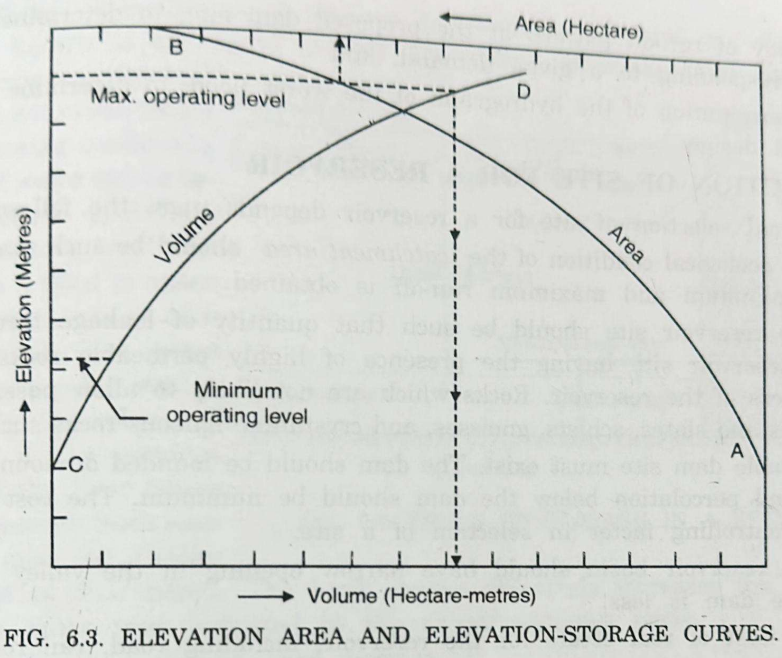


FIG. 6.3. ELEVATION AREA AND ELEVATION-STORAGE CURVES.

3.Hydrological investigations:

The hydrological investigation is a very important aspect of reservoir planning.

The capacity of the irrigation canals and/or the installed capacity of the power houses will depend upon the available supplies from the reservoir.

Factors affecting 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 dan 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, (ii) less evaporation losses because of reduction in the water spread area, and (iii) less likelihood of weed growth.

8. The reservoir site should be such that it avoids or excludes water from 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 storages and various water levels:

The following are various zones of storage in reservoir:

(1) Useful storage.

(2) Surcharge storage.

(3) Dead storage.

(4) Bank storage or Valley storage.

- The maximum level to which the water will rise in the reservoir during ordinary operation condition is called **normal pool level.**
- 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 the minimum pool level is known as the **useful storage.**
- The volume of water below the minimum pool level is known as the **dead storage.**
- The term **bank storage or valley storage** are referred to the volume of water stored in the pervious formation of the river banks and the soil above it. Such storage depends upon the geological condition of river banks.
- 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**.**

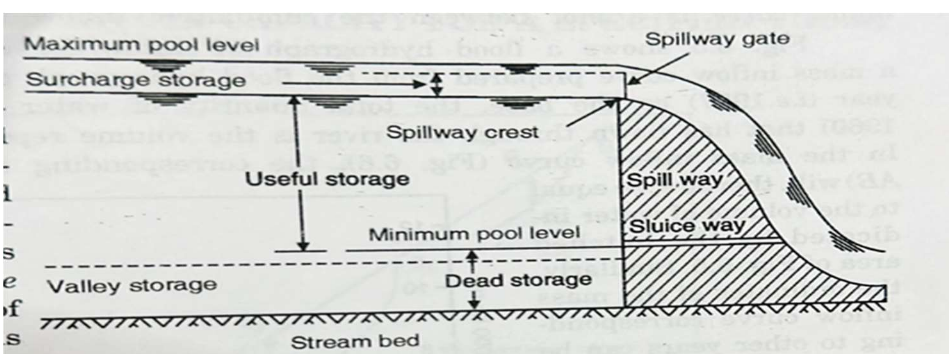


FIG. 6.4. ZONES OF STORAGE IN A RESERVOIR.

