CIVIL ENGINEERING

For

UPSC Engineering Services Examination, GATE,

State Engineering Service Examination & Public Sector Examination.

(BHEL, NTPC, NHPC, DRDO, SAIL, HAL, BSNL, BPCL, NPCL, etc.)

IRRIGATION ENGINEERING

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CONTENT

Irrigation & Methods of Irrigation

CHAPTER

1

Irrigation & Methods of Irrigation

GENERAL UNDERSTANDING

Plants require water and air for their survival. Different types of plants require different quantities of water, and at different times, till they grow up completely. Water is normally supplied to these plants by nature through direct rain or through the flood waters of rivers. The supply of water by nature does not match the requirement of crops. Thus to control the nature, man discovered various methods by which the water can be stored during the periods of excess rainfall and to use that stored water during periods of less rainfall or no rainfall. The art of the science by which it is accomplished, is generally, termed as *irrigation*.

The following are the main concerns on irrigation:

(1) *How to apply ?* --- i.e., what should be the method of irrigation: Border flooding method, furrow irrigation method, sprinkler irrigation method, drip irrigation method etc.

(2) *How much to apply ?* --- i.e., how much moisture the soil can hold in its pores (moisture holding capacity of the soil).

(3) *When to apply ?* --- i.e., when has the soil moisture level depleted to 50 to 60% of moisture holding capacity, and when is the time to irrigate. In other words, what should be the frequency of irrigation.

DEFINITION OF IRRIGATION

Irrigation may be defined as the science of artificial application of water to the land, in accordance with the *crop requirements* throughout the *crop period* for full-fledged nourishment of the crops.

CROP YIELD AND PRODUCTIVITY

The crop yield from irrigation is expressed as quintal/ha or tonnes/ha . The productivity of the crop is expressed as crop yield per mm of water applied.

The following table gives the data for productivity

Crops	Water applied, Δ (in cm)	Yield (qn/ha)	Productivity (kg/ha/mm)
Rice	120	45	3.75
Jowar	50	45	9.0
Bajra	50	40	8.0
Maize	62.5	50	8.0
Wheat	40	50	12.5

Increase of yield/productivity can be achieved by following methods:

(1) Land shaping or land levelling
(2) Suitable crop rotations and crop planning.
(3) Using high yielding varieties of seeds.
(4) Using chemicals and fertilizers: inputs like NPK, FYM, green manure (nitrogen-rich crops)
(5) Methods of irrigation such as sprinkler, drip, furrow, etc.
(6) Lining of canals, distributaries and water courses by an economical lining material
(7) Drainage of irrigated land by surface and subsurface drainage.

ADVANTAGES & DISADVANTAGES OF IRRIGATION

Advantages of Irrigation

Direct advantages

(i) **Increase in food production :** Increase in crop yield due to irrigation leads to increase in food production, thus developing people as well as society.

(ii) **Protection against drought :** The provision of adequate irrigation facilities in any region ensures protection against failure of crops from famine or droughts.

(iii) **Revenue generation :** When regular supply of water is assured for irrigation the cultivators can grow certain superior or high priced crops (like cash crops) in place of inferior or low priced crops. Thus revenue is generated.

(iv) **Mixed cropping :** Means sowing of two or more crops together in the same field. This practice is followed so that if weather conditions are not favourable for one crop it may be suitable for other crop. But if irrigation facilities are made available, the need of mixed cropping is eliminated.

Indirect advantages

(i) **Power generation :** Major river valleys projects are usually planned to provide hydroelectric power together with irrigation. However relatively small quantity of hydroelectric power may also be generated at a small cost on projects which are primarily planned for irrigation.

(ii) **Transportation :** Most of the irrigation canals are provided with unsurfaced roads primarily for purposes of inspection and maintenance. These roads provide a good pathway to the local people. The network of irrigation canals can be used as the most economical means of transportation of goods as well as human beings.

(iii) **Ground water table :** In areas where irrigation facilities are provided, due to constant percolation of a portion of water flowing in the canals and also that is supplied to the field, the ground water storage is increased and consequently ground water table is raised.

(iv) **Employment :** During the constructions of irrigation works, employment is provided

Disadvantages of Irrigation

- Abundant supply of irrigation water tempts the cultivators to use more water than required. Excess water supplied to the field would percolate into the soil. Hence, due to constant percolation ground water table would be raised and will lead to water logging.
- The ground water can get polluted due to seepage of the nitrates into the ground water (applied to the soil as fertilizers).

Note: If irrigation water is used judiciously with proper scientific consideration then there won't be ill effects of irrigation.

TYPES OF IRRIGATION PROJECTS

Projects	Irrigation Potential (CCA)	Cost of project
Major irrigation projects	> 10000 ha	> 5 crores
Medium irrigation projects	2000 – 10000 ha	0.25 – 5 crore
Small irrigation proejcts	< 2000 ha	0.25 – 0.5 crore

TYPES OF IRRIGATION

(1) Surface irrigation (2) Subsurface irrigation

(1) Surface irrigation

- Surface irrigation is defined as the group of water application techniques where water is applied and distributed over the soil surface either by gravity or by pumping.
- More than 75% of irrigated lands in India is supplied water by surface irrigation methods.
- This method is best suited to soils with low to moderate infiltration capacities and to lands with relatively uniform terrain (slopes less than 3%)

Surface irrigation can be further classified into:

(i) Flow irrigation (ii) Lift irrigation.

(i) When the water is available at a higher level, and it is supplied to lower level, by the mere action of gravity, then it is called **Flow Irrigation.**

(ii) If the water is lifted up by some mechanical or manual means, such as by pumps, etc. and then supplied for irrigation, then it is called **Lift Irrigation**. Use of wells and tubewells for supplying irrigation water fall under this category.

Flow irrigation can be further sub-divided into:

(a) *Perennial irrigation* (b) *Flood irrigation.*

(a) Perennial Irrigation

- In perennial system of irrigation, constant and continuous water supply is assured to the crops in accordance with the requirements of the crop, throughout the *crop period.*
- In this system of irrigation, water is supplied through canal distribution system taking-off from above a weir or a reservoir.
- When irrigation is done by diverting the river runoff into the main canal by constructing a diversion weir or a barrage across the river, then it is called as **direct irrigation.** But if a dam is constructed across a river to store water during monsoons, so as to supply water in the off-taking channel during periods of low flow, then it is termed as **storage irrigation.**

(b) Flood Irrigation

- In this method of irrigation, soil is kept submerged and thoroughly flooded with water, so as to cause thorough saturation of the land.
- It is usually practised in delta regions where the river water level during flood is sufficiently high to supply water to the land by flow, or partly by flow and partly by lift.
- This system of irrigation is also called *uncontrolled irrigation or inundation irrigation.*

(2) Sub-surface Irrigation

In this type of irrigation, water does not actually wet the soil surface rather it flows underground and nourishes the plant roots by capillarity.

It may be divided into the following two types

(i) Natural sub-irrigation (ii) Artificial sub-irrigation.

(i) Natural sub-irrigation

- Leakage water from channels, goes underground and during passage through the sub-soil, it may irrigate crops, sown on lower lands, by capillarity. Sometimes, leakage causes the water-table to rise up, which helps in irrigation of crops by capillarity.
- When underground irrigation is achieved, simply by natural processes without any additional extra efforts, it is called natural sub-irrigation.

(ii) Artificial sub-irrigation

- When a system of open jointed drains is artificially laid below the soil, so as to supply water to the crops by capillarity, then it is known as artificial sub-irrigation.
- It is a very costly process and hence, adopted on a very small scale.
- It may be recommended only in some special cases with favourable soil conditions and for cash crops of very high return.

ADVANTAGES OF LIFT IRRIGATION OVER FLOW IRRIGATION

(1) Lift irrigation can be installed directly on the farm (mostly it is installed centrally) at any time with a small investment. Thus the length of the field channels (water courses) is less and can be lined with an economic lining material locally available.

(2) Water can be applied to the required depth when it is time to irrigate.

(3) There are no pollution and evaporation losses (due to pumping from nearby tube wells, wells) Subsoil water table can be controlled and prevents water logging, soil salinity and alkalinity

(4) High irrigation efficiency and improved methods of irrigation like perforated pipe, sprinkler and drip can be practised.

(5) Cash crops can be grown, which pay back for the high initial investment made.

(6) Irrigation is possible even during a year of drought.

(7) Isolated patches of land can be easily irrigated where the canal water cannot reach.

(8) High yields can be obtained with good water management and fertilizer application.

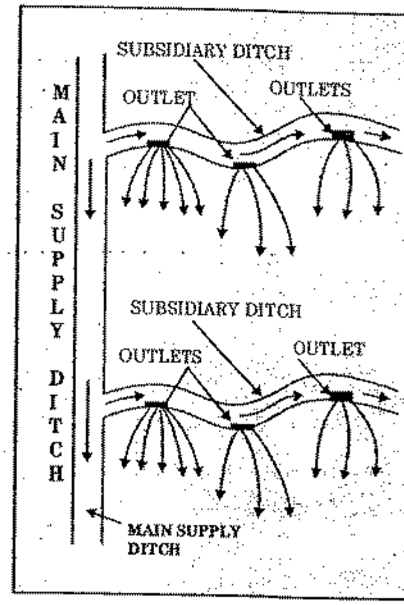
METHODS OF IRRIGATION

There are various ways in which the irrigation water can be applied to the fields.

Following are the main classifications :

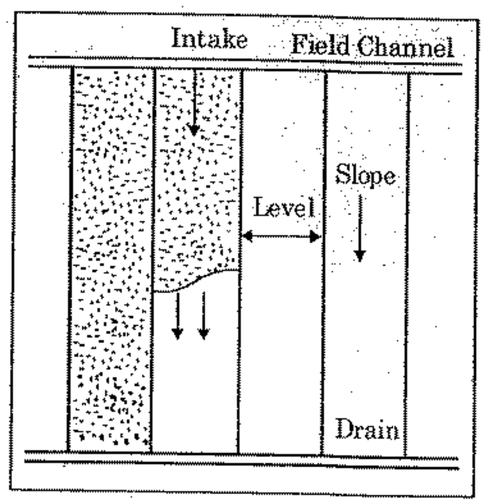
(1) Free flooding
(2) Border flooding
(3) Check flooding
(4) Basin flooding
(5) Furrow method or furrow irrigation
(6) Sprinkler irrigation method
(7) Drip irrigation method

(1) Free flooding or Ordinary flooding



- In this method, ditches are excavated in the field, and they may be either on the contour or up and down the slope.
- Water from these ditches, flows across the field.
- After the water leaves the ditches, no attempt is made to control the flow by means of levees, etc.
- It is sometimes called wild flooding (as the movement of water is not restricted).
- The initial cost of land preparation is low but labour requirements are usually high
- Water application efficiency is also low.
- Wild flooding is most suitable for close growing crops, pastures, etc., particularly where the land is steep
- This method may be used on rolling land (topography irregular) where borders, checks, basins and furrows are not feasible.

(2) Border flooding



- In this method, the land is divided into a no. of strips, separated by low levees called borders.
- Borders are long, uniformly graded strips of lands, separated by earth bunds. These bunds are to guide the flow of water down the field.
- The land areas confined in each strip is of the order of 10 to 20 metres in width, and 100 to 400 metres in length.
- To prevent water from concentrating on either side of the border, the land should be levelled perpendicular to the flow.
- Water is allowed to flow from the supply ditch into each strip. The water flows slowly towards the lower end, and it infiltrates into the soil as it advances.
- When the advancing water reaches the lower end of the strip, the supply of water to the strip is turned off.
- The supply ditch may either be in the form of an earthen channel or a lined channel or an underground concrete pipe having risers at intervals.
- The size of the supply ditch depends upon the infiltration rate of the soil, and the width of the

border strip.

- Coarse textured soils with high infiltration rates will require high discharge rate and therefore larger supply ditch, in order to spread water over the entire strip rapidly, and to avoid excessive losses due to deep percolation at the upper reaches whereas fine textured soils with low infiltration rates, require smaller ditches to avoid excessive losses due to run off at the lower reaches.
- This method is the most popular

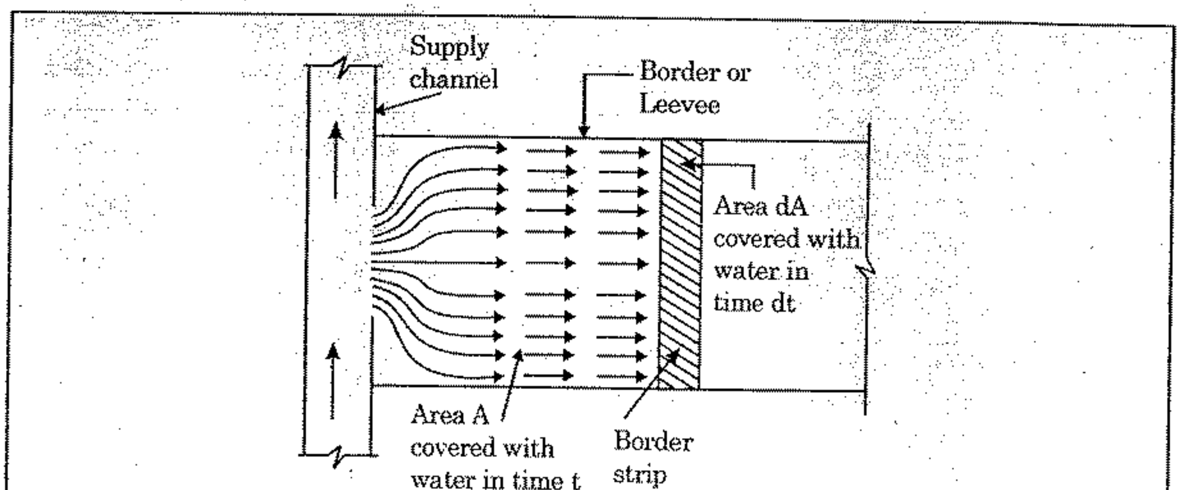
Relationship to obtain the maximum area irrigated with a supply ditcb

A relationship between the discharge through the supply ditch (Q), the average depth of water flowing over the strip (y), the rate of infiltration of the soil (f), the area of the land irrigated (A), and the approximate time required to cover the given area with water (t), is given by the equation

$$\boxed{t = 2.3 \cdot \frac{y}{f} \cdot \log_{10}\left(\frac{Q}{Q - fA}\right)} \quad \text{.... (i)}$$

where, Q = Discharge through the supply ditch ; A = Area of land strip to be irrigated
y = Depth of water flowing over the border strip ; f = Rate of infiltration of soil
t = Time required to cover the given area A.

Proof : If we consider a small area (dA) of the border strip of area A and assume that in time dt, water advances over this area dA. The volume of water that flows to cover this area would be $y \cdot dA$, because y is the water depth over this area. Also, during the same time dt, the volume of water that percolates into the soil over the area A would be $f.A\ dt$. The total quantity of water supplied to the strip during time dt would be $Q.\ dt$, and also equal to $y \cdot dA + f \cdot A\ dt$.



$$\therefore \quad Q.dt = y \cdot dA + f \cdot A \cdot dt \quad \text{... (ii)}$$

or

$$dt = \left(\frac{y.dA}{Q - f.A}\right) \quad \text{... (iii)}$$

Considering y, f, and Q as constants, and integrating the above equation, we get

$$\int dt = y\int \frac{dA}{Q - fA}$$

$$t = -\frac{y}{f} ln(Q - fA) + C$$

at $t = 0,\ A = 0$

$\Rightarrow$ $$C = \frac{y}{f} ln Q$$

$\Rightarrow$ $$t = -\frac{y}{f} ln(Q - fA) + \frac{y}{f} ln Q$$

$\Rightarrow$ $$t = \frac{y}{f} ln\left(\frac{Q}{Q - fA}\right)$$

or $$\boxed{t = 2.303\,\frac{y}{f}\log_{10}\left(\frac{Q}{Q - fA}\right)}$$

This equation can be further written as

$$\frac{t.f}{2.3y} = \log_{10}\left(\frac{Q}{Q - fA}\right)$$

Now, let $$\frac{t.f}{2.3y} = x \quad \Rightarrow \quad x = \log_{10}\left(\frac{Q}{Q - fA}\right)$$

$\Rightarrow$ $$10^x = \frac{Q}{Q - fA} \quad \Rightarrow \quad Q.10^x - f.A.10^x = Q$$

$\Rightarrow$ $$Q\ (10^x - 1) = f.A.10^x$$

or $$A = \frac{Q(10^x - 1)}{f.10^x}$$

The maximum value of $\frac{10^x - 1}{10^x} \approx 1$,

So, $$\boxed{A_{max} = \frac{Q}{f}} \qquad \ldots (iv)$$

This equation enables us to determine the maximum area that can be irrigated with a supply ditch of discharge Q and soil having infiltration capacity f.

In other words, discharge per unit area of the border strip should be varied according to the infiltration capacity of the soil in order to avoid the loss of water.

Example 1

Determine the time required to irrigate a strip of land of 0.04 hectares in area from a tube well with a discharge of 0.02 cumec. The infiltration capacity of the soil may be taken as 5 cm/hr, and the average depth of flow on the field as 10 cm.

Also determine the maximum area that can be irrigated from this tube well.

Sol. Given,

Area of the strip, $A = 0.04$ hectares $= 0.04 \times 10^4\ m^2 = 400\ m^2$.

Discharge, $Q = 0.02$ cumecs $= 0.02\ m^3/sec = 0.02 \times 60 \times 60\ m^3/hr = 72\ m^3/hr$.

Infiltration capacity of soil, $f = 5\ cm/hr = \frac{5}{100}\ m/hr = 0.05\ m/hr.$;

Average depth of flow on the field, $y = 10\ cm = 0.1\ m$.

Approximate time required to irrigate a strip of land, $t = 2.303\ \frac{y}{f}\log_{10}\left(\frac{Q}{Q - fA}\right)$

or

$$t = 2.303 \times \frac{0.10}{0.05} \times \log_{10}\left(\frac{72}{72 - 0.05 \times 400}\right)$$

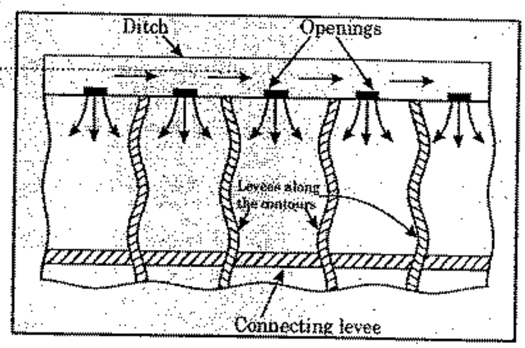
$$= 2.303 \times 2\ \log_{10}\ (72/52) = 0.651\ hr = 39.06\ \text{minutes.}$$

Maximum area that can be irrigated,

$$A_{max} = \frac{Q}{f} = \frac{72}{0.05}\ m^2 = 1440\ m^2 = \frac{1440}{10^4}\ ha = 0.144\ ha.$$

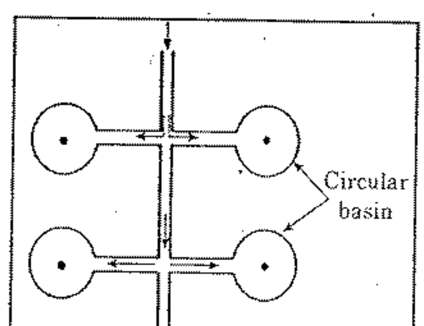
Note : *So, surface flow will stop after the irrigation of this area and deep percolation will start.*

(3) Check flooding



- Check flooding is similar to ordinary flooding except that the water is controlled by surrounding the check area with low and flat levees.
- In check flooding, each compartment is filled with water at a fairly high rate and allowed to stand until the water infiltrates.
- Close growing crops such as jowar or paddy are preferred for this method of irrigation.
- Deep homogenous loam or clay soils with medium infiltration rates are preferred for this method.
- This method is suitable for both more permeable and less permeable soils.
- Water can be quickly spread in case of highly permeable soils thus reducing percolation losses. Water can also be held for a longer time in case of less permeable soils for assuring adequate penetration.

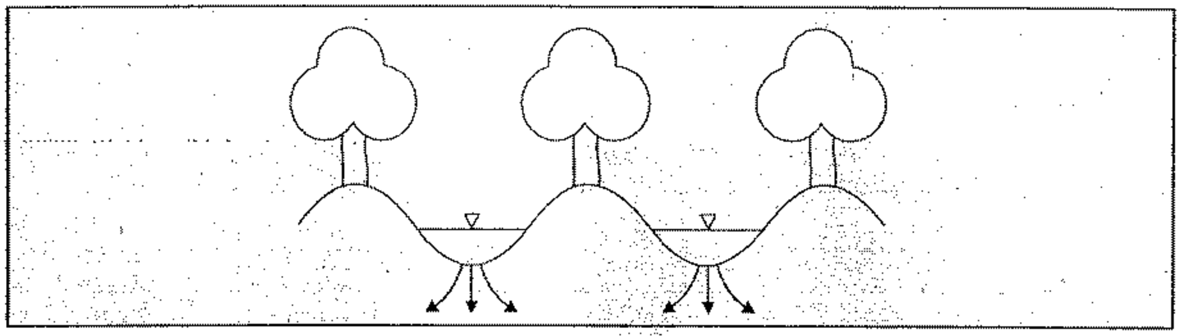
(4) Basin flooding



- This method is a special type of check flooding & adoptd specially for orchard trees.
- One or more trees are generally placed in the basin & surface is flooded as in check method
- Shape of the basin can be square, rectangular, circular or it may be irregular.
- Flatter the land surface, easier it is to construct the basins

- Water is supplied to the basins from a supply channel through small field channels connecting the basins with the supply channel.
- Coarse sands are not suitable for basin irrigation because of high percolation losses.
- The size and shape of basins are mainly determined by the land slope, the soil type, the available stream size, the required depth of irrigation water to the applied.
- If depth of applied water is large, then the size of the basin should also be large and vice-versa to obtain good distribution of water over the basin area.

(5) Furrow method or furrow irrigation



- In this method of irrigation, water is applied to the land to be irrigated by a series of furrows.
- Furrows are small, parallel channels, made to carry water for irrigating the crops.
- The water flowing in the furrows infiltrates into the soil and spreads laterally to irrigate the land between the furrows.
- The crops are usually grown on the ridges between the furrows.
- In the furrow method, only a part of the land varying from one-half to one-fifth is wetted which results in reduced evaporation losses.
- Furrow irrigation is suitable for a wide range of soil types, crops (especially row crops) and land slopes.
- Furrow irrigation is preferred on uniformly flat or gentle slopes which should not exceed 0.5%
- Furrows can also be level and thus very similar to long narrow basin.
- In this method, the requirements of labour for land preparation and irrigation are very much reduced as compared to the various methods of irrigation by flooding.

Note : *Corrugations are in general similar to furrows but furrows are the channels of relatively larger cross section compared to corrugations.*

(6) Sprinkler irrigation method

- In this method, the irrigation water is applied to the land in the form of spray, somewhat as in ordinary rain through a network of pipes and pumps.
- It is also sometimes known as *overhead irrigation* (over head system is required).
- The sprinkler irrigation can be used for all the crops except rice and jute and for almost all the soils except very heavy soils with very low infiltration rates.
- Best suited for very *light soils* as deep percolation losses are avoided.
- This system is flexible to suit undulating topography and hence land levelling is not necessary.

- This method is used mainly by cultivators of tea, coffee and vegetables in our country.

Note : (i) *For rice and jute, standing water is required.*

(ii) *Light soils are sandy & silty with very little clay. Generally easy to work, warm up quickly, dry out rapidly.*

Advantages of Sprinkler Irrigation

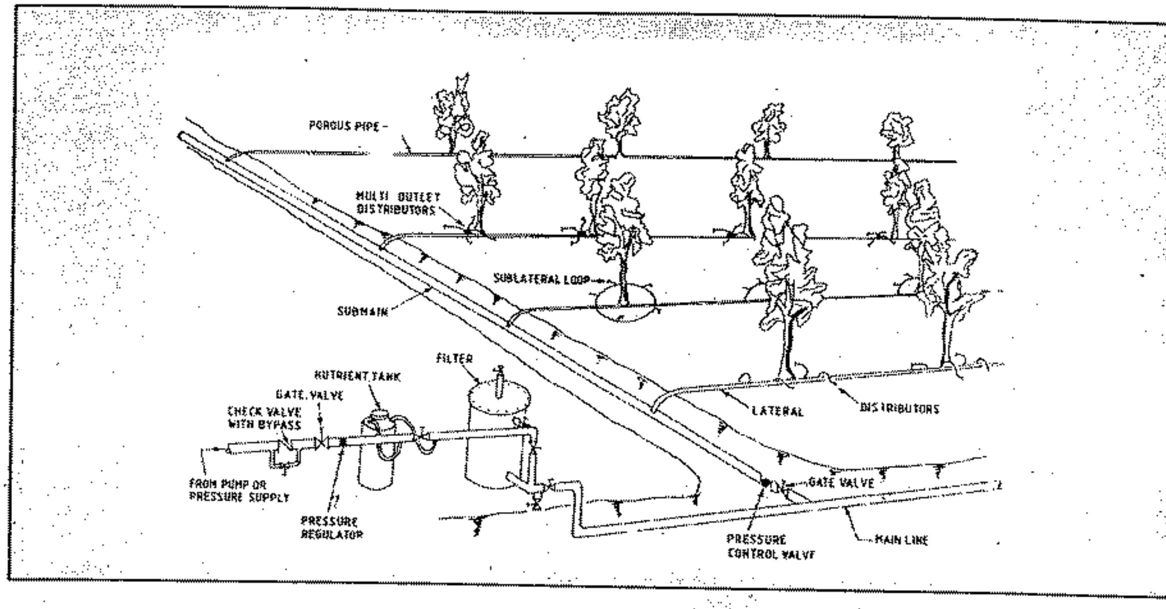
- It can be efficiently used for a wide range of topographic conditions, soils and crops.
- With use of sprinkler irrigation, erosion of soil can be controlled (as surface runoff is eliminated)
- Uniform application of water is possible with sprinkler irrigation.
- In this method, a better control on irrigation can be enforced and also light irrigation is possible which is required for seedlings and plants which are very young.
- Labour cost is reduced as no land preparations are required.
- Does not require borders, field channels etc. and hence more land is available for cropping.
- The time and amount of application of fertilizer can be better controlled in sprinkler irrigation.
- About 80% water application efficiency is possible (water application efficiency is high).
- The sprinkler irrigation method is especially adaptable to more humid regions
- Can be used when there is water scarcity because evaporation and percolation losses are less.

Disadvantages of Sprinkler Irrigation

- Under windy conditions and high temperatures, the water distribution and water application efficiencies are low.
- Saline water may cause leaf burns in many crops
- System is costly to install, operate and maintain.
- Continuous supply of power is generally required for operating the system.
- Corners remain under-irrigated and therefore uniformity of application is to some extent affected.

(7) Drip irrigation method

- One of the latest method of irrigation which is becoming increasingly popular in areas with acute scarcity of irrigation water and salt problems
- In this method, water and fertilizer is slowly and directly applied to the root zone of the plants, in order to minimize the losses due to evaporation and percolation.
- It is also known as trickle irrigation
- This is achieved with the help of specially designed emitters and drippers.
- Centrifugal pump is best suited for this method
- Drip irrigation is best suited for row crops and orchards such as tomatoes, grapes, corn, citrus melons, fruits, cauliflower, cabbage and turnips.



Advantages of drip irrigation

- Water requirement is very less (since water is delivered directly to the plant)
- Evaporation loss and wind loss are almost negligible
- There is least wetting of soil surface due to direct water supply to the plants
- Suitable for all types of soils, especially for coarse textured soil.
- Highest rate of vegetative growth
- Land levelling is not required.
- Roots stay within the moist zone
- Reduction of deleterious effects of salts.
- Low labour requirements, as most drip systems are permanent setups.
- Field operations are easier to manage
- There is no loss at the edge of the fields as can occur through wind drift of sprinkler systems or run off from surface system.

Disadvantages of drip irrigation

- Does not offer frost protection (as sprinklers do)
- Plastic drip-lines and sub-mains may be attacked by rodents and small animals.
- Requires regular flushing (to clear off the dirt collected near the ends of the drip lines) and supervision.
- High skill is required in the design, installation, operation and maintenance.

Note: Sprinkler and drip irrigation systems falls under a category known as pressurized irrigation system.

OBJECTIVE QUESTIONS

1. **Assertion (A):** Sprinkler method of irrigation has a higher water application/use efficiency.
   Reason (R): Sprinkler system causes less interference in cultivation and other farming operations.

2. A sprinkler irrigation system is suitable when
   (a) the land gradient is steep and the soil is easily erodible
   (b) the soil is having low permeability
   (c) the water table is low
   (d) the crops to be grown have deep roots

3. An Indian irrigation project designed to serve a command of more than 2000 hectares and up to 10000 hectares, is known as a :
   (a) major irrigation project
   (b) minor irrigation project
   (c) medium irrigation project
   (d) none of them, since irrigation projects are classified on the basis of their costs.

4. The method of growing crops on ridges, running on the sides of water ditches, is known as
   (a) flood irrigation (b) furrow irrigation
   (c) check irrigation (d) none of them

5. In a field under furrow irrigation, 'furrows' are referred to represent
   (a) ridges on which crops are grown (b) narrow ditches carrying water
   (c) both (a) and (b) (d) neither (a) nor (b)

6. Pinpoint the correct statement :
   (a) irrigation helps in adopting mixed cropping
   (b) 'mixed cropping' means sowing of a different crop after a particular crop has been grown
   (c) over-irrigation may lead to saving in fertilizers
   (d) irrigation helps in avoiding mixed cropping.

7. 'Flood irrigation' method of irrigation fields, works best on :
   (a) level or gently rolling terrain (b) steeply rolling terrain
   (c) both (a) and (b) (d) none of the above

8. In a mildly water scarce area, the drip irrigation could be preferred for growing:
   (a) wheat (b) fodder (c) rice (d) fruits & vegetables

9. Border method of irrigation is well suited to
   (a) Soil having infiltration rates that are neither low nor high
   (b) coarse sandy soils having high infiltration rates
   (c) clay soils having very low infiltration rates
   (d) soils having salts which require frequent leaching.

10. In an irrigation system, water was delivered to the field in ditches spaced about 30 m apart, and was allowed to seep into the ground to maintain the water table at such a height that the water is available to the crops through the capillary fringe. This method of irrigation is called
    (a) trickle irrigation (b) furrow irrigation (c) border irrigation (d) sub irrigation

11. In contour border irrigation method
   (a) the supply ditch runs along the contour
   (b) the drainage channel runs along the contour
   (c) the border strips are on the approximate contour and have uniform longitudinal gradient
   (d) the border strips are normal to the contour and level across the strip.

12. In border method of irrigation, the flow along the border is a case of
   (a) spatially varied, unsteady, open channel flow with decreasing discharges
   (b) steady, spatially varied open channel flow with decreasing discharges
   (c) unsteady, gradually varied, open channel flow
   (d) unsteady, uniform, open channel flow.

13. In an irrigation system, land was divided into a large no. of smaller size unit areas, having fairly level surface, by bunds & cross ridges. The basins thus created were filled with water to the desired depth & the water was retained for some time. This method of irrigation is known as
   (a) border method (b) check basin method (c) sub-irrigation (d) contour irrigation

14. Identify the *incorrect* statement pertaining to furrow irrigation :
   (a) Furrow irrigation is suitable for soils that have a very high infiltration rate
   (b) The furrow method can be adopted to lands having a wide range of natural slopes
   (c) Compared to check basin method, considerably less land is wasted in furrow irrigation
   (d) Evaporation losses in furrow irrigation is relatively small when compared to the check-basin method of irrigation.

15. For growing irrigated paddy, the ideal water application method is
   (a) furrow irrigation (b) check basin irrigation
   (c) border irrigation (d) sprinkler irrigation

16. The maximum application rate by sprinklers is limited by
   (a) the infiltration capacity of the soil (b) the prevailing wind velocity
   (c) the quantity of water available (d) the prevailing humidity and radiation.

17. Which of the statements given below is *not* correct?
   In a trickle irrigation system
   (a) deep percolation and runoff are practically eliminated
   (b) the water application efficiency is very high
   (c) the evapotranspiration is practically eliminated
   (d) the fertiliser can be applied economically along with the irrigation water.

18. Which of the following statements pertaining to sprinkler irrigaton is *not* correct?
   (a) No extra cost of land preparation is involved in sprinkler irrigation
   (b) Excessive soil erosion is initiated by sprinkler irrigation
   (c) Sprinklers can be used for the application of liquid fertilisers also
   (d) Sprinkler irrigation is particularly advantageous in hilly terrains.

ANSWERS

1.	(c)	5.	(b)	9.	(a)	13.	(b)	17.	(c)
2.	(a)	6.	(d)	10.	(d)	14.	(a)	18.	(b)
3.	(c)	7.	(a)	11.	(c)	15.	(b)		
4.	(b)	8.	(d)	12.	(a)	16.	(a)		

Soil-Moisture-Plant Relationships

CHAPTER

2

Soil-Moisture - Plant Relationships

SOIL MOISTURE

- Water added to a soil mass during irrigation is held in the pores of the soil and is termed as *soil moisture.*
- Soil moisture causes the soil to appear wet or even damp depending upon the amount of moisture present in the soil mass.
- Soil pores which do not contain liquid water remain filled with air or water vapour.

BASIC PHYSICAL PROPERTIES OF SOILS

- Soil provides the necessary medium for water to be used by plants through their roots that are present in the same medium.
- Water acts as a medium to carry large amounts of nutrients essential for the growth of a plant.
- The rate of entry of water into the soil, its retention and then its movement and availability to plant roots are physical phenomena that contribute to the growth of vegetation.

Note: Hence, there is need to understand the physical aspects of soil in relation to water for efficient management of irrigated agriculture.

WATER HOLDING CAPACITY OF SOIL

Water Holding capacity of soil is one of the dominant factors influencing irrigation. The water holding capacity of a soil mainly depends on its porosity.

Porosity, $n = \frac{V_V}{V_T}$ where, V_V = Volume of pores, V_T = total volume n = porosity

In general there are two types of soil pores viz..

(1) Capillary or small pores (2) Non-capillary or large pores.

- The capillary pores hold a large amount of water held by the soil at saturation due to capillarity and prevent it from being drained off under gravity.
- On the other hand, the non-capillary pores do not hold water tightly and hence a large amount of water held by the soil at saturation is drained off under gravity.
- Capillary pores induce greater water holding capacity while non capillary pores induce drainage and aeration.

- The relative magnitudes of these types of pores in a soil depend on its texture and structure.
- A sandy soil has more non-capillary pores which result in better drainage and aeration but low water holding capacity.
- On the other hand clayey soil has more capillary pores which result in better water holding capacity but poor drainage and aeration.
- The water held by the soil is extracted by the roots of the plants for their growth which is resisted by some forces. These forces are more in clayey soils than in sandy soils, so it is difficult for roots of the plants to extract water easily from the clayey soils compared to the sandy soils.
- An ideal soil for irrigation has its pore spaces equally divided between capillary and non-capillary pores.

CLASSIFICATION OF SOIL WATER

Soil water may be classified in the following three categories.

(1) Gravitational water (2) Capillary water (3) Hygroscopic water.

(1) Gravitational water

- It is that water which is not held by soil but drains out freely under the influence of gravity.
- When the water is added to the soil during irrigation, the water content of the soil is raised to a state of saturation
- At this point, the soil pores are completely filled with water and the soil contains the maximum possible water content, which thus constitutes the upper limit of the gravitational water.
- Some of this water is held very loosely by the soil & readily moves downward under the pull of gravity, unless prevented by an impermeable barrier such as hard pan or a high water table.

(2) Capillary water

- It is that the water which is retained in the soil after the gravitational water has drained off from the soil
- Capillary water is held in the soil by surface tension as a continuous film around the soil particles and in the capillary pores between the soils particles.
- The capillary water is held in the soil against the force of gravity.
- The plant roots gradually absorb the capillary water which thus constitutes the principal source of water for plant growth.
- The capillary water is also designated as available water.
- The water content in the soil corresponding to permanent wilting point constitutes the lower limit of the capillary water.

(3) Hygroscopic water

- It is that water which is adsorbed by the particles of dry soil from the atmosphere and is held as a very thin film on the surface of the soil particles due to adhesion or attraction between surface of particles and water molecules.
- Below the permanent wilting point, the soil contains only *hygroscopic water*.
- It cannot be removed easily from the soil particles as it is held with a considerable force.

- Hygroscopic water is in general not available for plant use.

Note : The soil water may also be classified on the basis of its availability to the plant roots or otherwise as unavailable, available and gravitational water.

SOIL MOISTURE TENSION

- Soil moisture tension is defined as the force per unit area that must be exerted in order to extract water from the soil. In other words, it is a measure of the tenacity with which water is retained in the soil.
- Soil moisture tension is usually expressed in terms of atmospheres.
- Several other terms such as capillary potential, capillary tension, soil pull, and the force of suction are often used with the same meaning as that of soil moisture tension.
- For a soil of given texture and structure, soil moisture tension is inversely proportional to its moisture content.
- A knowledge of the amount of moisture held by the soil at various tensions is required in order to determine the amount of moisture that would be available to the plants and also the amount of water that must be used for irrigation.

SOIL MOISTURE STRESS

- Soil moisture stress is defined as the sum of the soil moisture tension and osmotic pressure of soil solution.
- In many irrigated soils, the soil solution contains an appreciable amount of salts.
- Salts in soil water increase the force that must be exerted to extract water and thus affect the amount of water available to plants.
- The increase in the force (or tension) caused by salts is from *osmotic pressure.* If two salt solutions differing in concentration are separated by an impermeable membrane such as a cell membrane in a plant root, water moves from the solution of lower concentration to the one of higher concentration. The force with which water moves across such a membrane is called ***osmotic pressure*** and is measured in atmospheres.
- The osmotic pressure developed by the soil solution retards the uptake of water by plants.
- Plants growth is a function of both soil moisture tension as well as osmotic pressure (i.e. a function of soil moisture stress).
- Thus, for good growth of plants in soils having appreciable salts, the osmotic pressure of the soil solution must be maintained as low as possible by controlled *leaching* so that in the root zone soil moisture stress is maintained in range that will provide adequate moisture to the plants.

SOIL MOISTURE CONSTANTS

(1) Saturation Capacity

- Saturation capacity is defined as the total water content of a soil when all the pores of the soil are filled with water.
- This is also known as the maximum water holding capacity of the soil.

- At saturation capacity, soil moisture tension is almost equal to zero as it is equal to the surface tension at free water surface.

(2) Field Capacity

- Field capacity is defined as the maximum amount of moisture which can be held by a soil against gravity. Thus immediately after the gravitational water has drained off from a Saturated soil, the moisture content held by the soil is the field capacity of the soil.
- The field capacity is usually expressed as the weight of the maximum amount of moisture held by the soil against gravity per unit weight of the dry soil and is given as percentage.
- At field capacity the large or non-capillary pores of the soil are filled with air and the small or capillary pores are filled with water.
- Field capacity is the upper limit of the capillary water or the moisture content available to the plant roots.
- In coarse textured sandy soils, the field capacity may usually be achieved in about 1 to 3 days after the soil has been thoroughly wetted by irrigation.
- In medium textured sandy silt and sandy clay soils about 4 to 8 days may be required for the moisture content to reach field capacity.
- In fine textured soils containing large proportions of clay, somewhat longer periods are required for the moisture content to reach field capacity.
- The soil moisture tension at field capacity ranges between 1/10 to 1/3 atmospheres.

(3) Permanent wilting point

- Permanent wilting point is the moisture content at which the films of water around the soil particles are held so tightly that the plant roots cannot extract enough moisture at sufficiently rapid rate to satisfy transpiration requirements thus resulting in the wilting of the plants.
- The permanent wilting point is usually expressed as the weight of the moisture held by the soil per unit weight of the dry soil when the plants are permanently wilted.
- Permanent wilting points differ widely for different soils but have approximately the same values for different plants grown on the same soil.
- The value of permanent wilting percentage may be as low as 2% for light sandy soils and it may be as high as 30% for heavy clay soils.
- The soil moisture tension of a soil at the permanent wilting point ranges from 7 to 32 atmospheres depending on soil texture, kind and condition of the plants etc.

(4) Available moisture

- The difference in moisture content of the soil between the field capacity and the permanent wilting point is termed as the available moisture.
- This represents the moisture which is stored in the soil in the form of capillary water for being used subsequently by the plants.

(5) Readily available moisture

- It is that portion of the available moisture which is most easily extracted by plant roots.
- Only about 75% of the available moisture is usually readily available

(6) Moisture equivalent

- Moisture equivalent is defined as the percentage of moisture retained in an initially saturated sample of soil 10mm thick after being subjected to a centrifugal force of 1000 times gravity for a period of 30 minutes.
- The moisture equivalent of a soil can be quickly determined in a laboratory and it is used as an approximate measure of field capacity.

DEPTH OF WATER HELD BY SOIL IN ROOT ZONE AND AVAILABLE TO PLANTS

The field capacity of water is expressed as the ratio of the weight of water contained in the soil to the weight of the dry soil retaining that water

$$\text{Field Capacity} = \frac{\text{Wt. of water retained in a certain vol. of soil}}{\text{Wt. of the same volume of dry soil}} \times 100$$

If we consider 1 m^2 area of soil and d metre depth of root zone, then the volume. of soil is $d \times 1 = d$ cubic metres. If the dry unit wt. of soil is γ_d kN/m^3 then the Wt. of d cubic metres of soil is $\gamma_d \cdot d$ kN. If F is the field capacity, then

$$F = \frac{\text{Wt. of water retained in unit area of soil}}{\gamma_d . d}$$

Wt. of water retained in unit area of soil $= \gamma_d \cdot d \cdot F$

$\therefore$ Vol. of water stored in unit area of soil $= \dfrac{\gamma_d \cdot d \cdot F}{\gamma_w}$

or, Total water storage capacity of soil in (m depth of water) $= \dfrac{\gamma_d \cdot d \cdot F}{\gamma_w}$

where, F = the field capacity moisture content ; d = depth of root zone in m

γ_w = the unit wt. of water ; γ_d = the dry unit wt. of soil.

Depth of water stored in the root zone in filling the soil upto field capacity $= \dfrac{\gamma_d \cdot d \cdot F}{\gamma_w}$ metres.

Note: *γ_d is the unit wt. of the dried soil sample and not of the soil solids. Hence, it may sometimes be called as apparent unit wt.*

Example 1

The depth of moisture in root zone at field capacity and permanent wilting point per m depth of soil are 0.5 m/m and 0.2 m/m respectively. Compute the field capacity and permanent wilting point. Take dry weight of soil as 13.73 kN/m³.

Sol. Given, Depth of moisture in root zone at F.C. per metre depth of soil, $d_1 = 0.5$ m

Depth of moisture in root zone at P.W.P per metre depth of soil, $d_2 = 0.2$ m

Take depth of soil = 1m

$$\therefore \text{Field capacity} = \frac{\text{Weight of water retained in the root zone corresponding to F.C}}{\text{Weight of dry soil}}$$

$$= \frac{\gamma_w \times d_w}{\gamma_d .1} = \frac{9.81 \times 0.5}{13.73} = 0.3572 = 35.72\%$$

$$\text{Permanent wilting point} = \frac{\text{Weight of water retained in the root zone corresponding to PWP}}{\text{Weight of dry soil}}$$

$$= \frac{9.81 \times 0.2}{13.73} = 0.1429 = 14.29\%$$

Example 2

A loam soil has a field capacity of 25 percent and wilting coefficient of 10%. The dry unit weight of soil is 1.5g/cc. If the root zone depth is 60 cm, determine the storage capacity of the soil. Irrigation water is applied when moisture content falls to 15 percent. If the water application efficiency is 75%, determine the water depth required to be applied in the field.

Sol. Given, Field capacity, FC = 25% ; Wilting coefficient, WC = 10%

Dry unit wt. of soil, γ_d = 1.5 gm/cc ; Root zone depth, d = 60 cm

Water application efficiency = 75%

Moisture storage capacity of soil in the root zone depth

$$= \frac{\gamma_d}{\gamma_w} \times d \times (FC - WC) = \frac{1.5}{1} \times 60 \times (0.25 - 0.10) = 13.5 \text{ cm}$$

Now, when moisture content falls to 15%, the deficiency of water depth created

$$= \frac{\gamma_d}{\gamma_w} \times d \times (0.25 - 0.15) = \frac{1.5}{1} \times 60 \times (0.25 - 0.15) = 9 \text{ cm}$$

Hence, the net irrigation requirement = 9 cm

$$\therefore \quad \text{Water depth required in the field} = \frac{\text{Net irrigation requirement}}{\text{Water application efficiency}} = \frac{9}{0.75} = 12 \text{ cm}$$

Example 3

The following data pertains to healthy growth of a crop:

(i) Field capacity of soil = 30% (ii) Permanent wilting point : 11%

(iii) Density of soil = 1300 kg/m³ (iv) Effective depth of root zone = 700 mm

(v) Daily consumptive use of water for the given crop = 12 mm

For healthy growth moisture content must not fall below 25% of the water holding capacity between the field capacity and the permanent wilting point. Determine the watering interval in days.

Sol. Given,

Field capacity of soil (FC) = 30% ; Permanent wilting percentage (PWP) = 11%

Density of soil, (γ_d) = 1300 kg/m³; Effective depth of root zone, (d) = 700 mm

Daily consumptive use of water = 12 mm

Determine water interval (T) in days

Key : Moisture content does not fall below 25 %

Since for healthy growth, moisture content should not fall below 25% of the water holding capacity between the F.C and P.W.P.

Therefore, readily available moisture $= 75\%$ of $(F.C - PWP) = \frac{75}{100} \times (0.30 - .11) = 0.1425$

So, if the readily available moisture is lost completely then we have to irrigate the crop

$$\text{Depth of water lost} = \frac{\gamma_d}{\gamma_w} \times d \times (\text{readily available moisture}) = \frac{1300}{1000} \times 0.7 \times 0.1425 \; (\gamma_w = 1000 \text{ kg/m}^3)$$

$$= 129.7 \approx 130 \text{ mm}$$

$\because$ Daily consumptive use = 12 mm

$\therefore$ Watering interval = Depth of water lost so as to start next watering / Daily consumptive use

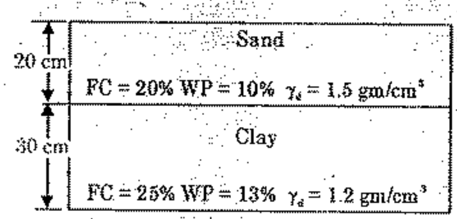
$$= \frac{130}{12} = 10.83 \text{ days.}$$

So, watering should be applied after every 10 days (As the value has to be rounded off to the nearest closer integer. Although 11 is the closest integer of 10.83, but irrigation should be done before wilting sets up).

Example 4

A soil 50 cm deep over rock has two horizons, the first being a fine sandy loam 20 cm thick and the second clay loam 30 cm thick. The field capacity, wilting point and weight/volume for the first horizon are 20%, 10% and 1.5 gm/cm³ respectively. The corresponding values for the second horizon are 25%, 13% and 1.2 gm/cm³. Determine the available moisture storage capacity of the soil profile. If consumptive use requirements of a crop in a particular season is 0.5 mm/day and the soil is initially at field capacity, how long will the crop survive without irrigation.

Sol.



$$\text{Moisture holding capacity of first horizon} = \frac{\gamma_d . d}{\gamma_w}(F.C - WP) = \frac{1.5}{1} \times 20(0.2 - 0.1) = 3 \text{ cm}$$

$$\text{Moisture holding capacity of second borizon} = \frac{1.2}{1} \times 30(0.25 - 0.13) = 4.32 \text{ cm}$$

Hence, Total max available moisture = 3 + 4.32 = 7.32 cm

Consumptive use requirement of crop in a particular season = 0.5 mm/day = 0.05 cm/day

Max. no. of days in which the entire stored moisture will be utilised

= Absolute no. of days for which crop will survive

$$= \text{Moisture storage capacity of the soil consumptive use} = \frac{7.32}{0.05} = 146.4 \text{ days}$$

Example 5

A sandy loam soil holds water at 140mm/m depth between field capacity and permanent wilting point. The root depth of the crop is 30 cm and the allowable depletion of water is 35%. The daily water use by the crop is 5 mm/day. The area to be irrigated is 60 ha and water can be delivered at 28 Lps. The surface irrigation application efficiency is 40%. There are no rainfall and ground water contribution.Determine:

(i) Allowable depletion depth between irrigations
(ii) Frequency of irrigation
(iii) Net application depth of water
(iv) Volume of water required
(v) Time to irrigate 4 ha plot.