Storage capacity and yield of reservoir:

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

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.

Dams: Various types of dams

TABLE 7.1. TYPES OF DAMS

Basis of Classification	*Types*	*Common Examples*
(a) Classification according to use	(*i*) Storage dam	Gravity dam, earth dam, rockfill dam, Arch dam etc.,
	(*ii*) Diversion dam	Weir, barrage
	(*iii*) Detention dam	Dike, water spreading dam, debris dam
(b) Classification by hydraulic design	(*i*) Overflow dam	Spillway
	(*ii*) Non-overflow dam	Gravity dam, earth dams, rockfill dam
(c) Classification by materials	(*i*) Rigid dams	Gravity dam, arch dam, buttress dam, steel dam, timer dam
	(*ii*) Non-rigid dams	Earth dam, rockfill dam

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. Storge 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, etc.

Diversion dam-The purpose of a diversion dam is essentially different, a diversion dam simply raises water level slightly in the river and thus provides head for carrying or diverting water into canals, or other conveyance systems to the place of use.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.

A detention dam is sometimes called water-spreading dam or dike.

In a multipurpose river valley project, the dam may serve the purposes of storage, flood protection and recreation. The stored water may also be used for irrigation, water power generation, municipal and industrial supply and other purposes.

CLASSIFICATION ACCORDING TO HYDRAULIC DESIGN

1.Overflow dam.

2.Non-overflow dam.

Non-overflow dam. A non-overflow dam is the one in which the top of the dam is kept at a higher elevation than the maximum expected high flood level. Water is not permitted to overtop the dam. A non-overflow dam may be constructed of wide variety of materials, such as earth, rockfill, masonry, concrete etc.

Overflow dam An overflow dam is the one which is designed to carry surplus discharge (including floods) over its crest. Its crest level is kept lower than the top of the other portion of the dam (i.e. non-overflow dam) Such dams are generally made of concrete or masonry. An overflow dam is commonly known as a spillway.