Sol. Given, Moisture holding capacity of soil (per metre depth of soil) = 140 mm

Root depth of the crop = 30 cm ; Allowable depletion of water = 35%

Daily water use by the crop = 5 mm/day ; Area to be irrigated = 60 ha = 60 $\times 10^4$ m^2

Rate of delivery of water = 28 lps ; Irrigation application efficiency = 40%

(i) Moisture storage capacity of soil = Depth of water stored in root zone

$$= 140 \text{ mm/m} \times \text{root depth of crop} = 140 \times \frac{30}{100} = 42 \text{ mm}$$

$$\therefore \quad \text{Allowable depletion depth between irrigations} = \frac{35}{100} \times 42 = 14.7 \text{ mm}$$

(ii) $$\text{Frequency of irrigation} = \frac{\text{Allowable depletion depth between irrigations}}{\text{Daily water use by the crop}} = \frac{14.7}{5} = 2.94 \text{ days}$$

(iii) $$\text{Net application depth} = \frac{14.7}{0.40} = 36.75 \text{ mm}$$

(iv) Volume of water required = Area to be irrigated × Net application depth of water

$$= 60 \times 10^4 \times 36.75 \times 10^{-3} = 22.05 \times 10^3 \text{ m}^3 = 22.05 \times 10^6 \text{ litres}$$

(v) Volume of water required to irrigate 4 ha plot

$$= 4 \times 10^4 \times 36.75 \times 10^{-3} = 1.47 \times 10^3 \text{ m}^3 = 1.47 \times 10^6 \text{ litres}$$

$$\text{Time to irrigate 4 ha plot} = \frac{\text{Vol. of water required}}{\text{Rate of delivery of water}} = \frac{1.47 \times 10^6}{28} = 5.25 \times 10^4 \text{ s}$$

$$= 14.583 \text{ hrs.}$$

Example 6

800 m^3 of water is applied to a farmer's rice field of 0.6 hectares. When the moisture content in the soil falls to 40% of the available water between the field capacity (36%) of soil and permanent wilting point (15%) of the soil crop combination, determine the field application efficiency. The root zone depth of rice is 60 cm. Assume porosity = 0.4

Sol. Given,

Water applied = 800m^3 ; Area (A) = 0.6 ha ; Field capacity(FC) = 36%

Wilting point (WP) = 15% ; Root zone depth (d) = 60 cm ; Porosity (η) = 0.4

We have defined Field Capacity m.c. (F) as :

$$F = \frac{\text{Wt. of water contained in a certain volume of soil}}{\text{Wt. of the same volume of dry soil (i.e. wt. of dry soil retaining that water)}}$$

If a saturated soil contains volume equal to V, and the volume of its voids is V_v, then the weight of water contained in this soil = $\gamma_\omega \cdot V_v$; where γ_ω is the unit wt. of voids with air, is given by $\gamma_d \cdot V$; where γ_d is the dry unit wt. of the soil.

$$\therefore \quad F = \frac{\gamma_w \cdot V_v}{\gamma_d \cdot V} = \frac{\gamma_w}{\gamma_d} \cdot n \qquad \left[\frac{V_v}{V} = n = \text{Porosity}\right]$$

$$\therefore \quad \frac{\gamma_d}{\gamma_w} = \frac{n}{F} = \frac{0.4}{0.36} = 1.11$$

Maximum quantity of water stored between field capacity (FC) and permanent wilting point (P.W)

$$= \left(\frac{\gamma_d}{\gamma_w}\right) d \cdot (w_F - w_p) = 1.11 \times 0.60\ [0.36 - 0.15] = 0.14\ m.$$

where, d = root zone depth = 0.6 m (given)

Deficiency of water created when irrigation is done = 60% × 0.14 m = 0.084 m

[∵ irrigation water is applied when m.c. falls to 40% of m.c. available between F.C. and P.W.]

Hence, irrigation water is supplied to fill up 0.084 m depth of water.

∴ Vol. of irrigation water required to fill up the created deficiency

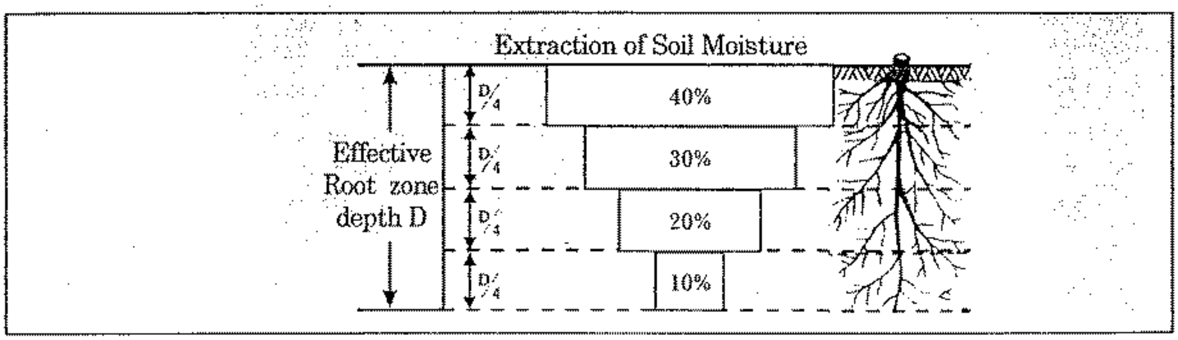
$$= 0.084\ m \times (0.6 ha) = 0.084\ m \times (0.6 \times 10{,}000) m^2 = 504 m^3.$$

Actual irrigation water supplied = 800 m^3

$$\therefore \quad \text{Efficiency of field application} = \frac{504}{800} = 63\% \text{ Ans.}$$

EXTRACTION PATTERN OF SOIL MOISTURE IN ROOT ZONE BY PLANT ROOTS

Moisture extraction pattern shows the relative amounts of moisture extracted by plant roots from different depths in the root zone.



- It may be observed that about 40% of total moisture used is extracted from the 1st quarter 30% from the 2nd quarter, 20% from the 3rd quarter & only 10% from the last quarter.
- The larger extraction of moisture from the upper layers is due to the fact that in a uniform soil generally greater root development takes place in the upper layers of the soil than elsewhere.
- In a layered soil, the moisture available per unit depth varies and hence the extraction pattern and the root development vary from the one in a uniform soil.

METHODS ADOPTED TO MAINTAIN FERTILITY OF A SOIL

The various methods adopted to maintain the fertility of a soil are as follows.

(1) **By keeping the land fallow:** If the land is kept fallow, (*i.e.*, it is left uncultivated for one or more crop seasons) then the soil is allowed to recover w.r.t. the nutrients which are deficient and thus regain its fertility.

(2) **Addition of manure and fertilizers:** If the soil is deficient in some nutrients then by adding manure as well as fertilizers the deficiency is removed and the fertility of the soil is improved.

(3) **Crop rotation**

- Crop rotation means changing the crops to be grown every year in the same field. In other words, it is a process of growing different crops in rotation in the same field.
- If the same crop is grown every year in any field then since the same type of nutrients are consumed the soil becomes deficient in these nutrients.
- On the other hand, if different crops are grown in the same field then since different crops require different nutrients and in different proportions a balanced utilization of the nutrients results and the soil does not become deficient in particular type of nutrients.
- Moreover, if a soil becomes deficient in some of the nutrients by a certain crop grown in the field then it is allowed to recoup when next time a different crop is grown in the same field.
- Different crops have different depths of the root zone. Thus by combining deep rooted crops and shallow rooted crops in the rotation of crops the optimum utilization of the nutrients available in the soil is made.

(4) **Mixed Cropping**

- Mixed Cropping is defined as the growing of two or more crops together in the same field during the same crop season. ***e.g.,*** Wheat and mustard or gram and barley may be grown simultaneously in the same field.
- In mixed cropping, the optimum utilization of the nutrients available in the soil is made.

IRRIGATION WATER QUALITY

Water containing impurities, which are injurious to plant growth, is not satisfactory for irrigation. The quality of suitable irrigation water is very much influenced by the constituents of the soil which is to be irrigated.

The various types of impurities, present in water that makes it unfit for irrigation, are classified as:

(1) Sediment concentration in water

(2) Total concentration of soluble salts in water

(3) Proportion of sodium ions to other cations

(4) Concentration of potentially toxic elements present in water.

(5) Bicarbonate concentration as related to the concentration of calcium plus magnesium

(6) Bacterial contamination.

(1) Sediment concentration

The effect of sediment present in the irrigation water depends upon the type of irrigated land. When fine sediment from water is deposited on sandy soils, the fertility is improved. On the other hand, if the sediment has been derived from the eroded areas, it may reduce the fertility or decrease the soil permeability.

(2) Total concentration of soluble salts

Salts of calcium, magnesium, sodium and potassium, present in the irrigation water may prove injurious to plants. When present in excessive quantities, they reduce the osmotic activities of the plants, and may prevent adequate aeration, causing injuries to plant growth. The effects of salts on plant growth depend largely upon the total amount of salts present in the soil solution.

The salinity concentration of the soil solution (C_s) after the consumptive water (C_u) has been extracted from the soil, is given by

$$\boxed{C_s = \frac{C.Q}{[Q-(C_u-R_e)]}}$$

where, Q = The quantity of water applied ; C_u = Consumptive use of water,

R_e = Effective rainfall or useful rainfall ; $C_u - R_e$ = Used up irrigation water

C = Concentration of salt in irrigation water ;

CQ = Total salt applied to soil with Q amount of irrigation water.

- The salt concentration is generally expressed by ppm or by mg/l, both units being equal.
- The critical salt concentration in the irrigation water in excess of 700 ppm are harmful to some plants, and more than 2000 ppm are injurious to all crops.
- The salt concentration is generally measured by determining the electrical conductivity of water. Salt concentration and electrical conductivity are directly proportional to each other.
- Electrical conductivity is expressed in micro mhos per centimetre.

Electrical conductivity at 25°C in m mhos/cm	Class	Uses
< 250	Low salinity water (C_1)	Can be used for all crops
250 - 750	Medium salinity water (C_2)	Can be used if leaching is done
750 - 2250	High salinity water (C_3)	High salt tolerant plant can be grown with special measures to control salinity
> 2250	Very high salinity water (C_4)	Not suitable for irrigation

(3) Proportion of sodium ions to other cations

- The percentage of the sodium ions is generally less than 5% of the total exchangeable cations.
- If this percentage increases to about 10% or more, the aggregation of soil grains breaks down. The soil becomes less permeable and of poorer tilth. It starts crusting when dry and its pH increases towards that of an alkaline soil.
- High sodium soils are, therefore, plastic, sticky when wet, and are prone to form clods, and

they crust on drying.

- The proportion of sodium ions present in the soils, is generally measured by a factor called Sodium-Adsorption Ratio (SAR) and represents the sodium hazards of water.

SAR is defined as : $\boxed{SAR = \dfrac{Na^+}{\sqrt{\dfrac{Ca^{++} + Mg^{++}}{2}}}}$

where, the concentration of the ions is expressed in equivalent per million (epm);

SAR	Types of water	Uses
0 – 10	Low sodium water (S_1)	Can be used for all soils & for all crops
10 – 18	Medium sodium water (S_2)	used in coarse grained soil
18 – 26	High sodium water (S_3)	May be harmful to all the soils & do require good drainage high leaching with gypsum.
> 26	Very high sodium water (S_4)	Not suitable

(4) Concentration of potentially toxic elements

- Elements like boron, selenium, processed may be toxic to plants.
- Traces of boron are essential to plant growth; its concentrations above 0.3 ppm may prove toxic
- Even for the most tolerant crops, the boron concentration should not exceed 4 ppm.
- Boron is generally present in various soaps.
- Selenium, even in low concentration, is toxic, and must be avoided.

(5) Bicarbonate concentration

High concentration of bi-carbonate ions may result in precipitation of calcium and magnesium bicarbonates from the soil-solution, increasing the relative proportion of sodium ions and causing sodium hazards.

(6) Bacterial contamination

- Bacterial contamination of irrigation water is not a grave problem, unless the crops irrigated with highly contaminated water are directly eaten, without being cooked.
- Cash crops like cotton, nursery stock, etc. which are processed after harvesting, can therefore, use contaminated waste waters, without any trouble.

Example 7

(a) What is the classification of irrigation water having the following characteristics : Conc. of Na, Ca and Mg are 22, 3 and 1.5 m-eq per litre respectively, and the electrical conductivity is 200 micro mhos per cm at 25°C? (b) What problems might arise in using this water on fine textured soil? (c) What remedies do you suggest to overcome this trouble?

Sol. (a) $$SAR = \frac{Na^+}{\sqrt{\frac{Ca^{++} + Mg^{++}}{2}}} = \frac{22}{\sqrt{\frac{3+1.5}{2}}} = \frac{22}{\sqrt{2.25}} = \frac{22}{\sqrt{2.25}} = \frac{22}{1.5} = 14.67$$

SAR is between 10 to 18 so, it is medium Sodium water and is represented.
Electrical conductivity is between 100 to 250 micro mhos per cm at 25°C. So, it is, C1 water.
Hence, the given water is classified as C1-S2 water.

(b) In fine-textured soils, the medium sodium (S2) water may create the following problems:
(i) Soil becomes less permeable. (ii) It starts crusting when dry.
(iii) Its pH increases towards that of alkaline soil. (iv) It becomes plastic and sticky when wet.
(c) Gypsum ($CaSO_4$) addition either to soil or to water.

OBJECTIVE QUESTIONS

1. The following data were recorded from an irrigated field:
   1. Field capacity : 20%
   2. Permanent wilting point : 10%
   3. Permissible depletion of available soil moisture: 50%
   4. Dry unit weight of soil : 1500 kgf/m^3
   5. Effective rainfall : 25 mm

   Based on these data, the net irrigation requirement per metre depth of soil will be

   (a) 75 mm (b) 125 mm (c) 50 mm (d) 25 mm

2. In a cultivated area, the soil has porosity of 45% and field capacity of 38%. For a particular crop, the root zone depth is 1.0 m, the permanent wilting point is 10% and the consumptive use is 15 mm/d. If the irrigation efficiency is 60%, what should be the frequency of irrigation such that the moisture content does not fall below 50% of the maximum available moisture?

   (a) 5d (b) 6d (c) 9d (d) 11d

3. The moisture tension for a soil is 8 atmospheres. The soil is then at

   (a) permanent wilting point (b) field capacity
   (c) optimum moisture content (d) equivalent moisture

4. A soil sample has an exchangeable sodium percentage of 16%, its electrical conductivity is 3.2 milli-mhos/cm and pH of 9.5. How is the soil classified?

   (a) Saline soil (b) Saline-alkaline soil
   (c) Alkaline soil (d) None of the above

5. Consider the following statements:
   Irrigation water has to be supplied to the crops when the moisture level falls

   1. below field capacity 2. to wilting point 3. below wilting point

   Which of these statements is/are correct?

   (a) 1 (b) 2 only (c) 3 only (d) 2 and 3

6. The most suitable water for irrigation is

   (a) C1–S1 (b) C2–S2 (c) C4–S1 (d) C1–S4

7. The electrical conductivity of medium saline water (C 2) at 25°C is of the order of

   (a) 50 to 100 $\mu\Omega/\text{cm}$ (b) 100 to 250 $\mu\Omega/\text{cm}$
   (c) 250 to 750 $\mu\Omega/\text{cm}$ (d) 750 to 1000 $\mu\Omega/\text{cm}$

8. The Sodium Absorption Ratio of an irrigation water is 8. This water will be called

   (a) low sodium water (b) medium sodium water
   (c) high sodium water (d) none of the above

9. Salinity in irrigation water is measured by

   (a) SAR value (b) Electrical-conductivity value
   (c) pH-value (d) none of the above

10. The water which can be utilised by the crops from the soil is called
   (a) field capacity water
   (b) capillary water
   (c) hygroscopic water
   (d) none of the above

11. Permanent wilting point moisture content for a crop represents the
   (a) hygroscopic water
   (b) capillary water
   (c) field capacity water
   (d) none of the above

12. Available moisture for a crop is equal to
   (a) field capacity moisture content–Wilting point moisture content
   (b) field capacity moisture content–hygroscopic moisture content
   (c) both (a) and (b)
   (d) none of the above

13. The optimum moisture content (m.c.) which is retained in the root zone of a soil, before applying irrigation water, is
   (a) equal to: (the field capacity m.c.–wilting point m.c.)
   (b) less than: (the field capacity m.c.–wilting point m.c.)
   (c) more than: (the field capacity m.c.–wilting point m.c.)
   (d) may be more or less than: (the field capacity m.c.–wilting point m.c.)

14. Frequency of irrigation is dependent upon the type of
   (a) soil and crop
   (b) soil and climate
   (c) soil, crop, and climate
   (d) soil, crop, climate, and fertilizer

15. Permanent wilting point is
   (a) a characteristic of the plant
   (b) a soil characteristic
   (c) a soil characteristic modified by the crop
   (d) dependent on soil-water-plant-fertiliser interaction.

16. Commonly adopted moisture tension, in atmosphere, of a soil at permanent wilting point is
   (a) 15 (b) 7 (c) 45 (d) 150

17. At field capacity, water is held in most of the soils at a tension of
   (a) 15 atmosphere (b) zero atmosphere (c) 1/3 atmosphere (d) 1 atmosphere

18. In a well-drained soil, the useful moisture for plant growth essentially comes from
   (a) gravity water
   (b) capillary water
   (c) hydroscopic water
   (d) water of adhesion.

19. Irrigation of a field is normally warranted when the available moisture content in the root zone of a crop is depleted by about
   (a) 10% (b) 100% (c) 0% (d) 50%

20. A clayey soil has a field capacity of 35% and permanent wilting point of 20%. If the specific weight of the soil is 12.75 kN/m^3, the available moisture holding capacity in 80 cm depth of soil, constituting the root zone depth of a crop, is
   (a) 15.6 cm (b) 17.5 cm (c) 20.8 cm (d) 36.4 cm

21. The available moisture holding capacity of soil is 13 cm per metre depth of soil. If a crop with a root zone of 0.8 m and consumptive use of 5 mm/day is to be grown, the frequency of irrigation for restricting the moisture depletion to 50% of a available moisture is
   (a) 6.5 days (b) 26 days (c) 13 days (d) 10 days

ANSWERS

1. (c)	6. (a)	11. (a)	16. (a)	21. (d)
2. (d)	7. (c)	12. (c)	17. (c)	
3. (a)	8. (a)	13. (b)	18. (b)	
4. (c)	9. (b)	14. (c)	19. (d)	
5. (a)	10. (b)	15. (c)	20. (a)	

Water Requirements of Crops & Canal Irigation

CHAPTER

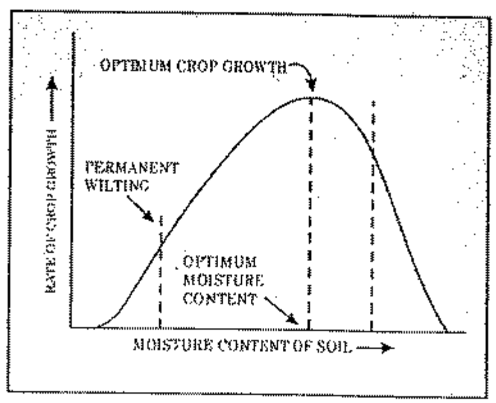
3

Water Requirements of Crops & Canal Irrigation

WATER REQUIREMENT OF CROPS

- The term *water requirements of a crop* means the total quantity and the ways in which a crop requires water, from the time it is sown to the time it is harvested.
- The water requirement varies with the crop as well as with the place.
- In other words, different crops will have different water requirements, and the same crop may have different water requirements at different places of the same country; depending upon the variations in climates, type of soils, methods of cultivation, and useful rainfalls, etc.

LIMITING SOIL MOISTURE CONDITIONS



- Growth of most crops is retarded by either excessive or deficient amounts of soil moisture content.
- Excessive moisture content in the soil results in filling the soil pore spaces completely with water, thus driving out air from the soil in the root zone.
- On the other hand, soils having deficient amounts of moisture hold it so tightly that plants are required to expend extra energy to obtain it.
- In between these two extreme soil moisture conditions, there is a moisture content designated as optimum moisture content at which plants grow most rapidly, resulting in the optimum growth of crops.

CROP SEASONS AND CROPS OF INDIA

(1) There are two main crop seasons of India viz., *Rabi and Kharif.* The crops grown during these crop seasons are designated as Rabi crops and Kharif crops. Rabi crops are also known as *winter crops* and Kharif crops are known as *monsoon crops (See the details in table).*

(2) Sometimes in between the Rabi and Kharif crops intermediate crops are grown, in which case the crops may be classified as *Hot weather crops, Kharif (or monsoon) crops* and *Rabi (or winter) crops.*

Often the Hot weather crops and the *Kharif crops* are combinedly designated as *Summer crops (See the details in table below).*

(3) There are certain crops which have a longer period between their sowing and harvesting times which extends from one crop season to the other *(See the details in table below).*

e.g., *Sugarcane, cotton* (sugarcan is called perennial crop).

(4) The crops may also be classified on the basis of their irrigation requirements as *Dry crops* and *Wet crops. Dry crops* are those crops which are ordinarily grown without irrigation, but they utilize the moisture stored in the soil during rainfall. On the other hand, *wet crops* are those crops which cannot normally be grown without irrigation.

(5) Kharif crops require 2-3 times the water than that of Rabi Crops.

Type	Sown time	Harvested time
Rabi	October	March (next year)
Kharif	April	September (same year)
Hot weather	Feb.	May (same year)
kharif	June	Sept. (same year)
Rabi	oct	Feb. (same year)
Sugarcan	Feb. to march	Nov.to mar. (next year)
Cotton	May to june	Dec. to jan. (next year)

Note : *(i) Season from March to June is called Zaid*

(ii) Sugarcane, tobacco, potato, jute, tea, coffee etc. are plantation crops.

(iii) Main Kharif cereals are : Jowar, soyabean, rice, bajra, cotton, maize, tobbaco and groundnut

(iv) Main Rabi crops : Potato, wheat, pulse, gram, mustard, barley, linseed etc.

CROP PERIOD AND BASE PERIOD

- **Crop period** is defined as total time that elapses b/w the sowing of the crop and its harvesting. Thus, crop period represents the total time during which the crop remains in the field.
- **Base period** is defined as the total time between the first watering done for the preparation of the land for sowing of a crop and the last watering done before its harvesting.
- Crop period is slightly more than the base period for any crop but for calculation purpose they are taken same.
- Both the crop period and the base period are expressed in days.
- Consideration of base period is essential for determining the total water requirement of a crop.

Note : The terms like *growth period, crop period, base period*, etc., can be used as synonyms, each representing crop period, and will be represented by B (in days.)

DUTY OF WATER & DELTA

Duty

- Duty of water is the relation between the area of the land irrigated and the quantity of water required to be supplied for growing a crop.

- It is usually defined as the area of land in hectares which can be irrigated for growing any crop if one cumec (one cubic meter per second) of water is supplied continuously to the land for the entire base period of the crop.

  e.g., If 1500 hectares of land can be irrigated for growing any crop by one cumec of water supplied continuously for the entire base period of the crop then the duty of water for this crop is 1500 hectares per cumec.

- The duty of water expressed in hectares per cumec is convenient in the case of *direct flow irrigation* from canals, because by knowing the duty of water and the area of the land to be irrigated for growing crop, the required discharge for the canal can be determined. The duty of water expressed in this manner is usually termed as *flow duty of water*.
- The duty of water may be expressed in terms of the total area of land in hectares which may be irrigated for growing a crop per million cubic metre of water stored in a tank or reservoir. This mode of expressing the duty of water is specially suitable for *tank irrigation* because in such cases the areas which can be irrigated are dependent on the amount of water stored in the tanks supplying them. The duty of water expressed in this manner is usually termed as *quantity duty of water* or *storage duty of water*.
- In the case of a well (or tube well), the area which can be irrigated by the well (or tube well) annually is called the *duty of the well (or tube well)*.

Note : Duty is used to find out the discharge require from canal if various crop and their area is known discharge require for them can be calculated & added up to obtain total discharge require from canal.

Delta

- It is the total depth of water applied over an irrigated land at different waterings throughout the entire base period of the crop. It is denoted by a symbol Δ and expressed in cm or m.
- The delta for any crop may be determined by dividing the total qty of water in ha-metres required by the crop for its growth by the area of the land in ha over which the crop is growing.
- It is expressed in cm or m.
- Since total water required by a crop is to be applied in stage and is distributed over the base period of the crop, the delta for the crop may be conveniently subdivided into the depths of water to be applied during each watering depending upon the crop requirements.

Crop	Duty (D) ha / m^3 / s	Delta Δ (cm)
Rice	775	120
Wheat	1800	30
Sugarcane	730	120
Vegetable	1000	45
Fodder	2000	22.2

RELATIONSHIP BETWEEN DUTY & DELTA

If D is the duty of water on the field in hectares per cumec, Δ, the total depth of water in metres supplied to a crop growing on the field during the entire base period and B, the base period of the crop in days.

Then, for a field of area D hectares corresponding to the depth of water Δ metres,

The total quantity of water supplied for growing a crop on the field = $D \times \Delta$ ha-m = $D \times \Delta \times$

10^4 cubic metre

Further, for the same field of area D hectares for growing a crop, if water is supplied at the rate of 1 cumec for the entire base period of B days,

The total quantity of water supplied to the field. = 1 × B × 24 × 60 × 60 cubic metre

Hence, $$D \times \Delta \times 10^4 = 8.64 \times 10^4 \times B$$

$$\boxed{D = \frac{8.64 \times B}{\Delta}}$$

FACTORS AFFECTING DUTY OF WATER

The duty of water mainly depends on the following factors.

(1) Types of crop

Different crops requires different quantities of water. So, duty of water varies from crop to crop. The crops which require large quantity of water have lower duty of water than for the crops which requires less quantity of water.

(2) Climatic condition of the area

The water requirement of a crop varies with the climatic condition of the area and hence it also affects the duty of water.The climatic conditions which affect the duty of water are (i) *temperature*, (ii) *wind velocity*, (iii) *humidity* and (iv) *rainfall*.

- Higher is the temperature, lesser is the duty of water (the loss of water due to evapotranspiration will be more)
- Higher is the wind velocity, lesser is the duty of water.
- Higher is the humidity, higher will be the duty of water since the loss of water due to evapotranspiration will be less.
- As duty will vary in different season. However duty is expressed as an average over the entire crop period.
- If rainfall contributes some water then amount of water require from irrigation will be less. Thus duty will become more.

(3) System of irrigation

In the perennial irrigation system the soil of the irrigated area remains continuously wet and hence less quantity of water is required for initial saturation of the soil. On the other hand in the inundation irrigation system there is a wasteful use of water. Hence for the perennial irrigation system the duty of water is higher than for the inundation irrigation system.

(4) Method of irrigation

- The duty of water is higher for furrow method than for any of the flooding methods.
- In *furrow method*, water is not applied to the entire land surface, hence water losses are considerably reduced.
- As compared to the *surface irrigation* methods, the duty of water is high for the *sprinkler and drip irrigation method*
- For permeable soil, duty is less as water is lost in percolation.

(5) Quality of irrigation water

If the irrigation water contains an appreciable amount of harmful salts and alkalies dissolved in it then it is required to be applied in large quantity so that the salts are leached off. This however results in a lower duty of water due to the wastage of considerable amount of water.

(6) Method of Cultivation

If the land is properly ploughed upto the required depth and made quite loose before irrigating, the soil will have high water retaining capacity in the root zone of the plants. This will reduce the number of waterings and hence result in a higher duty of water.

(7) Time of irrigation and frequency of cultivation

- In the initial stages, the land to be cultivated may not be properly levelled and hence more than the required quantity of water may be applied, which will result in a lower duty of water.
- Gradual rise of water table with time due to continuous irrigation will make water available in the root zone of the plants, thus relatively less quantity of water will be required to be applied which will result in a higher duty of water.
- Frequent cultivation of land reduces the loss of moisture through weeds and evaporation from soil and hence results in a higher duty of water

(8) Type of soil and sub-soil of the area through which canal passes

- If the canal is unlined and it passes through coarse grained soil then since there will be greater percolation, loss the duty of water will be low.
- On the other hand, if an unlined canal is passing through fine grained soil then the percolation loss will be less and hence the duty of water will be high.

(9) Canal conditions

In an earthen canal, the percolation loss is high which will result in a low duty of water. But if the canal is lined, the percolation loss will be less and the duty of water will be high.

(10) Topography of land

- If the land to be irrigated is properly levelled then uniform application of water will be possible which will result in economical use of water and hence a higher duty of water.
- If the land is not levelled, then the lower portion will receive more water than the higher portion, resulting in a wasteful use of water and hence the duty of water will be low.

(11) Base period of crop

In general when the base period of a crop is long, more water may be required thus resulting in a lower duty of water.

DUTY AT VARIOUS PLACES

If,

D_A = Duty on field

D_F = Duty at the head of

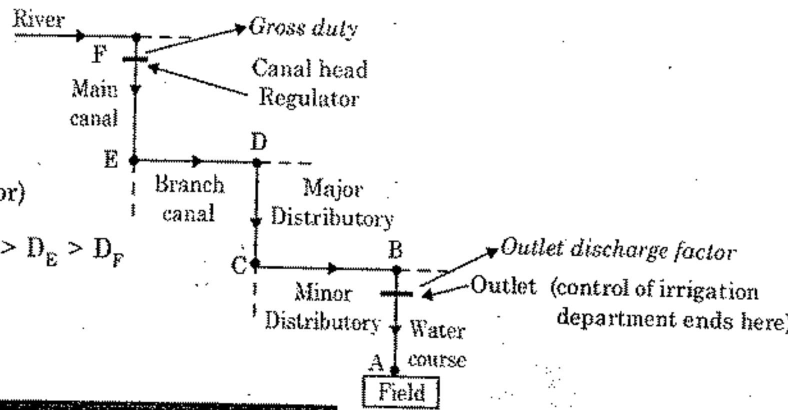
main canal

B = Outlet point of minor

D_B = Duty at B

(also called outlet factor)

Then, $D_A > D_B > D_C > D_D > D_E > D_F$



METHODS OF IMPROVING DUTY OF WATER

(1) The land should be properly ploughed to the required depth and levelled before sowing the crop.

(2) The land should be cultivated frequently because frequent cultivation reduces loss of moisture from the soil especially when the ground water is within capillary reach of the ground surface.

(3) Suitable methods of irrigation should be used.

(4) The canals should be lined. This will reduce the transmission losses.

(5) The rotation of crops must be practised.

(6) Volumetric assessment of irrigation water should be made so that efficient and economical use of water is made by the cultivators.

(7) The alignment of the canal either in sandy soil or in fissured rock should be avoided.

(8) The cultivators must be trained to use irrigation water efficiently and economically.

(9) The land should be redistributed among the cultivators by grouping together their small holding so that irrigation water may be used more efficiently

FEW IMPORTANT DEFINITIONS

(1) Commanded area

It is defined as the area which can be irrigated (or commanded) by a canal system.

(2) Gross commanded Area (GCA)

- The gross commanded area is defined as the total area which can be irrigated by a canal system such that unlimited quantity of water is available.
- A canal is usually aligned along water-shed in between two drainage valleys, so that water can flow from it on both sides under gravity to the maximum possible area.
- The area to which water can flow from a canal will be restricted by the drainage boundaries.
- It is the total area lying b/w the drainage boundaries which can be irrigated by a canal system.
- It includes culturable as well as unculturable area for example ponds, residential area, roads forest etc.

(3) Culturable Commanded Area (CCA)

- The culturable commanded area is that portion of the gross commanded area which is culturable or cultivable.

- Thus culturable commanded area may be obtained by subtracting the unculturable area from the gross commanded area.

  Thus, C.C.A = G.C.A. − unculturable area (area not fit for cultivation)

- The culturable commanded area may be subdivided into the following two categories.

  (a) *Culturable cultivated area:* It is that portion of the culturable commanded area which is actually cultivated during a crop season.

  (b) *Culturable uncultivated area:* It is that portion of the culturable commanded area which is not cultivated during a crop season.

(4) Intensity of Irrigation

- The intensity of irrigation is defined as the percentage of the culturable commanded area proposed to be irrigated annually.
- The area irrigated during each crop season (Rabi or Kharif) is expressed as a percentage of the culturable commanded area which represents the intensity of irrigation for the crop season.
- The yearly intensity of irrigation may be obtained by adding the intensities of irrigation for all the crop seasons.

  e.g., If the intensity of irrigation for Rabi is 50% and that for Kharif is 60% then the yearly intensity of irrigation will be be 110%.

(5) Crop Ratio

- Crop ratio is defined as the ratio of the areas of the land irrigated (or anticipated to be irrigated) during the two main crop seasons viz., *Rabi and Kharif.*
- It is also called *Rabi-Kharif ratio.*
- Crop ratio should be selected such that discharge of canal is almost uniform in all seasons. So that full capacity of canal can be utilized.
- If different crops are grown in Rabi such that

$$A_1 = Q_1D_1 + Q_2D_2 + Q_3D_3$$
$$Q = (Q_1 + Q_2 + Q_3)$$

and in kharif

$$A_2 = Q_1'D_1' + Q_2'D_2' + Q_3'D_3'$$
$$Q = (Q_1' + Q_2' + Q_3')$$

then crop ratio $= \dfrac{A_1}{A_2}$.

(6) Paleo Irrigation (or Paleo)

It is defined as the watering done prior to the sowing of a crop. This is done to prepare the land for sowing & to add sufficient moisture to the soil which would be required for the initial growth of the crop.

(7) Kor Watering, Kor Depth and Kor Period

- The total quantity of water required by a crop is applied through a number of waterings at certain interval during the base period of the crop.

- The quantity of water required to be applied during each of these waterings is not same.
- In general for all the crops during the first watering after the plants have grown a few centimetres high, the qty of water required is more than that during the subsequent watering.
- The first watering after the plants have grown a few centimetres high is known as *kor watering.*
- The depth of water applied during this watering is known as *kor depth.*
- The kor watering must be done in a limited period which is known as *kor period.*

Note : Since during kor watering certain quantity of water is required to be applied in a relatively short duration, the discharge capacity of the canal supplying irrigation water has to be maximum during this period.

(8) Outlet Factor

The irrigation water is supplied to the land to be irrigated by field channels (or water courses) which in the case of a flow irrigation system are supplied water from canals through the outlets. The duty of water at the outlet is known as the *outlet factor.*

In other words, outlet factor is the duty of water at the head of a field channel. (Also called outlet discharge factor)

(9) Capacity Factor

- A canal is designed for a certain maximum discharge capacity, but it need not carry the same discharge at all the time.
- Hence, ratio of the mean supply discharge of a canal for a certain duration to its maximum discharge capacity is defined as capacity factor.
- For example if during kharif season area sown is such that dishcarge required is 0.95 Q_{max}, then capacity factor = 0.95, where Q_{max} = maximum capacity of canal.

  During Rabi it will be say 0.67 Q_{max}. But to imprve the capacity factor more area can be sown in Rabi.

(10) Duty on Capacity (Full supply coefficient)

- It is design duty at the head of canal.

$$\text{full supply coefficient} = \frac{\text{Area estimated to be irrigated during base period}}{\text{Design full supply discharge at the head of canal}}$$

(11) Time Factor

- The ratio of number of days the canal has actually run during a watering period to the total number of days of the watering period is known as *time factor.*
- Thus if Q is the continuous discharge requirement of canal for the complete crop season but canal is closed for certain days. Then in this much left days only, all water is to be supplied. Thus discharge requirement will increase.

$$Q_0 = \frac{Q_{\text{if continuous}}}{\text{Time factor}},\ Q_0 = \text{discharge required at outlet}$$

- Normally canal supplying irrigation water must run on all the days during each watering period but often on account of some unavoidable circumstances the canal may have to be closed for some days during the watering period.

(12) Cumec day

- The total qty of water flowing for one day at the rate of 1 cumec is known as *cumec-day*.
- It is a unit for measurement of qty of water & it is used when relatively large qty of water is involved.

(13) Overlap Allowance

- Sometimes crop of one season may overlap the next crop season for some period.
- During such a period of overlapping, irrigation water is required to be supplied simultaneously to the crops of both the seasons.
- Due to extra demand of water during this period the canal discharge will have to be increased by some amount.
- The extra discharge of the canal required for this purpose is known as *overlap allowance*.

e.g., Sugarcane

(14) Cash Crops

- A cash crop may be defined as a crop which has to be encashed in the market for processing, etc. as it cannot be consumed directly by the cultivators.
- All *non-food crops* are included in cash crops.
- Crops like jute, tea, cotton, tobacco, sugarcane, etc. are excluded from the list of cash crops.

(15) Transpiration ratio

- Transpiration ratio is defined as the ratio of the mass of water transpired by the plant in its full growth period, to the mass of the dry matter produced.

Thus,
$$\text{T.R.} = \frac{\text{Total mass of water transpired by the plant during its full growth}}{\text{Mass of dry matter produced}}$$

"Mass of dry matter produced," is generally taken as the entire mass of the plant including its roots.

- Different plants will transpire different amount of water; and, hence, their water consuming characteristics are compared by transpiration ratio.

(16) Crop calender

- It gives us the information about various agronomic practices of the crops grown by farmers.
- Helps the farmers, civil society and the pvt. sector to make available quality seeds of specific crop varieties for a particular agro-ecological zone at the appropriate sowing planting season.
- The crop calender can also serve as a quick reference tool in selecting crop varieties to adopt to changing weather patterns accelerated by climate change.

(17) Crop factor

Ratio between crop evapotranspiration and reference crop evapotranspiration.

(18) Carry-over storage

The storage of water (say, about 5% of live storage) required for the next crop-year as a protection against the late monsoon break is called carry-over storage.

(19) Live storage

Live storage (LS) is defined as the water stored in a tank or reservoir between full-tank level (FTL) or full reservoir level (FRL) (i.e. RL of weir or spillway crest) and the lowest supply level (LSL) (sill level of sluice or dead storage level DSL).

(20) Dead storage

The dead storage (DS) is defined as the water stored betweend the LSL and the deepest river bed level (RBL) in a reservoir or of feeder nallah in a tank. This space is to allow for silting up and is about 10% of the gross storage.

(21) Gross storage

The gross storage (GS) is defined as the storage capacity, or the volume of water, between FTL or FRL and RBL. Mathematically, $GS = LS + DS = LS + 0.1\ GS$ or $GS = \frac{LS}{0.9}$

IRRIGATION EFFICIENCIES

The ratio of the water available for use to the water applied is defined as irrigation efficiency.

Various types of irrigation efficiencies are:

(1) Water Conveyance Efficiency (η_c)

It is defined as the ratio of the quantity of water delivered to the fields or the irrigated land to the quantity of water diverted into the canal system from the river or reservoir.

Thus, if W_f is the quantity of water delivered to the field and W_r is the quantity of water diverted into the canal system from the river or reservoir then

$$\boxed{\eta_c = \frac{W_f}{W_r} \times 100}$$

Note : *The water conveyance efficiency accounts for the water losses which occurs in conveyance from the point of diversion into the canal system to the fields.*

(2) Water Application Efficiency (η_a)

It is defined as the ratio of the quantity of water stored in the root zone of the plants to the quantity of water delivered to the field.

Thus if W_s is the quantity of water stored in the root zone and W_f is the quantity of water delivered to the field then $\boxed{\eta_a = \frac{W_s}{W_f} \times 100}$

The water application efficiency therefore accounts for the water losses which occur during the application of irrigation water to the field. the common sources of loss of water during its application to the field are surface runoff from the field and deep percolation.

Thus, if R_f is the quantity of water lost as surface runoff from the field and D_f is the quantity of water lost due to deep percolation to a level far below the root zone. then

$$\boxed{W_f = W_s + R_f + D_f}$$

and
$$\boxed{\eta_a = \frac{W_f - (R_f + D_f)}{W_f} \times 100}$$

e.g., In the case of sprinkler irrigation method the water application efficiency may be as high as 80% while in the case of a surface irrigation method it may not exceed 60%.

(3) Water Use Efficiency (η_u)

It is defined as the quantity of water used beneficially including the water required for leaching to the quantity of water delivered.

Thus, if W_u is the quantity of water used beneficially and W_f is the quantity of water delivered to the field then
$$\boxed{\eta_u = \frac{W_u}{W_f} \times 100}$$

(4) Water storage Efficiency (η_s)

It is defined as the ratio of the quantity of water stored in the root zone during irrigation to the quantity of water needed to bring the moisture content of the soil to the field capacity. Thus, if W_s is the quantity of water stored in the root zone during irrigation and W_n is the quantity of water needed to bring the moisture content of the soil to the field capacity (i.e., W_n = Field capacity – Available moisture in the soil prior to irrigation), then

$$\boxed{\eta_s = \frac{W_s}{W_n} \times 100}$$

Note : The presence of excess salts in the soil would require that water storage efficiency should be high in order to keep the salts washed out of the soil.

(5) Water Distribution Efficiency (η_d)

It is determined from the following expression

$$\boxed{\eta_d = \left[1 - \frac{y}{d}\right] \times 100}$$

where, 'y' is the average numerical deviation in depth of water stored from the average depth of water 'd' stored in the root zone during irrigation.

- Water distribution efficiency evaluates the degree to which water is uniformly distributed throughout the root zone during irrigation and hence it is also known as *uniformity coefficient.*
- Higher is the value of η_d, more uniformly is the water distributed in the root zone which in turn will result in a better crop response.

(6) Consumptive use Efficiency(η_{cu})

It is defined as the ratio of the normal consumptive use of water to the net amount of water depleted from the root zone.

Thus, if W_{cu} (or C_u or E_{t_c}) is the normal consumptive use of water or evapotranspiration and W_d is the net amount of water depleted from the root zone then

$$\boxed{\eta_{cu} = \frac{W_{cu}}{W_d} \times 100}$$

Note : The consumptive use efficiency, therefore, accounts for the loss of water by deep percolation and any excessive evaporation following an irrigation.

Example 1

The depths of penetrations along the length of a border strip at point 30 metres apart were probed. Their observed values are 2.0, 1.9, 1.8, 1.6 and 1.5 metres. Compute the water distribution efficiency

Sol. Given Data, the observed depths at five stations are

$x_1 = 2.0$, $x_2 = 1.9$, $x_3 = 1.8$, $x_4 = 1.6$ and $x_5 = 1.5$ metres, respectively.

Flow Diagram:

$$\boxed{\eta_d = \left[1 - \frac{y}{d}\right] \times 100} \longrightarrow \begin{cases} \text{d(Mean depth)} = \dfrac{\sum x_i}{n} \\ \text{y(average deviation)} = \dfrac{\sum \text{absolute value of } (x_i - d)}{n} \end{cases}$$

$$\text{Mean depth, } d = \frac{2.0+1.9+1.8+1.6+1.5}{5} = \frac{8.8}{5} = 1.76 \text{ metres}$$

Values of deviations from the mean are (2.0 – 1.76), (1.9 – 1.76), (1.8 – 1.76), (1.6 – 1.76), (1.5 – 1.76) *i.e.*, 0.24, 0.14, 0.04, – 0.16 and – 0.26.

The absolute values of these deviations from the mean, are 0.24, 0.14, 0.14, 0.16 and 0.26.

The average of these absoulte values of deviations from the mean

$$y = \frac{0.24+0.14+0.04+0.16+0.26}{5} \; \frac{0.84}{5} = 0.168 \text{ metre}$$

$$\text{The water distribution efficiency} = \left(1 - \frac{d}{D}\right) = \left[1 - \frac{0.168}{1.76}\right] = 1 - 0.095 = 0.905$$

Hence, the water distribution efficiency = 0.905.

Example 2

A stream of 150 litres per second was delivered from a canal and 110 litres per second were delivered to the field. An area of 2.2 hectares was irrigated in eight hours. The effective depth of root zone was 1.5m. The runoff loss in the field was 445 m^3. The depth of water penetration varied linearly from 1.5 m at the head end of the field to 1.1 m at the tail end. Available moisture holding capacity of the soil is 200 mm per metre depth of soil. Determine the water conveyance efficiency, water application efficiency, water storage efficiency, and water distribution efficiency. Irrigation was started at a moisture extraction level of 50%.

Sol. Given Data, $Q_{canal} = 150$ litres/sec ; $Q_{field} = 110$ litre/sec

Area (A_{field}) = 2.2 ha. ; Irrigation duration (T) = 8 hrs.

depth of water penetration (at the head end), $d_h = 1.5$ metre

depth of water penetration (at the tail end) $d_t = 1.1$ metre

Moisture holding capacity = 200 mm/metre ; Run off loss = 445 cum.
Determine :
water conveyance efficiency (η_c) , water application efficiency (η_a)
water storage efficiency (η_s) , water distribution efficiency (η_d)
Key : Irrigation was started at a moisture extraction level of 60%

Water conveyance efficiency, $\eta_c = \frac{W_f}{W_r} \times 100 = \frac{110}{150} \times 100 = 73.33\%$

Water application efficiency, $\eta_a = \frac{W_s}{W_f} \times 100$

Water delivered to the plot $= \frac{110 \times 60 \times 60 \times 8}{1000} = 3168 \text{ m}^3$; Runoff loss = 445 m^3

$\therefore$ Water stored in the root zone, $W_s = (3168 - 445) = 2723 \text{ m}^3$

$$\eta_a = \frac{2723}{3168} \times 100 = 86\%$$

Water storage efficiency, $\eta_s = \frac{W_s}{W_n} \times 100$

Water holding capacity of the soil = 200 × 1.5 = 300 mm

Moisture required in the root zone $= \left(300 - \frac{300 \times 50}{100}\right) = 150 \text{ mm} = \frac{150}{1000} \times 2.2 \times 10^4 = 3300\text{m}^3$

$$\eta_s = \frac{2723}{3300} \times 100 = 82.52\%$$

Water distribution efficiency, $\eta_d = \left(1 - \frac{y}{d}\right) \times 100$

Average depth of water stored in the root zone is

$$d = \frac{1.5 + 1.1}{2} = 1.3 \text{ m}$$

Numerical deviation from depth of penetration

at the head end of the field = (1.5 − 1.3) = 0.2 m

at the tail end of the field = (1.3 − 1.1) = 0.2 m

$\therefore$ Average numerical deviation in depth of water stored is

$$y = \frac{0.2 + 0.2}{2} = 0.2 \text{ m}$$

$$\therefore \quad \eta_d = \left(1 - \frac{0.2}{1.3}\right) \times 100 = 84.62\%$$

IRRIGATION REQUIREMENTS OF CROPS

Irrigation water requirement can be defined as the quantity of water that must be supplied by irrigation to satisfy evapotranspiration, leaching and other miscellaneous water requirements that are not provided by water stored in the soil and precipitation that enters the soil.

(1) Consumptive Irrigation Requirement (CIR)

It is defined as the amount of irrigation water that is required to meet the evapotranspiration needs of a crop during its full growth. If during the growth period of a crop rain occurs then since a part of it will be retained by the soil in the root zone and the same will be available to meet a part of the evapotranspiration requirements of the crop, the quantity of irrigation water required to be applied will be correspondingly reduced. This part of the rainfall is known as effective rainfall. Thus if E_{tc} or C_u is the evapotranspiration or consumptive use of water for a crop and R_e is the effective rainfall during the growth period of the crop then

$$\boxed{\text{CIR} = E_{t_c} - R_e \left(or\, C_u - R_e\right)} \quad \text{... (i)}$$

(2) Net Irrigation Requirement (NIR)

It is defined as the amount of irrigation water required to be delivered at the field to meet the evapotranspiration needs of a crop as well as the other needs such as leaching, presowing requirement and nursery water requirement.

$$\boxed{\text{NIR} = \text{CIR} + \text{LR} + \text{PSR} + \text{NWR}}$$

where, LR = leaching requirement ; PSR = presowing requirement;
NWR = nursery water requirement

Note : (i) *Presowing requirement (PSR):* Presowing irrigation is important for field preparation as availability of moisture is essential for good germination of seeds.

(ii) *Nursery Water Requirement (NWR):* The water requirement for nursery is required to be considered in the case of those crops which are sown on nursery beds and are transplanted within few days after sowing when the plants are a few cm tall.

(3) Field Irrigation Requirement (FIR)

It is defined as the amount of water required to meet the 'net irrigation requirements' plus the amount of water lost as surface runoff and through deep percolation.

$$\boxed{\text{FIR} = \frac{NIR}{\eta_a}}$$

(4) Gross Irrigation Requirement (GIR)

It is defined as the amount of water required to meet the *field irrigation requirements* plus the amount of irrigation water lost in conveyance through the canal system by evaporation and by seepage.

$$\boxed{\text{GIR} = \frac{FIR}{\eta_c}}$$

Example 3

A certain crop is grown in an area of 3000 hectares, which is fed by a canal system. The data pertaining to irrigation are as follows :

Field capacity of soil = 26% ; Optimum moisture = 12% ; Permanent wilting point = 10% Effective depth of root zone = 80 cm ; Apparent relative density of soil = 1.4

If the frequency of irrigation is 10 days and the overall irrigation efficiency is 22% find :

(i) Daily consumptive use (ii) Water discharge in m³/s required in the canal feeding the area

Sol. Flow diagram:

Water discharge in the canal ⟶ (Total irrigation required) × (Area) ⟶ Given

↓

Net irrigation required/$\eta_{irrigation}$ ⟶ Given

↓

Daily water consumed by plant

↓

$\frac{\gamma_d \times d}{\gamma_w}$ [FC − OMC] in 10 days

Depth of water used by plants for growth, which is supplemented by irrigation after every 10 days

$$= \frac{\gamma_d}{\gamma_w} \times d \times [\text{Field capacity m.c.} - \text{Optimum m.c.}]$$

$$= 1.4 \times 0.8 \times (0.26 - 0.12) = 0.1568 \text{ m} = 15.68 \text{ cm}$$

(i) Daily water consumed by plants; i.e., daily consumptive use $= \frac{15.68}{10}$ cm $= 1.568$ cm.

(ii) Total irrigation water required i.e losses in field and conveyance $= \frac{N.I.R}{\eta_{irrigation}} = \frac{1.568}{0.22}$ cm / day

$= 7.127$ cm/day

Field area $= 3000$ ha $= 3000 \times 10^4$ m²

$\therefore$ Discharge required in irrigation canal $= \frac{7.127}{100} \times \frac{3000}{24 \times 60 \times 60} \text{m}^3/\text{s} = 24.75$ m³/s.

Example 4

During a particular stage of the growth of a crop, consumptive use of water is 2.5 mm/day. Determine the interval in days between irrigations, and the depth of water to be applied when the amount of water available in the soil is 50% of the maximum depth of available water in the root zone, which is 80 mm. Assume irrigation efficiency to be 60%.

Sol. Given data :

Consumptive use = 2.5 mm/day ; max. depth of available water = 80 mm

Water available = 50 % of max. depth of available water = 40 mm; Irrigation efficiency = 60%

Determine : Frequency of irrigation (T), Depth of water to be applied

Flow Diagram :

(frequency of irrigation) ⟶ Water available = 50 % of max. available depth ⟶ Given

(frequency of irrigation) ⟶ Consumption rate ⟶ Given

Depth of water to be applied $\longrightarrow \dfrac{\text{Depth of water actually required}}{\eta_{irrigation}}$ $\rightarrow$ Total depth required – Available depth; $\rightarrow$ Given

Readily available moisture in root zone = 50% × 80 mm = 4cm

Consumptive use = 2.5 mm/day = 0.25 cm/day

$\therefore$ 0.25 cm of water is consumed by crop in 1 day

4 cm of water will be consumed by crop in $\frac{1}{0.25}\times 4 = 16$days.

Also, the depth of water to be recouped through irrigation = 4 cm.

Actual water required to be applied to the field = $\frac{4\text{cm}}{0.6}$ cm = 6.67 cm.

Hence, 6.67 cm depth of irrigation water shall have to be applied to the fields at an interval of 16 days.

Example 5

A water course commands an irrigated area 1000 hectares. The intensity of irrigation of rice in this area is 70%. The transplantation of rice crop takes 15 days and during the transplantation period total depth of water required by the crop on the field is 500 mm. During the transplantation period, the useful rain falling on the field is 120 mm. Find the duty of irrigation water from the crop on the field during transplantation, at the head of the field and also at the head of the water course assuming losses of water to be 20% in the water course. Also calculate the discharge required in the water course.

Sol. Culturable commanded area, CCA = 1000 ha ; Intensity of irrigation, I.I = 70%

So, rice area = I.I × CCA = 70% of 1000 = $\frac{70}{100}\times 1000$ = 700 ha

Total depth of water required for transplantation of rice = 500 mm = 50 cm

Useful rainfall on the field = 120 mm = 12 cm

Extra depth of water required to be supplemented by irrigation, Δ = 50–12= 38 cm

Transplantation period, B = 15 days

$$\text{Duty of irrigation water} = \frac{864B}{\Delta(\text{in cm})} = \frac{864\times 15}{38} = 341 \text{ ha/cumec}$$

So, Duty of irrigation water at the head of the field = 341 ha/cumec

If losses of water in the water course is 20%

Duty at the head of the water course = 80% of 341 = $\frac{80}{100}\times 341$ = 273 ha/cumec

Example 6

For the data given in table and taking Kor depth and Kor period for wheat as 13.5 cm and 4 weeks respectively and for rice as 19.0 cm and 2.5 weeks respectively, compute the average discharge requirement and peak demand :

Crop	Area under crop (hectare)	Total depth (cm)	Base period (days)	Average duty ha/cumec
Wheat	5000	37.50	140.0	3225.60
Rice	2500	120.00	120.0	864.00

Sol. Given, Kor depth for wheat = 13.5 cm ; Kor depth for rice = 19 cm

Kor period for wheat = 4 weeks ; Kor period for rice = 2.5 weeks

From table,

$$\text{Average discharge required for wheat} = \frac{\text{Area under wheat}}{\text{Average duty for wheat}} = \frac{5000}{3225.60} = 1.55 \text{ cumec}$$

$$\text{Average discharge required for rice} = \frac{\text{Area under rice}}{\text{Average duty for rice}} = \frac{2500}{864} = 2.89 \text{ cumec}$$

So, average discharge requirement = 2.89 cumec (max. of above two values)

$$\text{Duty for wheat (peak demand)} = \frac{8.64\times \text{kor period for wheat}}{\text{kor depth for wheat (in m)}} = \frac{8.64\times(4\times7)}{\frac{13.5}{100}} = 1792 \text{ ha/cumec}$$

$$\text{Duty for rice (peak demand)} = \frac{8.64\times \text{kor period for rice}}{\text{kor depth for rice (in m)}} = \frac{8.64\times(2.5\times7)}{\frac{19}{100}} = 795.8 \text{ ha/cumec}$$

$$\text{Hence, peak demand for wheat} = \frac{\text{Area under wheat}}{\text{Duty for wheat}} = \frac{5000}{1792} = 2.79 \text{ cumec}$$

$$\text{Peak demand for rice} = \frac{\text{Area under rice}}{\text{Duty for rice}} = \frac{2500}{795.8} = 3.14 \text{ cumec}$$

Hence, peak demand = 3.14 cumec (max. of above two values)

Example 7

The following cropping pattern is evolved under a reservoir.

Crop	Season	Base period (days)	Outlet factor (ha / cumec)	Intensity (%)	Crop ratio
Sugarcane	Perennial	360	800	60	4
Paddy	Kharif	120	800	80	2
Cotton	Kharif	210	1200	60	2
Wheat	Rabi	120	1800	75	3
Vegetable	summer	100	750	60	1

GCA = 30000 ha of which 80% is culturable.

Assuming the following:
(i) Extra water for the period of peak water use = 20% (ii) Time factor = 13/20
(iii) Conveyance losses = 20% (iv) Evaporation & seepage losses in the reservoir = 10%
(v) Allowance for carryover storage of the reservoir = 5%

Determine the following:
(i) Capacity factor for the main canal
(ii) The design discharge at the head of the main canal
(iii) The capacity of the storage reservoir
[Assuming 10% as dead storage]

Sol. Given CCA = GCA × 0.80 = 30000×0.80=24000ha

Now for different crops:

(i) Sugarcane

$$\text{Area for sugarcane} = \frac{4}{4+2+2+3+1}\text{ of CCA} = \frac{4}{12}\times 24000 = 8000\text{ ha}$$

and area actually irrigated = 8000 × 0.60 = 4800 ha

Then $$Q = \frac{A}{D} = \frac{4800}{800} = 6\text{ cumec}$$

and $$\Delta = \frac{8.64B}{D} = \frac{8.64\times 360}{800} = 3.88\text{ m}$$

(ii) Paddy

$$\text{Area for paddy} = \frac{2}{12}\times 24000 = 4000\text{ ha}$$

and Area actually irrigated = 4000 × 0.80 = 3200 ha

Then $$Q = \frac{A}{D} = \frac{3200}{800} = 4\text{ cumec}$$

and $$\Delta = \frac{8.64B}{D} = \frac{8.64\times 120}{800} = 1.296\text{ m}$$

(iii) Cotton

$$\text{Area for cotton} = \frac{2}{12}\times 24000 = 4000\text{ha}$$

and Area actually irrigated = 4000 × 0.60 = 2400 ha

Then $$Q = \frac{A}{D} = \frac{2400}{1200} = 2\text{ cumec}$$

and $$\Delta = \frac{8.64B}{D} = \frac{8.64\times 210}{1200} = 1.512\text{ m}$$

(iv) Wheat

$$\text{Area for wheat} = \frac{3}{12}\times 24000 = 6000\text{ ha}$$

and Area actually irrigated = 6000 × 0.75 = 4500 ha

Then $$Q = \frac{A}{D} = \frac{4500}{1800} = 2.5 \text{ cumec}$$

and $$\Delta = \frac{8.64B}{D} = \frac{8.64 \times 120}{1800} = 0.576 \text{ m}$$

(v) Vegetables

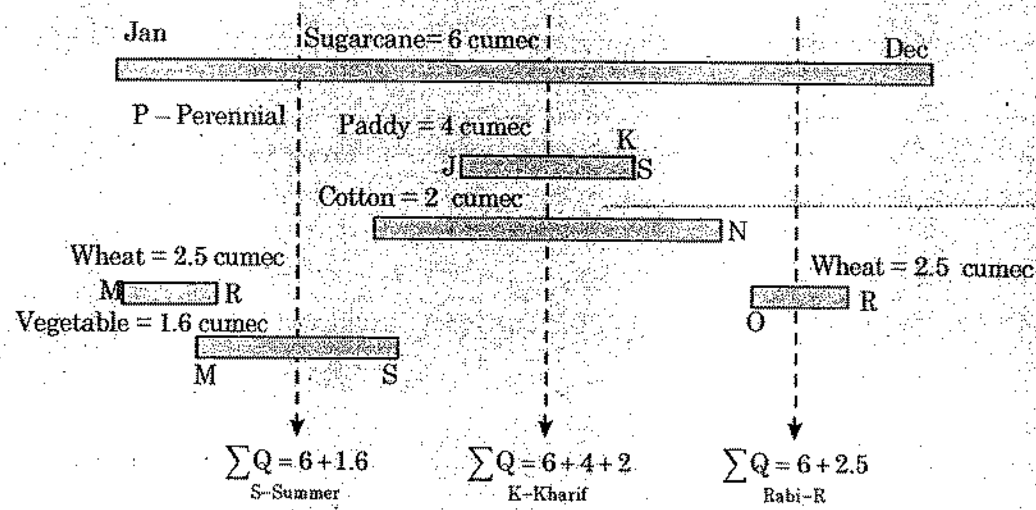
$$\text{Area for vegetables} = \frac{1}{12} \times 24000 = 2000\text{ha}$$

and $$\text{Area actually irrigated} = 2000 \times 0.60 = 1200 \text{ ha}$$

Then $$Q = \frac{A}{D} = \frac{1200}{750} = 1.6\text{cumec}$$

and $$\Delta = \frac{8.64B}{D} = \frac{8.64 \times 100}{750} = 1.15\text{m}$$

The cropping pattern is shown as in figure.



(i) Capacity factor for the main canal : There are three crop seasons:

During Kharif, $\sum Q = 6 + 4 + 2 = 12$ cumec

During Rabi, $\sum Q = 6 + 2.5 = 8.5$ cumec

During summer, $\sum Q = 6 + 1.6 = 7.6$ cumec

Hence, $\bar{Q} = \frac{1}{3}(12 + 8.5 + 7.6) = 9.37$ cumec

During Kharif, $Q_{max} = 12$ cumec. Therefore,

Capacity factor, $\text{C.F.} = \frac{\bar{Q}}{Q_{max}} = \frac{9.37}{12} = 0.78$ (high) which is satisfactory.

(ii) Discharge at the head of the main canal:

Maximum discharge in the three seasons, $Q_{max} = 12$ cumec.

Design discharge at the field outlet allowing for peak use and time factor is

$$Q_o = \frac{12 \times 1.2}{13/20} = 22.16 \text{ cumec}$$

Design discharge at the head of the main canal, allowing for conveyance losses, is

$$Q_{max} = \frac{Q_o}{1-0.2} = \frac{22.16}{0.8} = 27.7 \quad \text{(say, 28 cumec)}$$

(iii) Storage capacity of the reservoir : Live storage (without losses) is

$$\sum Qt = (6 \times 360 + 4 \times 120 + 2 \times 210 + 2.5 \times 120 + 1.6 \times 100)\ 86400$$
$$= 304 \times 10^6 \text{ m}^3 = 304 \text{ M-m}^3$$

Allowing for losses of 20% in conveyance and 10% in reservoir, we get total losses = 30%. Then

$$\text{Storage} = \frac{304}{1-0.3} = 434 \text{M} - \text{m}^3$$

Allowing 5% for carryover storage,

Live storage of the reservoir, $LS = 434 \times 1.05 = 456 \text{ M} - \text{m}^3$

Allowing 10% for dead storage,

Gross storage capacity of the reservoir, $GS = \frac{456}{0.9} = 506.7 = 507 \text{ M-m}^3$

The dead storage, $DS = 507 - 456 = 51 \text{ M-m}^3$

CONSUMPTIVE USE OF WATER OR EVAPOTRANSPIRATION

Consumptive use of water or Evapotranspiration is defined as the total quantity of water used by the vegetative growth of a given area in transpiration and building of plant tissue and that evaporated from the adjacent soil in the area in any specified time.

- **Transpiration** is the process in which the water that enters the plant roots and is used in building plant tissue, finally passes into the atmosphere in the vaporous form through the leaves of the plants.
- **Evaporation** is the process in which water from the adjacent soil passes into the atmosphere in the vaporous form.
- Since all the processes involve evaporation and transpiration that are not easily conceived as separate processes, they are thought of as combined process and called *Evapotranspiration*.
- Water deposited on the plant leaves in the form of dew a portion of rain water as well as sprinkler irrigation water intercepted by the plant leaves and subsequently evaporated without entering the plant system also constitutes a part of the consumptive use of water.
- Evapotranspiration or consumptive use is expressed as equivalent depth of water over an area at a given time period (mm/day).
- The value of the consumptive use of water varies from crop to crop and also for the same crop it varies with time as well as place.

Factors affecting consumptive use of water

The various factors affecting the consumptive use of water are as follows:

1. Evaporation from the soil
2. Temperature
3. Wind velocity
4. Relative humidity of air
5. Precipitation
6. Day time hours
7. Intensity of sunlight
8. Soil type and topography

9. Type of crop
10. Cropping pattern
11. Method of irrigation
12. Quantity of readily available moisture.

Potential Evapotranspiration (PET) and Actual Evapotranspiration (AET)

(1) **Potential Evapotranspiration:** If a crop completely covers the ground surface, evapotranspiration takes place entirely through the plants, and if the roots can absorb water at a sufficiently high rate, the water transfer is controlled by the climate alone. This rate of use of moisture is called potential evapotranspiration (PET).

(2) **Actual Evapotranspiration:** The water that is available to plants varies considerably, and the rate of actual evapotranspiration (AET) can fall below the potential rate under natural conditions. Thus, AET is not only a function of meteorological factors but also of factors related with plant and soil.

- If the supply of water to the plant is not limited, soil moisture will be at field capacity (max. amount of water that the soil can retain) and AET will be equal to PET.
- If the water supply is less than PET, then soil starts drying and the ratio AET/PET would be less than one.
- For clayey soils, AET/PET is 1.0 for nearly 50% of the available moisture.
- When the soil moisture status reaches the ultimate wilting point AET reduces zero.

Methods of Determining the Consumptive use of Water

The various methods adopted for determining the consumptive use of water may be broadly classified under the following two categories.

(a) Direct measurement of consumptive use of water

(b) Use of empirical formulae for determining consumptive use of water.

(a) Direct measurement of consumptive use of water

(i) Soil moisture studies on plots
(ii) Tank or Lysimeter method
(iii) Field experimental plots
(iv) Integration method
(v) Inflow and outflow studies for large areas.

(i) Soil moisture studies on plots

- This method is usually employed to determine the consumptive use of water in irrigated field plots in which the soil is fairly uniform and the depth of ground water is such that it does not affect the soil moisture fluctuations within the root zone.
- Soil samples are taken before and after irrigation with some samples between irrigations.
- Usually a large number of soil samples must be taken at different depths in the roots zone.
- The moisture content of the soil samples is determined by standard laboratory methods and it is expressed as a percentage of the oven dry weight of the soil.

(ii) Tank or Lysimeter method

- Tanks are watertight cylindrical containers which are open at one end and are set into the ground with their rim approximately flush with the surface. The tanks are 1 to 3 metres in diameter and 2 to 3 metres deep. The consumptive use of water is determined by measuring the quantity of water required to be supplied to maintain satisfactory growth of the plants within the tank.

- Lysimeters are similar to the tanks with the difference that lysimeters have pervious bottoms. In the case of lysimeters, also the plants are grown in the same manner as in the case of tanks but owing to their pervious bottom a part of the water applied is allowed to drain through it and the same is collected in a pan kept below it. The consumptive use of water in this case is therefore given by the difference between the water applied and that draining through the pervious bottom and collected in the pan.

Note: (i) The advantage in the case of lysimeter is that gravity flow is permitted and hence conditions more close to the natural ones are developed.

(ii) Phytometer are closed glass chambers in which plants are grown in order to find the consumptive use.

(iii) Field experimental plots

In this method, irrigation water is applied to selected field plots in such a way that there is neither runoff nor deep percolation but the quantity of water applied is sufficient for satisfactory growth of the crops grown. The consumptive use of water is then given by the quantity of water applied. However if there is some runoff resulting from the application of the irrigation water then the same should be measured and subtracted from the quantity of water applied to obtain the consumptive use of water.

(iv) Integration method

In this method, consumptive use of water is determined by the summation of the products of

(i) Consumptive use of water for each crop times its area, plus

(ii) Consumptive use of water for natural vegetation times its area, plus

(iii) Evaporation from water surface times water surface area, plus

(iv) Evaporation from bare land times its area.

(v) Inflow and outflow studies for large areas

In this method, the consumptive use of water is determined by measuring the inflow and outflow from a selected area.

If U is the consumptive use of water for a certain area then its value is given by

$$U = (I + P) + (G_s - G_e) - O$$

I = Total inflow into the area during a 12 months

P = annual rainfall of the area

G_s = Ground water storage of the area at the beginning of the year

G_e = Ground water storage of the area at the end of the year

O = Yearly outflow from the area

(b) Use of empirical formulae for determining consumptive use of water

Various formulas have been developed for determining potential evapotranspiration after rigorous studies. Some of these formulas are purely empirical but some are supported by theoretical analysis. Commonly used methods for determining evapotranspiration or consumptive use of water using these formulae are

(i) Blaney–Criddle method (ii) Hargreaves class A pan evaporation method.

(iii) Modified Penman method

(i) Blaney–Criddle method

- Blaney and Criddle (1950) developed a simplified formula in which the consumptive use of water is correlated with the temperature and daytime hours.
- By multiplying the mean monthly temperature t by the mean monthly percentage p of the maximum possible daytime hours of the year, a monthly consumptive use factor f is obtained as $f = (pt/100)$.
- The value of p depends on the latitude of the place and the period of the year.

Blaney-Criddle Formula: It states that the monthly consumptive use is given by

$$\boxed{C_u = \frac{k.p}{40}[1.8t+32]}$$

where, C_u = Monthly consumptive use in cm ; t = Mean monthly temperature in °C.
k = *Crop factor*, determined by experiments for each crop, under the environmental conditions of the particular area.
p = Monthly per cent of annual day light hours that occur during the period.

If $\frac{p}{40}[1.8t+32]$ is represented by f, we get $C_u = k.f$

The consumptive use of water for the entire crop season or the consumptive use of water for any given period is given by the sum of the monthly consumptive use values.
Thus, the formula was finally expressed as

$$\boxed{C_u = k\Sigma f}$$

where, C_u is the seasonal consumptive use (consumptive use during the period of growth for a given crop in a given area).

Note: The above formula involves the use of crop factor, the value of which is to be determined for each crop and for different places. This formula does not take into consideration the factors such as humidity, wind velocity, elevation, etc. on which consumptive use depends.

(ii) Hargreave's Class A pan evaporation method

In this method, evapotranspiration (consumptive use) is related to pan evaporation by a constant K, called consumptive use coefficient.

$$\frac{\text{Evapotranspiration } (E_t \text{ or } C_u)}{\text{Pan evaporation } (E_p)} = K$$

or

$$\boxed{E_t \text{ or } C_u = K.E_p}$$

Consumptive use coefficient (K) is different for different crops and is different for the same crop at different places. It also varies with the crop growth, and is different at different crop stages for the same crop. The above relationship is now available for various crops from many countries such as Israel, Philippines, U.S.A. and India.

(iii) Penman's Equation

- Penman equation (1998) has been recently introduced for determining the consumptive use of different areas or different segments of a basin, depending upon the type of vegetation covering each sub-basin.
- The advantage with equation lies in the fact that the different specified values of coefficient

of reflection, a factor used in this equation, are available for different types of areas, which can be used in Penman's equation to compute consumptive use (i.e. *Potential evapotranspiration*) values for different segments of command area.

- This equation has been derived by combining the energy balance and mass transfer approaches of the computations of transpiration and evaporation, respectively.
- Its use is becoming more and more popular, in today's modern computer age.
- Penman's equation, incorporating some of the modifications suggested by other investigators, is given as :

$$\boxed{E_t = \frac{A.H_n + E_a.\gamma}{A+\gamma}}$$

where, E_t = Daily potential evapo-transpiration

A = Slope of the saturation vapour pressure v_s Temp. curve at the mean air temperature

H = Net incoming solar radiation or energy, expressed in mm of evaporable water per day

E_a = A parameter including wind velocity and saturation deficit, in mm/day

γ = psychromatic constant = 0.49 mm of Hg/°C

Note: *(i) Hargreaves class A pan evaporation method is generally used in India.*

(ii) Annual evapotranspiration values obtained from Penman's equation are quite close to the values obtained from the actual field observations made in pan evaporation method.

Example 8

Determine the evapotranspiration and irrigation requirement for wheat, if the water application efficiency is 65% and the consumptive use coefficient for the growing season is 0.58 from the following data :

Month	Mean monthly temperature, °C	Monthly percentage of sunshine hours	Effective rain fall, cm
November	18	7.20	2.60
December	15	7.15	2.80
January	13.5	7.30	3.50
February	14.5	7.10	2.00

Sol. Using Blanney-Criddle formula the monthly consumptive use can be calculated as

$$C_u = \frac{kp}{40}(1.8t+32)$$

where, C_u = monthly consumptive use (in cm) ; k = crop factor or consumptive use coefficient t = mean monthly temperature (in °C); p = monthly percent of annual day light hours

If, $\frac{p}{40}(1.8t+32) = f$, then $C_u = kf$

Hence, seasonal consumptive use, $C_u = k\Sigma f$

Month	t(°C)	p	R_e(cm)	$f = \frac{p}{40}(1.8t+32)$
November	18	7.20	2.60	11.6
December	15	7.15	2.80	10.55
January	13.5	7.30	3.50	10.28
February	14.5	7.10	2.00	10.31
		$\Sigma R_e = 10.9$	$\Sigma f = 42.74$ cm	

Given, k = 0.58, water application efficiency $\eta_a = 65\%$

∴ Seasonal consumptive use or evapotranspiration

$$C_u = k\Sigma f = 0.58 \times 42.74 = 24.79 \text{ cm}$$

Consumptive irrigation requirement $= C_u - \Sigma R_e = 24.79 - 10.79 = 13.9$ cm

Field irrigation requirement $= \frac{CIR}{\eta_a} = \frac{13.9}{0.65} = 21.37$ cm

Example 9

Determine the evapotranspiration and irrigation requirement for wheat, if the water application efficiency is 65% and the consumptive use coefficient for the growing season is 0.8 from the following data:

Month	t	p	R_e
November	18	7.20	2.60
December	-15	7.15	2.80
January	13.5	7.30	3.50
February	14.5	7.10	2.00

Sol. The monthly consumptive use (in cm) is calculated by Blaney-Criddle formula i.e.

$$C_u = k \times \frac{p}{40}(1.8t + 32)$$

where, k is consumptive use coefficient ; t is mean monthly temperature in °C
p is monthly percent of annual day light hours

If $\frac{p}{40}(1.8t+32) = F$, then $C_u = kF$

Seasonal consumptive use, $C_u = k\Sigma F$

The calculations are tabulated below:

Month	t	p	R_e	$f = \frac{p}{40}(1.8t+32)$
November	18	7.20	2.60	11.6

December	15	7.15	2.80	10.5
January	13.5	7.30	3.50	10.3
February	14.5	7.10	2.00	10.3
			$\Sigma R_e = 10.9$ cm	$\Sigma f = 42.7$ cm

Seasonal consumptive use or evapotranspiration $C_u = k \times \Sigma f = 0.8 \times 42.7 = 34.16$ cm

Consumptive irrigation requirement $= C_u - \Sigma R_e = 0.8 \times 42.7 - 10.9 = 23.26$ cm

Field irrigation requirement $= \dfrac{CIR}{\eta_a} = \dfrac{23.26}{0.65} = \mathbf{35.8\ cm}$

CANAL IRRIGATION

- A canal may be defined as an artificial channel constructed to carry water from a river or from a tank or reservoir for various purposes such as irrigation, power generation, navigation, etc.
- A water conveyance system that carries water from the supply point to point where it is used comprises canals through open cuts, rocks or earth formations.
- Both canal and channel are synonymous terms; they mean one and the same thing.
- In general, the canals have trapezoidal cross-section.
- The canals are usually designated according to the purpose (or function) for which they are constructed as **irrigation canals, power canals, navigation canals,** etc.
- An **irrigation canal** is a canal which carries water from the source to the agricultural fields for irrigation. Example: Ganga canal, Western Yamuna canal, Bhakra canal.
- A **power canal** is a canal which supplies water to a power house for generation of hydro-electric power. The power house is usually located on the canal itself at a section where a fall is available on a canal. At most times, power generation and irrigation are combined on the same canal so that after the generation of power water is used for irrigation.
- A **navigation canal** is a canal which is constructed to provide various navigation facilities like transport of goods and men. The velocity of flowing water in a navigation canal should be small for easy movement of small ships and barges in the direction of flow of water as well as in the opposite direction. It is usually not possible to combine navigation canals with irrigation canals because the requirements of navigation canals are completely different from those of irrigation canals.

CLASSIFICATION OF IRRIGATION CANALS

The irrigation canals can be classified in different ways on the basis of various considerations as follows:

(1) Classification based on the Nature of Source of Supply

(i) Permanent canals (ii) Inundation canals

(i) Permanent canals

A permanent canals is the one which is fed by a permanent source of supply. It is a well graded channel and is provided with permanent regulation and distribution works. The permanent canals may be further classified as:

(a) *Perennial canals:* Canals which get continuous supplies from the source throughout the year.

(b) *Non-perennial canals:* Canals which get supplies only for a part of the year.

(ii) Inundation canals

An inundation canal is a canal which gets its supplies only when the water level in the river, from which it takes off, rises during floods. These canals are not provided with any headworks for diversion of river water to the canal but obtain their supplies through open cuts in the bank of the river. The inundation canals are therefore non-perennial and the flow in these canals depend on the periodical rise of water level in the river.

(2) Classification based on the Function of the Canal

(i) Feeder canals (ii) Carrier canals

(i) Feeder canals: A feeder canal is a canal which is constructed only to feed another canal. No direct irrigation is carried out from a feeder canal. ***e.g.***, Indira Gandhi feeder canal.

(ii) Carrier canal : A carrier canal is a canal which is used both for direct irrigation and for feeding water to another canal. Thus it acts as an irrigation canal as well as feeder canal. ***e.g.***, Upper Chenab Canal in West Punjab (Pakistan).

(3) Classification based on Discharge and Relative Importance in a given Network of Canals

(i) Main canal (ii) Branch canal (iii) Major distributary (iv) Minor distributary

(v) Water courses or Field channel

(i) Main canal

- It is the principal canal of a network of irrigation canals.
- It takes off at headworks directly from a river or a reservoir or from the tail end of a feeder canal.
- It is a large capacity canal which supplies water to branch canals and major distributaries.
- Direct irrigation is not usually carried out from it since a very high discharge is conveyed through the main canal .

(ii) Branch canal

- Branch canals (usually called branches) are the irrigation canals which takes off from the main canal on either side.
- From large branches, direct irrigation is not generally done, but from smaller branches direct outlets may be provided to cover large areas by irrigation.
- The branch canals generally carry a discharge of over 5 cumecs.
- The main function of branch canals is to supply water to major and minor distributaries.

(iii) Major distributary

- Major distributaries are the irrigation canals which take off from the branch canals and sometimes from the main canal.
- The major distributaries are generally used for direct irrigation and hence they supply water through outlets to water courses.
- They carry a discharge varying from 0.25 to 5 cumec.

(iv) Minor distributary

- Minor distributaries are the irrigation canals which take off from major distributaries and branch canals.

- They carry discharge less than 0.25 cumec.

(v) Water courses or field channels

- Water courses are small channels which carry water from the outlets of a major or minor distributary or a branch canal to the fields to be irrigated.
- Outlets are provided in the irrigation canal at appropriate places.
- Beyond the outlet, water is handled by the individual cultivators who directs it to various parts of its command.
- These are owned, constructed, controlled and maintained by the cultivators.

***Note:** (i) to (iv) are constructed and maintained by the state.*

(4) Classification based on Canal Alignment

(i) Ridge canal or Watershed canal (ii) Contour canal (iii) Side slope canal

(i) Ridge canal or Watershed canal

- A ridge or watershed canal is a canal which is aligned along the ridge or the natural watershed line.
- Canal running on a watershed can irrigate areas on both sides and hence a large area can be brought under cultivation i.e. (they have a very high command area).
- No drainage can intersect a ridge line or watershed, as all the drainage flows away from the ridge line.
- A ridge canal does not cross drainage line and for this canal, cross drainage works are not required.
- Ridge canals are quite economical and can be provided in plane areas.

(ii) Contour canal

- A contour canal is a canal which is aligned nearly parallel to the contours of the country.
- It can irrigate areas only on one side.
- Ground level on the other side in this type of canal is higher, so it is not necessary to construct a bank on that side. Such a canal with only one bank is known as single bank canal. However, when both the banks are provided it is known as double bank canal.
- A contour canal has to cross the drainage and hence for this canal, cross-drainage works are required to be provided.
- It is generally provided in hilly areas.

(iii) Side slope canal

- A side slope canal is a canal which is aligned at right angles to the contours of the country.
- Side slope canal is neither on the watershed nor in the valley but it is somewhere in between the two along the slope.
- This type of canal is nearly parallel to the natural drainage of the country as it does not intercept drainage and hence cross-drainage works are not required.

(5) Classification based on Financial Output

(i) Productive canals (ii) Protective canals

(i) Productive canals : A canal which when fully developed yields enough revenue to cover up its running cost and a net saving at the rate of more than six per cent of the capital invested initially for its construction.

(ii) Protective canals : A canal which is constructed as a relief work during famine to provide employment to the inhabitants of the famine affected area and to protect that area against famine in future.

(6) Classification based on the soil through which constructed

(i) Alluvial canals (ii) Non-alluvial canals

(i) Alluvial canals: Canals flowing through *alluvium* i.e. ground formations consisting of non cohesive sediments like sand, silt and gravel. These soils are readily scoured and deposited.

(ii) Non-alluvial canals (NAC) : Canals that have been lined with some suitable material to provide a rigid bed and banks so as to avoid the problem associated with alluvial boundaries.

(7) Classification based on the Lining

(i) Unlined canals : Canals which has its bed and banks made up of natural soil through which it is constructed and it is not provided with a lining of impervious material. The bed and banks of the canal may not be scoured.

(ii) Lined canals: Canals which are provided with a lining of impervious material on its bed and banks to prevent the seepage of water.

CANAL ALIGNMENT

For the alignment of an irrigation canal following considerations are made:

(1) An irrigation canal should be aligned in such a way that maximum area is irrigated with the least length of channel and its cost including the cost of cross-drainage works is minimum. A shorter length of the canal has less loss of head due to friction and smaller loss of water due to seepage and evaporation so that additional area can be brought under cultivation.

(2) The main canal which takes off from the river should mount the watershed or the ridge in the area at some point. The portion of the canal between the point where it takes off from the river to the point where it mounts the watershed is usually in heavy cutting and also in this portion a number of cross-drainage works will be required to be constructed which will make this portion of the canal very expensive.

(3) Once a canal is aligned along a watershed it will in general be kept on the watershed. However, if the watershed takes a sharp loop between some points, then considerable length of canal can be saved if it is aligned straight instead of following the watershed. Some cross-drainage works may be required. Also canal will be able to irrigate areas only on one side of the canal.

(4) In hilly areas, it may not be possible to align the canal along the watershed or ridge because in the hills the river flows in the valley below, while the watershed or the ridge may be hundred of metres above it.

(5) Curves should be avoided in the alignment of canals because the curves lead to disturbance of flow and a tendency to silt on the inside and to scour on the outside of the curves.

(6) Alignment of the canal should be such that idle length of the canal (*i.e.*, the portion of the canal from which no irrigation is carried out) is minimum.

(7) Alignment should be such that a balanced depth of cutting & filling is achieved.

Note: *Alignment of the canal* should be such that from the point where it takes off from the river it mounts the watershed in the shortest possible distance. For this if necessary the location of the diversion head works may also be suitably adjusted.

WARABANDHI ROTATIONAL METHOD

- Rotational system of water distribution is called warabandhi.
- 'Wara' means turn and 'Bandhi' means fixation (means fixation by turns).
- In warabandhi method, the available water is allocated to cultivator in proportion to their land holdings. The utilization of irrigation supply is left entirely to the cultivator, who is under no obligation to grow any particular crop on any area.
- In this system, the water from the source is carried by the main canal that feeds two or more branches, which operate by rotation and may or may not run full depending on the discharge available in the river for diversion into the main canal. Distributaries supply water to water course. Water course run at full supply level when the distributary is running at full supply level, and the water is allocated to farmers on a water course by a time roaster.
- Warabandhi is the system of equitable water distribution by turns according to a predetermined schedule specifying the day, time and duration of supply to each irrigator in proportion to land holdings in the outlet command.

  Roster of Turn : The cycle of turns on a water course or its branch starts from its head, proceeds d/s and ends at the tail.

CULTIVATION PRACTICES

Practices such as suitable tillage and land grading, fertilizer use, appropriate cropping patterns, mulching, weed control, pest and disease control and other improved agronomic measures helps in economising the use of water, and hence water use efficiency increases.

(1) Tillage : *Tillage* is the agricultural preparation of the soil by mechanical agitation of various types, such as digging, stirring, and overturning.

e.g., (i) *Human-powered tilling methods* using hand tools includes *shovelling, picking,* hoeing, and raking. (ii) Draft-animal-powered or mechanized work include ploughing, rolling with cultipackers or other rollers, harrowing, and cultivating with cultivator shanks (teeth).

Puddling is the tillage of rice paddies while flooded, an ancient practice that is used to prepare for rice cultivation. Historically, this has been accomplished by dragging a weighted harrow across a flooded paddy field behind a buffalo or ox, and is now accomplished using mechanized approaches, often using a walking tractor.

(2) **Use of Fertilizers :** Crop yield generally increases with increase in evapotranspiration (ET) by the crop. In low to high yield range, a large increase in crop yield with fertilizers is accompanied with relatively small increment in ET. This means that use of a fertilizer enhances water use efficiency; and, it also promotes deeper and more profuse root system which extracts more water stored in deeper soil layers

means fixation by turns

(3) **Mulching :** Mulching is the practice of spreading an extraneous material on the surface of soils to increase water infiltration, check evaporation, reduce soil erosion, improve soil environment and suppress weeds.

Note: Various types of mulches are paper, polyethene film, crop residues, petroleum products, etc.

OBJECTIVE QUESTIONS

1. Given that the base period is 100 days and the duty of the canal is 1000 hectares per cumecs, the depth of water will be

   (a) 0.864 cm (b) 8.64 cm (c) 86.4 cm (d) 864 cm

2. **Assertion (A):** Duty is an expression of the irrigating capacity of a unit volume of water.
   Reason (R): Duty at the head of a distributary will be less than that at the head of a watercourse and more than that at the head of a canal.

3. If the depth is 8.64 cm on a field over a base period of 10 days, then the duty is

   (a) 10 hectares per cum/s (b) 100 hectares per cum/s
   (c) 864 hectares per cum/s (d) 1000 hectares per cum/s

4. 10 m3/s of water is diverted to a 32 hectare field for 4 hours. Soil probing after irrigation showed that 0.3 m of water had been stored in the root zone. Water application efficiency in this case would be

   (a) 96% (b) 66.67% (c) 48% (d) 24%

5. A canal was designed to supply the irrigation needs of 1000 ha of land growing rice of 140 days base period and having a delta of 130 cm. If the canal water is used to irrigate wheat of base period 119 days and having a delta of 50 cm, the area that can be irrigated is

   (a) 452 ha (b) 904 ha (c) 1105 ha (d) 2210 ha

6. The delta for a crop having base period 120 days is 70 cm. What is the duty?

   (a) 2480 hectare/cumec (b) 1481 hectare/cumec
   (c) 148 hectare/cumec (d) 1.481 hectare/cumec

7. For a culturable command area of 1000 hectare with intensity of irrigation of 50%, the duty on field for a certain crop is 2000 hectare/cumec. What is the discharge required at head of water course with 25% losses of water?

   (a) 3/16 cumec (b) 1/4 cumec (c) 1/3 cumec (d) 1/2 cumec

8. What is the moisture depth available for evapotranspiration in root zone of 1 m depth soil, if dry weight of soil is 1.5 gm/cc, field capacity is 30% and permanent wilting point is 10%?

   (a) 450 mm (b) 300 mm (c) 200 mm (d) 150 mm

9. Consider the following terms relating to irrigation requirements:

   1. Consumptive irrigation requirement 2. Net irrigation requirement
   3. Field irrigation requirement 4. Gross irrigation requirement

   For a given set up, which one of the following is the correct relation?

   (a) $1 > 2 > 3 > 4$ (b) $1 < 2 < 3 < 4$ (c) $(1 = 2) < 3 < 4$ (d) $1 < (2 = 3) < 4$

10. If the discharge required for different crops is 0.4 cumecs in the field and the capacity factor and time factors are 0.8 and 0.5 respectively, then what is the design discharge of the distributary at its head ?

    (a) 0.80 cumecs (b) 0.16 cumecs (c) 1.0 cumecs (d) 1.24 cumecs

11. During a particular stage of the growth of a crop, consumptive use of water is 2.8 mm/day. If the amount of water available in the soil is 25% of 80 mm depth of water, what is the frequency of irrigation ?

(a) 9 days (b) 13 days (c) 21 days (d) 25 days

12. A groundwater basin consists of 10 km2 area of plains. The maximum groundwater table fluctuation has been observed to be 1.5 m. Consider specific yield of the basin as 10%. What is the available groundwater storage in million cubic metres?

(a) 1.0 (b) 1.5 (c) 2.5 (d) 2.0

13. In a canal irrigation project, 76% of the culturable command area (CCA) remained without water during Kharif season; and 58% of CCA remained without water during Rabi season in a particular year. Rest of the areas got irrigated in each crop respectively. What is the intensity of irrigation for the project in the year ?

(a) 134% (b) 76% (c) 66% (d) 58%

14. The discharge required for Rabi and Kharif crops are 0.4 m^3/s and 0.3 m^3/s respectively. The capacity and time factors are 0.8 and 0.5 respectively at each season. The design discharge of the distributary at its head is

(a) 0.8 m^3/s (b) 0.16 m^3/s (c) 1.0 m^3/s (d) 1.24 m^3/s

15. The total irrigation depth of water, required by a certain crop in its entire growing period (150 days), is 25.92 cm. The culturable command area for a distributary channel is 100,000 hectares. The distributary channel shall be designed for a discharge

(a) less than 2 cumecs (b) 2 cumecs (c) 20 cumecs (d) more than 20cumecs

16. The moisture content of soil in the root zone of an agricultural crop at certain stage is found to be 0.05. The field capacity of the soil is 0.15. The root zone depth is 1.1 m. The consumptive use of crop at this stage is 2.5 mm/day and there is no precipitation during this period. Irrigation efficiency is 65%. It is intended to raise the moisture content to the field capacity in 8 days through irrigation. The necessary depth of irrigation is

(a) 115 mm (b) 169 mm (c) 200 mm (d) 285 mm

17. A canal irrigates a portion of a culturable command area to grow sugarcane and wheat. The average discharges required to grow sugarcane and wheat are, respectively, 0.36 and 0.27 cumecs. The time factor is 0.9. The required design capacity of the canal is

(a) 0.36 cumecs (b) 0.40 cumecs (c) 0.63 cumecs (d) 0.70 cumecs

18. The culturable commanded area for a distributary is 2×10^8 m^2. The intensity of irrigation for a crop is 40%. If kor water depth and kor period for the crop are 14 cm and 4 weeks, respectively, the peak demand discharge is

(a) 2.63 m^3/s (b) 4.63 m^3/s (c) 8.58 m^3/s (d) 11.58 m^3/s

19. The consumptive use of water for a crop during a particular stage of growth is 2.0 mm/day. The maximum depth of available water in the root zone is 60 mm. Irrigation is required when the amount of available water is 50% of the maximum available water in the root zone. Frequency of irrigation should be

(a) 10 days (b) 15 days (c) 20 days (d) 25 days

20. The culturable command area for a distributed channel is 20,000 hectares. Wheat is grown in the entire area and the intensity of irrigation is 50%. The kor period for wheat is 30 days and

the kor water depth is 120 mm. The outlet discharge for the distributary should be

(a) 2.85 m^3/s (b) 3.21 m^3/s (c) 4.63 m^3/s (d) 5.23 m^3/s

21. An outlet irrigates an area of 20 ha. The discharge (L/s) required at this outlet to meet the evapotranspiration requirement of 20 mm occurring uniformly in 20 days neglecting other field losses is

(a) 2.52 (b) 2.31 (c) 2.01 (d) 1.52

22. An agricultural land of 437 ha is to be irrigated for a particular crop. The base period of the crop is 90 days and the total depth of water required by the crop is 105 cm. If a rainfall of 15 cm occurs during the base period, the duty of irrigation water is

(a) 437 ha/cumec (b) 486 ha/cumec (c) 741 ha/cumec (d) 864 ha/cumec

23. The most expected crops in a hot arid district of Rajasthan State in India, in the month of September, are

(a) Rice & Sugarcane (b) Bajra & Maize (c) Wheat & Maize (d) Tobacco & Cotton

24. The maximum irrigation requirement of Rice crop is exhibited by its

(a) maximum delta value (b) maximum duty value
(c) minimum duty value (d) none of the above

25. The crop among the following, which is expected to have the maximum duty, is

(a) Wheat (b) Rice (c) Sugarcane (d) Cotton

26. Kor-Watering is the irrigation water supplied to a crop

(a) at the time of its sowing (b) just before harvesting
(c) about three weeks after sowing (d) about three weeks before harvesting

27. The duty of irrigation water for a given crop is maximum

(a) on the field (b) at the head of the main canal
(c) at the head of the water-course (d) none of them

28. The duty at the end point of a canal minor, where the Govt. control usually ceases, is called

(a) duty on field (b) outlet duty (c) flow duty (d) storage duty

29. The first important watering of crops is usually called

(a) paleo watering (b) kor-watering (c) crop-watering (d) all of the above

30. The crop sequence, which cannot serve any useful purpose in 'Crop rotation', is

(a) Wheat–Jowar–Gram (b) Rice–Gram–Rice
(c) Cotton–Wheat–Gram (d) Rice–Wheat–Cotton

31. The optimum depth of kor watering for rice is about

(a) 25 cm (b) 19 cm (c) 13.5 cm (d) 9 cm

32. In India, the cultivated area under Rabi season is generally x times the area under Kharif season; where x is

(a) 1 (b) $\frac{1}{2}$ (c) 2 (d) none of them

33. The efficiency of water application does not depend upon

(a) climatic conditions
(b) type of the soil
(c) method of application
(d) geometry of the conveyance system

34. If the intensity of irrigation for Kharif is 45% and that for Rabi is 60%; then the annual intensity of irrigation, is

(a) 60% (b) 100% (c) 105% (d) none of them

35. A ridge canal is also called a

(a) watershed canal (b) contour canal (c) side slope canal (d) none of the above

36 The canal, which may frequently encounter cross-drainage works, will be a

(a) watershed canal (b) contour canal (c) side slope canal (d) none of the above

37. The canal, which can irrigate only on one side, is a

(a) watershed canal (b) contour canal (c) side slope canal (d) none of the above

38. 'Duty on capacity' is also called

(a) outlet duty
(b) capacity factor
(c) full supply coefficient
(d) quantity duty

39. A canal was designed to supply the irrigation needs of 1200 ha of land growing rice of 140 days base period and having a delta of 134 cm. If the canal waters are used to irrigate wheat of base period 120 days and having a delta of 52 cm, the area that can be irrigated is

(a) 2651 ha (b) 543 ha (c) 3608 ha (d) 399 ha

40. A tank has an available storage of 10 Mm^3. If the overall losses are 10%, this storage can irrigate a wheat crop of base period 120 days and duty 40 cm, planted in an area of

(a) 26 ha (b) 2500 ha (c) 2250 ha (d) 2592 ha

41. The discharge capacity required at the outlet to irrigate 2600 ha of sugarcane having a *kor* depth of 17 cm and a kor period of 30 days is

(a) 2.3 m^3/s (b) 0.18 m^3/s (c) 14.7 m^3/s (d) 1.71 m^3/s

42. A flow of 150 litre/s was supplied from a tank to irrigate 2 ha of land for 8 hours. If it was found that the actual delivery rate at the farm was 120 litre/s and the run off loss in the field was 1000 m^3, the wear application efficiency of this system is

(a) 80% (b) 61% (c) 77% (d) 71%

43. The water conveyance efficiency of an irrigation system is

(a) $\dfrac{\text{Crop yield}}{\text{Total amount of water used in the field}}$
(b) $\dfrac{\text{Water actually strored in the root zone}}{\text{Water delivered to the farm}}$

(c) $\dfrac{\text{Consumptive use}}{\text{Water delivered from the source}}$
(d) $\dfrac{\text{Water reaching the farm}}{\text{Water delivered form the source}}$

ANSWERS

1. (c)	10. (c)	19. (b)	28. (b)	37. (b)
2. (b)	11. (a)	20. (c)	29. (b)	38. (c)
3. (d)	12. (b)	21. (b)	30. (d)	39. (a)
4. (b)	13. (c)	22. (d)	31. (b)	40. (c)
5. (d)	14. (c)	23. (b)	32. (c)	41. (d)
6. (b)	15. (d)	24. (c)	33. (d)	42. (d)
7. (c)	16. (c)	25. (a)	34. (c)	43. (d)
8. (b)	17. (d)	26. (c)	35. (a)	
9. (b)	18. (b)	27. (a)	36. (b)	

Water Logging &
Reclamation of Saline Soils

CHAPTER

4

Water Logging & Reclamation of Saline Soils

WATER LOGGING

- *Water logging* is a phenomena in which productivity of land gets affected due to the high watertable leading to flooding of root zone of the plants and making the root zones of the plants ill-aerated.
- The life of a plant, depends upon the nutrients like nitrates, and the form in which the nitrates are consumed by the plants is produced by the bacteria, under a process called *nitrification*. These bacteria needs oxygen for their survival. The supply of oxygen gets cutoff when the land becomes ill aerated, resulting in the death of these bacteria, and fall in the production of plant's food (i.e. nitrates) and consequent reduction in the plant growth.

Other problems created by water logging are

(i) The normal cultivation operations, (such as tilling, ploughing, etc.) cannot be easily carried out in wet soils. In extreme cases, the free water may rise above the surface of the land, making the cultivation operation impossible.

(ii) Water logging also leads to salinity

If the watertable has risen up, or if the plant roots comes within the capillary fringe, water is continuously evaporated by capillarity. Thus, a continuous upward flow of water from the watertable to the land surface, gets established. With this upward flow, the salts which are present in the water, also rise towards the surface, resulting in the deposition of salts in the root zone of the crops. The concentration of these alkali salts present in the root zone of the crops has a corroding effect on the roots, which reduces the osmotic activity of the plants and checks the plant growth, and the plant ultimately fades away. Such soils are called **saline soils**.

Note : Whenever there is water logging, there is salinity.

CAUSES OF WATER LOGGING

(i) **Over and Intensive Irrigation:** When a policy of intensive irrigation is adopted, then the maximum irrigable area of a small region is irrigated. This leads to heavy irrigation in that region resulting in heavy percolation and subsequent rise of watertable. Hence, policy of extensive irrigation should be preferred.

(ii) Seepage of water from the Adjoining High Lands: Water from the adjoining high lands may seep into the sub-soil of the affected land and may raise the watertable.

(iii) Seepage of Water through the Canals; Water may seep through the beds and sides of the adjoining canals, reservoirs, etc., situated at a higher level then the affected land. resulting in high watertable. This seepage is excessive, when soil at the site of canals, reservoirs, etc., is very pervious.

(iv) Impervious Obstruction: Water seepage below the soil moves horizontally but may find an impervious obstruction causing the rise of watertable on the u/s side of the obstruction. Similarly, an impervious stratum may occur below the top layers of pervious soils. In such cases, water seepage through the pervious soils will not be able to go deep, and hence, quickly resulting in high watertable.

(v) Inadequate Surface Drainage; If proper drainage is not provided, the storm water falling over the land and the excess irrigation water will constantly percolate and will raise the level of the underground reservoir.

(vi) Inadequate Natural Drainage : Soils having less permeable sub-stratum below the top layers of pervious soils, will not be able to drain the water deep into the ground.

(vii) Heavy rains : Heavy rainfall may create temporary water logging, and in the absence of good drainage, it may lead to continued water logging.

(viii) Submergence due to Floods: If a land continuously remains submerged by floods, grasses, weeds, etc. may grow, and obstruct the natural surface drainage of the soil, this may lead to water logging

WATER LOGGING CONTROL

The various measures adopted for controlling water logging are as follow :

(i) Lining of Canals and Water Courses : There should be attempts for reducing the seepage of water from the canals and water courses. This can be achieved by lining them.

(ii) Reducing the Intensity of Irrigation: In area where there is a possibility of water logging, intensity of irrigation should be reduced. Only a small portion of irrigable land should receive canal water in one particular season. The remaining areas can receive water in the next season, by rotation.

(iii) By Introducing Crop rotation: Certain crops require more water and others require less water. If a field is always sown with a crop requiring more water, the chances of water logging are more. In order to avoid this, a high water requiring crop should be followed by one requiring less water, and then by one requiring almost no water.

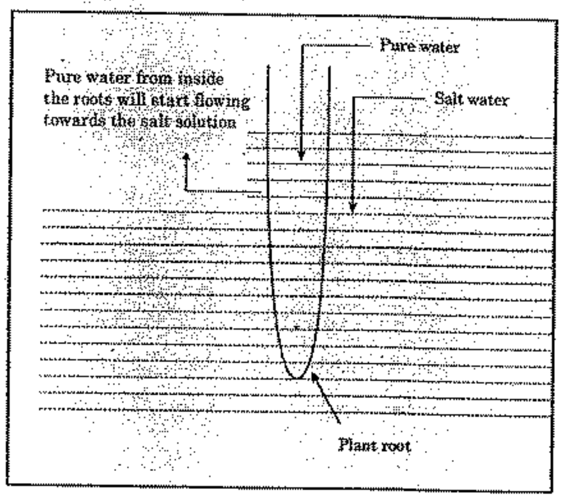
(iv) By Optimum Use of Water : Only a certain fixed amount of irrigation water gives best productivity. The unaware cultivators should be made aware about this technicality. Moreover, the revenue should not be charged on the basis of irrigated area but should be charged on the basis of the quantity of water utilised.

(v) By Providing Intercepting Drainage : Intercepting drains along the canals should be constructed, wherever necessary. These drains can intercept and prevent the seeping canal water from reaching the area likely to be water logged.

(vi) By Provision of an Efficient Drainage System : An efficient drainage system should be provided in order to drain away the storm water and the excess irrigation water A good drainage system consists of surface drains as well as sub surface drains.