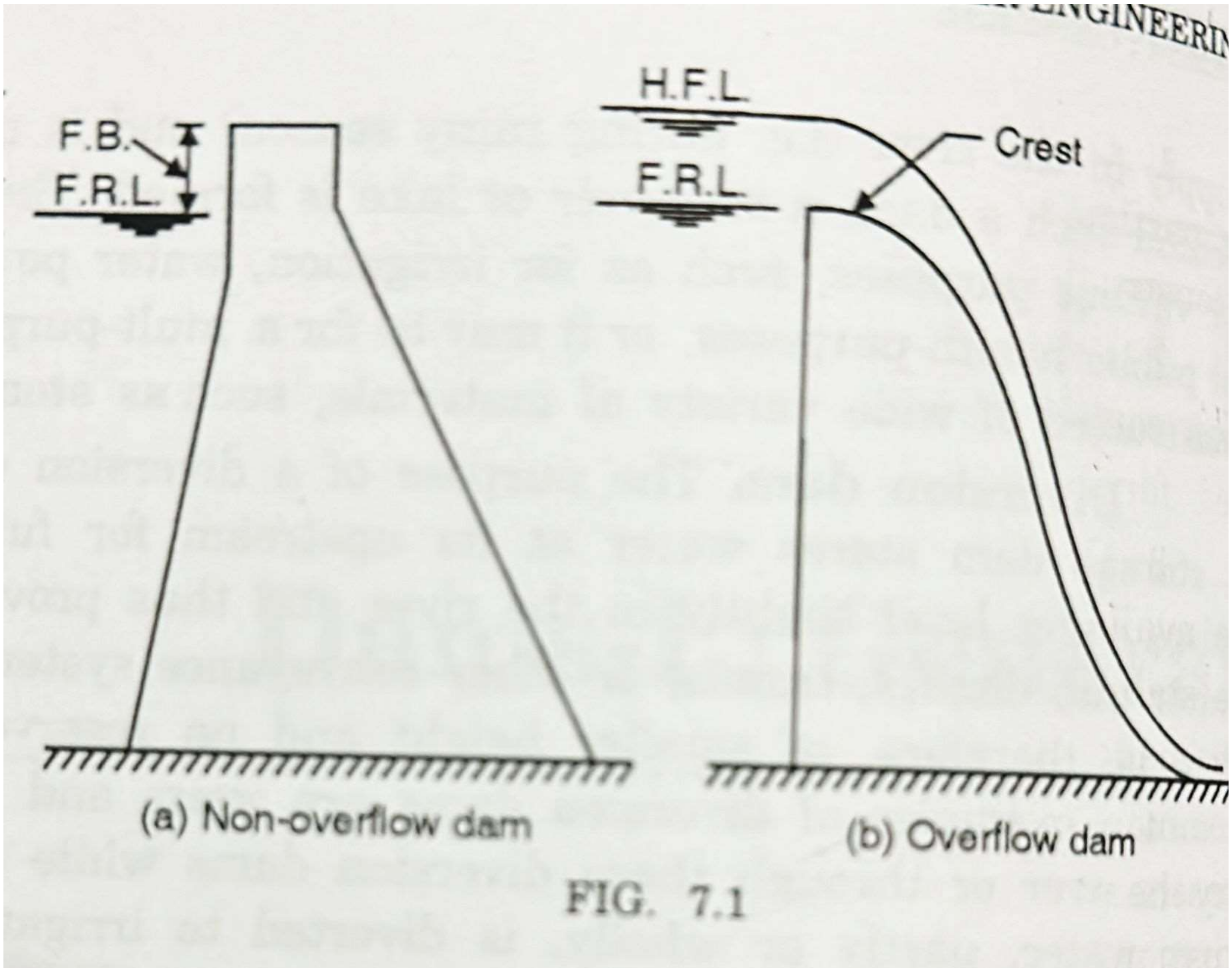


FIG. 7.1

Classification according to material:

Rigid Dam

Non-rigid Dam

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

A gravity dam is the one in which the external forces (such as water pressure, wave pressure, silt pressure, uplift pressure etc.) are resisted by the weight of the dam itself. Thus, the forces disturbing the stability of the dam are resisted by the gravity forces of the mass of the dam. A gravity dam may be constructed either of masonry or of concrete.

Masonry gravity dams are now-a-days constructed of only small heights.

All major and important gravity dams are now constructed of concrete only.

ARCH DAMS

An arch dam (Fig. 7.2) is a dam curved in plan and carries at major part of its water load horizontally to the abutments by arch action. This part of water load depends primarily upon the amount of curvature. The balance of the water load is transferred to the foundation by cantilever action.