RECLAMATION OF SALINE AND ALKALINE LANDS

- *Land reclamation* is a process by which an uncultivable land is made fit for cultivation. Saline and water logged lands give very less crop yields, and are therefore, almost unfit for cultivation, unless they are reclaimed.
- Every agricultural soil contains certain mineral salts in it. Some of these salts are beneficial for plants as they provide the plant foods, while others prove injurious to plant growth. These injurious salts are called alkali salts and their common examples are Na_2CO_3, Na_2SO_4, and NaCl. Na_2CO_3 is the most harmful; and NaCl is the least harmful. These salts are soluble in water.
- If the watertable rises up, or if the plants roots happen to come within the capillary fringe, water from the watertable starts flowing upward. The soluble alkali salts also move up with water and get deposited in the soil within the plant roots as well as on the surface of the land. This phenomenon of salts coming up in solution and forming a thin crust on the surface, after the evaporation of water, is called efflorescence.
- Land affected by efflorescence is called *saline soil.*
- The salt water surrounding the roots of the plants reduces the osmotic activity of the plants.
- Since the plant roots act as semi permeable membranes, so we have almost pure water on one side of the membrane (i.e., the water already extracted by the roots) and highly concentrated salt solution on the other side. So, we can conclude that pure water from within the roots will start flowing out of the roots by *osmosis* towards the salt solution, until the pressure on pure water side becomes equal to the osmotic pressure of the salt solution. Hence, the plant will die after roots has been dried up completely. Such a salt affected soil is unproductive and is known as *saline soil.*



- If the salt efflorescence continues for a longer period, a base exchange reaction sets up, particularly if the soil is clayey, thus sodiumising the clay, making it impermeable and, therefore, ill aerated and highly unproductive. Such soils are called *alkaline soils.*

Note : (i) All those measures which were suggested for preventing water logging hold good for preventing salinity of lands also. (ii) The reclamation of alkaline lands is more difficult.

Leaching

- In this process, land is flooded with adequate depth of water. The alkali salts present in the soil gets dissolved in this water which percolate down to join the watertable. The process is repeated till the salts in the top layer of the land are reduced to such an extent that some salt resistant crop can be grown. This process is known as *leaching.*
- High salt resistant crops like fodder, bajra etc. are grown on this leached land for one or two seasons or till the salinity is reduced to such an extent that an ordinary crop like wheat, cotton, citrus garden crops, etc., can be grown. The land is then said to have been *reclaimed.*

- When sodium carbonate (Na_2CO_3) is present in the saline soil, gypsum ($CaSO_4$) is generally added to the soil before leaching and thoroughly mixed with water. Na_2CO_3 reacts with $CaSO_4$ forming Na_2SO_4, which can be leached out as explained earlier.

Leaching requirement (LR) of a soil

- In order to avoid further increase in salinity, of a given soil after the land has been reclaimed it is necessary to apply water to the soil in excess of the consumptive use.
- This excess water will flow down beyond the root zone of the crop to the underground drainage system or to the underground reservoir, washing down the excess salts, which otherwise would have been deposited in the soil to further increase the salinity of the soil.
- This excess water, which is required to meet the leaching needs, is generally expressed as the percentage of the total irrigation water applied to the soil (field) to meet the consumptive use as well as the leaching needs.

$$\text{LR (Leaching Requirement)} = \frac{D_d}{D_i} = \frac{\textit{Depth of water drained out per unit area}}{\textit{Depth of irrigation water applied per unit of area}} \quad \text{... (i)}$$

where, D_i = Total irrigation water depth applied. = $C_u + D_d$

where, C_u = Consumptive use + Drained out water depth

$$\therefore \quad \boxed{L.R = \frac{D_d}{D_i} = \frac{D_i - C_u}{D_i}} \quad \text{... (ii)}$$

For salt equilibrium, the ratio D_d/D_i is found to be equal to C_i/C_d

where C_i = salt content of irrigation water, and C_d = salt content of drainage or leached water. Salt content is directly proportional to the Electrical conductivity (EC),

$\frac{C_i}{C_d}$ will be equal to $\frac{EC_{(i)}}{EC_{(d)}}$; where $EC_{(i)}$ is the electrical conductivity of irrigation water;

$EC_{(d)}$ is the electrical conductivity of drained water (leached water or leaching water).
Hence, equation (i) can be written as :

$$L.R = \frac{D_d}{D_i} = \frac{EC_{(i)}}{EC_{(d)}} \quad \text{... (iii)}$$

EC of drainage water or leaching water, i.e., EC(d), may be assumed on the basis of permissible salt tolerance limit of the grown crop, but is generally assumed to be twice the EC value of the saturation soil extract i.e., EC(e).

Hence, equation (iii) can also be written as:

$$\boxed{L.R = \frac{D_d}{D_i} = \frac{EC_{(i)}}{EC_{(d)}} = \frac{EC_{(i)}}{EC_{(e)}}} \quad \text{... (iv)}$$

LAND DRAINAGE

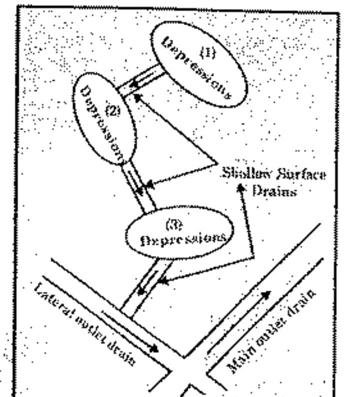
A properly designed drainage system is an effective means to prevent land from getting waterlogged. Before undertaking a drainage project few things should be kept in mind.

(i) Investigations such as topographical, geological and soil surveys should be carried out.
(ii) The nature of soil from the point of view of permeability should be studied.

(iii) A knowledge of water table and its fluctuations and the quality of ground in the area proposed for irrigation.

Two types of drainage can be provided, i.e., (i) Surface drainage, (ii) Sub-surface drainage.

Surface Drainage or Open Drainage

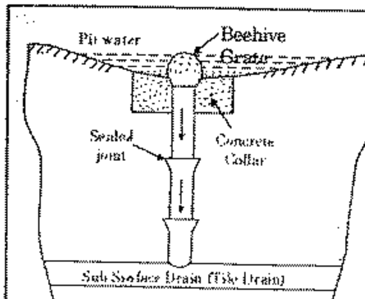


Random field-drain (shallow surface drain) system for surface drainage

- *Surface drainage* is the removal of excess rain water falling on the fields or the excess irrigation water applied to the fields, by constructing open ditches, field drains etc. Land is sloped towards these ditches or drains so as to make the excess water flow into these drains.
- In arid regions, drainage ditches become necessary to remove water required for leaching undesirable salts from the soil, and to dispose off the excess rainfall.
- The *open drains,* which are constructed to remove the excess irrigation water collected in the depressions on the fields, as well as the storm water, are broad and shallow, and are called *shallow surface drains.*
- These drains carry the runoff to the *outlet drains*, which are large enough to carry the flood water of the catchment area from the shallow surface drains, and are of sufficient depths to provide outlets even for the underground tile drains, if provided. These outlet drains may be called deep *surface drains.*
- *Surface drains* constructed for removing excess irrigation water applied to the farms and the storm water, should not be deep enough, as to interfere with the agricultural operations. They are, therefore, designed as *shallow surface drains.*
- The *shallow surface drain* are trapezoidal in cross-section. They should be strictly designed to carry the normal storm water into the fields, plus the excess irrigation water.
- Manning's equation is, generally used for the design of shallow as well as deep surface drains.
- *Deep surface drains* or *outlet drains* carry the storm water discharge from the shallow surface drains, and the seepage water coming from the underground tile drains.
- They are designed for the combined discharge of the shallow surface drains as well as that of the tile drains.

Surface Inlet

- The surface water from the pot holes, depressions road ditches, etc., may be removed either by connecting them with the shallow surface drains, or by constructing an intake structure called an open inlet or surface inlet.

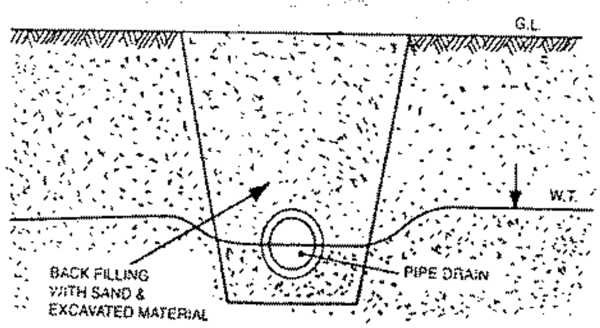


Surface inlet draining the surface water into a tile drain

- A cast iron pipe or manhole constructed of bricks can be used as surface inlets.
- At the surface of the ground, a concrete collar extending around the intake is constructed on the riser to prevent growth of vegetation and to hold it in place. On the top of the riser, beehive grate or some other suitable grate is provided, so as to prevent trash from entering the tile. When the inlet is constructed in a cultivated field, the area immediately around the intake should be kept in grass.

French Drain

- When the quantity of water to be removed from the pits or depressions is small, a blind inlet may be installed over the tile drain. The blind inlet is also called *French drain*.
- These are constructed by back filling the trench of the tile drain with graded materials, such as gravel and coarse sand.
- Such inlets are not permanently effective.
- The voids in the backfill of the blind inlet become filled up with the passage of time, thereby reducing its effectiveness.
- They are economical to be installed and do not interfere with the farming operations.



Cross-section of a tile drain in pervious soils (without any filter).

Bedding

- It is a method of surface drainage which makes use of dead furrows.
- The area between the two adjacent furrows is known as a bed.
- The depth of the bed depends on the soil characteristics and tillage practices.
- In the bedded area, the direction of farming may be parallel or normal to the dead furrows.
- Bedding is most practicable on flat slopes of less then 15%,

SUB-SURFACE DRAINAGE OR TILE DRAINAGE

- Sub surface drains are required for soils with poor internal drainage and a high watertable.
- Tile drains are usually pipe drains made up of porous earthenware and are circular in section.
- The diameters may vary from 10 to 30 cm
- The trenches in which they are laid are back filled with sand and excavated material
- The tile drains should not be placed below less pervious strata.
- If no impervious layer occurs below the land and the watertable is low (lower than about 3 m from the ground), internal soil drainage may be sufficient and no tile drains needed.
- For maximum productivity of most of the crops, both surface as well as sub surface drains may sometimes however become, essential, particularly in areas of higher water tables.

Advantage of Tile Drains:

(i) Removes the free gravity water that is not directly available to the plants

(ii) Increases the volume of soil from which roots can obtain food.

(iii) Increases air circulation.

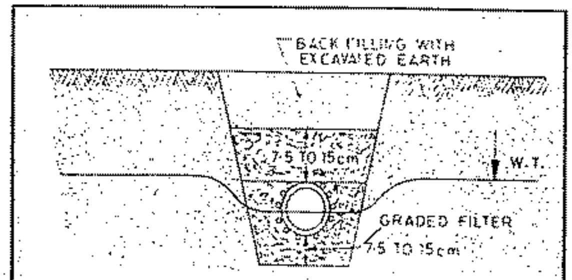
(iv) Permit deep roots development by lowering the watertable especially during spring months

(v) Increases bacterial activity in the soil, thus improving soil structure and making the plant food more readily available.

(vi) Lesser time and labour is required for tilling and harvesting.

Envelope Filters

- Enveloped filter are the graded gravel filters surrounding the tile drains when tile drains are placed in less pervious soils.
- The envelope filter serves two functions :
  (i) Prevents the inflow of the soil into the drain
  (ii) Increases the effective tile diameter, and thus increases the inflow rate.
- The filter consists of different gradations, such as gravel, coarse sand, bajra, etc.,
- Coarsest material is placed over the tile; size is gradually reduced towards the surface.
- The minimum thickness of the filter is about 7.5 cm.



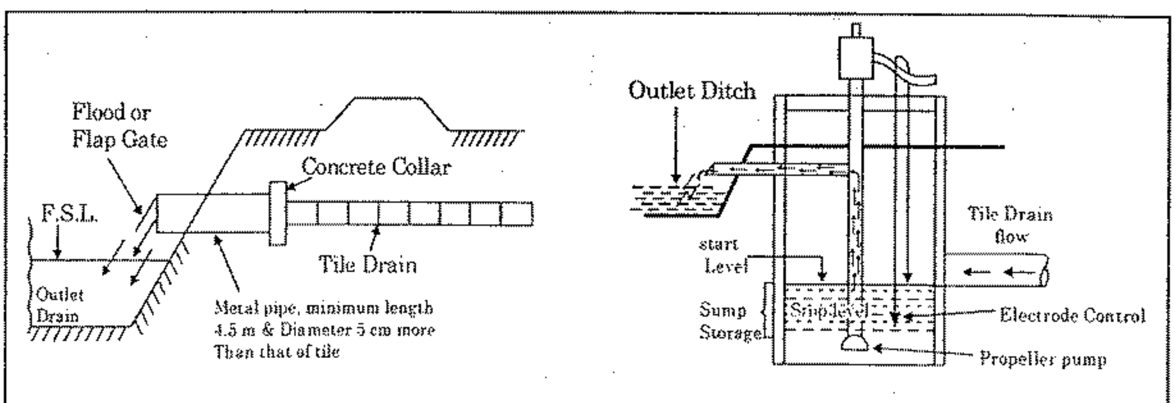
Cross-section of a tile drain in less pervious soils (*with graded filter*).

Outlets for Tile Drains or Closed Drains

The water drained by the tile drains is discharged into some bigger drains, called *deep surface drains*. The water from a tile drain may be discharged into an outlet drain either by gravity or by pumping,

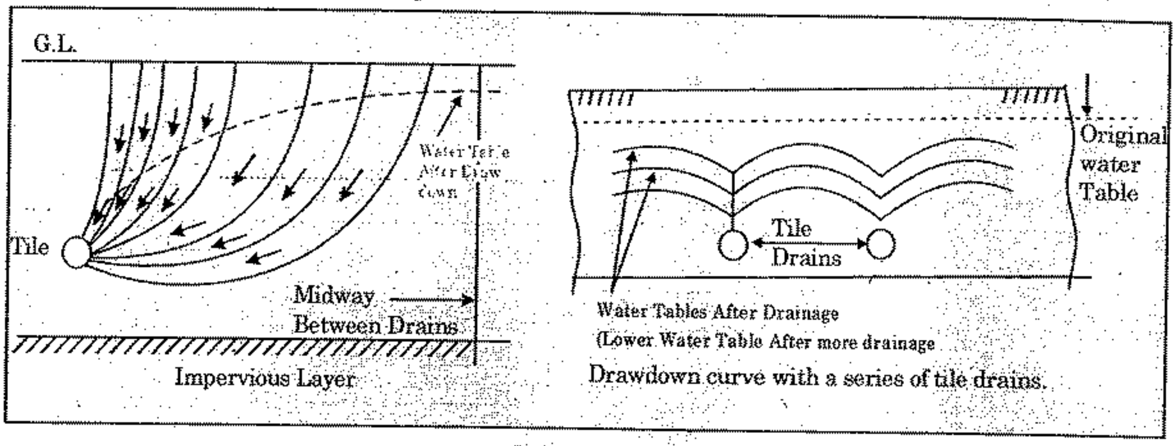
(a) Gravity outlet : If the bed level and the full supply level (FSL) of the outlet drain is lower than the invert level of the tile drain, then the water can be discharged easily into the outlet drain by the mere action of gravity.

(b) Pump outlet : When the bed level of the outlet drain is higher than that of the discharging tile drain; a pump outlet has to be installed. It consists of an automatic controlled pump with a small sump for storage. Pump outlets are costly and require technicality.



Drawdown Curve or Movement of Water into the Tile Drains

- In a fully saturated soil, water flows into the tile drain along the path as shown in fig.
- Since the quantity of water moving between any two flow lines is the same, the drawdown will be more near the tile than at the points farther away.
- After saturated soil has drained for a day, the resulting watertable will be as shown in fig.
- With series of tile drains, the sub soil water level directly over the drains, is lower than the level midway between them as shown in fig:



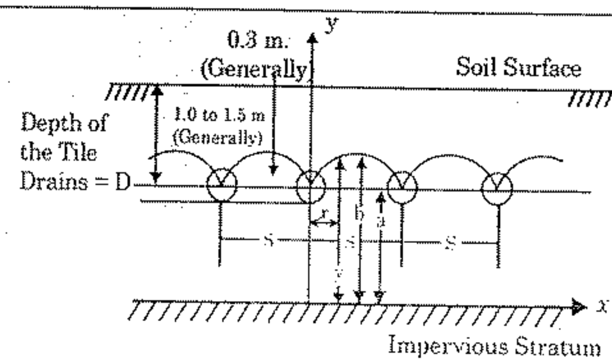
Drawdown curve with a series of tile drains.

- When a filter is provided around the tile drains to surround the drains with more pervious soil, then the overall drawdown will be more.
- Rate of drop of watertable mainly depends upon the soil permeability and spacing of the drains.

Depth and Spacing of the Tile Drains

- The closed drains are generally spaced at such a distance as to be capable of lowering the watertable sufficiently below the root zone of the plants.
- For most of the plants, the top point of the watertable must be at least 1.0 to 1.5 meters below the ground level
- Tile drains may be placed at about 0.3 m below the desired highest level of the watertable.

RELATION TO PREDICT THE SPACING BETWEEN THE TILE DRAINS IF DISCHARGE IS UNKNOWN



Spacing of tile drains.

Let S be spacing between the drains, and a be the depth of impervious stratum from the centre of the drains, as shown in figure. Let the maximum height of the drained watertable above the impervious layer be b. At any distance x from the centre of a drain, let the height of the watertable above the impervious stratum be y.

According to Darcy's Law, we have $Q = KIA$

where, K = permeability coefficient in m/sec.

∴ Discharge per unit length of the drain passing the section at y (q_y) is given as :

$$q_y = K \cdot \frac{dy}{ds} \cdot y$$

Assuming the inclination of the water surface to be small, such that the tangent$\left(\text{i.e. } \frac{dy}{dx}\right)$ can be used in place of sine$\left(\text{i.e. } \frac{dy}{ds}\right)$ for the hydraulic gradient,

we get $$q_y = K \cdot \frac{dy}{dx} \cdot y \qquad \text{... (i)}$$

when $x = S/2, \quad q_y = 0.$

and $x = 0, \quad q_y = q/2$

where, q is total discharge per unit length carried by the drain ($\frac{q}{2}$ enters the drain from either side).

Also assuming that q is linearly varying with the distance from the drain,

we can write $$q_y = \frac{1}{2}q - \frac{1}{2}\ q\ \frac{x}{s/2} = \frac{1}{2}\left(1 - \frac{x}{s/2}\right)$$

or $$q_y = \frac{q}{2S}\ [S - 2x] \qquad \text{... (ii)}$$

Equating equation (i) and (ii), we get

$$\frac{q}{2S}\ (S - 2x) = K\ y \cdot \frac{dy}{dx}$$

Rearranging and integrating, we get

$$\int \frac{q}{2SK}(S - 2x)dx = \int y\ dy$$

Assuming the soil permeability to be constant, we get

$$\frac{q}{2K.S}\left[SX - \frac{2x^2}{2}\right] = \frac{y^2}{2} + C \qquad \text{... (iii)}$$

when $x = 0, \quad y = a$

$$\therefore \quad \frac{q}{2KS}[0] = \frac{a^2}{2} + C \quad \text{or} \quad C = -\frac{a^2}{2}$$

Substituting $C = -\frac{a^2}{2}$, equation (iii) becomes

$$\frac{q}{2KS}[Sx - x^2] = \frac{y^2}{2} - \frac{a^2}{2}$$

or $$\frac{q}{2KS}x(S-x) = \frac{y^2 - a^2}{2}$$

or $$q = \frac{KS(y^2 - a^2)}{(S-x)x} \quad \text{... (iv)}$$

Also, when $x = S/2$, $y = b$, equation (iv) then becomes

$$q = \frac{KS.(b^2 - a^2)}{\left(S - \frac{S}{2}\right)\frac{S}{2}} = \frac{KS(b^2 - a^2)}{\frac{S^2}{4}}$$

or $$q = \frac{4K}{S}(b^2 - a^2) \quad \text{... (v)}$$

or $$\boxed{S = \frac{4K}{q}(b^2 - a^2)} \quad \text{... (vi)}$$

Equation (vi) can be used to predict the spacing (S) between the drains, if q is known.

Note : *q will depend on the infiltration discharge into the ground, which should be removed by the drains.*

Example 1

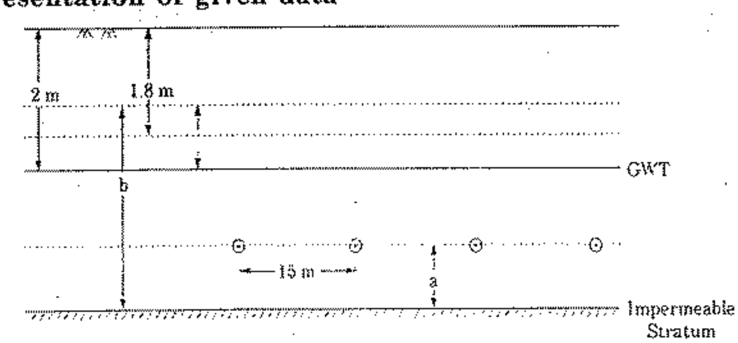
Sugar cane (root zone depth 1.8m) is grown in a particular area where the ground water table is 2.0 m below ground. If the size of soil pores is 0.08 mm in diameter and surface tension, $\sigma = 0.054$ N/m, is the field waterlogged? If so, determine the vertical location of closed drains below ground spaced at 15 m. Take drainage coefficient as 0.116 cumecs/ km^2, coefficient of permeability as 10^{-6} m/s and the impervious stratum to occur at 7.0 m below ground. Assume the capillary rise from ground water table not to interfere with the root zone of plant.

Sol. Given data, Root zone depth (d) = 1.8 m ; Ground water table = 2 m below GL

Soil pores diameter = 0.08 mm ; Surface tension (σ) = 0.054 N/m ; Spacing (S) = 15 m
Drainage coefficient = 0.116 cumec/km^2 ; Permeability (k) = 10^{-6} m/sec.

We have to determine field is water logged or not and Vertical location of drain

Pictorial representation of given data



Flow diagram:

If capillary rise (H_c) < 2 – 1.8 i.e. unable to reach root level then water logging does not occur.

where, $$H_c = \frac{4\sigma\cos\theta}{\gamma_w . d}$$

Vertical distance of tile drain from impermeable stratum (a) → CBCB

Given

$$q = \frac{4K}{S}(b^2 - a^2)$$

extreme point of capillary rise from impermeable stratum

Given

q = Drainage coeff. × S × length of tile drain

The height to which water rises by capillary action in a soil is given by

$$H_c = \frac{4\sigma\cos\theta}{\gamma_w . d}$$

where, $\theta = 0$, for max. capillary height ; d = diameter of pores = 0.08 mm = 0.08×10^{-3} m

γ_w = unit weight of water = 9.81×10^3 N/m^3 ; σ = Surface tension = 0.054 N/m

$$\therefore \quad H_c = \frac{4 \times 0.054 \times \cos 0}{9.81 \times 10^3 \times 0.08 \times 10^{-3}} = 0.275 \text{ m}$$

Thus, the capillary saturated zone stands at 2 – 0.275 = 1.725 m below the ground, thereby causing slight water logging, since the roots extend upto 1.8 m below the ground.

Now, we know that $$q = \frac{4k}{S}(b^2 - a^2)$$

where q is total discharge per unit length carried by the drain

k = coefficient of permeability (in m/s) ; s = spacing between the drains (in m)

a = depth of impervious stratum from the centre of the drains (in m)

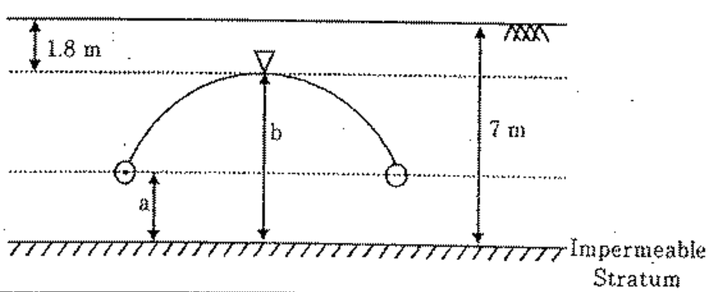
b = maximum height of the drained water table above the impervious layer (in m).

But, q = Drainage coefficient × S × length of tile drain $= \frac{0.116}{10^6} \times 15 \times 1$ ....

[considering unit length of drain]

$$\Rightarrow \quad q = 1.74 \times 10^{-6} \text{ cumecs/m}$$

Given that $k = 10^{-6}$ m/s, s = 15m,



water table should be rough down to such a level that oven the capillar water does not each the root zero depth

$$b = 7 - 1.8 - 0.275 = 4.925 \text{ m}$$

$$\therefore \quad q = \frac{4k}{S}(b^2 - a^2)$$

$$\Rightarrow \quad 1.74 \times 10^{-6} = \frac{4\times10^{-6}}{15}\left[(5.2)^2 - a^2\right]$$

$$\Rightarrow \quad \frac{1.74\times10^{-6}\times15}{4\times10^{-6}} = (5.2)^2 - a^2$$

$$\Rightarrow \quad a^2 = (5.2)^2 - 6.525$$

$$\Rightarrow \quad a = 4.529 \text{ m}$$

$$\therefore \quad \text{Depth of tile drains} = 7 - 4.529 = 2.471 \text{ m}$$

Thus the tile drains should be laid at 2.471 m below the ground.

DRAINAGE COEFFICIENT (D.C)

- The rate at which the water is removed by a drain is called the drainage coefficient.
- It is expressed as depth of water in cm or m, to be removed in 24 hours from the drainage area.
- Depends upon the rainfall but varies with type of soil & crop,degree of surface drainage etc.
- Its recommended value is 1% of the average annual rainfall to be removed per day.
- Values of 1 to 2.5 cm/day for mineral soils and 1.25 to 10 cm/day for organic soils for different crops, have been suggested for humid regions

Drainage Area : The area actually drained by the tile drain system is called its drainage area. Sometimes, the surface water is also to be removed by the tiles. In that case, the watershed area will be the drainage area even though it may not be entirely tiled.

Size of the Tile Drains : The tile drains are designed according the Manning's formula to carry a certain discharge decided by D.C. and drainage area. The drains are laid on a certain longitudinal slope varying from 0.05 to 3%. A desirable minimum working grade is 0.2%. 10 to 15 cm tiles are minimum recommended sizes.

Example 2

Determine the size of a tile at the outlet of a 6 hectare drainage system, if the D.C. is 1 cm and the tile grade is 0.3%. Assume the rugosity coefficient for the tile drain material as 0.011.

Sol. Given data : Area = 6 ha. ; Drainage coefficient (DC) = 1 cm.

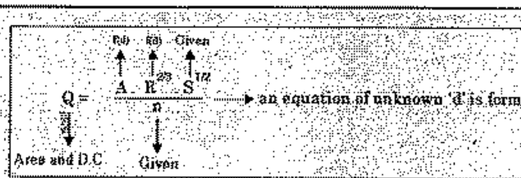
(1 cm D.C. means that 1 cm of water from an area of 6 hectares is entering the tiles per day).

Tile grade (S) = 0.3 % ; n = 0.011

Determine : Size of tile (d)

Flow diagram:

Since here 'n' is given so it is a great hint that we should use Manning's formula



$\therefore$ Volume of water passing the drain in 1 day $= \left(\dfrac{1}{100} \times 6 \times 10^4\right) = 600$ m³/day

$\therefore$ Volume of water passing the drain in 1 second $= \left(\dfrac{600}{24 \times 3600}\right) = \dfrac{1}{144} m^3/s$

$$\therefore \quad Q = \frac{1}{144} m^3/s$$

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

For a circular drain of diameter D, we have

$$A = \frac{\pi D^2}{4}, P = \pi D, R = \frac{D}{4}$$

or $$\frac{1}{144} = \frac{1}{0.011} \times \left(\frac{\pi D^2}{4}\right) \cdot \left(\frac{D}{4}\right)^{2/3} \cdot \left(\frac{0.3}{100}\right)^{1/2}$$

or $$\frac{1}{144} \times \frac{0.011 \times 4}{\pi} = \frac{D^2.D^{2/3}}{(4)^{2/3}} \times \frac{1}{\sqrt{333.3}}$$

or $$\frac{0.011 \times 4 \times 2.52 \times 18.26}{144 \times \pi} = D^{8/3}$$

or $$D = (0.00447)^{3/8} = 0.132 \text{ metre} = \mathbf{13.2\ cm}$$

Example 3

The extract of a saturated solution from from the soil has an EC of 10 m-mhos/cm and the EC of irrigation water is 1.5 m-mho/cm. What is the leaching requirement? If the consumptive use of crop is 6 cm, what depth of water has to be applied? Work from the first principles.

Sol. $$LR = \frac{D_d}{D_i} = \frac{EC_{(i)} \text{ given}}{EC_{(d)} \text{ given}}$$

LR can be calculated

$$\frac{D_d}{D_i} = LR \Rightarrow \frac{D_c + D_i}{D_i} = LR \Rightarrow \text{(DC) given, (LR) calculated, } \Rightarrow (D_i) \text{ can be calculated.}$$

The water to be applied over and above the irrigation requirement to leach out the excess salts (to keep the salinity of the soil below a specific limit is termed as leaching requirement (L.R). For salt balance in an irrigation area, $V_iC_i = V_dC_d$

where, V = volume of water, C = salt concentration.

Then $$(d_i \times 1)\, C_i = (d_d \times 1)C_d \qquad \text{(considering unit area)}$$

Hence, $$LR = \frac{d_d}{d_i} = \frac{C_i}{C_d} = \frac{EC_i}{EC_d} \quad \text{... (i)}$$

where, $d_i = d_c + d_d$ (d_c = consumptive use)

Using eqn. (i), $d_i = d_c + LR\, d_i$

or $$d_i = \frac{d_c}{1 - LR} \quad \text{... (ii)}$$

In the given problem, assuming EC of drainage water as twice that of the soil extract. Then

$$LR = \frac{E_{ci}}{E_{cd}} = \frac{1.5}{2 \times 10} = 0.075 \text{ (or 7.5\%)}$$

This means that 7.5% more water than the consumptive requirement is required.

Hence $$d_i = \frac{d_c}{1 - LR} = \frac{6}{1 - 0.075} = 6.5\text{cm}$$

and $$d_d = d_i - d_c = 6.5 - 6 = 0.5 \text{ cm}$$

Note: Here, $$LR = \frac{d_d}{d_i} = \frac{0.5}{6.5} = 0.075 \text{ (or 7.5\%)}$$

OBJECTIVE QUESTIONS

1. Pick up the incorrect statement from the following :
   (a) intensive irrigation should be avoided in areas susceptible to water-logging
   (b) extensive irrigation should be adopted in areas susceptible to water logging
   (c) lift irrigation can help alleviate water logging susceptibilities.
   (d) none of the above.

2. Point out the incorrect statement, out of the following :
   (a) salinity is caused by water-logging (b) water logging is not caused by salinity
   (c) salinity subsides when waterlogging is removed (d) none of the above

3. Alkaline soils are best reclaimed by :
   (a) leaching (b) addition of gypsum to soil
   (c) providing good drainage (d) addition of gypsum to soil & leaching

4. A soil has an Exchangeable Sodium Percentage (ESP) of 16%, and its Electrical conductivity (EC) is 32 milli-mhos/cm. If the pH of this soil is 9.5, then the soil is classified as :
   (a) alkaline soil (b) saline soil (c) saline alkali soil (d) none of the above

5. The soil becomes practically infertile when its pH value is above :
   (a) 0 (b) 7 (c) 11 (d) none of the above

6. While growing crops in high water-table areas, open jointed drains, discharging into outlet drains, are sometimes laid below the cropped land to lower down the water-table and to remove the rain water effectively. These drains are called :
   (a) tile drains (b) french drains (c) gravity outlets (d) surface drains

7. A tile drain is laid below a cropped land to remove excess irrigation water. The Drainage Coefficient of this drain, is usually expressed as;
   (a) cm of water depth removed from the drainage area per day
   (b) cum of water removed per second
   (c) percentage of applied water, which is intercepted by this drain
   (d) none of the above.

8. The method, which uses dead furrows on cropped farms for drainage of excess irrigation or rain water, is called :
   (a) surface inlet (b) tile drainage (c) bedding (d) french drain

9. **Assertion (A) :** Leaching requirement is defined as the fraction of irrigation water that must be leached through the root zone of the plant in order to prevent soil salinity exceeding a specified level.

   Reason (R) : The concept of leaching requirement can be used to compute the quantity of drainage water that must be removed from an identified land spread.

10. The spacing of tile drains to relieve water-logged land is directly proportional to the

(a) depth of drain below the ground surface
(b) depth of impervious strata from the drain
(c) depth of drain below the water level
(d) coefficient of permeability of the soil to be drained

11 Acidic soils are reclaimed by
(a) leaching of the soil
(b) using limestone as a soil amendment
(c) using gypsum as a soil amendment
(d) provision of drainage

12. Leaching is a process
(a) by which alkali salts present in the soil are dissolved and drained away
(b) by which alkali salts in soil come up with water
(c) of draining excess water of irrigation
(d) which controls water-logging

13. A drainage coefficient
(a) decides the choice of the method of the drainage
(b) decides the kind of crop that can be grown on the land
(c) is the depth of water that can be removed from the drainage area in unit time.
(d) is the flow of water from the soil into the tile laterals per unit time

14. Which one of the followings does not contribute to water logging?
(a) inadequate drainage
(b) seepage from unlined canals
(c) frequent flooding
(d) excessive tapping of ground water

15. A recently reclaimed alkaline soil should preferably be sown with a salt resistant crop, like:
(a) wheat (b) cotton (c) barseem (d) any of the above

ANSWERS

1. (d)	4. (a)	7. (a)	10. (d)	13. (c)
2. (c)	5. (c)	8. (c)	11. (b)	14. (d)
3. (d)	6. (a)	9. (b)	12. (a)	15. (c)

Canal Design

CHAPTER

5

Canal Design

SEDIMENT TRANSPORT

Water flowing in a channel has a tendency to scour its surface. Silt, gravel or even large boulders are detached from the bed or sides of the channel and moved d/s by the moving water. This phenomenon is known as *Sediment transport.*

IMPORTANCE OF SEDIMENT TRANSPORT

- Sediment transport phenomena causes large scale scouring and siltation of irrigation canals.
- Bed levels may change by direct scouring or deposition of sediment, and thereby leading to change in flood levels
- Scouring & silting of the river banks may create sharp and irregular curves, which increases the flow resistance of the channel, thereby, flood levels gets raised for the same discharge.
- Silting of reservoir and rivers is an important aspect of sediment transport. The storage capacity of the reservoir is reduced by silting, thereby, reducing use and life of the reservoir. Natural rivers used for navigation are frequently silted up. This leads to drastic reduction in the clear depth for navigation, thus, requiring costly dredgings.

SEDIMENT LOAD

Sediment load is the burden of sediment carried by the flowing water in a canal. The sediment moving in water has been classified as:

(1) Bed load (2) Suspended load.

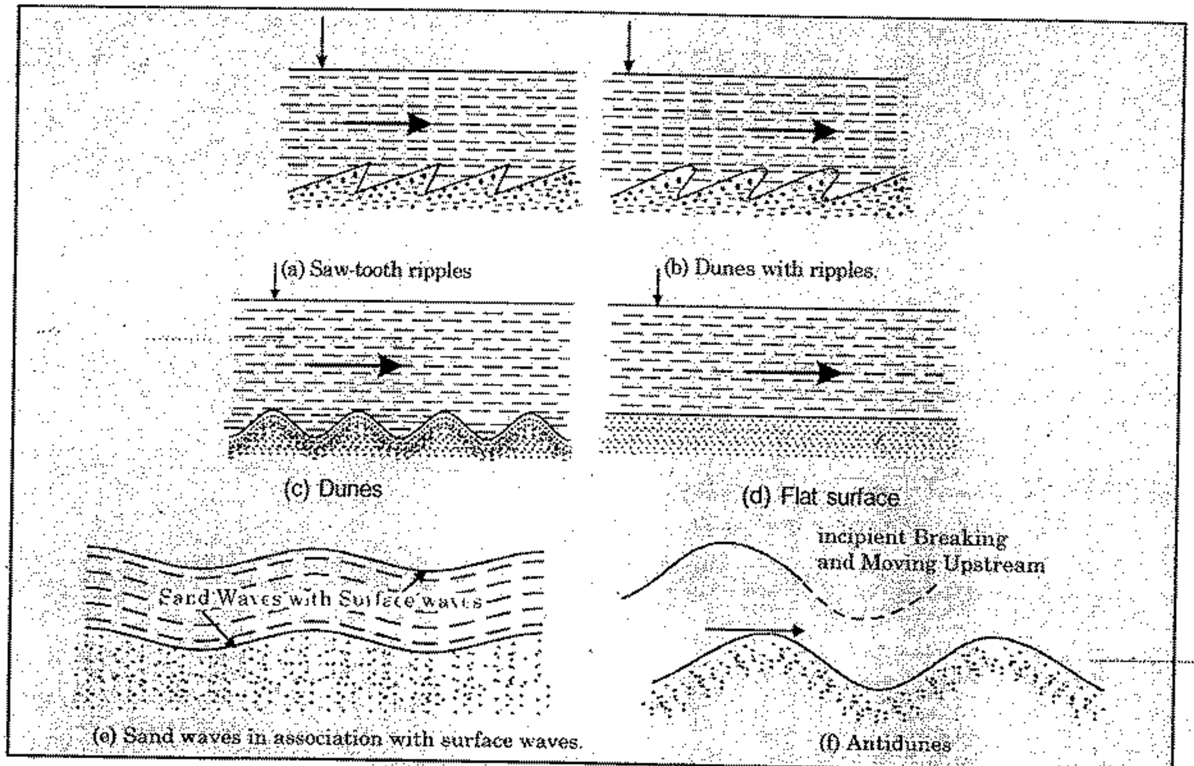
(1) Bed Load: Sediment load moves along the bed with occasional jumps into the channel.

(2) Suspended Load: Sediment load is maintained in suspension due to the turbulence of the flowing water.

MECHANISM OF BED FORMATION

- The channel bed may get distorted into various shapes by the moving water, depending upon the discharge or the velocity of the water.
- At low velocities, the bed does not move at all, but it goes on assuming different shapes as the velocity increases.
- When velocity is gradually increased then a stage is reached when the sediment load comes just at the point of motion. This stage is known as *threshold stage of motion.*

- On further increase of velocity, bed develops the saw-tooth type ripples.
- As the velocity is increased further, larger periodic irregularities appear, and are called *dunes*.
- When they first appear, ripples are superimposed on them. But at higher velocities, the ripples disappear and only the dunes are left.



- When the velocity is increased beyond formation of dunes, the dunes are erased by the flow, leaving very small undulations or virtually a flat surface with sediment particles in motion.
- Now, increase in velocity results in formation of sand waves in association with surface waves.
- As velocity is further increased, Froude no. $\left(i.e \dfrac{V}{\sqrt{gy}}\right)$ exceeds unity, the flow becomes super-critical, and the surface waves become so steep that they break intermittently and move u/s, although the sediment particles keep on moving d/s only. Sand Waves are then called *antidunes*, since direction of movement of bed forms in this regime is opposite to that of the dunes.

Note: (i) Dunes may form in any grain size of sediment, but ripples do not occur if the size of the bed particles is coarser than 0.6 mm. (ii) In case of canals and natural streams, anti-dunes rarely occur. (iii) Dunes are much larger and more rounded than ripples.

MECHANICS OF SEDIMENT TRANSPORT

- We assume the soil to be incoherent and study each soil grain individually.
- We confine ourselves to the mechanism of movement of sands and gravels only as most of our river beds are made up of sands and gravels.
- **Basic mechanics involved:** Drag force exerted by water in direction of flow on channel bed.
- This force is known as *Tractive Force or Shear Force or Drag Force*

Let us consider a channel of length L and cross-sectional area A.

The volume of water stored in this channel reach = AL

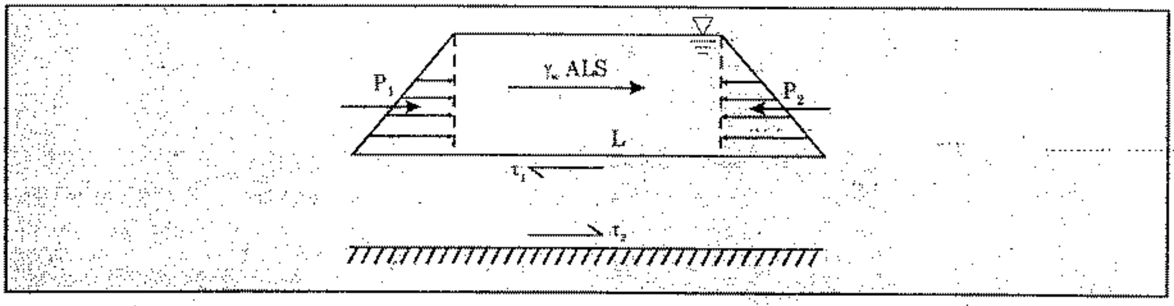
Wt. of water stored = $\gamma_w AL$

where, γ_w = unit weight of water = $\rho_w g$ (ρ_w is the density of water).

Horizontal component of this weight = $\gamma_w AL = \gamma_w ALS$ ($\because$ $\sin\theta \approx \tan\theta$; for small values of θ)

where, S = channel bed slope.

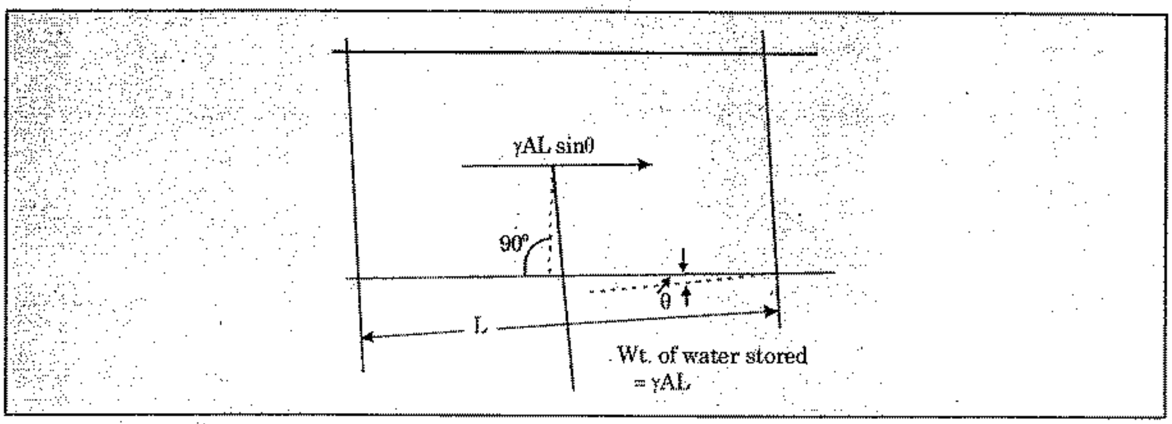
If we consider a control volume as shown below



Then for uniform flow, $P_1 = P_2$

$\Rightarrow$ $\tau_1 = \gamma_w$ ALS

$\Rightarrow$ $\tau_2 = \tau_1 = \gamma_w$ ALS = Tractive force on the wetted perimeter



This horizontal force exerted by the water is the *Tractive force.*

Average Tractive force per unit of wetted area

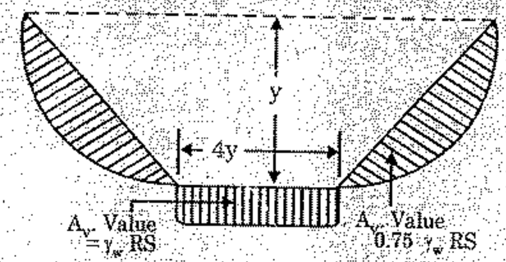
$$= \text{Unit Tractive Force } (\tau_0) = \frac{\gamma_w ALS}{\text{Wetted area}} = \frac{\gamma_w ALS}{\text{Wetted perimeter} \times \text{Length}} = \frac{\gamma_w ALS}{P.L}$$

$$= \gamma_w \left(\frac{A}{P}\right) S = \gamma_w R.S \qquad \left(\because \frac{A}{P} = R\right)$$

where, R = the hydraulic mean depth of channel ; S = channel bed slope

γ_w = unit weight of water ; P = wetted perimeter

Shear stress, $\boxed{\tau_0 = \gamma_w RS}$, here τ_0 is average drag. ... (i)



Actual distribution of tractive force in trapezoidal channel

Note: (i) Incoherent soil are cohesionless soils such as sands or gravels. (ii) The unit tractive force in channels is not uniformly distributed along the wetted perimeter (except for wide open channels).

Threshold Motion of the Sediment (For Non Scouring Channels)

- When velocity of flow through a channel is very small, the channel bed does not move at all, and the channel behaves as a *rigid boundary channel.*
- As the flow velocity increases steadily, a stage is reached when the shear force exerted by the flowing water on the bed particles will just exceed the force opposing their movement leading to intermittent movement of few particles on the bed. This condition is called the *incipient motion condition* or simply the *critical condition* or the *threshold point.*
- A knowledge of the velocity at which such a critical condition occurs is quite helpful in designing *stable non-scouring channels* admitting clear waters, since this velocity will help us in fixing the hydraulic mean depth (R) and bed slope (S_0) of the channel.

Note : *R* helps us in fixing depth (*y*) and bed width (*b*).

DESIGN OF NON-SCOURING STABLE CHANNELS HAVING PROTECTED SIDE SLOPES IN ALLUVIUMS : SHIELD'S ENTRAINMENT METHOD

Shield provided a semi-theoretical analysis of the problem of incipient condition of bed motion, and used it for designing *non-scouring channels.*

According to him, the bed particle begins to move when the *drag force* exerted by the fluid on the particle, just equals or exceeds the *resistance* offered by the particle to its movement.

(1) The *drag force* (F_1) exerted by the flow is given by

$$F_1 = K_1\left[C_D . d^2 . \frac{1}{2} . \rho_w . V_0^2\right] \quad \text{... (ii)}$$

where, K_1 = a factor depending on the shape of the particle ; C_D = coefficient of drag.

d = The dia of the particle ; ρ_w = The density of the flowing fluid *i.e.*, water.

V_0 = The velocity of flow at the top of the particle (*i.e.*, at the bottom of the channel).

The velocity of flow at the bottom of the channel (V_0) can be expressed as:

$$\frac{V_0}{V^*} = f_1 . \left(\frac{V^*d}{\nu}\right) = f_1 . R_e^* \quad \text{(Karman Prandtl equation)} \quad \text{... (iii)}$$

or

$$V_0 = V^* f_1 . R_e^* \quad \text{... (iv)}$$

where, $V^* =$ Shear friction velocity $= \sqrt{\frac{\tau_0}{\rho_w}}$

where, τ_0 = shear stress acting on the boundary of the channel

ν = Kinematic viscosity of the flowing fluid ; R_e^* = Particle Reynold Number = V^*d/ν

Also, the coefficient of drag C_D is given by:

$$C_D = f'\left(\frac{V_0.d}{\nu}\right)$$

or
$$C_D = f_2.\left(\frac{V^*d}{\nu}\right) = f_2.R_e^* \quad \text{... (v)}$$

Substituting values of V_0 and C_D from equation (iv) and (v) in equation (ii), we get

$$F_1 = K_1\left[\left(f_2.R_e^*\right).d^2.\frac{1}{2}\rho_w.\left(V^* f_1.R_e^*\right)^2\right]$$

or
$$F_1 = \left[K_1.f_2.f_1^2.\frac{1}{2}d^2.\rho_w.V^{*2}R_e^{*3}\right] \quad \text{... (vi)}$$

(2) The particle resistance (F_2) is further given by

$$F_2 = K_2\left[d^3.\left(\rho_s - \rho_w\right)g\right] \quad \text{... (vii)}$$

where ρ_s = density of particle ; ρ_w = density of fluid or water.

S_s = Specific gravity of particle ; γ_w = unit weight of fluid or water.

K_2 = a factor dependent on the shape of the particle and internal friction of soil.

$$F_2 = K_2\left[d^3.\left(\frac{\rho_s}{\rho_w} - 1\right)\rho_w.g\right] = K_2\left[d^3.\left(S_s - 1\right).\gamma_w\right]$$

$$= K_2.\gamma_w d^3\left(S_s - 1\right) \quad \text{.... (viii)}$$

At critical condition, equating equation (vi) and (viii), and introducing subscript (c) to denote critical conditions, we get

$$K_1 f_2.f_1^2\frac{1}{2}.d^2.\rho_w.V_{(c)}^{*2}R_{e(c)}^{*3} = K_2.\gamma_w.d^3.\left(S_s - 1\right) \quad \text{... (ix)}$$

or
$$\frac{\rho_w.V_{(c)}^{*2}}{\gamma_w.d\left(S_s - 1\right)} = \left(\frac{2K_2}{K_1.f_2 f_1^2}\right).R_{e(c)}^{*-3};$$

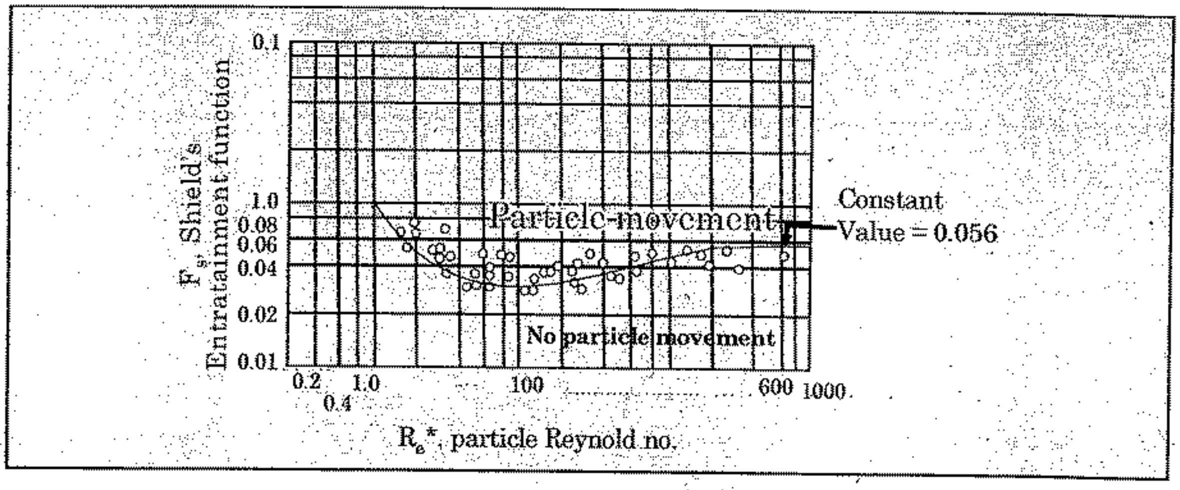
$$\rho_w.V_{(c)}^{*2} = \tau_c \qquad \because V^* = \sqrt{\frac{\tau_0}{\rho_w}}$$

$$\therefore \quad \frac{\tau_c}{\gamma_w.d.\left(S_s - 1\right)} = F.R_{e(c)}^* \quad \text{... (x)}$$

$\therefore$ Shield's Entrainment function, $F_s = F.(R_e^*)$

- From above mathematical analysis we can conclude that F_s is a function of R_e^* at critical stage of bed movement in a channel in alluviums.