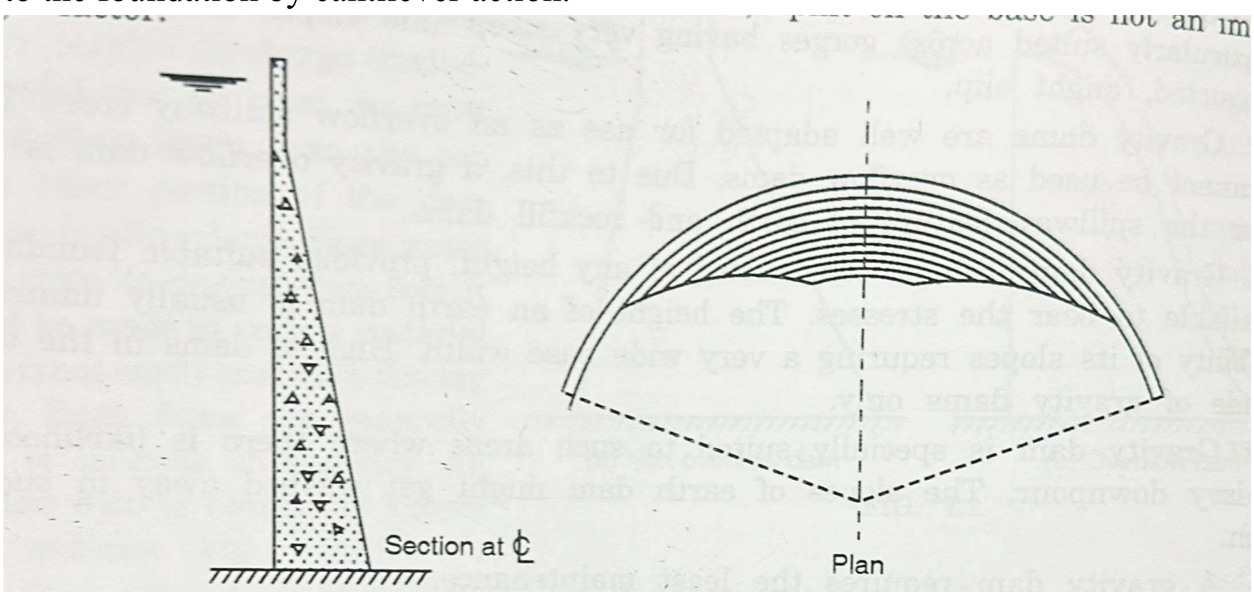


FIG. 7.2. ARCH DAM.

BUTTRESS DAMS

A buttress dam (Fig.7.3) consists of a number of buttresses or piers dividing the space to be dammed into a number of spans. To hold up water and retain the water between these buttresses, panels are constructed of horizontal arches or flat slabs. When the panels consist of arches, it is known as multiple arches type buttress dam. If the panels consist of flat slab, it is known as deck type buttress dam.

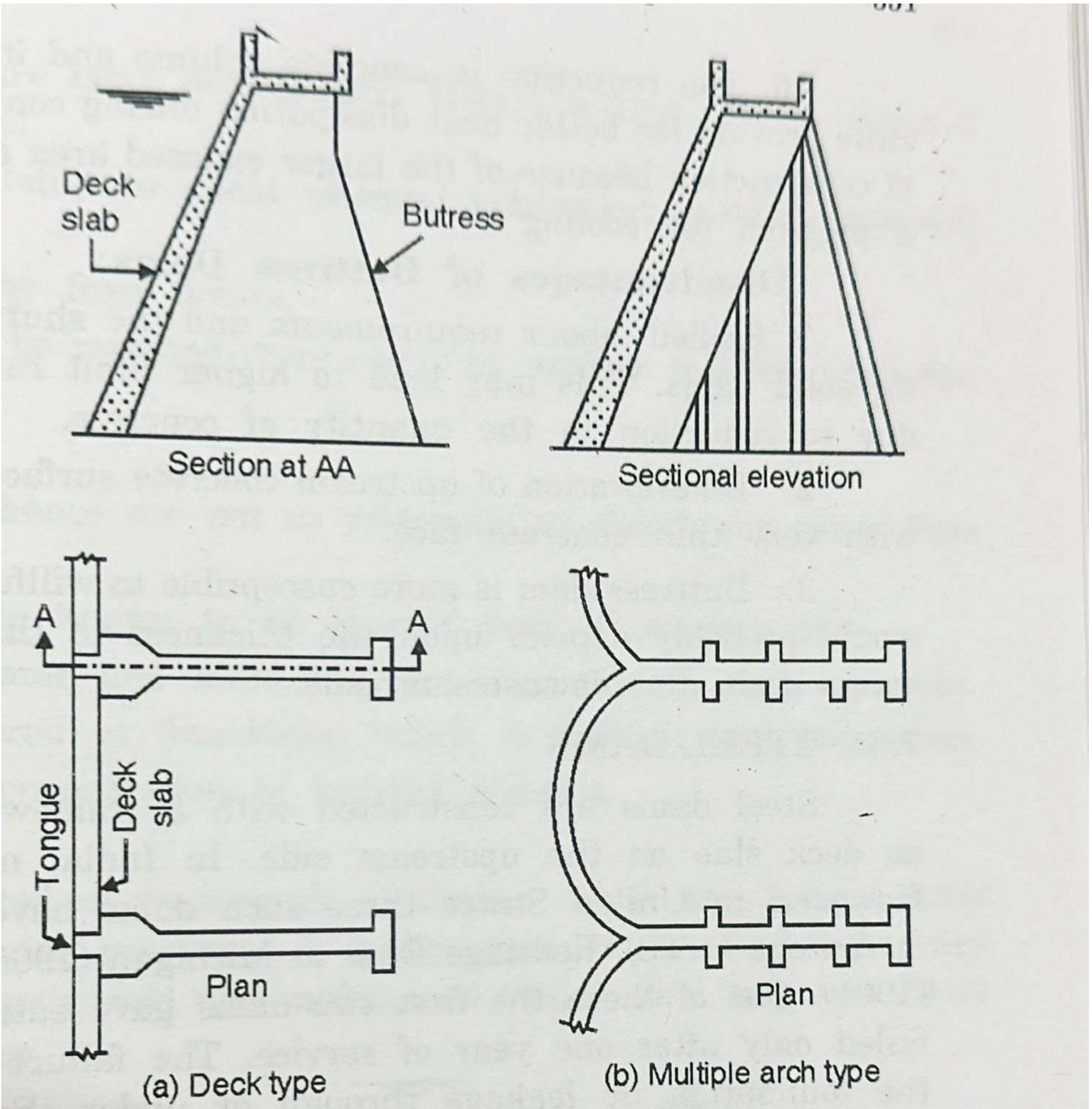


FIG. 7.3. BUTTRESS DAMS.

Steel Dams: Steel dams are constructed with a framework of steel with a thin skin plate as deck slab on the upstream side.

Steel dams (Fig. 7.5) are generally of two types

(i) direct strutted type (ii) cantilever type.

In the **direct strutted type**, the load on the deck plate is carried directly to the foundations through inclined struts.

In the **cantilever type**, the section of the bent supporting the upper part of the deck is formed into a cantilever truss.

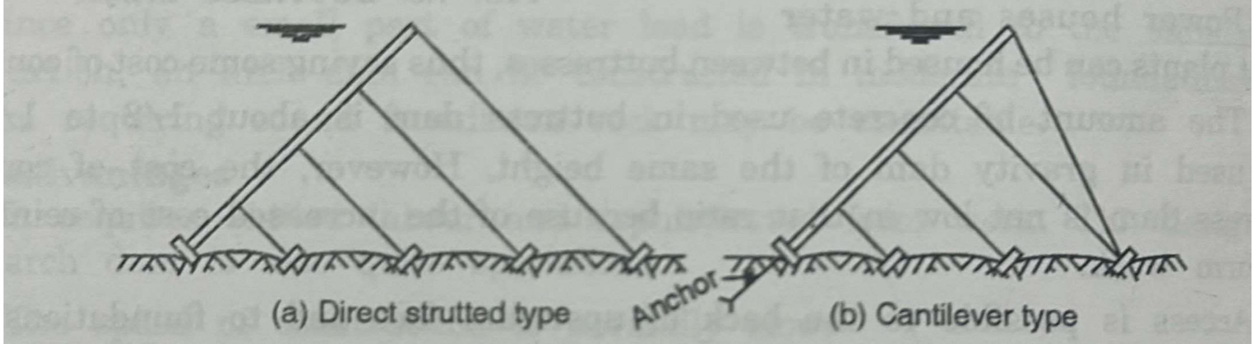


FIG. 7.5. STEEL DAMS.

Timber Dams:

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.

Timber dams are normally found to be of three types:

1. A-frame type
2. Rock-filled crib type
3. Beaver type

EARTH DAMS AND ROCKFILL DAMS

Earth dams are made of locally available soils and gravels and therefore, are most common types of dams used up to moderate heights. Their construction involves utilization of materials in the natural state requiring a minimum of processing. With the advancing knowledge of soil mechanics and with the more sophisticated earth moving equipment, earth dams are now becoming more common, even for higher heights. The foundation requirements of earth dams are less stringent than for other types.