- Graphs plotted between F_s and R_e^* forms a sound basis for the design of channels, where it is required to prevent bed movement or to keep it to a minimum.



In the above curve, for simplicity of calculation we take that part of curve where

- Particle Reynold's number representing roughness R_e > 400, so that F_s becomes a constant value (equal to 0.056)
- The particle size exceeds 6 mm (for coarse alluvium soil) for R_e > 400.

Hence, for designing *non-scouring channels* in coarse alluviums

$$\frac{\tau_c}{\gamma_w d(S_s-1)} = 0.056 \text{ (for d > 6 mm)},$$

here τ_c is resisting shear against movement of particle

$$\tau_c = 0.056\ \gamma_w d(S_s-1) \qquad \text{----(xi)}$$

and $\tau_0 = \gamma_w RS$ [from equation (i)]

Moreover, $\tau_0 \le \tau_c$;

$\therefore \quad \tau_0 \le \gamma_w d(S_s-1)(0.056)$ or $\gamma_w RS \le \gamma_w d(S_s-1)(0.056)$

or $RS \le d(S_s-1)(0.056)$ or $RS \le d(2.65-1)(0.056)$

or $RS \le \dfrac{d}{10.8}$ or $\boxed{d \ge 10.8RS}$

Above equation gives us the minimum size of the bed material that will remain at rest in a channel of given R and S.

✦ ***Strickler's formula*** : Manning's Rugosity Coefficient (n) according to Strickler's formula is given as: $\boxed{n = \frac{1}{24} d^{1/6}}$ d in m

- It is only applicable to the rigid boundary channels
- It does not account for *form roughness*, which is caused by the undulations in the bed of a

movable boundary channel.

- This formula is valid for rivers with beds of coarse materials, practically free from ripples.

✦ **Mittal and Swamee** has worked out a general relation between τ_c and d which gives results within +5% of the values given by *Shield's curve*, for all values of d. The relation for water and soil of S_s = 2.65, is given by equation

$$\boxed{\tau_c(\text{N/m}^2) = 0.155 + \frac{0.409 d^2_{mm}}{\sqrt{1+0.177 d^2_{mm}}}}$$

Note : The true value of *n* in Manning's equation will represent roughness of bed consisting of *grain roughness* as well as the *form roughness*, together.

For Unprotected Side Slopes

$$\boxed{\frac{\tau_c'}{\tau_c} = \sqrt{1-\frac{\sin^2\theta}{\sin^2\phi}}}$$

where, τ_c' = shear stress required to move the single grain of weight W on the side slopes.

τ_c = critical shear stress or the shear stress required to move a similar grain on a horizontal bed

θ = angle of side slope with the horizontal ; ϕ = angle of repose of soil

Hence, $\tau_c' < \tau_c$

Shear stress required to move a grain on the side slopes is less than the shear-stress required to move the grain on canal bed.

Actual shear stress on channel bed, $\tau_0 = \gamma_w RS$
On slopes, this value is given by, $\left(\tau_0'\right) = 0.75\gamma_w RS$

Example 1

An unlined irrigation canal has its bed and side composed of cohesionless material having mean diameter 6 mm. Angle of repose of the material is 40°. The bed width of the canal is 5 m and the side slope 1.5 (H) : 1 (V). Determine the maximum discharge that can be admitted into the canal without any sediment movement. Longitudinal slope of the canal is 1 in 5000 and Manning's n = 0.025.

Sol. Given, The mean diameter of particle = 6 mm

Angle of repose, ϕ = 40° ; Bed width, B = 5 m ; Side slope = 1.5 H : 1V

$\therefore \qquad \cot\theta = 1.5 \quad \Rightarrow \quad \theta = 33.69 = 0.588$

Longitudinal slope of the canal = 1 : 5000, n = 0.025
According to Shield's equation, the critical tractive stress acting on bottom of the canal is

$$\frac{\tau_c}{\gamma_w d(S_s-1)} = 0.056 \text{ (for d} \geq 6 \text{ mm)}$$

where, γ_w = unit wt. of water ; S_s = specific gravity of sediment particles

Considering cohesionless material as sand, $S_s = 2.65$

$$\therefore \quad \tau_c = 0.056\ \gamma_w\ d\ (S_s - 1) = 0.056 \times 9.81 \times \frac{6}{1000}\ (2.65 - 1)$$

$$= 5.439 \times 10^{-3}\ \text{kN/m}^2 \quad \text{... (i)}$$

Now, if τ_o is the avg. unit tractive force acting on bottom of the canal, then for nearly critical condition (i.e no sediment movement) at bed.

$$\tau_o \le \tau_c$$

$$\Rightarrow \quad \gamma_w RS \le 5.439 \times 10^{-3} \qquad (\because\ \tau_o = \gamma_w\ RS)$$

$$\Rightarrow \quad 9.81 \times R \times \frac{1}{5000} \le 5.439 \times 10^{-3}$$

$$\therefore \quad R \le 2.732\ \text{m} \quad \text{.... (ii)}$$

Now for particle movement as side slopes.

If τ_o' is the average unit tractive force acting or sides, then for critical condition.

$$\tau_o \le \tau_o'$$

where, τ_o' = critical tractive slope acting on sides

So, from the relation between τ_c and τ_o', we get

$$\frac{\tau_c'}{\tau_c} = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}}$$

$$\Rightarrow \quad \frac{\tau_c'}{\tau_c} = \sqrt{1 - \frac{\sin^2 33.69}{\sin^2 40}} = \sqrt{1 - 0.745}$$

$$\Rightarrow \quad \tau_c' = 0.505 \times 5.439 \times 10^{-3} = 2.748 \times 10^{-3}$$

[From (i)]

$$\tau_o' = 0.75\ \gamma_w RS$$

$$\therefore \quad \tau_o' \le \tau_c' \qquad \text{(critical condition)}$$

$$\Rightarrow \quad 0.75\ \gamma_w\ RS \le 2.748 \times 10^{-3}$$

$$\Rightarrow \quad 0.75 \times 9.81\ R \times \frac{1}{5000} \le 2.748 \times 10^{-3}$$

$$\Rightarrow \quad R \le 1.868\ \text{m} \quad \text{... (iii)}$$

So, for calculating maximum discharge that will not cause any sediment movement we go for the lesser value of (ii) and (iii)

$$\therefore \quad R = 1.868$$

$$\therefore \quad A = \frac{1}{2}\ [\ B + (B + 3y)] \times y = (B + 1.5\ y)\ y$$

$$P = B + 2 \times 1.8\ y = B + 3.6\ y$$

$$\therefore \quad R = \frac{A}{P} = \frac{(B+1.5y)y}{B+3.6y}$$

$$\Rightarrow \quad 1.868 = \frac{(5+1.5y)y}{5+3.6y}$$

$$\Rightarrow \quad 9.34 + 6.725\,y = 5y + 1.5y^2$$

$$\Rightarrow \quad 1.5y^2 - 1.725\,y - 9.34 = 0$$

$$\Rightarrow \quad y = 3.136 \text{ m}$$

$$\therefore \quad A = (5 + 1.5 \times 3.136) \times 3.136 = 30.43 \text{ m}^2$$

$$V = \frac{1}{n} R^{2/3} S^{1/2} = \frac{1}{0.025} \times (1.868)^{2/3} \times \sqrt{\frac{1}{5000}} = 0.858 \text{ m/s}$$

So, min. discharge, $Q = AV = 30.43 \times 0.858 = 26.11 \text{ m}^3/\text{s}$

Example 2

A canal is to be designed to carry a discharge of 56 cumec. The slope of the canal is 1 in 1000. The soil is coarse alluvium having a grain size of 5 cm. Assuming the canal to be unlined and of a trapezoidal section, determine a suitable section for the canal, ϕ may be taken as 37°.

Sol. First of all, let us choose suitable side slopes, such that $\theta < \phi$. Let θ be 30°.

Now $$\frac{\tau_c'}{\tau_c} = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}} = \sqrt{1 - \frac{\sin^2 30^\circ}{\sin^2 37^\circ}} = \sqrt{1 - \left(\frac{0.50}{0.62}\right)^2} = 0.557$$

$$\therefore \quad \frac{\tau_c'}{\tau_c} = 0.557$$

Therefore, minimum shear stress required to dislodge the grain on side slope is given by

$$\tau_c' = 0.557\,\tau_c.$$

Hence, for stability, the shear stress actually going to be generated on the slopes of a channel of given R and S must be less than or equal to $0.557\,\tau_c$

i.e., $\tau_0' \le 0.557\ \tau_c$ (for stability)

But the shear stress actually going to be generated on the side slopes of a channel of given R and S is

$$\tau_0' = 0.75\ \gamma_w RS$$

$$0.75\ \gamma_w RS < 0.557\ \tau_c$$

But $$\tau_c = \frac{\gamma_w . d}{10.8}$$

$$\therefore \quad 0.75\ \gamma_w RS \le 0.557 . \frac{\gamma_w d}{10.8}$$

or $$RS \le \frac{0.557}{0.75 \times 10.8} d$$

or $RS \le 0.0668d$

or $RS \le (0.0688 \times 0.05)$m.

or $R \times \frac{1}{1000} \le (0.0688 \times 0.05)$ when R is in metres

or $R \le 3.44$ m

or $y \le 3.44$ m. ($\because y \approx R$)

With 20% factor of safety, use y = 2.75 m ≈ 2.8 m

Hence, choose the depth as 2.8 m. Let us now choose the base width 'b' in such a way as to have a discharge of 56 cumec.

Let us use hit and trial method to determine b.

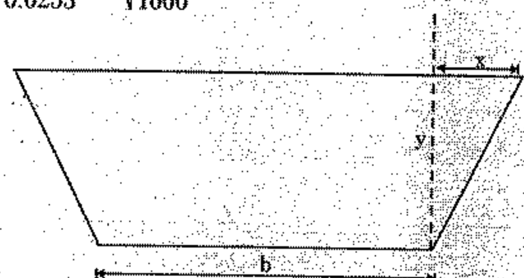
$$A = y\,(b + x) = 2.8\,(b + 4.85) \qquad \{x = y\cot\theta = 2.8\cot 30^\circ = 4.85\}$$

and

$$P = b + 2\sqrt{x^2 + y^2} = b + 2\,\sqrt{23.6 + 7.84} = b + 11.22$$

$$n = \frac{1}{24}d^{1/6} = \frac{1}{24}\times(0.05)^{1/6} = 0.0253$$

$$V = \frac{1}{n}R^{2/3}S^{1/2} = \frac{1}{0.0253}R^{2/3}\sqrt{\frac{1}{1000}} = 1.25\ R^{2/3}$$



Choose a number of trial values of b and assuming a depth of 2.8m, proceed by the table, till a discharge of 56 cumec is reached at b = 6.3 m.

b (in m)	$A = 2.8\times(b+4.85)$ (in m^2)	$P = (b+11.22)$ (in m)	$R = \frac{A}{P}$	$R^{2/3}$	$V = 1.25R^{2/3}$ (in m/s)	$Q = A\times V$ (in cumecs)
3	22	14.22	1.544	1.337	1.67	36
4	24.8	15.22	1.625	1.382	1.673	42
5	27.6	16.22	1.695	1.422	1.777	48.3
6	30.4	17.22	1.764	1.470	1.825	54.8
6.15	30.94	17.37	1.773	1.464	1.837	56.4; say 56

Hence, use 6.15 m base width and 2.8 depth **Ans.**

IMPORTANT EQUATIONS TO ESTIMATE THE TRANSPORTED SEDIMENT IN A CANAL

(1) Rouse equation

$$\boxed{\frac{c}{c_a} = \left[\frac{a(D-y)}{y(D-a)}\right]^{w_0/KV^*}}$$

where, c = sediment concentration at a distance y ; y = any distance from the bed

c_a = sediment concentration at a known distance '*a*' apart;

D = total water depth ; w_0 = fall velocity of a grain in still water,

and V^* = shear friction velocity, given by $\boxed{V^* = \sqrt{\frac{\tau_0}{\rho_w}}}$

where, τ_0 = shear stress at the bottom; ρ_w = density of water.

K = Von-Karman Universal Constant = 0.4

The equation is known as the ***suspended load concentration equation.***

Limitation of Rouse equation.

- It cannot be used directly to predict the sediment concentration at any point, unless the sediment concentration at some known distance 'a' above the channel bed is preknown.
- This equation has been derived on the assumptions of:

(i) 2-D steady flow **(ii)** Constant fall velocity (w_0) **(iii)** Constant value of Karman's constant (K)

Note: Both w_0 as well as K vary with sediment concentration and turbulence.

(2) Shield's Formula

A dimensionally homogeneous equation for sediment of uniform size, taking into account the effect of specific gravity of sediment (S_s), was proposed by Shield as:

$$\boxed{\frac{qS_s}{q_b} = 10\left[\frac{\tau_0 - \tau_c}{\gamma_w . d(S_s - 1)}\right]}$$

where, q_b = Bed load transported in m³/sec per m width of channel

S_s = Specific gravity of the bed grain ; q = Discharge per unit width in m²/sec

γ_w = Unit weight of fluid in kN/m³ ; d = Dia. of bed grain in m

τ_0 and τ_c are stresses in kN/m²

(3) Meyer-Peter's Formula

Meyer and Peter has suggested that the unit tractive force causing bed load to move, is reduced by ripples, in the ratio of

$$\tau_0' = \tau_0 . \left(\frac{n'}{n}\right)^{3/2}$$

The effective unit tractive force going to cause bed load transportation, is then given by

$$\tau_{eff} = \left[\tau_0 \left(\frac{n'}{n}\right)^{3/2} - \tau_c\right]$$

On these concepts, Meyer and Peter has suggested the following formula

$$\boxed{g_b = 0.417\left[\tau_0 \left(\frac{n'}{n}\right)^{3/2} - \tau_c\right]^{3/2}}$$

where , g_b is the rate of bed load transport (by wt.) in N per m width of channel per second.

($g_b = q_b . \gamma_w . S_s$)

where, q_b = Vol. of sediment transport per meter width of channel per second.

γ_w = Unit weight of water ; S_s = Specific gravity of grain.

n' = Manning's coefficient pertaining to grain size on an unrippled bed and given by

Note : *(i) Channel design will not be successful unless full provision is made for effects produced by the actual quantity of sediment moving in the channel. (ii)* ***Eddy Viscosity*** *: The rate of mass exchange per unit area between the adjacent layers of fluid*

DESIGN OF STABLE CHANNELS IN INDIA

- Resistance equations given by Chezy & Manning is well applicable to ***rigid boundary channels.***
- The channel in which the average shear stress (τ_0)acting on the boundary of an alluvial channel is less than the critical shear stress (τ_c) and the channel shape remains unchanged can be considered of ***rigid boundary.***
- But as soon as the sediment movement starts, undulations develop on the bed, which increases the boundary resistance of the channel.
- Besides this, some energy is required to move the grains.
- The suspended load, carried due to turbulence in the flow, further affects the resistance of the alluvial streams.
- All these factors makes the evaluation of resistance of alluvial streams a very complex problem and complexity further increases if one includes the effects of the channel shape, non-uniformity of sediment size, discharge variation, and other such factors is included.
- In India, alluvial channels are designed on the basis of hypothetical theories given by *Kennedy* and *Lacey.*
- For designing a properly functioning channel, one must think to design a channel in which neither silting nor scouring takes place. Such channels are known as stable channels or regime channels.

Regime Channels: A channel is said to be in a state of *'Regime'* if in the flow there is no need to worry about phenomena of silting and scouring. Such a state is possible only in artificial channels.

The basis for designing an ideal, non-silting, non-scouring channel is that whatever silt has entered the channel at its head is kept in suspension, so that it does not settle down and deposit at any point of the channel. Moreover, the velocity of the water should be such that it does not produce local silt by erosion of channel bed and slopes.

Regime conditions : Following conditions should be established for a channel to be in *regime.*

(i) The channel should flow uniformly in unlimited incoherent alluvium of the same character as that transported by the channel.

(ii) Silt grade and silt charge should be constant. (iii) Discharge should be constant.

KENNEDY'S THEORY

- R.G. Kennedy (an Executive Engineer of Punjab P.W.D.) in 1895 carried out extensive investigations on some of the canal reaches in the Upper Bari Doab Canal System.
- He selected some straight reaches of the canal section, which had not posed any silting and scouring problems for a long time in the past

- On basis of these observations, he concluded that the silt supporting power in a channel cross-section was mainly dependent upon the generation of the eddies, rising to the surface.
- These eddies are generated due to the friction of the flowing water with the channel surface. The vertical component of these eddies try to move the sediment up, while the weight of the sediment tries to bring it down, thus keeping the sediment in suspension. So silting will be avoided if the velocity is sufficient to generate these eddies, so as to keep the sediment just in suspension.

Kennedy defined the critical velocity (V_0) in a channel as the mean velocity (across the section) which will just keep the channel free from silting or scouring, and related it to the depth of flow

by the equation $$V_0 = c_1 y^{c_2}$$

where c_1 and c_2 are constants depending upon silt charge.

(c_1 and c_2 were found to be 0.55 and 0.64 in M.K.S. or S.I. units), respectively.

Therefore, $$V_0 = 0.55 y^{0.64}$$

V_0 in m/sec, y in m

- Above formula was worked out for a special case (i.e. Upper Bari Doab Canal system).
- For other canals as this specific case would not have been applicable, Kennedy introduced a factor in this equation to account for the type of soil through which the canal was to pass. This factor came to be known as *critical velocity ratio* (C.V.R) and was denoted by m

  The equation for critical velocity was modified as

$$\boxed{V_0 = 0.55\ m\ y^{0.64}}$$

where; V_0 = critical velocity in the channel (in m/s) ; y = water depth in channel (in metre)

m = C.V.R

Recommended Values of C.V.R. (m)

Type of silt	Value of m
Sands coarser than the standard	1.0 to 1.2
Sands finer than the standard	1.0 to 0.7.

Design procedure

Determine the critical velocity V_0 by the above equation by assuming a trial depth, and then determine area by dividing discharge by velocity (V), for this area determine channel dimensions.

Velocity is calculated by using Kutter's formula, Manning's formula, etc. If the two velocities V_0 and V are same, then the assumed depth is all right, otherwise change it and repeat the procedure, till V and V_0 become equal.

The items which must be known are:

(1) Discharge, Q (2) Rugosity coefficient, n (3) C.V.R, m (4) Bed slope S or B/D ratio

Kutter's Formula

$$\boxed{V = \left[\frac{\frac{1}{n} + \left(23 + \frac{0.00155}{S}\right)}{1 + \left(23 + \frac{0.00155}{S}\right)\frac{n}{\sqrt{R}}}\right]\sqrt{RS}}$$

Manning's Formula $\boxed{V = \frac{1}{n} R^{2/3} S^{1/2}}$

where V = Velocity of flow in metres/sec ; R = Hydraulic mean depth in metres.

S = Bed slope of the channel ; n = Rugosity coefficient

Note: *(i) The values of n in both these equations depend upon channel condition and also upon the discharge.*
(ii) Generally Kutter's equation is used with Kennedy's theory.

Drawbacks in Kennedy's Theory

The various drawbacks in Kennedy's theory are as follows

(1) This theory is aimed to design only an average regime channel

(2) The design of channel by this method involves trial and error

(3) Kutter's equation is used for determining the mean velocity of flow and hence the limitations of Kutter's equation are incorporated in Kennedy's theory.

(4) The significance of B/D ratio is not considered in this theory.

(5) No equation for the bed slope has been given which may lead to varied designs of the channel with slight variation in the bed slope.

(6) Silt charge (or silt concentration) and silt grade are not considered.

(7) The value of m is decided arbitrarily since there is no method given for determining its value.

✦ *A lot of mathematical calculations are required in designing irrigation channels by the use of Kennedy's method. To save mathematical calculations, graphical solution of Kennedy's and Kutter's equations, was evolved by Garret.*

Example 3

Design an irrigation channel by Kennedy's method, to carry 50 cubic metres per sec of discharge, with base width to depth ratio as 2.5. The critical velocity ratio is 1.1. Assume Kutter's rugosity coefficient as 0.025 and side slopes of the channel as 0.5 horizontal to 1.0 vertical.

Sol. Given: Discharge, $Q = 50\ m^3/s$; Base width to depth ratio, $\frac{B}{D} = 2.5$

Critical velocity ratio (CVR), $m = 1.1$; Kutter's rugosity coefficient, $n = 0.025$

Side slope of the channel $= 0.5\ H : 1V$

Critical velocity, $V_o = 0.55\ my^{0.64} = 0.55 \times 1.1 \times y^{0.64}$

We consider the section of the channel to be trapezoidal

Area, $A = (B + y/2)\ y$ ... (i)

$P = B + 2.236y$ ... (ii)

also, $Q = AV_o$

$$\Rightarrow \quad 50 = \left(B + \frac{y}{2}\right) y \times (0.55 \times 1.1 \times y^{0.64})$$

$\Rightarrow \quad 50 = \left(\frac{B}{y}+\frac{1}{2}\right)y^2 \times (0.55 \times 1.1 \times y^{0.64})$

$\Rightarrow \quad 50 = (2.5 + 0.5)\, y^{2.64} \times (0.55 \times 1.1)$

$\Rightarrow \quad y^{2.64} = 27.55$

$\Rightarrow \quad y = (27.55)^{1/2.64} = 3.51 \text{ m}$

Now, width $B = 2.5\ D = 8.78$ m

Putting the values of y in equation.

So, $$A = \left(8.78+\frac{3.51}{2}\right) \times 3.51 = 36.98 \text{ m}^2$$

$$P = (8.78 + 2.236 \times 3.51) = 16.63 \text{ m}$$

So, hydraulic mean depth, $R = \frac{A}{P} = \frac{36.98}{16.63} = 2.22$ m

Hence, critical velocity, $V_o = 0.55 \times 1.1 \times (y)^{0.64} = 0.55 \times 1.1 \times (3.51)^{0.64} = 1.351$ m/s

Now, mean velocity of the channel (by Kutter's formula)

$$V = \left[\frac{\frac{1}{n}+\left(23+\frac{0.00155}{S}\right)}{1+\left(23+\frac{0.00155}{S}\right)\frac{n}{\sqrt{R}}}\right]\sqrt{RS} \qquad \text{... (iii)}$$

Assume $S = \frac{1}{3000}$

Putting the value of S in equation (iii) we get.

$$V = \left[\frac{\frac{1}{0.025}+\left(23+\frac{0.00155}{\frac{1}{3000}}\right)}{1+\left(23+\frac{0.00155}{\frac{1}{3000}}\right)\frac{0.025}{\sqrt{2.22}}}\right]\sqrt{2.22\times\frac{1}{3000}} = \frac{67.65}{1.464}\times\frac{2.72}{100} = 1.257 \text{ m/s}$$

$\therefore \quad V < V_o$

Again assuming $S = \frac{1}{2600}$

$$V = \left[\frac{\frac{1}{0.025}+\left(23+\frac{0.00155}{\frac{1}{2600}}\right)}{1+\left(23+\frac{0.00155}{\frac{1}{2600}}\right)\frac{0.025}{\sqrt{2.22}}}\right]\sqrt{2.22\times\frac{1}{2600}}$$

$$= \frac{67.03}{1.45} \times (2.922 \times 10^{-2}) = 1.351$$

$$V = V_o \text{ at Slope } \quad S = \frac{1}{2600}$$

So, we can go upto a limiting slope of $S = \frac{1}{2600}$

Hence, the section should have

Base width = 8.78 m ; Depth = 3.51 m ; Bed slope = $\frac{1}{2600}$.

LACEY'S THEORY

Lacey (an eminent civil engineer of U.P. Irrigation Department), in 1939, carried out extensive investigation on the design of stable channels in alluviums. He found many drawbacks in Kennedy's Theory and he put forward his new theory.

Lacey's Regime channels

It was stated by Kennedy that a channel is said to be in a state of *'regime'* if there is neither silting nor scouring in the channel. But Lacey came out with the statement that even a channel showing no silting no scouring may actually not be in regime.

He differentiated between three regime conditions :

(1) True regime (2) Initial regime (3) Final regime.

(1) True regime

A channel shall be in regime, if there is neither silting nor scouring. In order to satisfy this condition, the silt load entering the channel must be carried through, by the channel section. Moreover, there can be only one *channel section* and one *bed slope* at which a channel carrying a given discharge and a particular quantum and type of silt, would be in regime.

Hence, an artificially constructed channel having a certain fixed section and a certain fixed slope can behave in regime only if the following conditions are satisfied.

(i) Discharge is constant;

(ii) Flow is uniform;

(iii) Silt charge is constant (constant amount of silt)

(iv) Silt grade is constant (the type and size of silt is always the same)

(v) Channel is flowing through a material which can be scoured as easily as it can be deposited (such soil is known as *incoherent alluvium*), and is of the same grade as is transported.

Hence, a designed channel shall be in *'true regime'* if the above conditions are satisfied. But in practice, all these conditions can never be satisfied. And, therefore, artificial channels can never be in *'true regime'*; they can either be in initial regime or final regime

Note: The *alluvial soil* is defined as the soil formed by continuous deposition of silt by the moving water. Such sandy soil deposits which do not possess any cohesion and are formed by the agency of water are called *incoherent alluvium*.

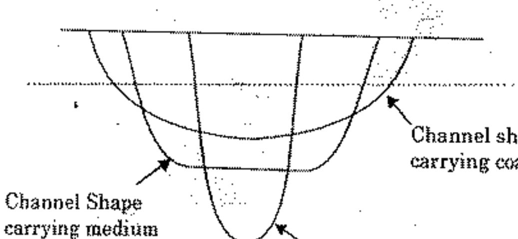
(2) Initial Regime

- It is the first stage of regime attained by a channel after it is in service.
- If a channel is excavated with smaller width and flatter bed slope, then as the flow takes place in the channel, bed slope of the channel is increased due to deposition of silt on the bed of the channel to develop an increased flow velocity. Hence, the given discharge is allowed to flow through the channel of smaller width.
- With increase in the bed slope , the depth may also vary but the width of the channel does'nt change because the sides of the channel are usually cohesive and hence they resist erosion.

So, keeping the *discharge, silt grade, silt charge* and *width* fixed and only by varying *bed slope* and *depth*, the channel attains stability. This condition is known as *initial regime.*

(3) Final Regime

- It is the ultimate state of regime attained by a channel when in addition to bed slope and depth, the width of the channel is also adjusted as per requirement.
- In this condition, the resistance of the sides of the channel is ultimately over come due to continuous action of water.
- So, the channel adjust its width, depth and bedslope in order to obtain a stable channel. This condition is known as *final regime.*



- Such a channel in which all variables are equally free to vary, has a tendency to assume a semi-elliptical section. The coarser the silt, the flatter is the semi-ellipse, i.e. greater is the width of the water-surface. The finer the silt, the more nearly the section attains a semicircle.

Note : (i) The various equations developed by Lacey are applicable to channels which has attained final regime.
(ii) Hence, we can conclude that total no. of independent equations that form the Lacey's regime theory is 3.

Design procedure for Lacey's theory

(1) Velocity can be calculated as

$$\boxed{V = \left[\frac{Qf^2}{140}\right]^{1/6} \text{ m/sec.}}$$

where, Q is in cumec : V is in m/s

f is the silt factor, given by $\boxed{f = 1.76\sqrt{d_{mm}}}$

where d_{mm} = Average particle size in mm

(2) Hydraulic mean depth (R) can be calculated as

$$\boxed{R = \frac{5}{2}\left(\frac{V^2}{f}\right)}$$ where, R is in metre

(3) Area of channel section can be computed as $A = \dfrac{Q}{V}$

(4) Wetted perimeter can be computed as , $\boxed{P = 4.75\sqrt{Q}}$

where P is metre

(5) Finally the bed slope S is determined after knowing the above values by the equation

$$\boxed{S = \left[\frac{f^{5/3}}{3340Q^{1/6}}\right]}$$

Note : If not given assume final side slope as 0.5 H : 1V generally observed field value.

- Lecey regime width & scous depth for alluvial river regime width = $4.75\sqrt{Q}$, for such stream, Lecey's regime scour depth in m = $0.48\left(\frac{Q}{f}\right)^{1/3}$, Q in m/sec.
- Above scour depth formula is applicable only when river width equals regime width. For any other river width normal scour depth is given by $1.35\left(\frac{q^2}{f}\right)^{\frac{1}{3}}$, $q = \frac{Q}{B} = \frac{\text{dsicharge}}{\text{width}}$

Drawbacks in Lacey's Theory

The various drawbacks in Lacey's theory are as follows:

(1) The characteristics of a regime channel are not precisely defined.

(2) The true regime conditions defined by Lacey are only theoretical and may not be achieved in actual practice.

(3) The derivation of various equations by considering a single factor called silt factor f is not satisfactory as there can be different value of f for the bed and the sides.

(4) Silt charge and silt grade have not been defined properly by Lacey.

(5) Lacey indicated that a true regime channel has a semi-elliptical section but the same is not supported by any of his equations

(6) The actual dimensions of stable channels are often found to be vastly different from those given by Lacey's equations with f based on the size of the bed material only. Moreaver, the values of f obtained from various equations of Lacey are often quite divergent.

(7) Lacey's equations are highly empirical and it is based on the data obtained from channels flowing in a particular type of material. So before these equations can be applied in general it would be necessary to determine the values of the constants by making observations on

existing stable channels flowing in other types of material.

Example 4

Design a regime channel for a discharge of 35 m^3/sec with silt factor of 0.9 by Lacey's theory, taking side slopes as 1 H : 2V.

Sol. Given Q = 35 m^3/s ; Silt factor, f = 0.9 ; Side slopes, = 1H : 2V

So, for a regime channel using Lacey's theory, we have

(i) Velocity, $V = \left(\frac{Qf^2}{140}\right)^{1/6} = 0.766$ m/s

(ii) Hydraulic mean depth, $R = \frac{5}{2}\left(\frac{V^2}{f}\right) = 2.5 \times 0.653 = 1.631$ m

(iii) Area of the channel, $A = \frac{Q}{v} = \frac{35}{0.766} = 45.692\ m^2$

(iv) Wetted perimeter, $P = 4.75\sqrt{Q} = 4.75\sqrt{35} = 28.10$ m

(v) Bed slope, $S = \frac{f^{5/3}}{3340Q^{1/6}} = \frac{(0.9)^{5/3}}{3340\times(35)^{1/6}} = 1.388 \times 10^{-4} = \frac{1}{7200}$

Now, for a trapezoidal channel, $A = \left(B+\frac{y}{2}\right)y$

$$\Rightarrow \quad 45.692 = (B + 0.5y)y \qquad \text{... (i)}$$

$$P = B + 2.236\ y$$

$$\Rightarrow \quad 28.10 = B + 2.236\ y \qquad \text{... (ii)}$$

$$\Rightarrow \quad 28.10 - 2.236y = B \qquad \text{... (iii)}$$

Putting the value of B in to equation No. (i)

$$45.692 = (28.10 - 2.236y + 0.5\ y)y$$

$$\Rightarrow \quad 45.692 = (28.10 - 1.736\ y)\ y$$

$$\Rightarrow \quad 1.736\ y^2 - 28.10y + 45.692 = 0$$

$$\Rightarrow \quad y = 1.834\text{m},\ 14.35\text{m}$$

$$\therefore \quad y = 1.834\ \text{m} \quad \text{(neglecting higher non feasible value)}$$

Putting value of $y = 1.834$m in equation (iii)

$$B = 28.10 - 2.236 \times 1.834 = 23.99\text{m} \simeq 24\text{m}$$

Hence, section should have: Bed width, B = 24m ; depth, y = 1.83m ; Bed slope. S = 1/7200

DIFFERENCE B/W LACEY & KENNEDY THEORY

	Kennedy		Lacey
1.	Trapezoidal shape	1.	Semielliptical
2.	Silt is kept in suspension due to eddies generated from the bottom.	2.	silt is kept in suspension due to eddies generated through out the perimeter.
3.	Recommended Kutter eq. to find velocity.	3.	Gave his own eq.
4.	No. eq. for bed slope	4.	Gave eq. to calculate bed slope.
5.	Trial & error procedure	5.	Direct procedure

LINING OF CANAL

Introduction

Irrigation water is a costly commodity and it should not be wasted while conveying from the reservoirs to the fields. Most of the canals, constructed in India to carry irrigation water, are unlined, and hence, a large part of the costly irrigation water is lost in percolation and absorption as seepage loss. Such seepage loss of the costly irrigation water must be minimized and that is done by *lining* the irrigation canals.

By lining the canal, we mean that the earthen surface of the channel is lined with a stable lining surface, such as concrete, tiles, asphalt, etc.

Advantages of Lining

(1) Seepage losses are considerably reduced if the channels are lined. A lined canal costs about 2 to 2.5 times as much as an unlined canal, but where seepage is heavy, the saving of costly irrigation water may itself be sufficient to fully justify the capital expenditure on lining.

(2) It helps in prevention of water logging.

(3) The capacity of a chosen canal section can be considerably increased by lining it. The lining presents a smooth surface and causes less resistance to the flow of water. The water flows faster by carrying more of it per second than that in an unlined canal. And since capacity is a function of velocity, the higher the velocity, the greater is the capacity of the channel.

(4) A lined canal can be designed not only smaller in cross-section but also shorter in length. The steeper gradients can be provided because higher velocities are permissible and a shorter alignment can, therefore, be selected. On the other hand, flatter slopes can be provided without silting on a lined channel compared to these on an unlined channel.

(5) Maintenance of unlined canals involve huge expenditure. This expenditure may be required on.

   (i) periodical removal of silt deposited on the bed and sides of the canal section;

   (ii) minor repairs like plugging of cracks, cuts and uneven settlements of banks; and

   (iii) removal of weeds and water plants.

   Lining reduces these charges considerably

(6) An unlined canal founded on weaker foundations is always in danger, and a breach may occur at any time. Small breaches in unlined canals can result in washing away of considerable length of embankment, leading to flooding of certain areas and causing scarcity of irrigation water in other areas. A strong concrete lining eliminates all such dangers.

Types of linings, their construction, and uses

Various types of canal linings, which are commonly adopted are enumerated below:

(1) Hard surface linings

1. Cast in-situ Cement Concrete lining
2. Shotcrete or Plaster lining
3. Cement Concrete tile lining or Brick lining
4. Asphaltic Concrete lining
5. Boulder lining

(2) Earth Type Linings

1. Compacted earth lining
2. Soil cement lining

Requirements of Good Lining

(1) **Economy:** The selection of a suitable type of lining for any canal project is mainly a question of economics and availability of material, skilled and unskilled labour, construction machinery and equipment, and time available for completing the work.

The type of lining selected should not only be economical in initial cost, but also in repair and maintenance cost.

(2) **Structural stability:** The lining should be able to withstand the differential sub-soil water pressure from behind the lining due to sub-grade getting saturated through seepage or rain or due to sudden drawdown of the canal. The lining should also be sufficiently heavy and strong to withstand the effect due to local cavity formation, if any, behind the lining.

(3) **Durability:** The canal lining should be able to withstand the natural wear and tear, such as the effect of velocity of water, rain, sunshine, frost, thawing (applicable only in cold countries), thermal and moisture changes, and chemical action of salts, etc. It should also be able to withstand the damaging effects caused by cattle traffic weed and rodent growth, etc.

(4) **Repairability:** Since the lining will get damaged with its use over a period of time, it should be such that it can be repaired easily and economically. Brick tile or concrete tile or stone boulder linings, or precast slab lining can be easily repaired, as compared to cast in situ concrete lining.

(5) **Impermeability:** The permeability of lining may decide the quantum of seepage loss from a canal, which also is governed by the depth of water in the canal, and the type of subgrade soil. the permissible values of seepage losses from a canal for a particular area will depend upon the local conditions, such as the values of land and water, population intensity, etc.

(6) **Hydraulic Efficiency:** The hydraulic efficiency of a canal, generally reduces with time, since the surface of lining gets eroded, increasing the friction factor (n), and thereby reducing its carrying capacity.

(7) **Resistance to erosion:** Sometimes, a canal may have to transport a considerable amount of sediment load, which may damage the lining by abrasion. Hence, in such a canal, the lining chosen should be able to withstand such abrasion. Cement concrete and stone boulder linings may provide better abrasion resistance, as compared to brick tile lining.

ECONOMIC JUSTIFICATION OF LINING OF CANAL

If average annual benefit is greater than average annual investment lining of the canal is desirable.

(1) Average Annual benefits

Let m cumecs of water be saved per year by preventing seepage and it is sold to farmers at the rate of Rs. R_1 per cumecs.

Then the total annual saving = $m R_1$

If Rs R_2 per year is the maintenance cost for an unlined canal and if by lining p fraction of it is saved.

Then the total money saved will be pR_2.

Therefore, total annual benefit will be $mR_1 + pR_2$ total saving per year.

(2) Annual expense/cost

Let C be the total initial investment required for lining the canal whose service life is y years.

Then, the total average annual expense for lining will be $\left(\frac{C}{y}\right)$

If 'r' percent is the rate of annual simple interest, then a locked up capital of C rupees would earn, annually $C\left(\frac{r}{100}\right)$ rupees as interest charges, and since the capital value of the asset decreases from C to zero in y years.

$\therefore$ Average annual interest cost $= \left(\frac{C+0}{2}\right)\frac{r}{100} = \frac{Cr}{2\times 100}$

Therefore, total annual expense of cost. $\frac{C}{y} + \frac{1}{2}\frac{Cr}{100}$

Lining of canal will be justified if $\frac{\text{Benefit}}{\text{cost}} \geq 1$

Assumptions

(i) Make the calculation for 1 km/length of canal and find the cost and the benefit for 1 year.

(ii) Interest rate is taken between 5 – 8 %

(iii) Percentage saving in maintenance cost is approximately 40%.

(iv) Life of canal after lining is 40 – 60 year.

(v) Seepage loss from unlined canal is 30 - 35% and from that of lined canal it may be 1 - 2%.

Example 5

An unlined canal giving a seepage loss of 4.0 cumecs per million square meters of wetted area is proposed to be lined with 12 cm thick cement concrete lining, which costs Rs. 20.00 per sq.m. Work out the economics of lining and show if the scheme is justified on the basis of the following data:

(i) Annual revenue from crops = Rs. 4,00,000 per cumec of water

(ii) Discharge of the channel = 100 cumecs

(iii) Area of the channel = 50 sq.m
(iv) Wetted perimeter of the channel = 22.4 m
(v) Wetted perimeter of the lining = 22.0 m
(vi) Annual maintenance cost of unlined channel = 12 paise per sq.m
(vii) Seepage loss in lined channel = 0.01 cumec per million sq.m of wetted perimeter
(viii) Saving in annual maintenance as a consequence to lining = 40%.
(ix) Life of lining = 40 years.
(x) Rate of interest = 6%.

Assume additional suitable data, if required.

Sol. Consider 1 Km reach of canal

$\therefore$ Wetted surface area (per Km) = Wetted perimeter × Length of canal = 22.4 × 1000 = 22400 sqm

(i) Annual Benefits

(a) *Seepage*

$$\text{Seepage loss in unlined canal} = 4 \text{ cumecs}/10^6 \text{ sqm} = \frac{4}{10^6}\times 22400 = \frac{89600}{10^6} \text{ cumec/Km}$$

Given, $$\text{Seepage loss in lined channel} = 0.01 \text{ cumec}/10^6 \text{ sqm} = \frac{0.01}{10^6}\times 22400 = \frac{224}{10^6} \text{ cumec/Km}$$

$$\text{Net saving} = \frac{89600-224}{10^6} = \frac{89376}{10^6} = 0.089376 \text{ cumecs/Km}$$

$\therefore$ Annual revenue saved per Km of channel = Rs. $0.089376 \times 4 \times 10^5$ = Rs. 35750

(b) *Saving in maintenance*

Annual maintenance cost of unlined channel = 12 paise per sqm.

Total wetted area per km length = 22400 sqm.

Annual maintenance charge for unlined channel per sq. km = Rs. $22400 \times \frac{12}{100}$ = Rs. 2688

Saving in annual maintenance because of lining = 40%

Hence, annual saving in maintenance charges = Rs. $\frac{40}{100} \times 2688$ = Rs. 1075.2

Total annual benefits per km = Rs. 35750 + 1075.2 = Rs. 36825.2

(ii) Annual costs

Area of lining per Km. of channel = wetted perimeter of lining × length of channel
= 22 × 1000 = 22000 sqm.

Cost of lining per km of channel (at Rs 20 per sqm) = 20 × 22000 = Rs. 44×10^4

Given, life of lining= 40 years.

$$\text{Depreciation cost per year} = \frac{44\times10^4}{40} = 11000$$

Given, rate of interest, r = 5%

$$\text{Avg. annual interest} = \frac{1}{2}\times C\times\frac{r}{100} = \frac{1}{2}\times 44\times10^4\times\frac{5}{100} = \text{Rs. } 11000$$

Total annual cost = 11000 + 11000 = 22000

Hence, Benefit to cost ratio $= \frac{\textit{Annual benefit}}{\textit{Annual cost}} = \frac{36825.2}{22000} = \mathbf{1.674}$

Benefit to cost ratio > 1 Hence, lining is justified.

DESIGN OF LINED IRRIGATION CHANNELS

Irrigation canals should be aligned and laid out, so that the velocity of flow is uniform under all conditions, and so that the water reaches the irrigated area at an elevation sufficient to ensure even and economical distribution. High velocities of flow can be permitted by taking the advantage of hard wearing surface, so as to ensure a hydraulically efficient channel.

In case of channels lined with hard surfaced materials, two types of channel sections are adopted

(1) Triangular channel section for smaller discharges, (with circular bottom)

(2) Trapezoidal channel section for larger discharges, (with rounded corners)

In order to increase A/P ratio, the corners are rounded and attempts are made to use deeper sections by limiting depth, etc.

(1) Triangular section

Let central depth = radius of circle = y

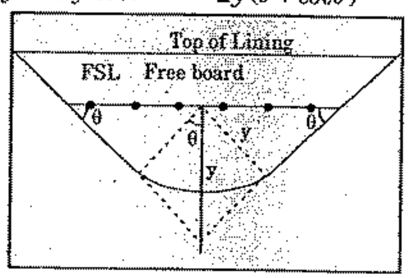
Area,
$$A = \pi . y^2 \frac{2\theta}{2\pi} + 2\times\frac{1}{2} y.y\cot\theta$$

$$A = \frac{\pi y^2\theta}{\pi} + y^2\cot\theta = y^2[\theta+\cot\theta]$$

Perimeter,
$$P = 2\pi y\times\frac{\theta}{\pi} + 2y\cot\theta = 2y.\theta + 2y.\cot\theta = 2y(\theta+\cot\theta)$$

Hence,
$$\text{Area} = y^2(\theta+\cot\theta)$$
$$\text{Perimeter} = 2y(\theta+\cot\theta)$$

$$\boxed{\text{Hydraulic mean depth} = \frac{y^2(\theta+\cot\theta)}{2y(\theta+\cot\theta)} = \frac{y}{2}}$$



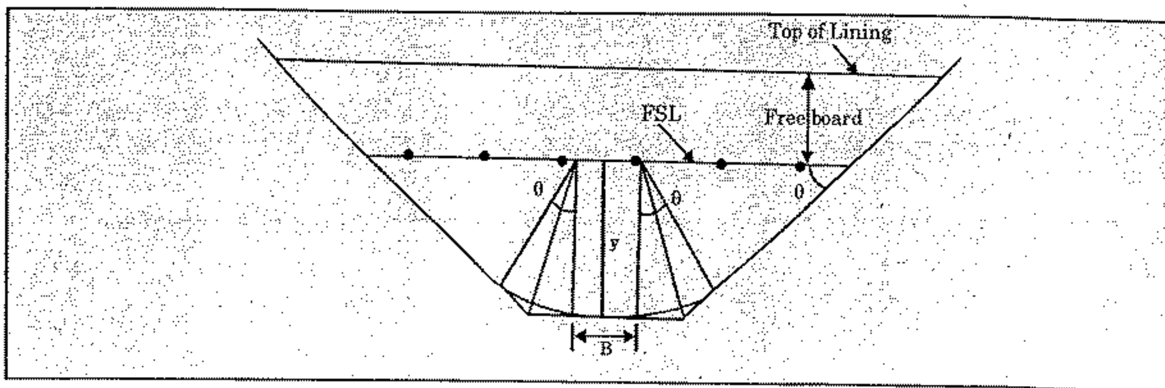
(2) Trapezoidal section

Area,
$$A = B.y + 2\left(\pi . y^2\frac{\theta}{2\pi}\right) + 2\left(\frac{1}{2}.y.y\cot\theta\right) = B.y + y^2\theta + y^2\cot\theta$$

$$\boxed{A = y(B + y\theta + y\cot\theta)}$$

Perimeter,
$$P = B + 2\left(2\pi y\frac{\theta}{2\pi}\right) + 2(y\cot\theta) = B + 2y\theta + 2y\cot\theta$$

$$\boxed{P = B + (2y\theta + 2y\cot\theta)}$$



MAX. PERMISSIBLE VELOCITIES IN DIFFERENT TYPES OF LININGS

Type of Lining	Permissible Velocity
Cement concrete lining (Unreinforced)	2.0 to 2.5 m / sec
Burnt clay tile lining	1.8 m / sec
Boulder lining	1.5 m / sec

Example 6

Design a concrete lined channel to carry a discharge of 500 cumecs at a slope of 1 in 4000. The side slopes of the channel may be taken as 1 : 1. The Manning's roughness coefficient for the lining is 0.014. Assume permissible velocity in the section as 2.5 m/s.

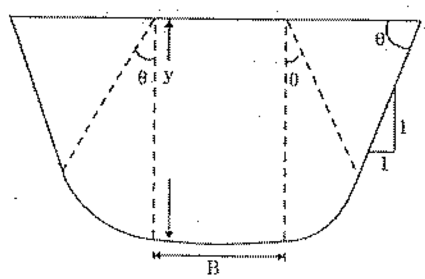
Sol. Given: Discharge, Q = 500 cumecs = 500 m^3/s ; Bed slope, $S = \frac{1}{4000}$; Side slope = 1 : 1

Manning's roughness coefficient, n = 0.014 ; Permissible velocity, v = 2.5 m/s

$$v = \frac{1}{n}R^{2/3}\ S^{1/2}$$

$$\Rightarrow \quad 2.5 = \frac{1}{0.014}\times\left(\frac{A}{P}\right)^{2/3}\left(\frac{1}{4000}\right)^{1/2}$$

$$\Rightarrow \quad \frac{A}{P} = (2.21)^{3/2} = 3.29 \text{ m} \qquad \text{... (i)}$$



We assume channel sector to be trapezoidal

$\because \quad \cot\theta = 1 \Rightarrow \theta = 45° = 0.785$ radian

Using the formula for lined trapezoidal section

$$A = y(B + y\theta + y\cot\theta) = (B + 0.785y + y \times 1)\,y$$

$$= (B + 1.785y)\,y \qquad \text{... (ii)}$$

$$P = B + 2y\theta + 2y\cot\theta = (B + 2 \times y \times 0.785 + 2 \times y \times 1)$$
$$= B + 3.57y \quad \text{... (iii)}$$

But,
$$A = \frac{Q_1}{v} = \frac{500}{2.5} = 200 m^2 \quad \text{... (iv)}$$

From (ii) and (iv), we have
$$(B + 1.785y)\, y = 200$$

$$\Rightarrow \quad B = \frac{200}{y} - 1.785y \quad \text{... (v)}$$

From (i), (iii) and (iv), we get
$$\frac{200}{B + 3.57y} = 3.29$$

$$\Rightarrow \quad \frac{200}{\frac{200}{y} - 1.785y + 3.57y} = 3.29 \text{ [from (v)]} \quad \text{... (vi)}$$

$$\Rightarrow \quad \frac{200}{\frac{200}{y} + 1.785y} = 3.29 \quad \Rightarrow \quad 60.79 = \frac{200}{y} + 1.785y$$

$$\Rightarrow \quad 1.785\, y^2 - 60.79y + 200 = 0 \quad \Rightarrow \quad y = 30.37 \text{ m and } 3.69 \text{ m}$$

$$y \approx 3.7 = 3.69 \text{ m (neglecting the higher non feasible value)}$$

Putting the value of y in equation (v), we get
$$B = \frac{200}{3.69} - 1.785 \times 3.69 = 47.61 \text{ m} \approx 48 \text{ m}$$

Hence, the section should have, Bedwidth = 48 m ; Depth = 3.7 m

Example 7

Design a lined canal to carry 100 m^3/sec on a slope of 1 in 2500. The maximum permissible velocity is 2 m/sec, n = 0.013 in Manning's formula and the slope = 1.25 H : 1.0 V.

Sol. Given, discharge, $Q = 100\ m^3/s$; Bed slope, $S = \frac{1}{2500}$

Max. permissible velocity, v = 2 m/s ; Manning's coefficient, n = 0.013

Side slope = 1.25 H : 1 V

$$\cot\theta = 1.25 \Rightarrow \tan\theta = \frac{1}{1.25} = 0.8$$
$$\theta = 38.66° = 0.6747 \text{ rad.}$$

From Manning's equation, we get
$$v = \frac{1}{n} R^{2/3}\, S^{1/2}$$
$$\Rightarrow \quad 2 = \frac{1}{0.013} \times R^{2/3} \times \left(\frac{1}{2500}\right)^{1/2}$$
$$\Rightarrow \quad R = 1.4822 \text{ m}$$

Area
$$A = \frac{Q}{v} = \frac{100}{2} = 50 \text{ m}^2$$

$\therefore \quad R = \frac{A}{P} \quad \therefore$ Perimeter, $P = \frac{A}{R} = \frac{50}{1.4822} = 33.73$ m

Since for a lined canal (trapezoidal section)

$$A = By + y^2\ (\theta + \cot\theta)) \quad \text{and} \quad P = B + 2y\ (\theta + \cot\theta)$$

$$\therefore \quad 50 = By + y^2\ (\theta + \cot\theta) \Rightarrow 50 = By + y^2\ (0.6747 + 1.25)$$

$$\Rightarrow \quad 50 = By + 1.925\ y^2 \quad \text{---- (i)}$$

$$\therefore \quad 33.73 = B + 2y\ (\theta + \cot\theta) \Rightarrow 33.73 = B + 2 \times y\ (0.6747 + 1.25)$$

$$\Rightarrow \quad 33.73 = B + 3.85\ y$$

$$\Rightarrow \quad B = 33.73 - 3.85\ y \quad \text{---- (ii)}$$

Putting the value of B in equation (i), we get

$$50 = y\ (33.73 - 3.85y + 1.925y)$$

$$\Rightarrow \quad 50 = y\ (33.73 - 1.925\ y) \Rightarrow 1.925\ y^2 - 33.73 + 50 = 0$$

$$\Rightarrow \quad y = 15.9\ \text{m},\ 1.635\ \text{m}$$

$$\therefore \quad y = 1.635\ \text{m} \quad \text{(neglecting larger non feasible values)}$$

Putting the value of y in equation (ii), we get

$$B = 33.73 - 3.85 \times 1.635 = 27.43\ \text{m}$$

Hence, Bed width, B = 27.43m ; Depth, y = 1.635 m

Example 8

A lined irrigation canal with trapezoidal cross-section has 5 m bed width, 2.5 m depth and 1.5 H : 1V side slope. Longitudinal bed slope of the canal is 1 in 1000 and Manning's n = 0.016. What is the maximum carrying capacity of the canal? What area of land is hectares the canal can irrigate if the crop has 150 mm field irrigation requirement in a core period of 10 days?

Sol. Given, Bed width, B = 5 m ; Depth, y = 2.5 m ; Side slope = 1.5 H : 1V

$$\Rightarrow \quad \cot\theta = 1.5 \Rightarrow \tan\theta = \frac{1}{1.5} \Rightarrow \theta = 0.588$$

Bed slope, $S = \frac{1}{1000}$; Manning's coefficient, n = 0.016

For a lined trapezoidal channel

$$\therefore \quad A = 5 \times 2.5 + 2.5^2\ (0.588 + 1.5) = 25.55\text{m}^2$$

$$P = 5 + 2 \times 2.5\ (0.588 + 1.5) = 15.44\ \text{m}$$

$$\therefore \quad \text{Hydraulic mean depth,} \quad R = \frac{A}{P} = \frac{25.55}{15.44} = 1.651\ \text{m}$$

From Manning's equation we get

$$v = \frac{1}{n} R^{2/3} S^{1/2} = \frac{1}{0.016} \times (1.651)^{2/3} \left(\frac{1}{1000}\right)^{1/2} = 2.761\ \text{m/s}$$

$$\text{and} \quad Q = Av = 25.55 \times 2.761 = 70.54\ \text{m}^3\text{/s}$$

Hence, max. carrying capacity of the canal = 70.54 m^3/s

Since, Kor period = 10 days = 10 × 86400 s

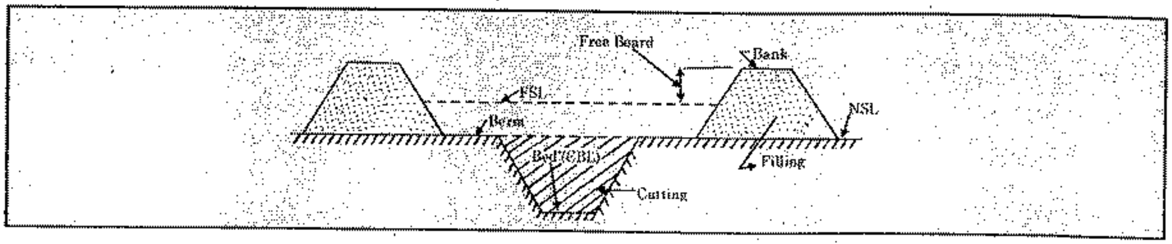
$\therefore$ Vol of water that can be supplied by the channel = 70.54 ×(10 × 86400) = 60.95 × 10^6 m^3

FIR = 150 mm

$\therefore$ Avg level which can be irrigated $= \frac{\text{Vol of water}}{\text{FIR}} = \frac{60.95\times10^6}{150\times10^{-3}} = 4.063 \times 10^8 = 40630\text{ha}$

CROSS-SECTION OF AN IRRIGATION CANAL

A typical section of a canal is shown in Fig. This section is 'partly in cutting and partly in filling', and aims at balancing the quantity of earth work in 'excavation' with that in 'filling'.



Sometimes, when the natural surface level (i.e. NSL) is above the top of the bank, the entire canal section will have to be in cutting, and it shall be called 'canal in cutting'. Similarly, when the NSL is lower than the Bed level of the canal, the entire canal section will have to be built in filling, and it is called 'canal in filling, or 'canal in banking'.

(1) Side Slopes

- The stability of side slopes depends upon the type of the soil.
- A comparatively steeper slope can be provided in cutting rather than in filling

Generally adopted slopes

In cutting: $1\ H : 1\ V\ (1:1)$ to $1\frac{1}{2}\ H : 1\ V \left(1\frac{1}{2}:1\right)$,

In filling: $1\frac{1}{2}\ H : 1V$ to $2\ H : 1V$

Note: In case of channels with silt laiden water, actual capacity of the channel is worked out with $\frac{1}{2}$: 1 side slopes, even though flatter slopes such as 1 : 1 to $1\frac{1}{2}$: 1 may be constructed at the time of execution.

(2) Berms

Berm is the horizontal distance left at ground level between the toe of the bank and the top edge of cutting.

The Berms serve the following purposes:

(i) The silt deposited on the sides is very fine and impervious. It, therefore, serves as a good lining for reducing losses, leakage and consequent breaches, etc.

(ii) They help the channel to attain regime conditions, as they help in providing a wider water, if required.

(iii) They give additional strength to the banks & provide protection against erosion and breaches.

(iv) They protect the banks from erosion due to wave action.

(v) They provide a scope for future widening of the canal.

(vi) Berms can be used as borrow pits for excavating soil to be used for filling.

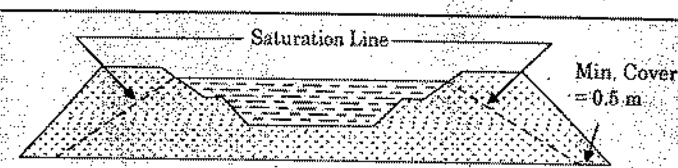
(3) Freeboard

The margin between FSL and bank level is known as freeboard. The amount of freeboard depends upon the size of the channel. The generally provided values of freeboard are given in Table.

Discharge (in cumecs)	Extent of freeboard (in metres)
1 to 5	0.50
5 to 10	0.60
10 to 30	0.75
30 to 150	0.90

(4) Banks

The primary purpose of banks is to retain water. They can be used as means of communication and as inspection paths. They should be wide enough, so that a minimum cover of 0.5 metre is available above the saturation line, as shown in Fig. High banks will have to be designed as earth dams.



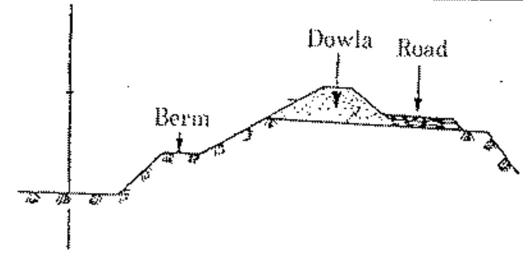
Canal Section in Filling

(5) Service Roads

Service roads are provided on canals for inspection purposes, and may simultaneously serve as the means of communication in remote areas. They are provided 0.4 m to 1.0 m above FSL, depending upon the size of the channel.

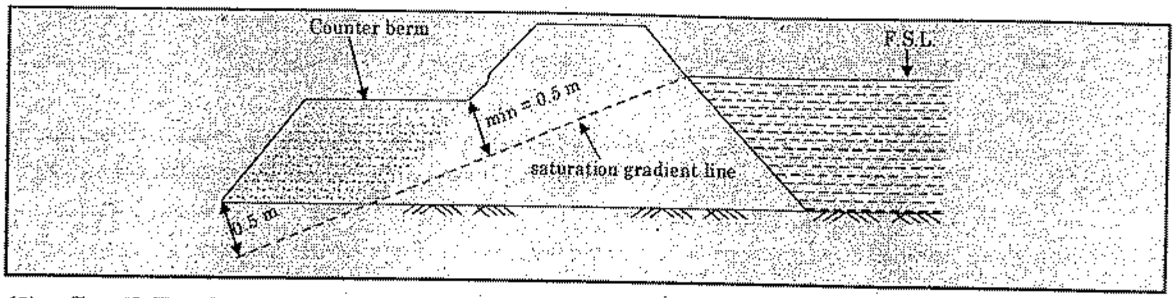
(6) Dowlas

As a measure of safety in driving, dowlas 0.3 m high and 0.3 to 0.6 m wide at top, with side slopes of $1\frac{1}{2}$: 1 to 2 : 1, are provided along the banks, as shown in Fig. They also help in preventing slope erosion due to rains, etc.



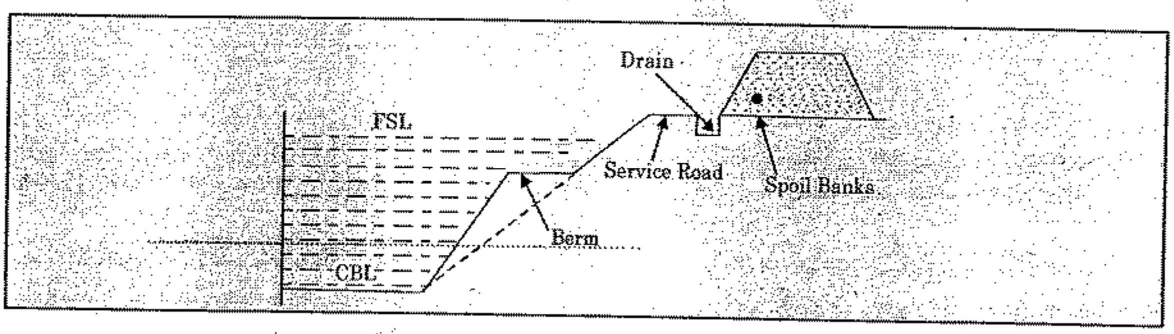
(7) Back Berm or Counter Berms

Even after providing sufficient section for bank embankment, the saturation gradient line may cut the d/s end of the bank. In such a case, the saturation line can be kept covered at least by 0.5 metre with the help of counter berms.



(8) Spoil Banks

When the earthwork in excavation exceeds earthwork in filling, even after providing maximum width of bank embankments, the extra earth has to be disposed off economically. To dispose of this earth by mechanical transport, etc. may become very costly, and an economical mode of its disposal may be found in the form of collecting this soil on the edge of the bank embankment itself. The soil is, therefore, deposited in such a case, in the form of heaps on both banks or only on one bank. These heaps of soil are discontinued at suitable intervals and longitudinal drains running by their sides are excavated for the disposal of rain water.



(9) Borrow Pits

When earthwork in filling exceeds the earthwork in excavation, the earth has to be brought from somewhere. The pits, which are dug for bringing earth, are known as borrow pits. If such pits are excavated outside the channel, they are known as *external borrow pits*, and if they are excavated somewhere within the channel, they are known as *internal borrow pits*. It is a very costly affair to bring soil from distances.

OBJECTIVE QUESTIONS

1. In the alignment of an irrigation channel where from offtakes have to be provided at regular intervals, changes in the given channel parameters are made use of. The correct sequence of the decreasing order of preference of these parameters is
   (a) width, slope, depth (b) width, depth, slope
   (c) depth, slope, width (d) depth, width, slope

2. Assertion (A) : Irrigation canals are constructed at a maximum grade.
   Reason (R) : It is advantageous to command as much arid land as possible.

3. For medium silt whose average grain size is 0.16 mm, Lacey's silt factor is likely to be
   (a) 0.30 (b) 0.45 (c) 0.70 (d) 1.32

4. When a river starts meandering, the sediment carrying capacity
   (a) first decreases and ultimately increases
   (b) first increases and ultimately decreases
   (c) remains unaffected as the plan shape changes continuously
   (d) changes erratically all the time leaving permanent braids

5. In order to ensure that no scouring takes place in the bed of a channel of bed slope 'S' constructed on alluvial soil of particle size 'd' m, the flow velocity should be restricted to
   (a) $4.85\, d^{1/2} S^{-1/6}$ (b) $4.85\, d^{-1/2} S^{1/6}$
   (c) $0.48\, d^{1/2} S^{-1/6}$ (d) $0.48\, d^{-1/2} S^{1/6}$

6. The Lacey's silt factor for a particular alluvium is 2.0. This alluvium would comprise
   (a) medium sand of size 0.5 mm (b) coarse sand of size 0.75 mm
   (c) medium bajri of size 1.3 mm (d) coarse bajri of size 2.4 mm

7. Consider the following statements:
   Garret's diagram for the design of irrigation channel is based on:
   1. Kennedy's theory 2. Lacey's theory
   3. Kutter's formula 4. Manning's formula
   Which of these statements are correct?
   (a) 1 and 3 (b) 1 and 4 (c) 2 and 3 (d) 2 and 4

8. The total number of independent equations that form the Lacey's regime theory is
   (a) 2 (b) 3 (c) 4 (d) 6

9. Balanced depth of cutting of canal is
   (a) half the total depth of a canal
   (b) half of full supply depth
   (c) the maximum cut that an excavator can take
   (d) where volume of cutting is equal to volume of filling

10. **Assertion (A):** With tile lining of canals, permissible velocity of flow is lower than that with concrete lining.

    Reason (R): The surface of tile lining becomes rough due to loss of surface material with high velocity.

11. Which of the following pairs are correctly matched?

1. Silt factor : Average size of silt particles
2. Silt load : Volume of suspended sediments flowing with water in unit time.
3. Silt charge : Weight of silt per unit volume of water
4. Silt grade : Gradation between different silt particles

Select the correct answer using the codes given below :

(a) 1 and 4 (b) 3 and 4 (c) 2 and 3 (d) 1 and 2

12. Which one of the following is the correct sequence in the increasing order of the Froude number of flow assumed by the bed form of an alluvial stream with movable bed material?

(a) Ripple-Plane bed-Dune-Plane bed-Antidune
(b) Dune-Ripple-Plane bed-Antidune-Plane bed
(c) Plane bed-Ripple-Dune-Plane bed-Antidune
(d) Plane bed-Ripple-Antidune-Dune-Plane bed

13. What is the regime scour depth for a channel in soil with silt factor of unity and carrying 8 m^2/s of discharge intensity in accordance with Lacey's regime theory?

(a) 3.6 m (b) 4 m (c) 5.4 m (d) 25. 6 m

14. The channel section can be designed on the basis of Lacey's Theory. The steps are mentioned below:

1. Finding out the perimeter
2. Finding out the velocity
3. Calculation of the silt factor
4. Finding out the area

What is the correct sequence of the steps?

(a) 4-2-3-1 (b) 3-1-4-2 (c) 4-1-3-2 (d) 3-2-4-1

15. In Lacey's regime theory, the velocity of flow is proportional to

(a) Qf^2 (b) Q/f^2 (c) $(Qf^2)1/6$ (d) $(Q/f^2)1/6$

16. Match List-I with List-II and select the correct answer using the codes given below the lists:

(S = bed slope, q = discharge intensity, Q = Discharge)

List-I	List - II
A. Mean velocity in a Lacey's regime channel	1. $S^{1/2}$
B. Mean velocity in a lined channel	2. $S^{1/3}$
C. Normal scour depth in an alluvial channel	3. $Q^{2/3}$
D. Wetted perimeter of a Lacey regime channel	4. $Q^{-2/3}$
	5. $Q^{1/2}$

Codes:

	A	B	C	D
(a)	2	5	3	1
(b)	3	1	4	5
(c)	2	1	3	5
(d)	3	5	4	1

17. The permissible tractive force in an erodible channel depends upon which of the following?

1. Angle of the repose of the material
2. Particle size
3. Sediment content of water
4. Wetted perimeter of channel

Select the correct answer using the codes given below:

(a) 1, 2 and 4 (b) 1, 2 and 3
(c) 1 and 3 only (d) 2 and 4 only

18. For calculating the maximum flood discharge in an alluvial stream, which is the best suited relation?
(a) $v \propto R^{2/3} S^{1/3}$ (b) $v \propto R^{2/3} S^{1/2}$ (c) $v \propto R^{1/2} S^{1/2}$ (d) $v \propto D^{0.64}$

19. Garret's diagrams are used to
(a) separate base flow from total runoff
(b) correct inconsistency in rainfall data
(c) determine reservoir capacity
(d) design channels

20. **Assertion (A):** Lining a canal is always beneficial and economical.
Reason (R): The seepage losses are greatly reduced and extra water is available for irrigation.

21. On which of the canal systems, R.G. Kennedy, executive engineer in the Punjab Irrigation Department made his observations for proposing his theory on stable channels?
(a) Krishna Western Delta canals (b) Lower Bari Doab canals
(c) Lower Chenab canals (d) Upper Bari Doab canals

22. As per the Lacey's method for design of alluvial channels, identify the TRUE statement from the following:
(a) Wetted perimeter increases with an increase in design discharge.
(b) Hydraulic radius increases with an increase in silt factor.
(c) Wetted perimeter decreases with an increase in design discharge.
(d) Wetted perimeter increases with an increase in silt factor.

23. A stable channel is to be designed for a discharge of Q m^3/s with silt factor f as per Lacey's method. The mean flow velocity (m/s) in the channel is obtained by

(a) $\left[\frac{Qf^2}{140}\right]^{1/6}$ (b) $\left[\frac{Qf}{140}\right]^{1/3}$ (c) $\left[\frac{Q^2f^2}{140}\right]^{1/6}$ (d) $0.48 \times \left[\frac{Q}{f}\right]^{1/3}$

24. The depth of flow in a alluvial channel is 1.5 m. If critical velocity ratio is 1.1 and Manning's n is 0.018, the critical velocity of the channel as per Kennedy's method is
(a) 0.713 m/s (b) 0.784 m/s (c) 0.879 m/s (d) 1.108 m/s

25. The ratio of the average value of the shear stress produced on the bed, and that on the banks of an alluvial canal, due to the action of the following water, is:
(a) 0.75 (b) 1.0 (c) 1.33 (d) 0

26. Lacey's silt theory is not applicable when:
(a) silt grade consist of pure sand
(b) silt amount is of the order of 500 ppm
(c) the canal is lined
(d) none of the above

27. The hydraulic radius of a standard trapezoidal lined canal with 1.25 H : 1V side slopes, 12 m bed width and 2 m FSD, is :
(a) 1.11 m (b) 1.41 m (c) 1.61 m (d) none of them

28. Which one of the following statements is not true about cement concrete lining?
(a) that is develops frequent cracks due to temperature changes
(b) that it develops frequent cracks due to settlement of subgrade
(c) that it is likely to be damaged by alkaline water
(d) that it can be easily punctured by weed growth

29. Rigid boundary canals, whose bed and banks are made with non-erodible materials, are in:
(a) initial regime
(b) final regime
(c) true regime
(d) permanent regime

30. Lacey's silt theory is not applicable to channels, which are in:
(a) final regime
(b) permanent regime
(c) true regime
(d) none of the above

31. A perfectly lined canal is a :
(a) mobile boundary canal
(b) rigid boundary canal
(c) neither (a) nor (b)
(d) either (a) or (b)

32. The arrangements made along a canal, to help the unfortunate victims being swept away by water current, are called:
(a) canal crossings
(b) cattle crossings
(c) canal ladders
(d) none of them

33. The bed of an alluvial channel along the flow will always be
(a) flat
(b) wavy
(c) duned and rippled
(d) all of the above are possible

34. Lacey's scour depth for a stream, carrying a discharge of 3 cumecs per metre width and having a silt factor 1.2, is :
(a) 1.32 m (b) 2.64 m (c) 3.96 m (d) 4.32 m

35. Which one of the following is not the requirement of an ideal regime conditions in Lacey's regime-theory?
(a) the discharge in the channel is constant
(b) the channel flows through the same soil grade, as that of the sediment entering the channel from the headworks
(c) the sediment grade and its amount entering the channel is constant.
(d) the silt grade should consist of clay sized particles.
(e) the flow should be uniform

36. Two irrigation channels, A and B, are designed on Lacey's theory to carry the same discharge. The alluvium through which canal A has to pass, however, is coarser than that for canal B. In such a design, we expect :
(a) channel A to be deeper
(b) channel B to be deeper
(c) channel A to have large wetted perimeter
(d) channel B to have larger cross sectional area.

37. The major drawback of Lacey's regime theory, as used for the design of irrigation canals, is that:
(a) It does not consider the quantum of sediment load, which is likely to flow into the canal
(b) It is applicable only to non-cohesive soils
(c) It is applicable only to alluvial areas of North India or Pakistan.
(d) None of the above

38. Stable channels for a given bed-load transport, can be best designed on the basis of
(a) Lacey's theory
(b) Kennedy's theory
(c) Einstein's theory
(d) all of the above

39. In an irrigation canal, berms are provided at :

(a) NSL (b) FSL (c) Bank level (d) none of the above

40. Counter berms are provided in an irrigation canal :

(a) at the outer sides of the canal banks to allow movement of cattle

(b) at the inner sides of the canal banks at the ground level, to make bank line and bed line parallel

(c) at the outer sides of the canal banks to keep the saturation gradient line well within the bank section

(d) none of the above

41. An irrigation canal is lined along its 20 m perimeter @ Rs. 40 per m^2 of lining. the lining has resulted in net water saving @ 3 cumecs per M-m^2 of lined area, whereas each cumec of water increases the annual crop yield by Rs. 4 lakh. Assuming the life of the interest on investment, the benefit cost ratio for the project is :

(a) 1.2 (b) 1.0 (c) 0.83 (d) none of them.

42 Under drainage arrangements in canals are necessarily required in :

(a) unlined canals (b) lined canals

(c) both (a) and (b) (d) lined canals constructed on sandy soil

43. When elaborate under-drainage arrangements cannot be provided, and the soil strata is sandy, the preferred material for canal lining, would be :

(a) cement concrete lining (b) soil cement lining

(c) brick lining (d) shotcrete lining.

44. Two irrigation channels M and N are designed using Lacey's theory. If the median diameter of the silt particles are the same for both the channels, and if the discharge in channel M is 50% greater than at N, then between these two channels

(a) channel N has smaller longitudinal slope

(b) channel M has smaller longitudinal slope

(c) channel M has smaller hydraulic radius

(d) channel N has larger wetter perimeter.

45. The Rouse equation for the suspended sediment concentration profile in an open channel, relating the concentration C at a height y from the bed to the reference concentration C_a at a height a in a channel of depth d is ($z = w_0/kV^*$)

(a) $\frac{C}{C_a} = \left[\left(\frac{y}{d-y}\right)\left(\frac{d-a}{a}\right)\right]^z$ (b) $\frac{C}{C_a} = \left[\left(\frac{d-y}{y}\right)\left(\frac{a}{d-a}\right)\right]^z$

(c) $\frac{C}{C_a} = \left[\left(\frac{y}{d-y}\right)\left(\frac{y-a}{d-y}\right)\right]^z$ (d) $\frac{C}{C_a} = \left[\left(\frac{d-y}{y}\right)\left(\frac{y-a}{d-a}\right)\right]^z$

46. In rectangular channel of bed width to depth ratio of 1.0, the maximum shear stress on the bed is

(a) $\gamma D S_o$ (b) $> \gamma D S_o$ (c) $< \gamma D S_o$ (d) $\gamma D S_o^{1/2}$

ANSWERS

1. (c)	2. (d)	3. (c)	4. (a)	5. (a)	6. (c)	7. (a)
8. (b)	9. (d)	10. (a)	11. (c)	12. (c)	13. (c)	14. (d)
15. (c)	16. (c)	17. (b)	18. (a)	19. (d)	20. (a)	21. (d)
22. (a)	23. (a)	24. (b)	25. (c)	26. (c)	27. (c)	28. (d)
29. (d)	30. (b)	31. (b)	32. (c)	33. (d)	34. (b)	35. (d)
36. (b)	37. (a)	38. (c)	39. (a)	40. (c)	41. (a)	42. (b)
43. (c)	44. (b)	45. (b)	46. (c)			

Canal Regulation Work

CHAPTER

6

Canal Regulation Work

CANAL REGULATION

The water entering the main canal from the river has to be divided into different branches and distributaries, w.r.t the relative urgency of demand on different channels. This process of distribution is called *Canal regulation.*

The amount of water which can be diverted from the river into the main canal depends upon

(i) The water available in the river (ii) The capacity of the main canal

(iii) The share of other canals taking off from the river.

CANAL REGULATION WORKS

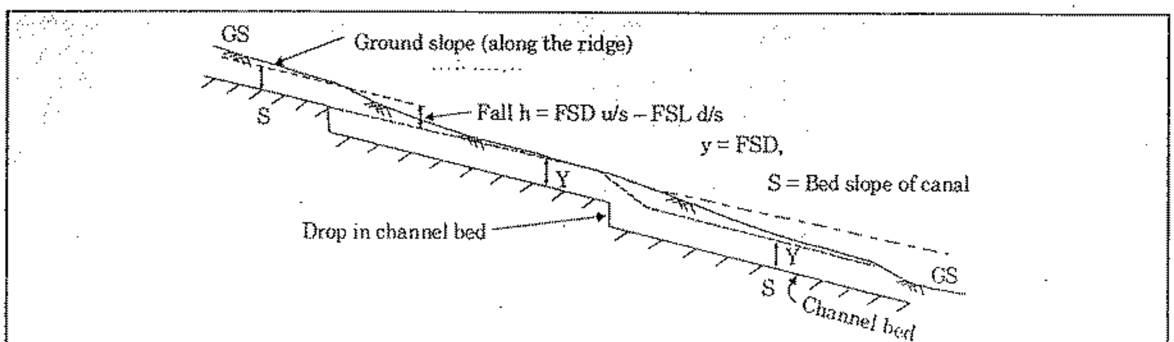
The works which are constructed in order to control and regulate discharges, depths, velocities etc. in canals, are known as *Canal Regulation Works.* These structures ensure the efficient functioning of a canal irrigation system by giving full control upon the canals.

Important Canal Regulation Works are

(1) Canal Falls (2) Canal Regulators (3) Canal Escapes.

(4) Metering Flumes, (5) Canal outlets and Modules.

CANAL FALLS



- A fall is a structure constructed across a channel to permit lowering down of water level in order to dissipate the surplus energy possessed by the falling water which may other wise scour the bed and banks of the channel.
- When the available natural ground slope is steeper then the designed bed slope of the

channel, the difference is adjusted by constructing vertical falls or drops in the canal bed at suitable intervals. Such a drop in a natural canal bed will not be stable and therefore, in order to retain this drop we have to construct these falls.

Necessity and Location of Falls

The location of a fall is decided according to various considerations as indicated below.

(i) A fall may be provided at a location where the F.S.L. of the channel outstrips the ground level but before the bed of the channel comes into filling.

(ii) A fall should be so located that as far as possible there is no loss of the commanded area of the channel.

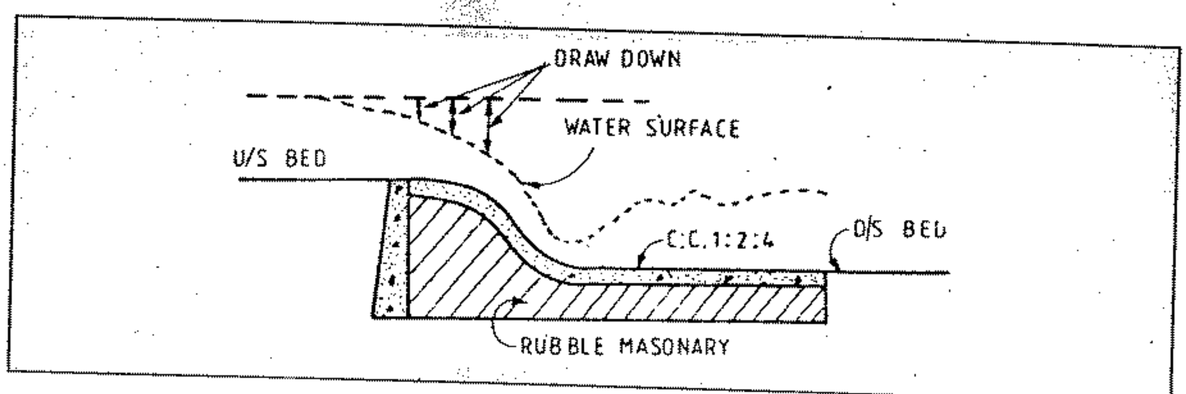
(iii) A fall should be such that below the fall the F.S.L. of the channel remains below the ground level for 0.5 to 0.75 kilometer.

(iv) The location of a fall may also be affected by the possibility of combining it with a regulator or a road bridge and also may serve as a metering device.

TYPES OF FALLS

(1) Ogee falls

- The Ogee fall was constructed in olden days on projects like Ganga canal.
- The water was gradually led down by providing convex and concave curves.



The performance of an Ogee fall had few major defects such as

(i) There was heavy draw-down on the u/s side, resulting in lower depth, higher velocities and consequent bed erosion.

(ii) Due to smooth transition, K.E. of the flow was not dissipated, causing erosion of d/s bed and banks.

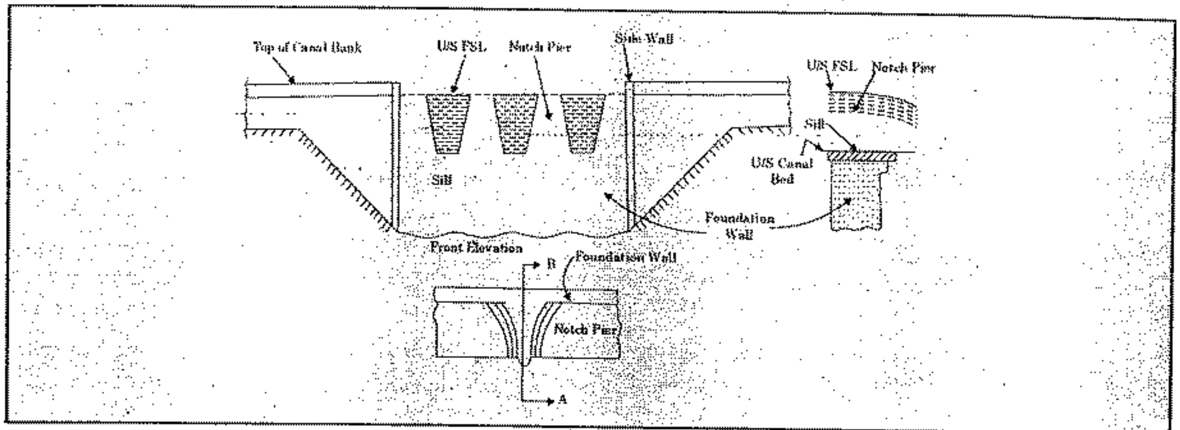
(2) Rapids

- Long rapids at slopes of 1 : 15 to 1 : 20 with boulder facings, were provided in Western Yamuna Canal.
- These falls were very expensive, and hence became obsolete.

- The long glacis assured the formation of hydraulic jump.
- These falls were quite satisfactory.

(3) Trapezoidal Notch Falls

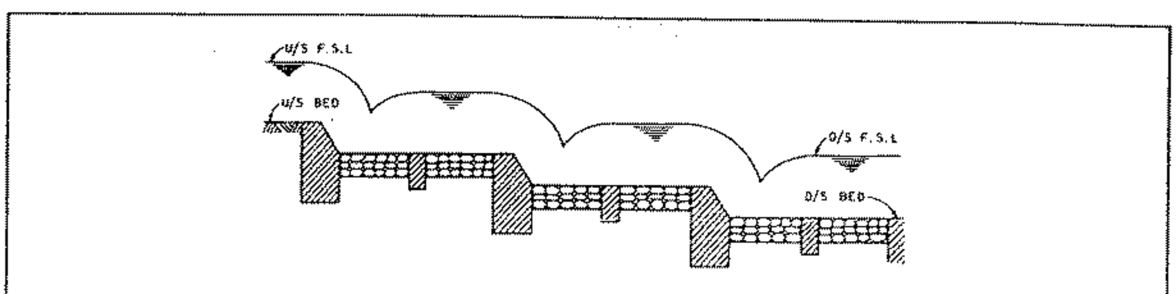
- It consists of a number of trapezoidal notches constructed in a high crested wall across the channel with a smooth entrance and a flat circular lip projecting d/s from each notch to spread out the falling jet.
- In these falls, energy is dissipated by turbulent diffusion.



- The notches could be designed to maintain the normal water depth in the u/s channel at any two discharges, as the variation at intermediate values is small.
- Depth discharge relationship of the channel remains practically unaffected by the introduction of the fall i.e. there would neither be drawdown nor heading up of water, as the channel approaches the fall.
- These falls were quite popular, simpler, and economical.

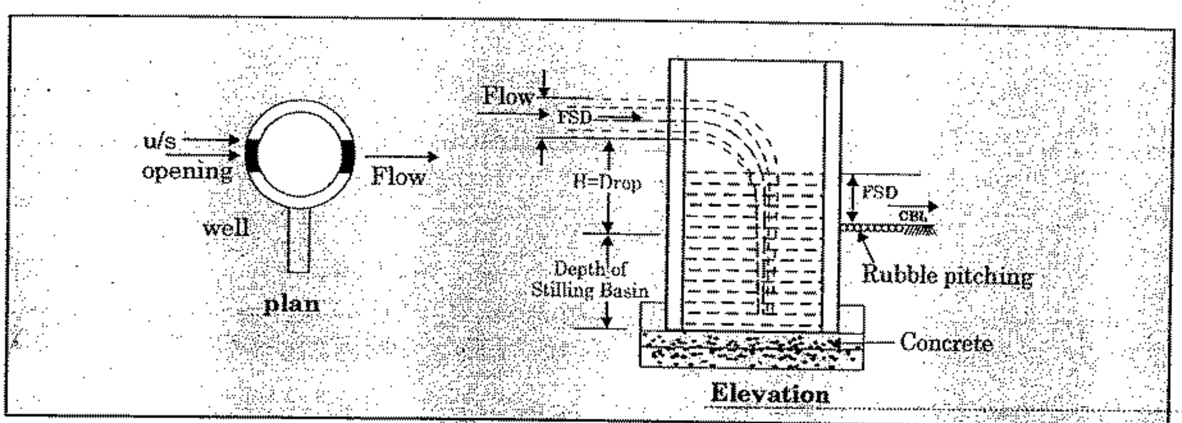
(4) Stepped falls

- Stepped falls were the modified form of rapids or rapid falls.
- The long glacis of the rapid falls were replaced by floors in steps in the stepped falls.
- After the development of stepped falls it was recognised that better dissipation of energy could be achieved through vertical impact of falling jet of water on the floor.



(5) Well Type Fall or Cylinder Falls or Syphon Well Drops

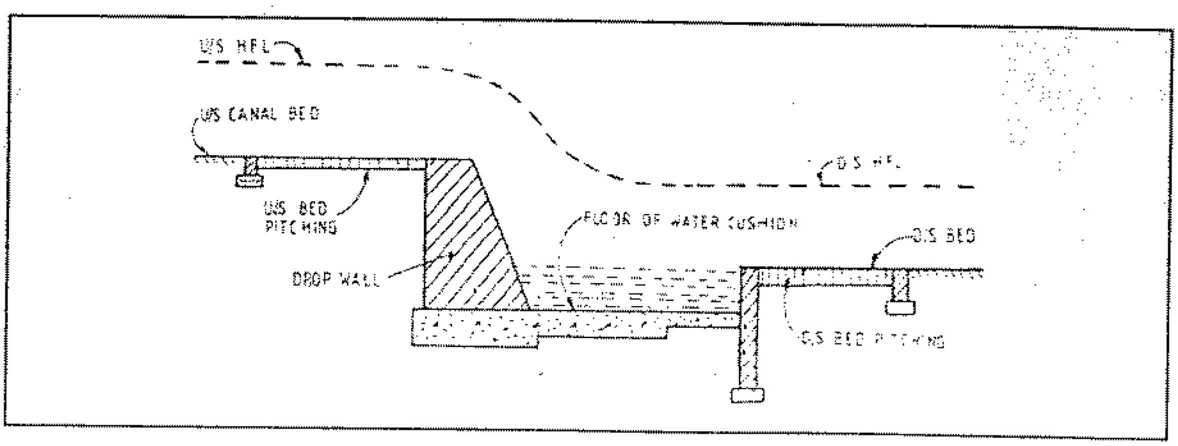
- This type of a fall consists of an inlet well with a pipe at its bottom, carrying water from the inlet well to a d/s well or a cistern.
- It has a concrete bottom and two openings.
- The u/s opening in the well is at a higher level and allows water into the level higher than the d/s opening.
- The portion of the well between the bed level of d/s opening and still level of well acts as a stilling basin for the dissipation of the energy.
- In case of falls greater than 1.8 m and discharges greater than 0.29 cumecs, this type of d/s well is necessary.



- This type of falls are very useful for affecting larger drops for smaller discharges.
- They are commonly used as tail escapes for small canals.

(6) Simple Vertical Drop Type and Sarda Type Falls

- A raised crest fall with a vertical impact was first of all introduced on Sarda Canal System in U.P.
- It was simple and quite economical.



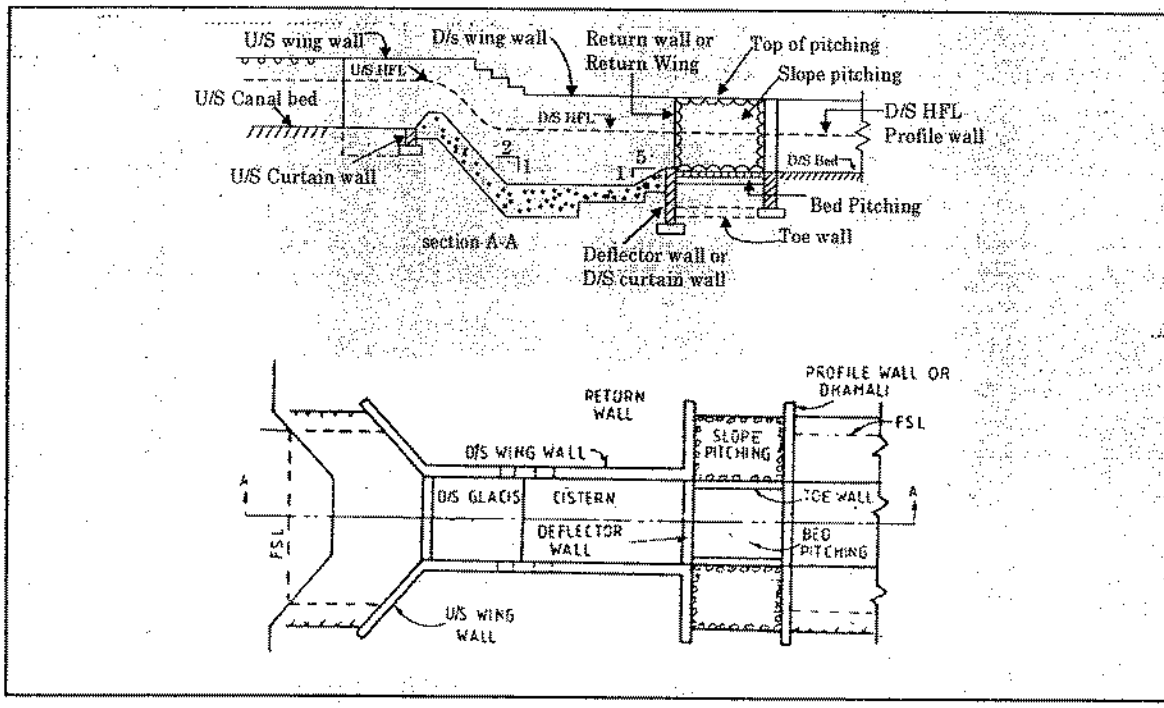
- In this type of a high crested fall, the nappe impinges into the water cushion below.

- There is no clear hydraulic jump and the energy dissipation is brought about by the turbulent diffusion.
- Not quite suitable for canals in which discharge varies within a wide range.

Note : A trapezoidal notch fall, although costlier than Sarda type or glacis type fall, is free from such troubles and, therefore, preferred for canals where the discharge is very small and also varies over a wide range

(7) Straight Glacis Falls

- In this type of a modern fall, a straight glacis is provided after a raised crest.
- Usually slope of glacis is 2 : 1.
- The hydraulic jump is made to occur on the glacis, causing sufficient energy dissipation.



- They are suitable up to a discharge of 60 cumecs and 1.5 m drop

(8) Montague Type Falls

- In *Montague type falls*, the problem of considerable residual energy below hydraulic jump is solved by giving such a shape to the glacis that maximum horizontal acceleration is imparted to the flowing water in a given length of the glacis.
- In this type of fall, an improvement in energy dissipation may be brought about by replacing the straight glacis by a parabolic glacis, commonly known as 'Montague Profile'

Montague derived analytically the equation for such a glacis profile which is as follows :

$$\boxed{x = U\sqrt{\frac{4y}{g}} + y}$$

where

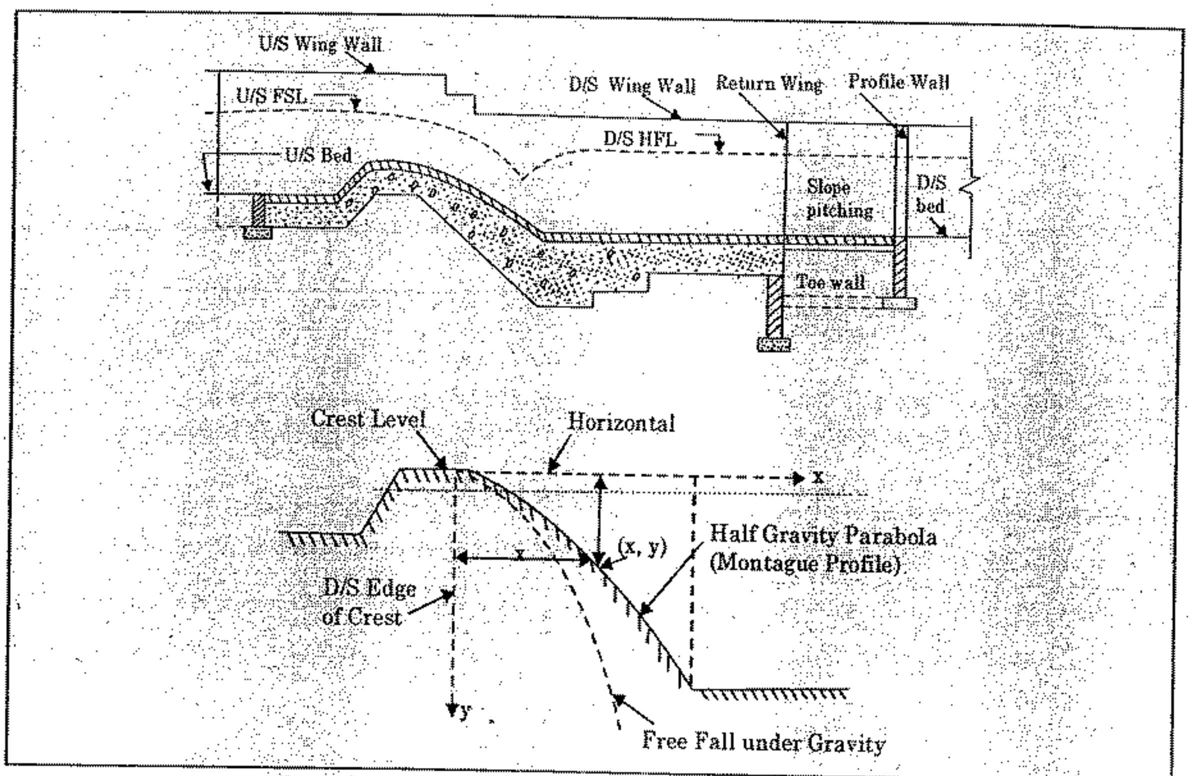
U = initial horizontal velocity of water leaving the crest;

x = horizontal distance of any point on the profile measured from the crest

y = vertical distance of any point on the profile measured from the crest

g = acceleration due to gravity.

The energy dissipation on a straight glacis remains incomplete due to vertical component of velocity remaining unaffected.



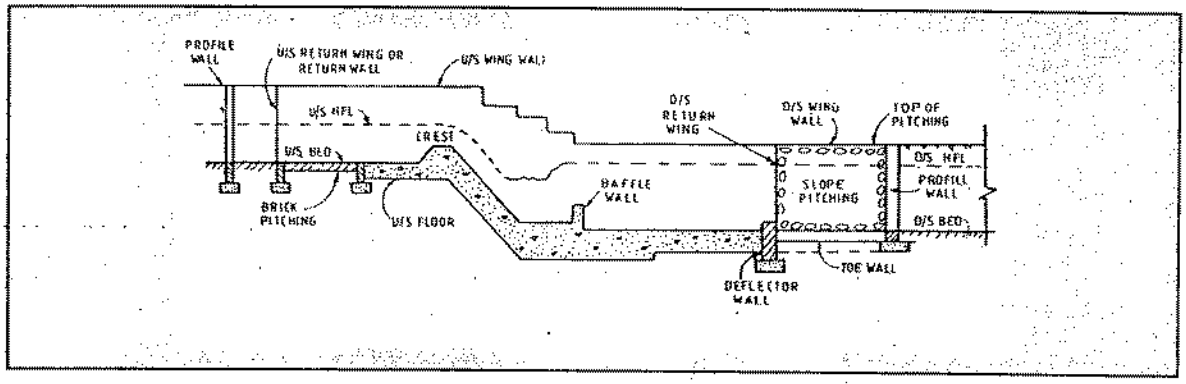
- This is a costlier affair, since, the curved glacis is difficult to construct

(9) Inglis Falls or Baffle Falls

- A straight glacis type fall when added with a baffle platform and a baffle wall was developed by Inglis, and is called 'Inglis Fall' or 'Baffle Fall'.

  They are quite suitable for all discharges and for drops of more than 1.5 m.
- It makes use of a horizontal impact for energy dissipation.
- The design consists of a standard long throated weir flume followed by a glacis and a horizontal platform or pavement on which a baffle is fixed to hold the hydraulic jump stable on the platform.
- On the d/s of the baffle, a cistern is provided and at the d/s end of the cistern, a deflector is provided. The impervious floor is provided only upto the end of the deflector.
- On the d/s of the deflector, often a second cistern is provided which is surfaced with stone or brick pitching only.

- The maximum dissipation of energy by a hydraulic jump occurs when the jump forms at the toe of the glacis. As such the horizontal platform is provided at such a level that for normal discharge conditions the jump forms at the toe of the glacis.
- The cistern and deflector d/s of the baffle are meant to restore normal distribution of velocities in the channel.
- They can be flumed easily ..



(10) Meter and Non-Meter Falls

- Meter falls are those type of fall which can be used to measure the discharge of the canal.
- If the discharge cannot be measured accurately at the site of the fall, then it is called a non-meter fall.
- Vertical drop fall is not suitable as a meter due to formation of partial vaccum under the nappe.
- Glacis type fall is actually quite suitable as a metering device.
- Since a sharp crest does not give constant coefficient of discharge with varying heads, while a broad crest does so reasonably; a fall to be used as meter must be provided with a broad crest.
- Generally, a flumed glacis fall or a flumed baffle fall. is used as meter, while an unflumed glacis fall is used as a non meter fall.

CISTERN OR CISTERN ELEMENTS

The cistern or cistern element is that portion of the fall on the d/s of the crest wall in which the surplus energy of water leaving the crest is dissipated and the subsequent flow stilled before it passes on to the lower level channel.