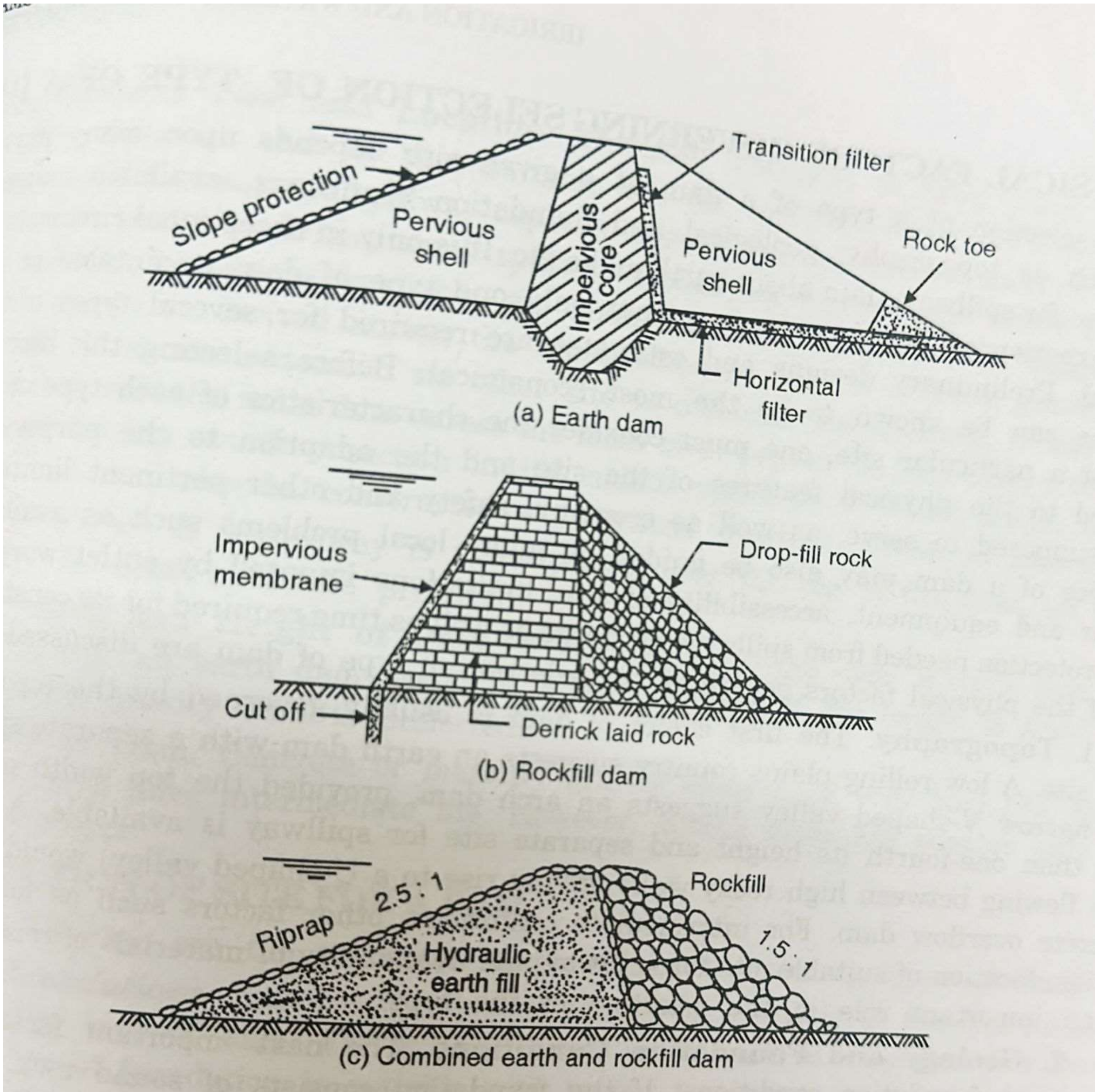


FIG. 7.9 EARTH AND ROCKFILL DAMS.

Factors governing the selection of type of dam

The selection of a type of a dam at a given site depends upon many physical factors such as topography, geological and foundation conditions, available materials, suitable site for spillway, data about earthquake etc.

The choice of a dam may also be guided by many local problems such as availability of labor and equipment, accessibility of site, limitations etc.

1. Topography-The first choice of dam is usually governed by the topography for the site. A low rolling plains country suggests an earth dam with a separate spillway.

A low narrow V-shaped valley suggests an arch dam, provided the top width of valley is less than one-fourth its height and separate site for spillway is available. A narrow stream flowing between high rocky walls (giving rise to a U-shaped valley) would suggest a concrete overflow dam.

2. Geology and Foundation Conditions. The next important factor is the geology and foundation conditions. If the foundation consists of sound rock, with no fault or fissures, any type of dam can be constructed on it.
Poor rock or gravel foundations are suitable for earth dam, rockfill dam or low concrete gravity dam. Since there will be considerable under-seepage in this case, effective water cutoffs or seals have to be provided. Silt or fine sand foundations have the problems of settlement, seepage and toe-erosion. Hence, such foundations are suitable only for either earth dam or low concrete gravity dam but not rockfill dams.

3. Materials of Construction. The next important factor is the availability of materials of construction for dam. The cost of construction of a particular type of dam will depend upon the availability of the materials in nearby area so that transportation charges are reduced. If sand, gravel and crushed stone is available, a concrete gravity dam may be more suitable. If, however, coarse- and fine-grained soils are available an earth dam may be suitable.

4. Spillway Size and Location. The safe discharge of flood water through dam is very essential, and for that suitable site for spillway should be available. If the area is such that a large spillway capacity is required, an overflow concrete gravity dam should be preferred.

5. Roadway. If a roadway is to be passed over the top of the dam, an earth dam or gravity dam would be preferred.

6. Length and Height of Dam. If the length of the dam is very long and its height is low, an earth dam would be a better choice. If the length is small but height is more, gravity dam is preferred.

7. Life of Dam. Concrete or masonry gravity dams have very long life. Earth and rockfill dams have intermediate life. However, timber dams are adopted only for temporary storages.

Factors for selection of site for a dam

The following are the requisites of good sites for various types of dams:

1. Foundations. Suitable foundations should be available at the site selected for a particular type of dam. For gravity dams, sound rock is essential. For earth dams, any type of foundation is suitable with proper treatment.

2. Topography. (i) The river cross-section at the dam site should preferably have a narrow gorge to reduce the length of the dam.

(ii) A major portion of the dam should preferably be on high ground, as this would reduce the cost and facilitate drainage.

3. Site for Spillway. Good site for the location of a separate spillway is essential especially in the case of earth or rockfill dam.

4. Materials. Materials required for a particular type of dam should be available nearby, without requiring much of transportation. This would very much reduce the cost of construction.

5. Reservoir and Catchment Area. (i) The side should ensure adequate storage capacity of reservoir basin at a minimum cost.

(ii) The cost of land and property submerged in the water spread area should be minimum.

(iii) The reservoir site should be such that quantity of leakage through its side and bed is minimum. Reservoir site having the presence of higher permeable rocks reduce the water-tightness of the reservoir.

(iv) The geological conditions of the catchment area should be such that percolation losses are minimum and maximum run-off is obtained.

(v) The reservoir site should be such that it avoids or excludes water from those tributaries which carry a high percentage of silt in water.

6. Communication. It would be preferable to select a site which is connected by a road or rail link or can be conveniently connected to the site for transportation of cement, labor, machinery food and other equipment.

7. Locality. The surroundings near the site should preferably be healthy and free of mosquitoes etc., as labor and staff colonies have to be constructed near the spillway may be located at its middle.

Student assignment

Q. Comparison of earthen and gravity dams with respect to foundation, seepage, Construction and maintenance.