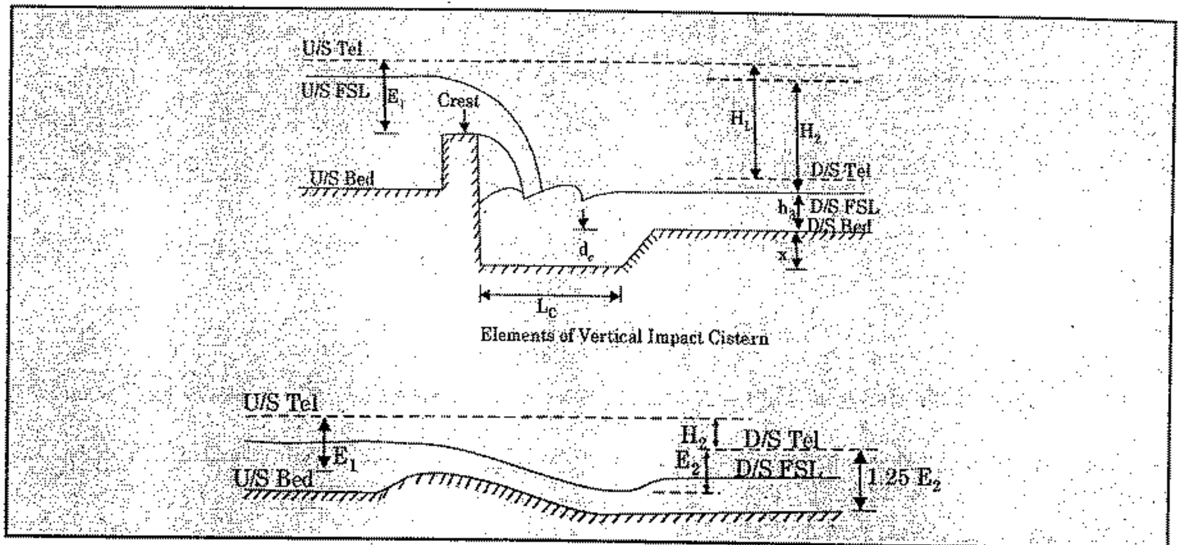
The object of providing a cistern is :

- To reduce the intensity of impact of the dropping jet of water against the d/s floor.
- To provide water cushion to dissipate the energy of the falling jet.
- To produce reverse flow by providing a suitable end wall to ensure an impact in the cistern.

The complete cistern element consists of: sloping glacis, cistern, roughening devices

(1) Cisterns

Cisterns are the water cushion provided by depressing the floor below the downstream bed of the channel in order to protect the floor from the impact of stream of water falling freely under gravity. The cisterns are of four types depending on the impact



Elements of Vertical Impact Cistern

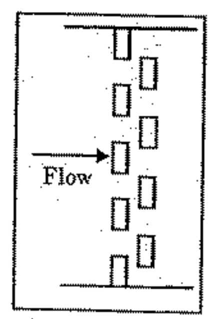
(1) Vertical Impact cistern (2) Horizontal Impact cistern

(3) Inclined Impact cistern (4) No Impact cistern

- To protect the floor from the impact of falling water, water cushion is provided by depressing the floor below the d/s bed of the channel.
- Vertical impact cistern is the most efficient while the inclined impact cistern is the least effective.

(2) Roughening Devices

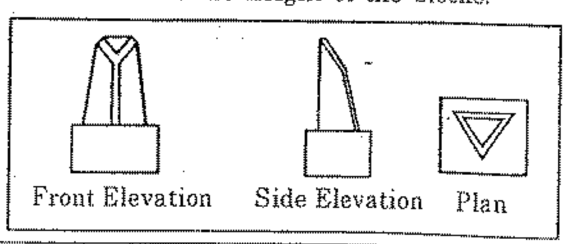
- Roughening devices are provided in the cisterns for the dissipation of energy.
- In the case of cisterns with impact, the roughening devices may be used as secondary means for dissipating residual energy of water while in the case of cisterns without impact, the roughening devices are the only means available for dissipation of energy.
- These devices depend on turbulence and boundary friction



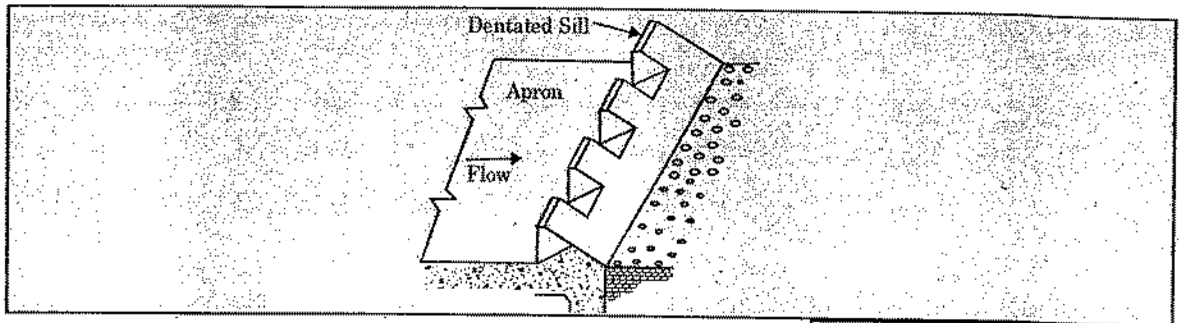
Commonly used roughening devices

(i) **Staggered friction blocks:** Staggered friction blocks are one of the most useful and simple devices used for dissipation of energy. They consists of rectangular blocks of concrete. The blocks are securely anchored into the floor and projecting upto a height of 0.25 times of the depth of water. The spacing between the block is about twice the height of the blocks.

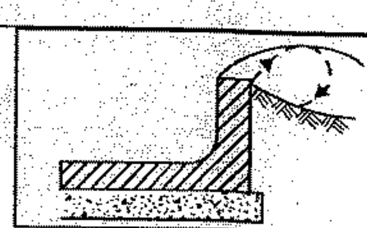
(ii) **Arrows:** Arrows are specially shaped friction blocks which serve the same purpose as rectangular blocks. They are able to withstand the action of the high velocity jet striking against them in a better way.



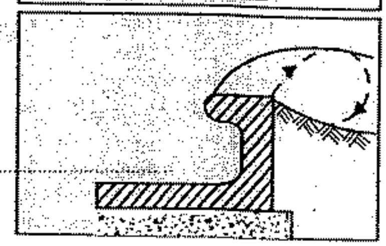
(iii) Dentated sill: This device is also provided where the high velocity flow continues till the end of the cistern. The dentated sill breaks up the stream jet into smaller jets. It causes reverse rollers.



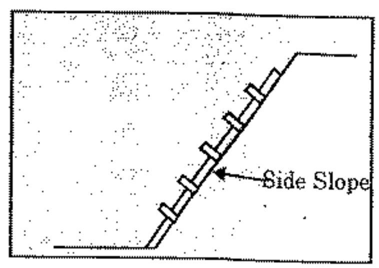
(iv) Deflector: A deflector (or baffle wall) is provided if the high velocity flow continues up to the end of the cistern, which helps in dissipating the residual energy.



(v) Biff Wall: The biff wall at the d/s edge of the cistern produces a reverse roller which causes a controlled scour away from the wall and piles up the scoured material against the toe of the structure, and thus prevent subsequent damage. This is provided where the high velocity flow continues unabated upto the end of the cistern.



(vi) Ribbed Pitching: The bed and/or sides of the channel may be provided with bricks alternately laid flat and on edge to dissipate surplus energy of flow. The bricks projecting into the flow cross-section increases the boundary friction.



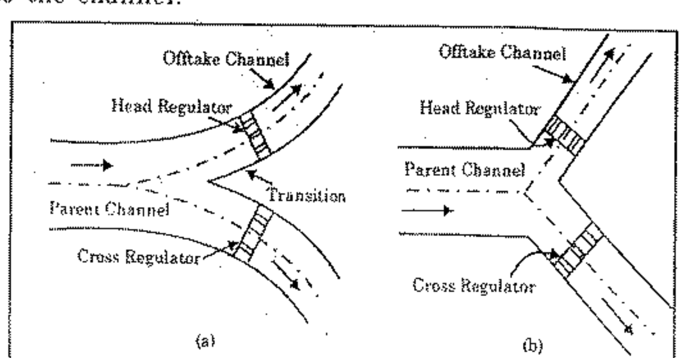
CANAL REGULATORS

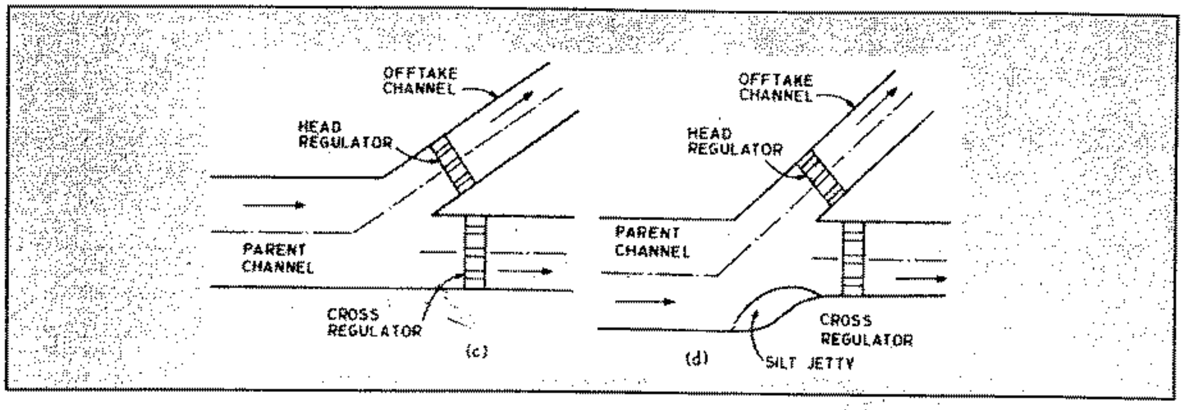
A canal head regulator which is located just u/s of a barrage (or placed appropriately near about a reservoir) is provided in order,

(a) to regulate the discharge flowing into the offtaking channel, and

(b) to control the entry of sediment into the channel.

A head regulator is generally aligned at an angle of 90° to 110° to the axis of the barrage in order to minimise the entry of sediment entering into the channel, besides preventing backflow and stagnant pools in the undersluice pocket that lies in the vicinity of the regulator.





The best alignment of offtake is when the offtake channel makes zero angle with the parent channel initially and then separates out along transition curves as shown in figure.

Cross regulators & Distributary head Regulators

- ***Cross regulator and distributary head regulator*** are provided to control the supplies passing down the parent channel and the offtaking channel respectively.
- A ***cross regulator*** is provided on the parent channel at the d/s of the offtake to head up the parent channel at the channel to draw the required supply.
- A ***distributary head regulator*** is provided at the head of the offtaking channel (or distributary) to control the supplies entering the offtaking channel.

Function of cross regulators

(i) Cross regulator enable effective regulation of the entire canal system.

(ii) They help to raise the water level and feed the offtaking channels to their full demand in rotation when the water level in the parent channel is low.

(iii) They help in closing the supply to the d/s of the parent channel for the purposes of repairs and construction works.

(iv) In conjunction with escapes they help water to escape from the channels.

(v) They facilitate communication, since a road can be taken over them with a little extra cost.

(vi) They help to absorb fluctuations in the various sections of the canal system, and hence to prevent possibilities of breaches in the tail reaches.

(vii) They help to control discharge at an outfall of canal into another canal or lake.

(viii) They help to control water surface slope for bringing the canals to regime slope and section.

Functions of distributary head regulators

(i) They regulate or control the supplies to the offtaking channel from the parent channel.

(ii) They control the entry of silt in the offtaking channel.

(iii) They serve as a meter for measurement of discharge entering the offtaking channel.

(iv) They help in shutting off the supplies when not needed in the offtaking channel or when the offtaking channel is required to be closed for repairs.

CANAL ESCAPES

- Canal escape is a structure constructed on an irrigation channel for the disposal of surplus water from the channel. It is also called *surplus water escape* or *canal surplus escape*.
- Sometimes escapes are provided in the head reaches of main canals to scour out bed silt deposited in the head reaches.

Necessity of Surplus water Escape

There will be necessity of surplus water at any point in an irrigation channel in the following circumstances.

(i) Mistake or difficulty in regulation at the head of a channel.

(ii) Heavy rainfall in upper reaches of a channel.

(iii) Sudden closure of outlets by cultivators due to sudden stoppage of demand.

(iv) Sudden closure of any offtaking channel due to breach.

If the surplus water is allowed to go to the lower reaches the water may overflow the bank and damage the same. Although the supplies may be reduced from the head of the channel, but the effect of such reduction would be felt only after a certain time depending on the distance of the affected reach from the head. As such immediate action is necessary to prevent damage and it is done by escapes.

✦ *The surplus water escapes are called the safety valves of an irrigation channel system.*

TYPES OF ESCAPES

The various types of escapes may be classified according to two different considerations as indicated below.

(1) Classification based on tbe purpose for which escape is provided

(i) Surplus water escapes or canal surplus escapes.

(ii) Canal scouring escapes. (iii) Tail escapes.

(i) Surplus water escapes or Canal surplus escapes

- The surplus water escapes or Canal surplus escapes may be either *Regulator type* or *Weir type* escapes.
- These escapes are provided in the banks of the channel at intervals depending on the importance of the channel and in the vicinity of a suitable natural drain or river for disposal of escaped water.
- The channel leading surplus water from escape to natural drain is called *escape channel* or *outfall channel*.
- The escape must be so located that the surplus water is led through the shortest possible escape channel to a natural drain which can safely take the maximum discharge of the escape without flooding.
- The discharging capacity of surplus water escape may be 0.33 to 0.5 of the capacity of the channel at the site of escape.
- Usually there should be a cross regulator across the channel on d/s side of the location of escape.

(ii) Canal scouring escapes

- The canal scouring escapes are *Regulator type escapes.*
- These escapes are also provided in the banks of the channel but are usually provided only in head reaches of main canals.
- The discharging capacity of the canal scouring escape should be about 0.5 to 0.66 of the capacity at the head of the main canal.
- There should be a *cross regulator* provided across the channel just on the d/s side of the location of escape.

(iii) Tail escapes

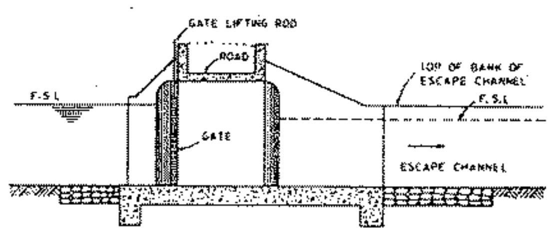
- The tail escapes are *Weir type escapes.*
- These escapes are provided across the channels at the tail ends in the case of irrigation channels which end in natural drains.
- The tail escapes are provided to maintain the required F.S. L. (full supply level) at the tail end of the channel.

(2) Classification based on the structural design of escape.

(i) Regulator (or Sluice) types escapes. (ii) Weir type escapes.

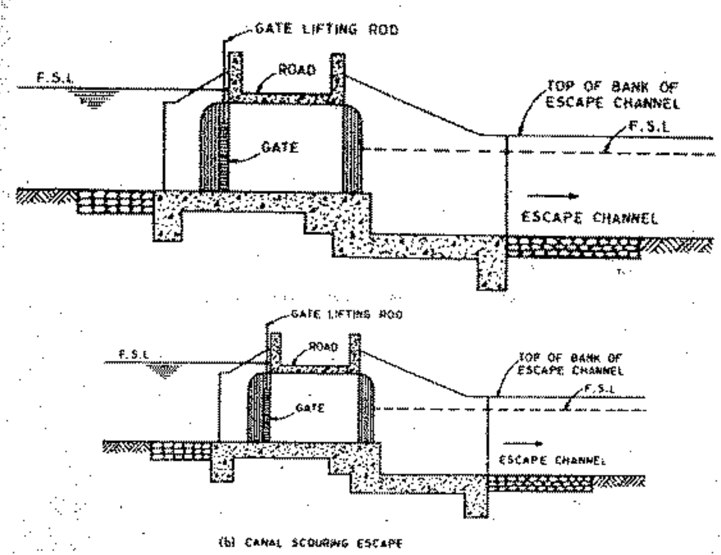
(i) Regulator (or Sluice) type escapes

- A regulator (or sluice) type escape is like a small regulator which is provided in the canal bank and has gates supported on pier.
- The flow through the escape is controlled by the gates which are operated from a raised platform.
- The regulator type escapes are used as surplus water escapes as well as canal scouring escapes.
- When a regulator type escape is used as surplus water escape, its sill (or crest) is kept at the bed level of the channel and the bed of the escape channel is kept at or below the bed level of the channel.



(a) SURPLUS WATER ESCAPE

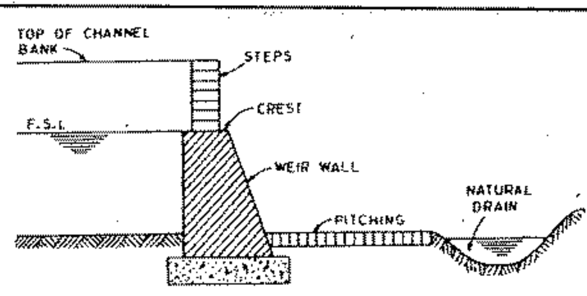
- When a regulator type escape is used as canal scouring escape its sill (or crest) is usually kept about 0.3 m below the bed level of the channel.
- The bed of the escape channel is kept below the bed level of the channel and is provided with such a slope that it is able to carry the silt scoured from the channel bed.



(b) CANAL SCOURING ESCAPE

(ii) Weir type escape

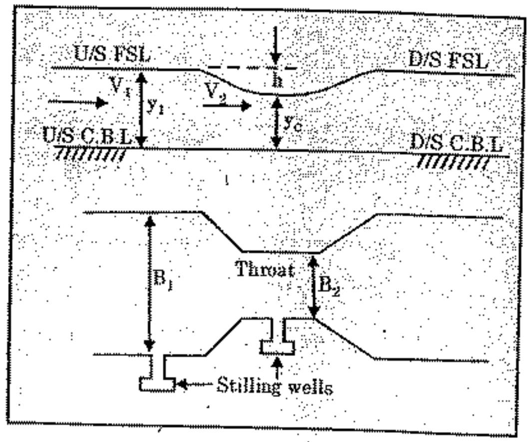
- A weir type escape is like a weir with the crest of the weir wall at the full supply level (F.S.L.) of the channel at the site of escape.
- The weir type escapes are used as surplus water escapes as well as tail escapes.
- When a weir type escape is used as surplus water escapes, the weir wall is provided in the bank of the channel with the crest of the weir wall being kept at the F.S.L. of the channel at the site of escape. As soon as the water level in the channel rises above F.S.L. the water starts flowing over the crest of the escape and is led through escape channel to the nearest natural drain.
- When a weir type escape is used as tail escape, the weir wall is constructed across the channel at its tail end. The crest of the weir wall is kept at the F.S.L. of the channel at the tail. A few sluices are provided at the centre of the weir wall with their sills at the bed level of the channel near the tail.



METERING FLUMES

A meter is a structure constructed in a canal for measuring the discharge in the canal accurately. A metering flume is an artificially flumed (narrowed) section of the channel, which can be utilized for calculating the discharge in the channel.

- The normal u/s section of the channel is narrowed by masonry walls with a splay of 1 : 1 to 2 : 1 to a rectangular section called Throat.
- The channel is slowly diverged from here with a splay of 2 : 1 to 10 : 1 in order to attain its normal section by means of masonry wing walls
- More gradual the convergence and divergence, less will be the loss head in the flume.
- Metering flumes works on the principle of venturimeter.



TYPES OF METERING FLUMES

There are two types of metering flumes which are generally used. They are:

(1) Venturi flume (*also called non-modular venturi flume or drowned venturi-flume*).

(2) Standing wave flume (*also called modular venturi flume or free flow venturi flume*).

(1) Venturi Flume

- A venturi flume consists of a gradually contracting channel leading to throat and a gradually expanding channel leading away from it.
- Stilling wells are provided for measuring head at the entrance and at the throat.

  If h is the difference of head between the two wells, then the discharge is given by

$$\boxed{Q = C_d \frac{a_1 . a_2}{\sqrt{a_1^2 - a_2^2}} \sqrt{2gh}} \quad [C_d \text{ varies from } 0.95 \text{ to } 1.0]$$

where, a_1 = Area at entrance = $B_1 \cdot y_1$; a_2 = Area at throat = $B_2 \cdot y_2$

(2) Standing Wave Flume

- When a standing wave, (i.e., hydraulic jump) forms on the d/s glacis in the diverging channel, the flume is called a *standing wave flume.*
- It is superior to venturi flume because its discharge depends only upon the u/s head over the crest of the throat, and also for the same u/s head, its discharging capacity is more than that of a venturi flume.
- The length of its throat is at least 2 to 3 times the head over the crest.
- It requires greater loss of head and where this loss of head is not available, this flume will act like a venturi flume

Discharge formula for such a flume is given by

$$\boxed{Q = 1.7\, C_d \,.\, B.\, H^{3/2}}$$

where, B is the width of the Throat

CANAL OUTLETS OR MODULES

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

In other words, canal outlets are devices to regulate the flow of water from a bigger channel into a smaller one.

The control and maintenance of the entire network of canals upto the module falls under the jurisdiction of the State Government, and beyond the module, the entire working of the water of the water courses or field channels is taken care of by the cultivators themselves.

Module is a connecting link between the Government and the cultivators. Outlets play a very important role in controlling the flow of water to different areas, so as to effect an equitable distribution of available water in accordance with the needs of the whole area.

Note : Sluices are outlets provided in dams.

Requirements of a Good Module

(1) Module should fit well to the decided principles of water distribution.

e.g., if the supply is to be fixed in accordance with the cultivable area commanded by the outlet, the outlet must be able to pass a constant and a fixed discharge. Similarly, if the supply is to be regulated in accordance with the area irrigated in the past year, the capacity of the outlet should be capable of being changed from year to year.

(2) Module should be simple, so that it can be easily constructed or fabricated by local masons or technicians.

(3) Module should work efficiently with a small working head.

(4) The outlet should be cheaper, since they are required in large number.

(5) The outlet should be sufficiently strong with no moving parts,so as to avoid periodic maintenance.

(6) The outlet should be such as to avoid interference by cultivators, thus preventing under tapping of water by cultivators.

(7) It should draw its fair share of silt.

TYPES OF OUTLETS (MODULES)

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

(1) Non-modular outlets

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

(2) Semi-modular outlets (or Flexible)

- SM are those through which the discharge is independent of the distributary so long as a minimum working head is available.
- The discharge through such an outlet will increase with a rise in the distributary water

surface level and vice versa.

- The common examples of this type of modules are : *pipe outlet, venturi flume, open flume* and *orifice semi-module.*

(3) Modular outlets (or Rigid modules)

- Modular outlets are those through which the discharge is constant and fixed within limits, irrespective of the fluctuations of the water levels of either the distributary or of the water course or of both.
- Common example of this type of module is *Gibb's module.*

CRITERIA FOR JUDGING THE PERFORMANCE OF MODULES

The behaviour and functioning of a module can be judged by the following important terms and definitions :

(1) Flexibility: Flexibility is defined as the ratio of the change of discharge of the outlet to the rate of change of discharge of the distributary channel.

Thus,
$$\boxed{F = \left[\frac{dq/q}{dQ/Q}\right]}$$

where, F = Flexibility of the outlet ; q = Discharge passing through the outlet
Q = Discharge in the distributary channel.

Now, if H is the head acting on the outlet, the discharge through the outlet (q) may be expressed as

$$q = C \cdot H^m \quad \text{... (i)}$$

where C and m are constants depending upon the type of outlet.

Similarly, the discharge passing down the distributary channel may be expressed as

$$Q = K \cdot y^n \quad \text{... (ii)}$$

where K and n are constants and y is the depth of water in the distributary

Hence,
$$\boxed{\text{Flexibility} = F = \frac{m}{n} \cdot \frac{y}{H}}$$

Note : (i) For a modular outlet, flexibility = 0 Hence, it known as a rigid outlet or module
(ii) For a semi modular outlet, flexibility $\neq$ 0 Hence, it is known as flexible module

(2) Proportionality: The outlet is said to be proportional when the rate of change of outlet discharge equals the rate of change of channel discharge. In other words, the outlet is 'proportional' when 'flexibility' equals unity.

Hence for a proportional outlet, we get

$$F = \frac{m}{n} \cdot \frac{y}{H} = 1$$

or
$$\boxed{\frac{H}{y} = \frac{m}{n} = \frac{\text{outlet index}}{\text{channel index}}}$$

(3) Setting : The ratio H/y, i.e. the ratio of the depth of the sill level of the outlet below the FSL of the distributary, to the full supply depth of the distributary, is known as setting.

For a proportional outlet $\boxed{H/y = \dfrac{\text{outlet index}}{\text{channel index}}}$

For a wide trapezoidal channel, the discharge is proportional to $y^{5/3}$. Therefore, for such channels $n = 5/3$

Hence, the channel index is generally 5/3. The discharge through an orifice type outlet is proportional to $\sqrt{H}$; and thus for such an outlet, index m = 1/2.

$$\therefore \quad \text{Setting} = \frac{H}{y} = \frac{m}{n} = \frac{1/2}{5/3} = 0.3$$

Hence, for such a combination of an orifice type outlet and a trapezoidal channel, the setting must be equal to 0.3. In other words, a pipe or orifice type outlet shall be proportional, if the outlet is fixed or set at 0.3 times the depth below the water surface.

Similarly, for a weir type outlet, m = 3/2 ($\because$ the discharge is proportional to $H^{3/2}$).

Hence, the setting for a combination of a weir type outlet and a trapezoidal channel

$$= \frac{m}{n} = \frac{3/2}{5/3} = 0.9$$

Hence, for the weir type outlet to be proportional, the outlet should be set at 0.9 times the depth below the water surface.

(4) Hyper-proportional outlet: An outlet is known to be hyper-proportional, if its flexibility is greater than unity.

$$\text{Thus,} \quad F = \frac{m}{n}\cdot\frac{y}{H} > 1 \quad \text{or} \quad \frac{m}{n} > \frac{H}{y}$$

$$\text{or} \quad \frac{H}{y} < \frac{m}{n} \quad \text{or} \quad \text{Setting} < \frac{m}{n}$$

Hence, the outlet will be hyper-proportional, if the numerical value of its setting (H/y) is less than m/n. In other words, the outlet is hyper-proportional if set higher (i.e., head acting on the outlet, H, is less then what is required for proportionality).

(5) Sub-proportional outlet: An outlet is known to be sub-proportional if its flexibility is less than unity. Thus

$$F = \frac{m}{n}\,\frac{y}{H} < 1 \quad \text{or} \quad \frac{m}{n} < \frac{H}{y} \quad \text{or} \quad \frac{H}{y} > \frac{m}{n}.$$

Hence, the outlet will be sub-proportional, if the numerical value of its setting (H/y) is more than m/n. In other words, the outlet is sub-proportional if set lower (i.e. H is more than what is required for proportionality).

(6) Sensitivity: Sensitivity is defined as the ratio of the rate of change of discharge through the outlet to the rate of change of water level of the distributary, referred to the normal depth of the channel. For rigid modules, the distributary is referred to the normal depth of the channel. For rigid modules, the discharge is fixed and hence the sensitivity is zero.

For flexible modules, where the discharge through the outlet is independent of the water level of the watercourse and depends only upon the water level of the distributary, a gauge can be fixed and calibrated so as to indicate its reading G = 0 when q = 0. Thus

$$\boxed{\text{Sensitivity} = S = \left[\frac{dq/q}{dG/y}\right]}$$

Relation between Sensitivity and Flexibility

We know that, $\text{Flexibility} = F = \left[\frac{dq/q}{dQ/Q}\right]$

But $\frac{dQ}{Q} = \frac{n}{y}dy$

Equation then becomes

$$F = \left[\frac{dq/q}{(n/y)dy}\right] = \frac{1}{n}\left[\frac{dq/q}{dy/y}\right]$$

But since, $dG = dy$ we get $F = \frac{1}{n}.S.$

or $\boxed{S = n.F}$

If $n = \frac{5}{3}$, for wide trapezoidal channels, then, $S = (5/3)\ F$.

Note : (i) Sensitivity of a modular outlet is zero

(ii) Sensitivity of an outlet for a wide trapezoidal or rectangular distributary channel is equal to $\frac{5}{3}$F.

Certain other Important Definitions Connected with Modules

(i) Minimum modular head : The minimum difference between the u/s and d/s water levels, which is required to be maintained so as to enable the module to pass the designed discharge is known as minimum modular head or minimum modular loss.

(ii) Efficiency of an outlet : The ratio of the head recovered (i.e. the head remaining after all the losses in the outlet has been accounted) to the head put in. Lesser is the head required for functioning of the outlet; more efficient the outlet will be. Efficiency is a measure of the conservation of head by the outlet.

(iii) Drowning Ratio : It is the ratio of the depth of water level over crest on the d/s of the module to the depth of water level over crest on the u/s of module. In case of a weir type outlet, the efficiency is the same as the drowning ratio.

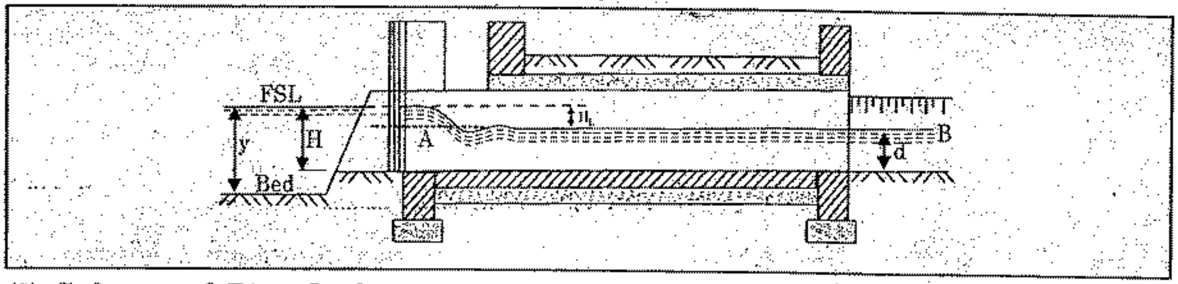
(iv) Modular limit and modular range : The modular limits are the extreme values of any one or more variables, beyond which an outlet becomes incapable of acting as a module or a semi-module. The range between the lowest and the highest limiting values of various such factors is known as modular range.

TYPES OF NON-MODULAR OUTLETS

A non-modular outlet may be in the form of a rectangular opening or open sluice, or a simple submerged pipe. Pipe outlet is a very simple type of a non-modular outlet and is extensively used in South India.

(1) Open Sluice

- An open sluice is a rectangular pucca opening created across the bank of the distributary by raising two abutments at 2.5 to 3 m apart and with a horizontal pucca floor.
- The width of the opening and height of the opening are computed to pass the given discharge by using the appropriate discharge formula.
- The sill level of the outlet (i.e., the sill level of the pucca floor) is kept somewhat above the DBL (designed bed level) of the distributary.



(2) Submerged Pipe Outlet

- Pipes are generally embedded in concrete and are generally fixed horizontally at right angles to the direction of flow.
- They may also be laid sloping upwards by depressing the u/s end of the pipe so as to increase silt conductivity.
- The pipe diameter varies from 10 to 30 cm.

  The velocity through the pipe can be precisely computed by using the relation:

  H_L = Total loss of head = Entry loss + Frictional loss + Velocity head at exit

Note: (i) A non-modular outlet may be controlled by providing a shutter on its u/s end since the loss of head in a non-modular outlet is less than that in a modular outlet, the former is useful where much loss of head is not available. (ii) Flexible modules (semi modules) and rigid modules are preferred to non-modular outlets, irrespective of their being at high or low levels. However, such outlets can work as modular outlets, only within certain limits of water level in the distributary and the water course.

TYPES OF SEMI-MODULES OR FLEXIBLE OUTLETS

The common types of semi-modules are :

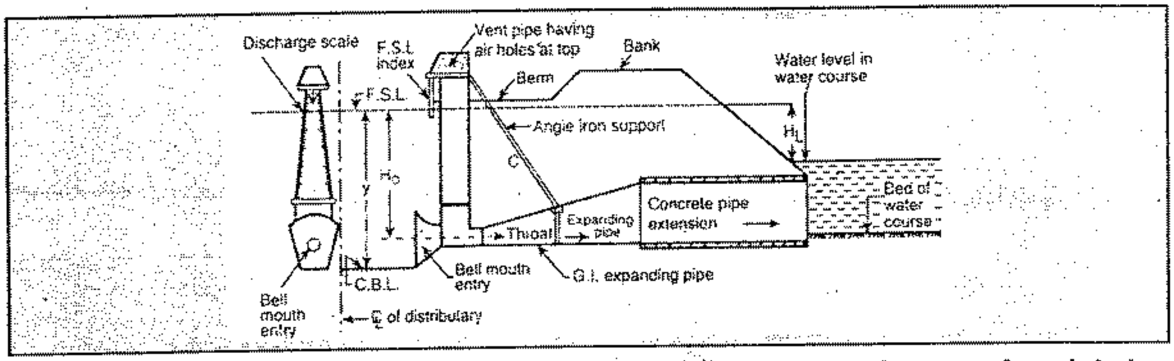
(i) Pipe outlet discharging freely into the air (ii) Venturi-flume outlet or Kennedy's Gauge outlet.
(iii) Open flume outlet (iv) Adjustable orifice semi-module.

(1) Free Pipe Outlet

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

(2) Venturi Flume Outlet or Kennedy's Gauge Outlet

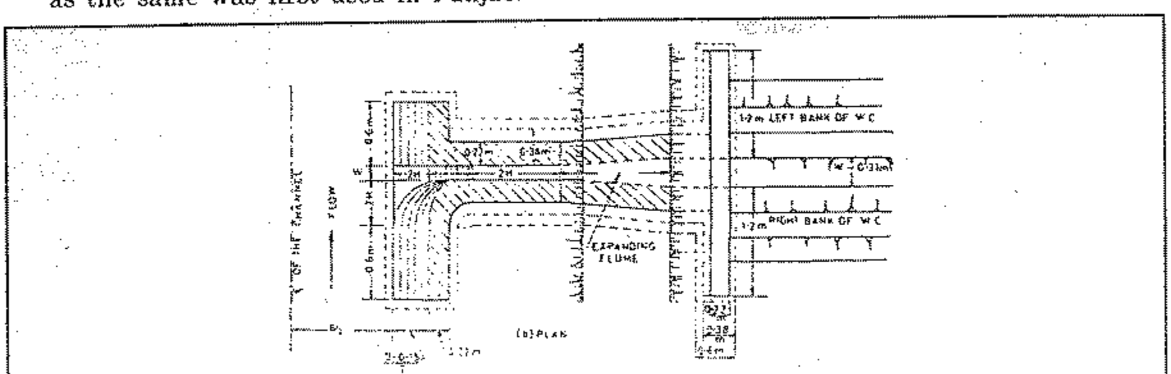
- Kennedy's Gauge Outlet is of a Venturi flume type.
- It is made of cast iron and consists of three main parts :
  (i) an orifice with a bell mouth entry; (ii) a long expanding delivery pipe;
  (iii) an air-vent connecting the throat of the delivery pipe to the atmosphere.
- Water from the distributary enters this module through the *bell-mouthed entry*. The diameter is narrowest at the throat and the water jet shoots across the air in the base of the vertical air bent pipe. The base of the vent pipe is kept conical and air holes are provided at the top, so as to allow free circulation of air around the jet.



- Due to tampering defect and due to its high cost of construction, this type of module has almost become obsolete these days.

(3) Open Flume Outlet

- It is a *weir type outlet* with a constricted throat and an expanding flume on the d/s.
- Due to the constriction, a super-critical velocity is ensured in the throat & thereby allows the formation of jump in the expanding flume. Formation of hydraulic jump makes the outlet discharge independent of water level in the water-course; thus making it a semi-module.
- The most commonly used type of such an outlet is called the *Punjab Open Flume Outlet*, as the same was first used in Punjab.



(4) Adjustable Orifice Semi Modules

- An adjustable orifice module consists of an orifice provided with a gradually expanding flume on the d/s side of the orifice.

- The flow through the orifice is super critical, resulting in the formation of a hydraulic jump in the expanding flume position. The formation of jump makes the discharge independent of the water level in the water course.
- The principal features of an adjustable orifice module are similar to those of a flumed regulator with horizontal crest and curved water of the water course.
- The module is thus perfectly rigid, and at the same time adjustable in dimensions at a slight cost of re-doing the masonry.
- It is considered to be the best of all the modules and is mostly adopted.

TYPES OF RIGID MODULES

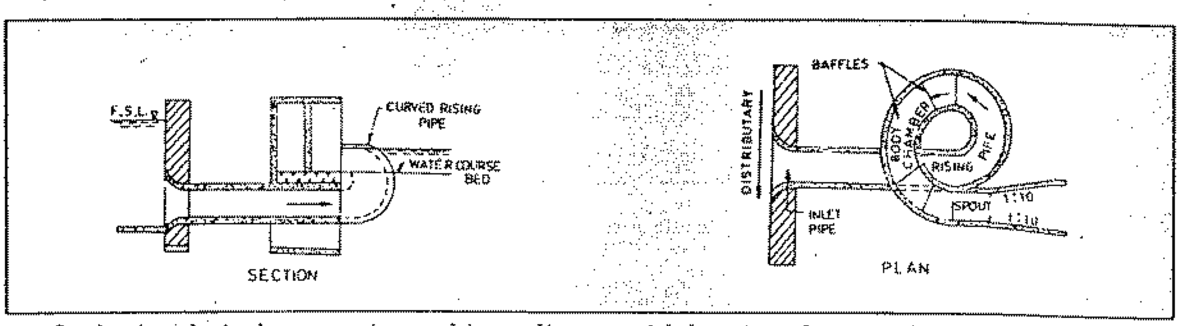
There are few types of rigid modules which have no moving parts, such as:

(1) Gibb's module (2) Khanna's rigid module

Out of all these modules, Gibb's module is the most important and widely used.

(1) Gibb's Rigid Module

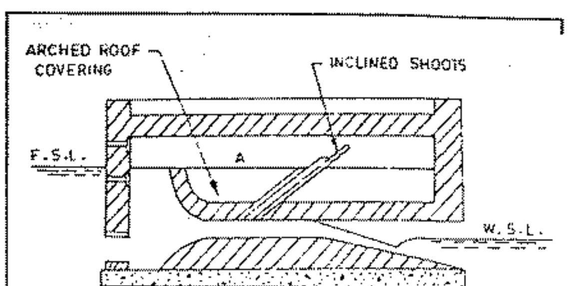
- Gibb's module has an inlet pipe below the distributary bank. The pipe takes water from the distributary to a rising spiral which is connected to the eddy chamber. This produces free vortex motion owing to which there is heading up of water near the outer wall of the rising pipe. The water surface thus slopes towards the inner wall.
- A series of baffle plates of appropriate size are attached to the roof of the eddy chamber such that their lower ends slope against the direction of flow. As the head increases, water banks up at the outer wall of the eddy chamber and strikes against the baffles and spins round in the compartment between two adjacent baffle plates. This result in dissipation of excess energy and release of a constant discharge.



- Outlet is relatively expensive and its sediment withdrawing characteristic is also not good.

(2) Khanna's rigid module

It is similar to an orifice semi module with additional shoots fixed to the roof block. The shoots result in back flow and thus keep the outlet discharge constant. If the water level in the distributary is at or lower than its normal level, the outlet functions like an orifice semi module.



Khanna's Module

OBJECTIVE QUESTIONS

1. Match **List-I** (Control structures) with **List-II** (Functions of the control structures) and select the correct answer using the codes given below the lists:

List-I	List-II
A. Canal drop	1. Control of flow depth
B. Canal escape	2. Control of bed grade
C. Canal cross regulator	3. Control of full supply level
D. Canal outlets	4. Control of discharge

Codes:

	A	B	C	D
(a)	2	3	4	1
(b)	2	3	1	4
(c)	3	2	1	4
(d)	3	2	4	1

2. **Assertion (A):** In a venturi flume, the specific energy values at the normal and throat sections are always individually constant, whether the flow in the normal section is subcritical or supercritical.

   Reason (R): In a canal transition formed by providing a 'hump' the total energy is constant at both the sections irrespective of the type of flow in the normal section.

3. Gibb's module is a type of outlet which ensures
   (a) constant discharge even if the water levels in the supply channel and water course fluctuate
   (b) variable discharge as per the need
   (c) constant discharge into the water course when the water levels in the supply channel vary
   (d) constant discharge for varying water levels in the water course for a given water level in the supply channel

4. Modular limit of a canal outlet is the ratio of
   (a) rate of change of discharge of outlet to that of distributary.
   (b) water depth above outlet crest to the full supply depth of the channel
   (c) water depth above the crest on d/s to that on u/s of outlet
   (d) rate of change of discharge of an outlet to the rate of change of water level of the channel.

5. **Assertion (A):** Canal escape serves as a safety valve for a canal.

   Reason (R): Canal escape discharges the excess water in the parent canal due to sudden closure of outlets by the farmers.

6. If the sensitivity of an irrigation module is 0.5, then what percent variation in outlet discharge will be caused by 50 per cent variation in canal water depth?
   (a) 100% (b) 50% (c) 25% (d) 12.5%

7. What are the recommended setting options of an adjustable proportional module worked with an open flume type outlet?

(1) 3/10 (2) 9/10 (3) 1/2 (4) 5/3

Select the correct answer using the codes given below:

(a) 1 and 2 (b) 1 and 3 (c) 3 and 4 (d) 2 and 4

8. Semi-module outlets are those outlets in which

(a) discharge gets affected by the change in water level of field channel
(b) discharge gets affected by the change in water level of the distributing channel but not with the change in water level of field channel
(c) discharge is independent of water levels in the distributing channel and the field channel
(d) None of the above

9. A submerged pipe outlet is an example of

(a) semi-modular outlet (b) non-modular outlet
(c) rigid module (d) adjustable proportional module

10. A canal fall is a control structure

(a) located at a place where the country slope is flatter than the canal bed slope
(b) located most economically where the depth of cutting is less than the balancing depth
(c) the location of which is independent of the command to be served
(d) designed to secure raising of water surface on its u/s

11. Which type of fall can be generally used for a moderate discharge of 40–60 cumecs and low fall heights of 1 to 1.5 m?

(a) Vertical drop fall (b) Ogee fall (c) Glacis fall (d) Baffle fall

12. The depth discharge relationship of the u/s canal remains practically unaffected by the introduction of a fall of the type :

(a) Ogee fall (b) Sarda type vertical fall
(c) Trapezoidal notch fall (d) none of them

13. For low to moderate discharge of the order of 10 to 15 cumecs and full height of 1 to 15 m, cheaper Sarda type canal falls can be recommended :

(a) universally without any if and buts
(b) if the canal runs with highly variable discharge
(c) when the canal fall is not to be used as a meter and discharge is fairly constant
(d) when the canal fall is not to be used as a mete, irrespective of variation in discharge

14. The type of fall, which you may recommend for very high drops and very low discharges, is :

(a) Sarda type fall (b) Siphon well drop (c) Straight glacis fall (d) Inglis fall.

15. The canal fall, involving parabolic glacis, is called :

(a) Straight glacis fall (b) Glacis fall (b) Inglis fall (d) Montague fall.

16. An Inglis fall, also called a Baffle fall, can be recommended for all discharges, provided :

(a) the fall is more than 1.5 m (b) the fall is undrowned
(c) the fall is either flumed or unflumed (d) all of the above

17. A trapezoidal crest in a Sarda type canal fall is preferred and used in comparison to a rectangular crest, when :

(a) the discharge is less then 14 cumecs
(b) the discharge is equal to or more than 14 cumecs
(c) the discharge is less then 30 cumecs
(d) the discharge is equal to or more than 30 cumecs

18. A cylinder fall, popularly known as syphon well drop, is suitable and economical for:

(a) low discharges and low drops
(b) low discharges and high drops
(c) high discharges and low drops
(d) high discharges and high drops

19. The canal regulator, which is constructed at a diversion headworks, is called a:

(a) cross regulator
(b) distributary head regulator
(c) canal module
(d) none of the above

20. The gated regulator, which is constructed in the parent canal near the site of an offtaking canal, is called a :

(a) canal head regulator
(b) distributary head regulator
(c) cross regulator
(d) none of the above

21. Point out the choice among the following, which is not a function of a distributary head regulator:

(a) it serves as a meter for measuring discharge in the offtaking canal
(b) it serves to control silt entry into the offtaking canal
(c) it helps in controlling and regulating supplies in the entire d/s canal network.
(d) it helps in controlling supplies in the offtaking canal.
(e) none of the above

22. Khosla's theory of independent variables is used in the design of :

(a) weirs and barrages
(b) cross regulators and head regulators
(c) modules
(d) both (a) and (b) above
(e) all (a), (b) and (c) above.

23. The arrangement made in a canal network, which acts as its safety valve, is :

(a) cattle crossing (b) canal ladder (c) canal escape (d) canal module.

24. Canal modules help in :

(a) modulating and varying the canal discharge
(b) releasing desired quantity of water into the water courses
(c) releasing desired quantity of water into the minors
(d) all of the above

25. The type of irrigation module, which makes equitable distribution of water more difficult, is a :

(a) non-modular outlet
(b) semi-module
(c) rigid module
(d) none of them.

26. A good irrigation module is the one, which :

(a) draws heavy silt from the canal
(b) draws clear water from the canal
(c) draws fair share of silt from the canal
(d) none of the above

27. If the rate of change of discharge from an irrigation outlet is equal to the rate of change of discharge in the distributary, then the outlet is called:
   (a) flexible
   (b) proportional
   (c) sensitive
   (d) none of these

28. The rate of change of discharge through an irrigation outlet becomes equal to the rate of change of water depth in the channel, when its :
   (a) flexibility is 1
   (b) sensitivity is 1
   (c) setting is 1
   (d) sensitivity is zero.

ANSWERS

1. (b)	**2. (c)**	**3. (a)**	**4. (b)**	**5. (a)**	**6. (a)**
7. (a)	**8. (b)**	**9. (b)**	**10. (b)**	**11. (c)**	**12. (c)**
13. (c)	**14. (b)**	**15. (d)**	**16. (d)**	**17. (a)**	**18. (b)**
19. (d)	**20. (c)**	**21. (c)**	**22. (d)**	**23. (c)**	**24. (b)**
25. (a)	**26. (c)**	**27. (b)**	**28. (b)**		

Canal Head Works & Seepage Theory

CHAPTER

7

Canal Head Works & Seepage Theory

INTRODUCTION

An irrigation canal takes its supplies from a river or a stream. In order to divert water from the river into the canal it is necessary to construct certain works or structures across the river and at the head of the offtaking canal. These works are termed as *canal headworks* or *headworks*.

The *canal headworks* may be classified into the following types

(1) Storage headworks (2) Diversion headworks.

- ***A storage headworks*** consists of a dam constructed across the river to create a reservoir in which water is stored during the period of excess flow in the river.
- From the reservoir water is supplied to the canal in required quantity as per the demand.
- *A storage headwork* stores water in addition to its diversion into the canal.
- ***A diversion headworks*** raises the water level in the river and divert the required quantity into the canal.

Note : Our prime focus will be to study in detail about *diversion headworks*.

The various purposes served by a diversion headworks are as follows:

(i) It raises the water level in the river in order to increase the commanded area.

(ii) It regulates the supply of water into the canal.

(iii) It controls the entry of silt into the canal.

(iv) It provides some storage of water for a short period.

(v) It reduces the fluctuations in the level of supply in the river.

TYPES OF DIVERSION HEADWORKS

The diversion headworks may be classified into the following types.

(i) Temporary diversion headworks (ii) Permanent diversion headworks.

- *Temporary diversion headworks* consists of a spur or bund constructed across the river to raise the water level in the river and divert it into the canal.

- These bunds are constructed almost every year after the floods, because they may be damaged by the floods.
- ***Permanent diversion headworks*** consists of a permanent structure such as *weir or barrage* constructed across the river to raise the water level in the river and divert it into the canal.
- In our country, most of the diversion head works for important canal system are *permanent diversion headworks.*

LOCATION OF CANAL HEADWORKS

The location of canal headworks depends on the stages of flow of river. Most of the large rivers in our country have the following four stages of flow.

(1) Rocky stage or Hilly stage

- In this stage, the river is in the hills.
- The bed slope and velocities are high in this stage.
- The cross-section of the river is made up of rock or very large boulders.

(2) Boulder stage

- From the rocky stage, the river passes on to the boulder stage.
- In this stage, the bed and banks of the river are composed of boulder and gravel.
- The river cross-section is usually well defined and confined between non-submersible banks on either side which are close to the main current of the river.
- In this stage, the bed slope and velocity are less than those in the rocky stage.
- There is large subsoil flow in the boulder region because of high permeability.

(3) Trough stage or Alluvial stage

- From boulder stage, the river passes on to the alluvial plain created by itself.
- The bed slope and the velocity are small in this stage.
- In this stage, cross-section of the river is made up of alluvial sand and silt.

(4) Delta Stage

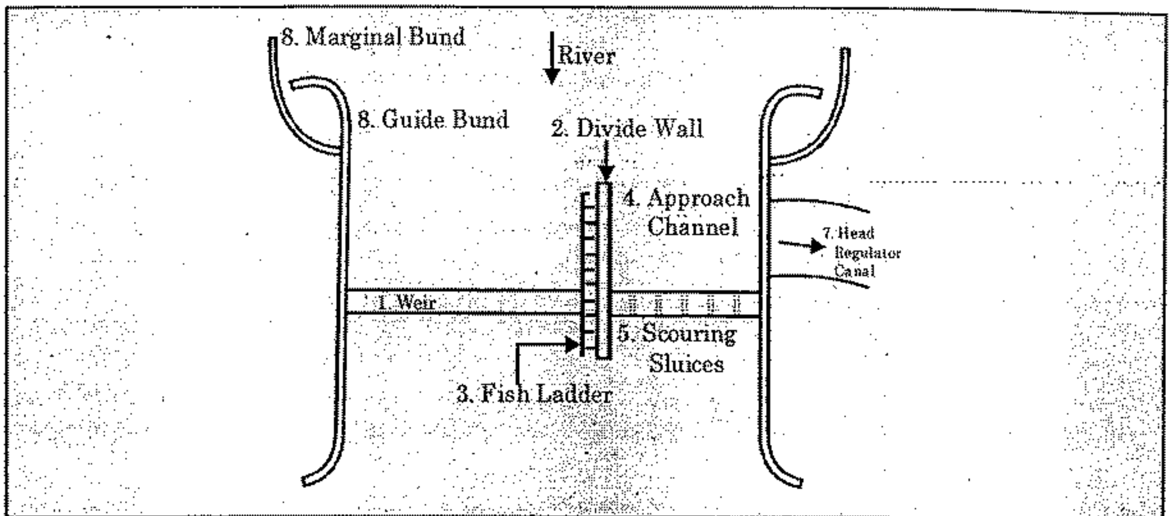
- From trough stage, the river passes on to the delta stage as it approaches the ocean.
- In this stage, the bed slope and velocity are reduced so much that it is unable to carry its sediment load.
- It drops down the sediment and gets divided into channels on either side of the deposit resulting in the formation of a *delta*.

Note : For the construction of the *canal headworks* both the *rocky* and the *delta* stages are not suitable. The *canal headworks* may be located either in the *boulder stage* or in the *trough stage* of the river.

COMPONENTS OF DIVERSION HEADWORKS

The various components of a diversion headworks are

(1) Weir or Barrage (2) Divide wall or Divide groyne
(3) Fish ladder (4) Pocket or Approach channel
(5) Undersluices or Scouring sluices (6) Silt excluder
(7) Canal head regulator (8) River training works (Marginal bunds & Guide bunds).



WEIR

- A weir is an obstruction constructed across a river to raise its water level and divert the water into the canal.
- Shutters are usually provided on the crest and only small part of the ponding of water is carried out by shutters.
- Major part of ponding of water is achieved by the raised crest
- During floods, shutters are dropped down to allow water to flow over the crest of the weir.
- Weirs are usually aligned at right angles to the direction of flow of the river.

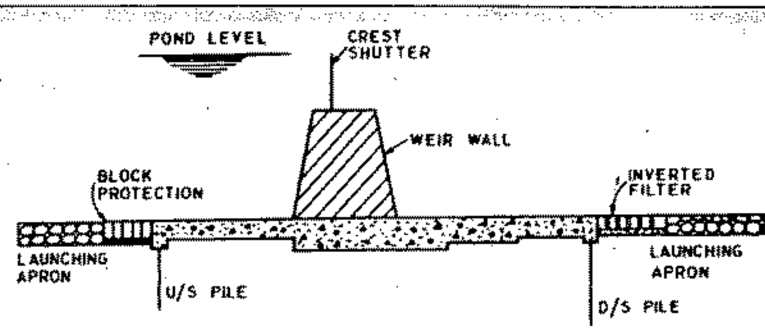
  Weirs may be classified into the following three types

(1) Masonry weirs with vertical drop or vertical drop weirs
(2) Rockfill weirs with sloping aprons (3) Concrete weirs with a d/s glacis.

(1) Masonry weirs with vertical drop or vertical drop weirs

- This type of weir consists of an impervious horizontal floor or apron and a masonry weir wall with either both u/s and d/s faces vertical; or both faces inclined; or u/s face vertical and d/s face inclined.
- *Curtain walls* and *cutoffs* or *piles* are provided at the u/s and the d/s ends of the floor.
- Immediately at the u/s end of the floor a *block protection* and at the d/s end a *graded inverted filter* is provided.

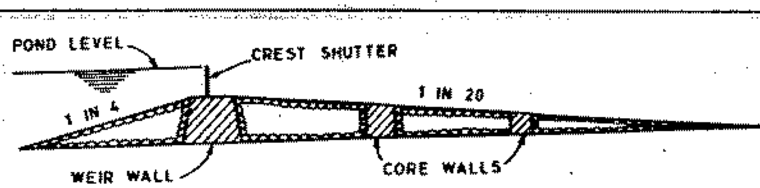
- After the block protection and the inverted filter, launching aprons are provided.
- This type of weir is suitable for any type of foundation.
- This is an old type of weir for which floor design was usually based on Bligh's theory.



(2) Rockfill weirs with sloping aprons

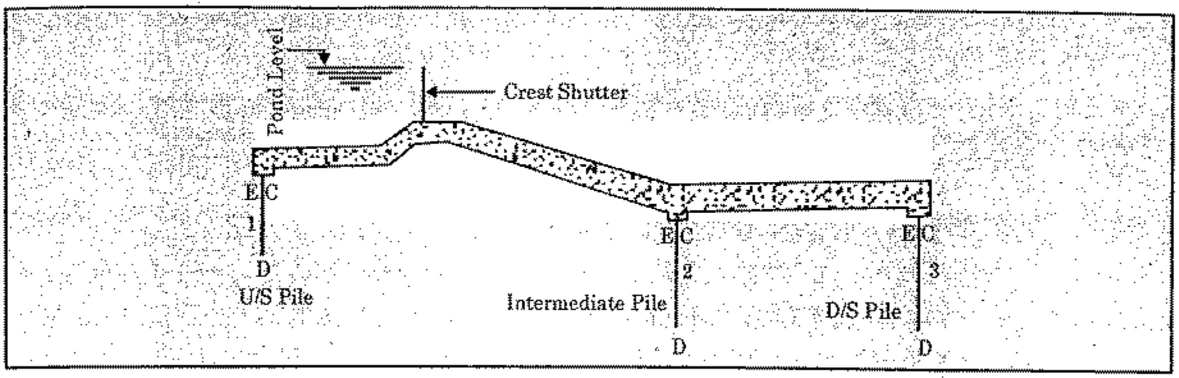
- It consists of a masonry weir wall and dry packed boulders laid in the form of glacis or sloping aprons on the u/s and d/s sides of the weir wall with a few intervening core walls.
- It is the simplest type of construction.
- It requires a very large quantity of stone.

  e.g., Okhla weir across River Yamuna near Delhi.



(3) Concrete weirs with a d/s glacis

- This type of weir is of recent origin based on the design concepts of Khosla's theory for subsoil flow.
- Sheet piles of sufficient depths are provided at the u/s and d/s ends of the floor.
- Sometimes an intermediate pile is also provided.
- Hydraulic jump is developed on the glacis due to which considerable energy of the flowing water is dissipated.
- Various protective measures such as *block protection*, *inverted filter* and *launching aprons* are adopted.
- This type of weir may be constructed on pervious foundations and are commonly adopted these days.

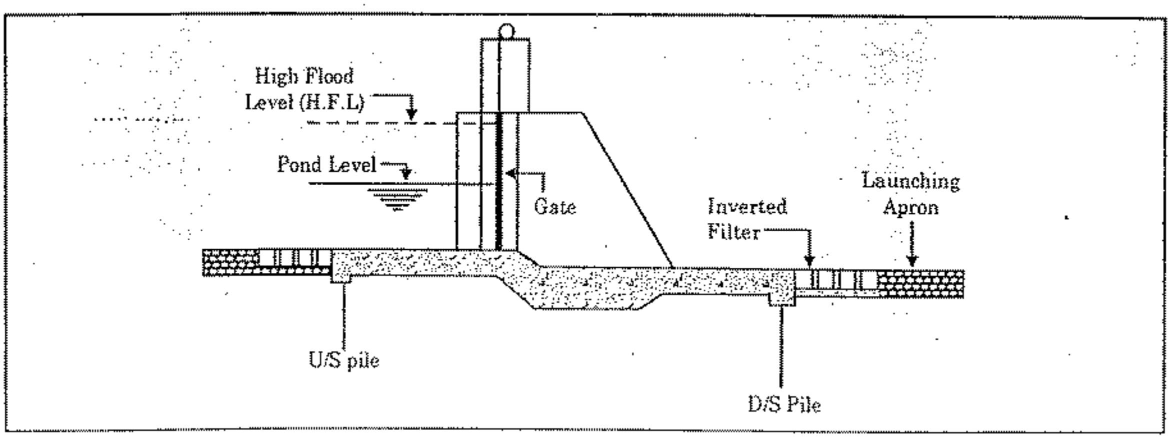


Gravity and Non-Gravity Weirs

- When the weight of the weir balances the uplift pressure caused by the head of the water seeping below the weir, it is called a *Gravity weir.*
- If the weir floor is designed continuous with the divide piers as reinforced structure, such that the weight of concrete slab together with the weight of divide piers keep the structure safe against the uplift then the structure may be called as a *Non-gravity Weir.*

BARRAGE

- Barrage is a structure similar to weir with the only difference that the crest is kept at a low level and the ponding of water is accomplished mainly by means of gates. During floods these gates can be raised above the HFL (high flood level) and thus enable the high flood to pass with minimum of afflux
- A barrage provides better control on the water level in the river but it is comparatively more costly.
- It is also known as river regulator.
- The crest of undersluice portion of the weir is kept at a lower level than the crest of the normal portion of the weir.



Note : (i) The design of a barrage involves the same procedure as a concrete weir.

(ii) The difference between a barrage and weir is only qualitative. In the former, gates provide the larger part of the ponding while in the latter the crest carries out most of the ponding.

✦ **Afflux** : The rise in the max. flood level u/s of the weir, caused due to construction of the weir across the river.

✦ **Pond level** : The water level required in the under sluice pocket u/s of the canal head regulation in order to feed the canal with its full supply.

UNDERSLUICES OR SCOURING SLUICES

- The undersluices are the openings provided in the weir wall with their crest at a low level.
- Openings are fully controlled by gates.
- They are located on the same side as the offtaking canal.
- If two canals take off, one on either side of the river, then it would be necessary to provide undersluices on either side.
- The spans of the undersluices should be wide enough (usually 10 to 20m) in order to be efficient in scouring action.
- Undersluices are also called *scouring sluices* as they help in removing the silt near the head regulator.
- By construction of undersluice portion of the weir a comparatively less turbulent pocket of water is created near the canal head regulator.

Functions of undersluices

The functions of undersluices are as follows:

(i)They preserve a clear and well defined river channel towards the canal head regulator,

(ii) They scour the silt deposited on the river bed in the pocket u/s of the canal head regulator.

(iii) They pass low floods without the necessity of dropping the weir crest shutters.

(iv) They help to lower the high flood level by supplementing the discharge over the weir

Note: The discharging capacity of the undersluices is provided as the maximum of the following:

(i) Two times the maximum discharge of the offtaking canal (ii) Maximum-winter discharge

(iii) 20% of the maximum flood discharge

DIVIDE WALL OR DIVIDE GROYNE

- A divide wall is a long masonry or concrete wall which is constructed at right angles to the axis of the weir to separate the undersluices from the rest of the *weir* or *weir proper*. The divide wall extends a little u/s side upto a distance little beyond the beginning of the canal head regulator and on the d/s side upto the end of the loose protection of the undersluices.
- These walls are likely to be subjected to maximum differential pressure when the full discharge of the river is passing through undersluices and no discharge is passing through the weir.
- If two canals take off, one on either side of the river, then two divide walls are required, one on each side.
- The top width of divide wall is about 1.5 to 2.5 m.

Functions of divide wall

The functions of a divide wall are as follows:

(i) Divide wall separates the floor level of the undersluices from the floor level of the weir as the floor level of the undersluices is generally lower than the floor level of the weir.

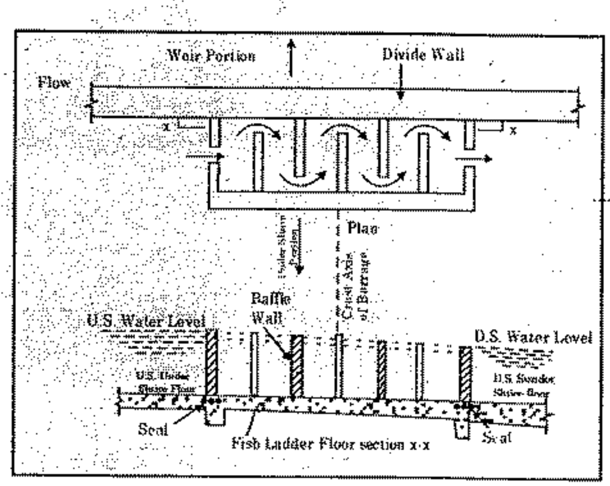
(ii) It provides a comparatively quiet pocket in front of the canal head regulator resulting in deposition of silt in the pocket and entry of clear water into the canal.

(iii) It provides a straight approach through the pocket and thus helps to concentrate scouring action of the undersluices for washing out the silt deposited in the pocket.

(iv) It keeps the cross currents away from the weir. A cross current will develop when the main current in the river tends to approach the bank opposite the canal head regulator and the weir forces the water to flow towards the regulator. The cross current cause formation of vortices and result in deep scour.

Note : The divide walls can be designed as cantilever retaining walls subjected to silt pressure and water pressure from the undersluice side.

FISH LADDER



- A fish ladder is generally provided to enable the fish ascend the head waters of the river and thus reach their spawning grounds for breeding or to follow their migratory habits in search of food.
- Large rivers have various types of fish many of which are migratory. In our country generally, *anadromous fish* move from u/s to d/s in the beginning of winter in search of warmth and return u/s before monsoon for clearer water.
- Due to the construction of a weir or barrage across the river such migration of the fish will be obstructed and if no arrangement is made in the weir or barrage for this migration, large scale destruction of the fish life may take place in the river.
- Fish ladder is a device by which the flow energy can be dissipated in such a manner so as to provided smooth flow at sufficiently low velocity around 3.0 to 3.5 m/s.
- The various types of fish ladders are

  (a) Pool type (b) Steep channel type (c) Fish lock
- To check the velocity flow in a fish ladder, baffles or other staggering devices are provided.
- Fish ladder is generally located adjacent to divide wall near undersluices

CANAL HEAD REGULATOR

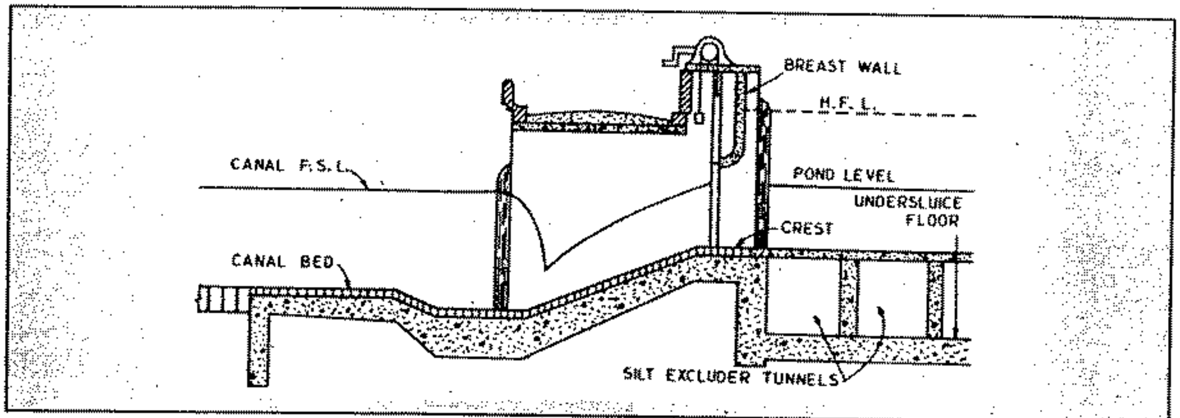
- A canal head regulator is a structure constructed at the head of a canal taking off from the u/s of a weir or a barrage.
- It consists of a number of spans separated by piers which support the gates provided for

regulation of flow into the canal. The spans of 6 to 8 m are commonly used with counterbalanced steel gates which are operated manually by winches.

- The regulator is generally aligned at right angle to the weir, but slightly larger angles between 90° to 110° are preferred for providing smooth entry of water into the regulator.
- Control of silt entering the canal is provided by keeping the crest of the head regulator about 1 to 1.5 m higher than the crest of the undersluices.

A canal head regulator serves the following functions.

(i) It regulates the supply of water into the canal. (ii) It controls the entry of silt into the canal.
(iii) It completely excludes the high flood from entering into the canal.



Breast Wall

- Breast wall is an RCC wall provided from the pond level upto river HFL (highest flood level) to avoid spilling of the water over the canal regulator gates.
- As we know that during high floods, the water in the river pocket will be much higher than the pond level so we tend to provided a breast wall in this case.
- Breast wall spans for the entire length of the regulator and will rest over the piers of the regulator bays.
- Breast wall is subjected to vertical self weight and horizontal water pressure acting against it from the u/s side.

WEIR OR BARRAGE REGULATION

- Silt entering the canal which originates from a river has to be controlled to avoid frequent closures of canal for silt removal in order to allow the design discharge to be carried without any hindrance along the canal.
- The silt can be removed from the entering water by operating the undersluices of the barrage or weir in an appropriate manner or by incorporating suitable structures at the head works (silt excluders) or in the canal (silt ejectors).

The supplies entering a canal which takes off from the u/s of a weir or a barrage can be regulated in the following two ways.

(1) Still pond regulation (2) Semi-open flow regulation

(1) Still pond regulation

Procedure:

(i) In this method of regulation, all the gates of the undersluices are kept closed while the canal is running. Hence the undersluice pocket draws only as much discharge as is required for the canal.

(ii) The velocity of flow in the undersluice pocket, gets reduced leading to deposition of silt in the pocket to a level of about 0.5 m below the crest of the regulator.

(iii) Canal is then closed and the gates of the undersluices are opened so that the deposited silt gets scoured and discharged into the river on the d/s side

(iv) The scouring process takes about a day and the canal is closed during this period

(v) The undersluices are closed as soon as the deposited silt gets washed away and the supply in the canal is restored.

- This is very useful method to control the amount of silt entering the canal.
- The main draw back of this method is that the supply in the canal has to be stopped during the cleaning operation of the undersluice pocket which leads to loss of irrigation during that period.
- This method is possible only when the crest of the canal head regulator is high above the u/s floor of the undersluices.
- The supply that is allowed to enter into the pocket is the same as the discharge into the canal. The discharge above that is moved d/s from other route.

(2) Semi-open flow regulation

Procedure:

(i) In this method of regulation, the gates of the undersluices are always kept partially open so that water in excess of the canal requirement enters the undersluice pocket and the same is allowed to be discharged to the d/s side through the undersluices.

(ii) The water entering the pocket gets divided into two parts the top water (above the crest of the regulator) which is relatively clean enters the canal through the regulator, and the bottom silt laden water (below the crest of the regulator) escapes through the undersluices to the d/s side.

(iii) Certain velocity is maintained in the pocket due to continuous flow through the undersluices which keeps silt in suspension and the same is discharged to the d/s side without being deposited in the pocket.

- This method does not provide proper control on entry of silt into the canal because turbulence created in the pocket tend to raise the coarser material upwards & enter the canal.
- The advantage of this method is that there is continuous scouring of silt from the undersluice pocket and the canal need not be closed for this purpose.

Note : (i) A *smooth bed* should be provided in the main canal u/s of the offtaking channel to help the sediment to concentrate in the lower layers due to the decrease occurring in the intensity of turbulence.

(ii) Lower layers of water are more easily directed into the off taking channel as compared to the upper layers because the lesser velocities in the lower layer.

(iii) Near the bed, sediment concentration is generally very high.

SILT CONTROL DEVICES

Special devices are required to be provided to control the entry of silt into a canal. These devices are of two types :

(1) Silt excluders (2) Silt extractors or silt ejectors

Fundamental principle of operation **:** In a stream of water carrying silt in suspension, the concentration of silt in the lower layers 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 which is relatively clear is allowed to flow in the canal while the bottom water which is heavily silt laden is allowed to go as a waste.

(1) Silt excluder

- Silt excluders are those silt control devices which exclude silt from water entering the canal.
- These devices are constructed on the river bed in front of the head regulator.
- A silt excluder consists of a number of rectangular tunnels resting on the floor of the undersluice pocket.
- The bottom of the tunnels is formed by the floor of the undersluice pocket.
- The top level of the roof of the tunnels is kept at same level as of the crest (or sill) of the canal head regulator.
- These tunnels are parallel to the axis of the canal head regulator and are of varying lengths.
- The ends of the tunnels terminate at the end of the undersluice bay while the positions of their entry points vary.
- The water approaching the head regulator is separated into two parts by the roof slab of the tunnels. The part on the top enters into the canal through the head regulator, while that at the bottom flows through the tunnels and is discharged from the undersluices to the d/s side.
- A minimum velocity of 2 to 3 m/s must be maintained through the tunnels to keep them free from silt deposit.
- The height of the opening of the tunnel is equal to the height of the crest of the head regulator above the floor of the undersluice pocket minus the thickness of the roof slab of the tunnel.

(2) Silt extractors or Silt ejectors

- Silt extractors are those silt control devices which remove the silt which has already entered the canal from the head.
- These devices are provided in the canal a little distance d/s from the head regulator.
- A silt extractor or ejector consists of a horizontal diaphragm slab a little above the canal bed which separates out the bottom layers.
- Under the diaphragm slab there are tunnels to eject the heavily silt laden bottom water into an escape channel.

- The canal bed is slightly depressed under the diaphragm and the height of the tunnels is about 0.5 to 0.6 m. The acceleration of velocity is achieved by quickly but steadily reducing the cross-sectional area by streamlined vanes.
- Silt ejector should not be located either too close u/s to the regulator or too far d/s off the regulator.
- The ejector consists of a *diaphragm, tunnels, control structures* and *an outfall channel.*

FAILURE OF WEIRS ON PERMEABLE FOUNDATION

Various causes of failure of the weirs on permeable foundations may be classified into the two broad categories:

(1) Due to seepage or subsurface flow (2) Due to surface flow

The seepage or subsurface flow may cause the failure of a weir in the following two ways.

(1) Seepage flow

(i) By piping or undermining: If the water percolating through the foundation has sufficient force when it emerges at the d/s end of the impervious floor it may lift up the soil particles at the end of the floor. With the removal of the surface soil there is further concentration of flow into the resulting depression and more soil is removed. This process of erosion progressively extends backwards towards the u/s side and results in the removal of soil and developing pipe like formation beneath the floor. The floor may subside in the hollows so formed and fail which is known as failure due to piping or undermining.

Remedies

(a) Provide sufficient length of the impervious floor so that the path of percolation is increased and exit gradient is reduced.

(b) Provide piles at the u/s and the d/s ends of the impervious floor.

(ii) By uplift pressure:

The water percolating through the foundation exerts an upward pressure on the impervious floor. This pressure is known as uplift pressure. If the uplift pressure is not counterbalanced by the weight of the floor, it may fail by rupture.

Remedies

(a) Provide sufficient thickness of the impervious floor.

(b) Provide pile at the u/s end of the impervious floor (uplift pressure is reduced on the d/s side).

(2) Surface flow

(i) **By suction due to standing wave or hydraulic jump:** The standing wave or hydraulic jump developed on the d/s side of the weir causes suction or negative pressure which also acts in the direction of uplift pressure. Suction happens due to the conversion of high pressure before jump to low pressure after jump.

Remedies

(a) Provide additional thickness of the impervious floor to counterbalance the suction pressure due to standing wave.

(b) Construct floor as monolithic concrete mass instead of in different layers of masonry

(ii) **By scour on the u/s and d/s of the weir:** Both at u/s and d/s ends of the impervious floor the bed of the river may be scoured to considerable depths during floods. If no preventive measures are taken, these scours may cause considerable damage

(a) Provide deep piles both at u/s and d/s ends of the impervious floor. The piles are to be driven upto a depth much below the calculated scour depth.

(b) Provide launching aprons of suitable length and thickness at u/s and d/s ends of the impervious floor.

Note : *Diversion headworks are usually loaded in boulder and trough stages of a river where only permeable foundations are available for the construction of the weirs.*

BLIGH'S CREEP THEORY

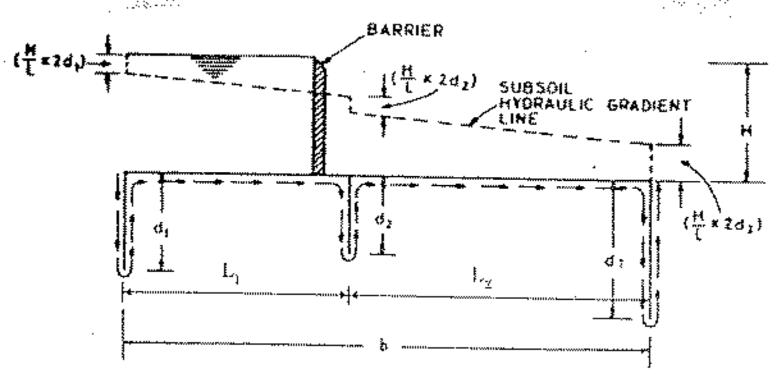
- Bligh developed a theory for the subsurface flow which came to be known as Bligh's creep theory.
- Bligh assumed that the percolating water follows the outline of the base of the structure which is in contact with the subsoil.
- The length of the path traversed by the percolating water is called the length of creep or creep length.
- Bligh further assumed that the head loss per unit length of creep which is called the hydraulic gradient is constant throughout the percolating passage i.e. the loss of head is proportional to the length of creep.

From figure, the percolating water will follow a path indicated by arrows and the creep length L will be given by $L = b + 2d_1 + 2d_2 + 2d_3$

Hence, the hydraulic gradient or the loss of head per unit length of creep will be given by

$$\frac{H}{L} = \frac{H}{b + 2d_1 + 2d_2 + 2d_3}$$

(1) As hydraulic gradient is constant, if L_1 is the creep length upto any point, then head loss upto this point will be $(H/L)\,L_1$ and the residual head at this point will be $[H - (H/L)\,L_1]$. Also there will be losses of head equal to $(H/L)\,2d_1$, $(H/L)\,2d_2$ and $(H/L)\,2d_3$ respectively in the planes of the three vertical cutoffs and the hydraulic gradient line will be drawn as shown in figure.



Bligh's path of creep and sub-soil hydraulic gradient line.

The reciprocal of hydraulic gradient i.e., (L/H) is known as Bligh's coefficient of creep C. Hence

$$L = CH.$$

According to Bligh, to ensure the safety of the impervious floor against the two possible ways in which failure may be caused by subsurface flow, following criteria are required to be satisfied.

(i) Safety against piping. The length of creep should be sufficient to provide a safe hydraulic gradient according to the type of soil.

The hydraulic gradient (H/L) is then equal to (1/C) and according to Bligh if the hydraulic gradient $\leq$ (1/C) there will be no danger of piping.

$L_{req} \geq CH$, value of C lies between 5-15 and depends on the types of soil.

(ii) Safety against uplift pressure : The ordinate of the subsoil hydraulic gradient line above the bottom of the floor at any point represents the residual seepage head or the uplift pressure at that point. If at any point, h' is the ordinate of the hydraulic gradient line above the bottom of the floor then at that point the uplift pressure exerted by the percolating water is wh', where w is specific weight of water. If at this point the floor thickness is t and the specific gravity of the material of the floor is G, the downward force per unit area due to the weight of the floor is (wG) t.

For equilibrium the uplift pressure must be counterbalanced by the weight of the floor.

$$wh' = (wG)\,t$$

or $$\boxed{t = \frac{h'}{G}} \quad \text{... (i)}$$

Hence, from equation (*i*) we have $h' = tG$

Deduct 't' from both sides

$$\therefore \quad h' - t = tG - t = t\,(G - 1)$$

From which $$\boxed{t = \frac{h'-t}{G-1}}$$

or $$t = \frac{h}{G-1} \quad \text{... (ii)}$$

where, h (= h' − t) is the ordinate of the hydraulic gradient line measured above the top of the floor.

The floor on the d/s side of the barrier must be designed in accordance with equation (ii)

Note : (i) It may be noted that on the u/s side of the barrier i.e., weir wall or the gates of the barrage, which holds up the water and creates the seepage head, the uplift pressures are counterbalanced by the weight of the water standing on the floor. However, when there is no water on the u/s side of the barrier there will be no seepage head and no uplift pressure. As such for the u/s floor, only a nominal thickness needs to be provided to resist wear, impact of flowing water or development of crack. (ii) According to Bligh's theory, a vertical cutoff at the u/s end of the floor is more useful than the one at the d/s end of the floor.

Limitation of Bligh's creep theory

The various limitation of Bligh's creep theory are as follow.

(i) Bligh made no distinction between horizontal and vertical creep

(ii) Bligh's method holds good so long as the horizontal distance between the cutoffs or pile lines is greater than twice their depth.

(iii) Bligh did not indicate any significance of exit gradient.

(iv) According to Bligh, the loss of head is proportional to the creep length, but it is not true.

(v) Bligh did not specify the absolute necessity of providing a cutoff at the d/s end of the floor, whereas it is absolutely essential to provide a deep vertical cutoff at the d/s end of the floor to prevent undermining.

LANE'S WEIGHTED CREEP THEORY

Lane, stipulated that the horizontal creep is less effective in reducing uplift (or in causing loss of head) than the vertical creep as Bligh made no distinction between the two creeps. He suggested a weightage factor of $\frac{1}{3}$ for the horizontal creep, as against 1.0 for the vertical creep.

Thus, the total Lane's creep length (L_l) is given as:

$$L_l = (d_1 + d_1) + \frac{1}{3} L_1 + (d_2 + d_2) + \frac{1}{3} L_2 + (d_3 + d_3)$$

$$= \frac{1}{3} \cdot (L_1 + L_2) + 2 \ (d_1 + d_2 + d_3) = \frac{1}{3} \cdot b + 2\ (d_1 + d_2 + d_3)$$

To ensure safety against piping, the creep length L_l must not be less than $C_l\, H_L$, where H_L is the head causing flow, and C_l is Lane's creep coefficient.

Note : Lane's theory was an improvement over Bligh's theory but was purely empirical. Bligh's theory is still used but Lane's theory is practically nowhere used .

THEORY OF SEEPAGE FLOW

It has been established both theoretically as well as experimentally that if Darcy's law holds good then steady seepage through a homogeneous soil can be represented by

$$\boxed{\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} = 0} \qquad \text{[Laplacian equation]}$$

in which ϕ is flow potential given by $\phi = kh$, where k is coefficient of permeability as defined by Darcy's law and h is the seepage head at any point in the soil. Equation (i) represents two sets of curves intersecting each other orthogonally or at right angles. One set of curves is called streamlines or flow lines and the other set of curves is called equipotential lines.

KHOSLA'S THEORY FOR DESIGN OF WEIR ON PERMEABLE FOUNDATIONS

For quite a long time, Bligh's theory was the accepted basis for designing the structures on permeable foundations. But few failures led to the investigation by a researcher named Khosla. Dr. A. N. Khosla and his associates carried few and came to the following conclusions.

(i) The outer face of the end sheet piles were much more effective than the inner ones and the horizontal length of the floor.

(ii) The intermediate sheet piles, if smaller in length than the outer ones were ineffective except for local redistribution of pressures.

(iii) Undermining of floors started from the tail end. If the hydraulic gradient at the exit was more than the 'critical gradient' for the particular soil the soil particles would move with the flow of water thus causing progressive degradation of the subsoil, resulting in cavities and ultimate failure.

(iv) It was essential to have a deep vertical cutoff at the d/s end of the floor to prevent undermining.

Khosla carried out research to find an ultimate solution to the problem of subsurface flow and provided a complete rational solution of the problem which is known as Khosla's theory.

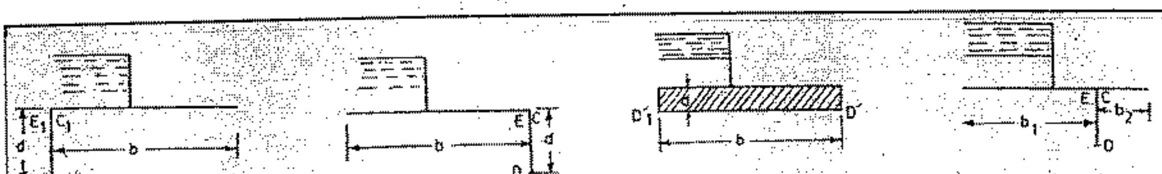
Khosla's Method of Independent Variables

In this method a composite weir or barrage section is split up into a number of simple standard form for which mathematical solutions have been obtained. The most useful standard forms are as follows.

(a) A straight horizontal floor of negligible thickness with a sheet pile at either end

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

(c) A straight horizontal floor depressed below the bed but without any vertical cutoff



$$\phi_{C_1} = 100 - \phi_E$$
$$\phi_{D_1} = 100 - \phi_D$$

(a)

$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$$
$$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right)$$

where $\lambda = \dfrac{1 + \sqrt{1 + \alpha^2}}{2}$

$\alpha = b/d$

(for respective cases)

(b)

$$\phi'_D = \frac{2}{3}(\phi_E - \phi_D) + \frac{3}{\alpha^2}$$
$$\phi'_{D_1} = 100 - \phi'_D$$

(c)

$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right)$$
$$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$$
$$\phi_C = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda}\right)$$

where $\lambda = \dfrac{\sqrt{1 + \alpha_1^2} + \sqrt{1 + \alpha_2^2}}{2}$

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

$\alpha_1 = b_1/d$

and $\alpha_2 = b_2/d$

(d)

These cases have been analyzed by Khosla and expressions have been derived for determining residual seepage head or uplift pressure head at the key points and the exit gradient.

The key points are the junction points of pile and floor, the bottom point of pile and the bottom corners of depressed floor.

The percentage pressure at these key points for the simple form into which the complex profile has been broken is valid for the complex profile itself if corrected for

(1) Correction for mutual interference of piles. (2) Correction for thickness of floor

(3) Correction for the slope of the floor.

(1) Correction for mutual interference of pile

The correction for the mutual interference of piles is given by

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

where, C = correction to be applied as percentage of head

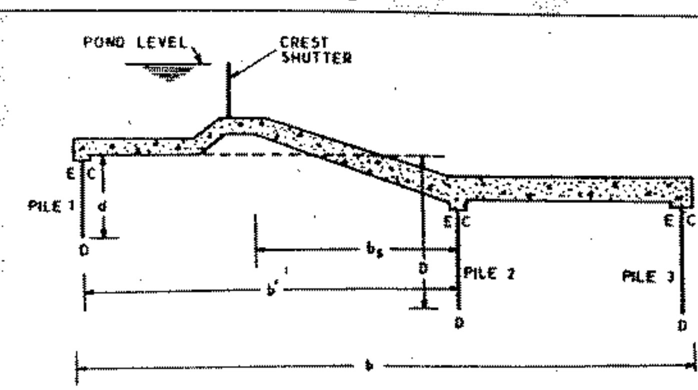
b' = distance between the piles; b = total length of floor;

D = depth of pile whose effect is reqd. to be determined on neighbour pile of depth d

d = depth of pile on which the effect of pile of depth D is required to be determined.

Both D and d are to be measured below the level at which the effect of interference is required

The correction is additive for points in the rear or backwater and subtractive for point forward in the direction of flow. Thus as shown in figure.

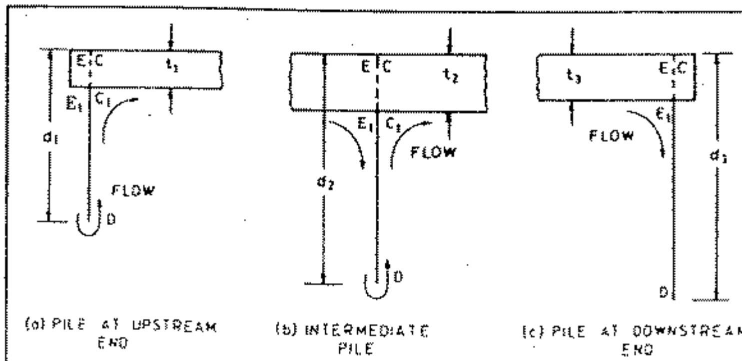


Correction for mutual interference of piles.

(2) Correction for thickness of floor

In the standard forms with vertical cutoffs the thickness of the floor is assumed to be negligible. Pressures at the junction points E and C pertain to the level at the top of the floor whereas the actual junction is with the bottom of the floor.

- The pressure at the actual junction points E_1 and C_1 are interpolated by assuming a linear variation from the hypothetical point E to D and also from D to C.



Correction for floor thickness

- For different locations of piles the corrections to be applied are as follows.

For pile at the u/s end

Correction for $C_1 = \frac{\phi_D - \phi_C}{d_1} \times t_1$ (additive)

where, ϕ = ratio of the residual seepage head or the uplift pressure head at any point and the total seepage head H.

ϕ is usually expressed in percentage.

$\therefore$ Pressure at C_1, $\phi_{C_1} = \phi_C + \frac{\phi_D - \phi_C}{d_1} \times t_1$

where, t_1 = Floor thickness ; d_1 = Depth of pile.

For intermediate pile ,

Correction for $E_1 = \frac{\phi_E - \phi_D}{d_2} \times t_2$ (subtractive)

$\therefore$ Pressure at E_1, $\phi_{E_1} = \phi_E - \frac{\phi_E - \phi_D}{d_2} \times t_2$

Correction for $C_1 = \frac{\phi_D - \phi_C}{d_2} \times t_2$ (additive)

$\therefore$ Pressure at C_1, $\phi_{C_1} = \phi_C + \frac{\phi_D - \phi_C}{d_2} \times t_2$

where, t_2 = Floor thickness ; d_2 = Depth of pile.

For pile at the d/s end

Correction for $E_1 = \frac{\phi_E - \phi_D}{d_3} \times t_3$ (Subtractive)

$\therefore$ Pressure at E_1, $\phi_{E_1} = \phi_E - \frac{\phi_E - \phi_D}{d_3} \times t_3$

where, t_3 = floor thickness ; d_3 = depth of floor.

(3) Correction for the slope of the floor

Correction is taken positive for downslopes and negative for upslopes in the direction of flow. The correction factor given in the above table is to be multiplied by the horizontal

Slope (*Vertical to Horizontal*)	*Correction* *% of pressure*
1 in 1	11·2
1 in 2	6·5
1 in 3	4·5
1 in 4	3·3
1 in 5	2·8
1 in 6	2·5
1 in 7	2·3
1 in 8	2·0

length of the slope and divided by the distance between the two pile lines between which the sloping floor is located

It is applicable only to the key points of the pile line fixed at the start or the end of the slopes.

Exit gradient

It may be defined as the hydraulic or pressure gradient of subsoil flow at the d/s or the exit end of the floor. For a standard form consisting a floor of length b, a vertical cutoff depth d at its d/s end. Khosla derived an expression for the exit gradient G_E which is given as

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

where, H = The total seepage head;

$$\boxed{\lambda = \frac{1+\sqrt{1+\alpha^2}}{2};\quad \alpha = b/d}$$

It is seen from equation (i) that if no cutoff is provided at the d/s end of the floor i.e., d = 0, the exit gradient G_E is infinite.

Note : The sheet pile must be taken upto the level of possible deepest scour below the bed of the river using the Lacey's scour depth formula in alluvial soils given by

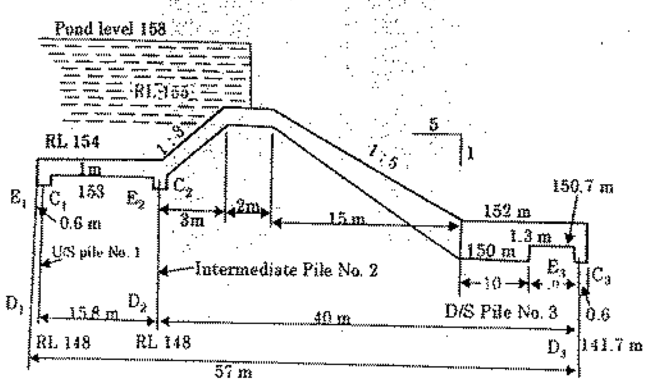
$$R = 1.35\left(\frac{q^2}{f}\right)^{1/3};\quad q = \text{discharge per unit weir length};\ f = \text{silt factor}$$

For additional safety, we consider scour depth as 1.25 - 2 times R.

However, in conservative design an intermediate pile line is provided to serve as an additional factor of safety.

Example 1

Determine the percentage pressures at various key points in figure. Also determine the exit gradient and plot the hydraulic gradient line for pond level on u/s and no flow on d/s.



Sol. (i) For u/s pile line no. 1 $\quad b = 57$ m; $d = 154 - 148 = 6$ m

$$\alpha = \frac{b}{d} = \frac{57}{6} = 9.5$$

From the cases related to Khosla's weir profile

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

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

$$\therefore \quad \lambda = \frac{1+\sqrt{1+9.5^2}}{2} = 5.276$$

$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{5.276-2}{5.276}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{3.276}{5.276}\right) = 0.287 = 28.7\%$$

$$\therefore \quad \phi_{C_1} = 100 - \phi_E = 100 - 28.7 = 71.3\%$$

and $$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right)$$

where $$\phi_0 = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{5.276-1}{5.276}\right) = 0.199 = 19.9\%$$

$$\therefore \quad \phi_{D_1} = 100 - 19.9 = 80.1\%$$

Correction for ϕ_{C_1}

(a)For mutual interference of piles.

ϕ_{C_1} is affected by intermediate pile no. 2

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

where, D = depth of pile no. 2 = 153 – 148 = 5 m

d = depth of pile no. 1 ; b' = distance between two piles = 15.8 m

b = Total floor length = 57 m

$$\text{Correction} = 19\sqrt{\frac{5}{15.8}}\left(\frac{5+5}{57}\right) = 1.875\ \%$$

Point C_1 is in the rear direction of flow,

So, correction is positive

$\therefore$ Correction due to pile interference on C_1 = 1.88% (positive)

(b) *For thickness of floor*

Pressure at C_1 will be more than C_1' at the direction of flow is from C_1 to C_1'

Hence, Correction will be positive

$$\text{Correction due to thick weir of floor} = \left(\frac{80.1-71.3}{154-148}\right)\times(154-153) = 1.467\ \text{(positive)}$$

(c) *For the slope of the floor*

Point C_1 is situated neither at the start nor at the end of the slope.

So correction at C_1 will be nil.

$\therefore$ Corrected $\phi_{C_1} = 71.3 + 1.875 + 1.467 + 0 = 74.64\%$

$$\phi_{D_1} = 80.1\%$$

(2) For intermediate pile no. 2

$$\phi_{E_2} = \phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right)$$

$$\phi_{C_2} = \phi_C = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda}\right)$$

$$\phi_{D_2} = \phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$$

where,
$$\lambda = \left(\sqrt{1+\alpha_1^2} + \sqrt{1+\alpha_2^2}\right)/2$$

$$\lambda_1 = \left(\sqrt{1+\alpha_1^2} - \sqrt{1+\alpha_2^2}\right)/2$$

$$\alpha_1 = \frac{b_1}{d}, \quad \alpha_2 = \frac{b_2}{d}$$

d = depth of intermediate pile =154 – 148 = 6 m

b_1 = Floor length u/s of intermediate pile = 15.8 + 0.6 = 16.4 m

b_2 = Floor length d/s of intermediate pile = 40 + 0.6 = 40.6 m

$$\alpha_1 = \frac{16.4}{6} = 2.73$$

$$\alpha_2 = \frac{40.6}{6} = 6.77$$

$$\lambda = \frac{\sqrt{1+2.73^2} + \sqrt{1+6.77^2}}{2} = \frac{2.91+6.84}{2} = 4.875$$

$$\lambda_1 = \frac{\sqrt{1+2.73^3} - \sqrt{1+6.77^2}}{2} = \frac{2.91-6.84}{2} = -1.965$$

$$\therefore \quad \phi_{E_2} = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{-1.965-1}{4.875}\right)$$

$$= 0.708 = 70.8\%$$

$$\phi_{C_2} = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{-1.965+1}{4.875}\right)$$

$$= 0.563 = 56.3\%$$

$$\phi_{D_2} = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_2}{\lambda}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{-1.965}{4.875}\right) = 0.632 = 63.2\%$$

Correction for ϕ_{E_2}

(a) *For mutual interference of piles*

Pile no. 1 will affect the pressure at E_2 and since E_2 is in the direction of flow, the correction will be negative.

$$\therefore \quad \text{Correction} = 19\sqrt{\frac{D}{b'}}\left[\frac{d+D}{d}\right]$$

where, D = depth of pile no. 1 = 153 – 148 = 5 m ; d = depth of pile no. 2 = 153–148 = 5 m

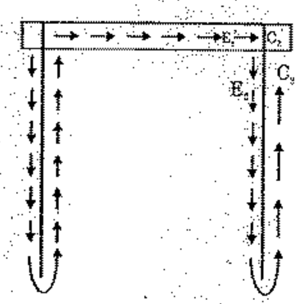
b' = Distance between the two piles = 15.8 m ; b = Total floor length = 57 m

$$\text{Correction} = 19\sqrt{\frac{5}{15.8}}\left(\frac{5+5}{5}\right)57 = 1.875\% \text{ (negative)}$$

(b) *For thickness of floor*

Pressure at E_2 will be less than the pressure at E_2'. Hence the correction will be negative

$$\text{Correction} = \frac{70.8-63.2}{154-148} \times (154 - 153) = 1.267\ \% \text{ (negative)}$$



(c) *For the slope of the floor*

As the point E_2 is situated neither at the start of a slope nor at the end of a slope.

$$\therefore \quad \text{Correction } \phi_{E_2} = 70.8 - 1.875 - 1.267 = 67.66\%$$

Correction for ϕ_{C_2}

(a) *For mutual interference of pile*

Pressure at C_2 is affected by pile No. 3 and since C_2 is in the direction upflow, correction is positive.

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

where, D = depth of pile no. 3, the effect of which is considered below the level at which interference is decided = 153 – 141.7 = 11.3 m

d = depth of pile no. 2 = 153 – 148 = 5m ;

b' = Distance between the pile no. 2 and pile no. 3 = 40 m ; p = Total floor length = 57 m

$$\text{Correction} = 19\sqrt{\frac{113}{40}}\left(\frac{11.3+5}{57}\right) = 2.84\% \text{ (positive)}$$

(b) For thickness of the floor.

Pressure at C_2 will more than that at $C_2{}'$ correction will be positive.

$$\text{Correction} = \frac{70.8-63.2}{154-148} \times (154 - 153) = 1.267\% \text{ (positive)}$$

(c) *For the slope of the floor*

Point C_2 is situated at the start of a up slope of 3 : 1. So correction will be negative.

Correction factor for 3 : 1 slope from the table = 4.5

Horizontal length of the slope = 3 m

Distance between two pile lines between which the sloping from is located = 40 m

$$\therefore \text{ Actual correction} = 4.5 \times \frac{3}{40} = 0.3375\% \text{ (negative)}$$

Corrected $\phi_{C_2} = 56.3 + 2.84 + 1.267 - 0.3375 = 60.07\%$

(c) For the slope of the floor

As the point E_2 is situated neither at the start of a slope nor at the end of a slope.

$\therefore$ Corrected $\phi_{C_2} = 70.8 - 1.875 - 1.267 = 67.66\%$

Correction for ϕ_{C_2}

(a) *For mutual interference of pile*

pressure at C_2 is affected by pile no. 3 and since C_2 is in the direction of flow, correction is positive.

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

where, D = depth of pile no. 3 the effect of which is corr. below the level at which interference is derived.

d = depth of pile no. 2 = 153 − 148 = 5 m

b' = Distance between the pile no. 2 and pile no. 3 = 40 m ;

b = Total floor length = 57 m

$$\text{correction} = 19\sqrt{\frac{11.3}{40}}\left(\frac{11.3+5}{57}\right) = 2.84\% \text{ (positive)}$$

b) *For thickness of the floor*

Pressure at C_2 will more than that at $C_2{}'$ correction will be positive

$$\text{Correction} = \frac{70.8-63.2}{154-148} \times (154 - 153) = 1.267\% \text{ (positive)}$$

(c) *For the slope of the floor*

Point C_2 is situated at the start of at up slope of 3 : 1. So correction will be negative.

Correction factor for 3 : 1 slope from the table = 4.5

Horizontal length of the slope = 3 m

Distance between two pile lines between which the sloping floor is located = 40 m

$\therefore$ Actual correction $= 4.5 \times \frac{3}{40} = 0.3375\%$ (negative)

Corrected $\phi_{C_2} = 56.3 + 2.84 + 1.267 - 0.3375 = 60.07\%$

(3) For d/s pile no. 3

$$d = 152 - 141.7 = 10.3 \text{ m} \; ; \; b = 57 \text{ m}$$

$$\alpha = \frac{b}{d} = \frac{57}{10.3} = 5.534$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = \frac{1+\sqrt{1+3.0.62}}{2} = 0.37045 = 37.04\%$$

$$\phi_{D_3} = \phi_D = \frac{1}{\pi} \cos^{-1}\left(\frac{h-1}{\lambda}\right) = 0.2540 = 25.40\%$$

Correction for ϕ_{E_3}

(a) *For mutual interference of piles*

Pressure at point E_3 is affected by pile no.2

and since E_3 is the forward direction, correction wil be negative.

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

where, D = depth of pile no. 2 = 150.7 – 148.0 = 2.7 m ;

d = depth of pile no. 3 = 150.7 – 141.7 = 9m

b' = Distance between piles = 40 m ; b = Total floor length = 57 m

$$\text{Correction} = 19 \text{ x}\sqrt{\frac{2.7}{40}}\left(\frac{9+2.7}{57}\right) = 1.013\% \text{ (negative)}$$

(b) *For thickness of the floor*

Pressure at E_3 will be less then pressure at E_3'

Correction will be negative

$$\text{Correction} = \frac{37.04-25.40}{152-141.7} \times 1.3 = 1.47 \text{ (negative)}$$

(c) *For slope of the floor*

Point E_3 is situated neither at the start nor at the end of any slope

So, Correction is zero

$$\text{Corrected } \phi_{E_3} = 37.04 - 1.013 - 1.47 + 0 = 34.56\%$$

upstream pile no.1	Intermediate pile no.2	Downstream pile no.3
$\phi_{E_1} = 100\%$	$\phi_{E_2} = 67.66\%$	$\phi_{E_3} = 34.56\%$
$\phi_{D_1} = 80.1\%$	$\phi_{D_2} = 63.2\%$	$\phi_{D_3} = 25.40\%$
$\phi_{C_1} = 74.64\%$	$\phi_{C_2} = 60.07\%$	$\phi_{C_3} = 0$

Example 2

A river discharges 1000m^3/sec of water at high flood level of RL = 103. A weir is constructed for flow diversion with a crest length of 225 m and total length of concrete floors as 40 m. The weir has to sustain the under seepage at a maximum static head of 2.4 m. The silt factor and the safe exit gradient for the river bed material are 1.1 and 1/6 respectively. Determine the depth of cut-off required at the d/s end of the concrete floor. Take the level of d/s concrete floor as RL = 100. Check for exit gradient.

Safe exit gradient = $\frac{1}{6}$; Length of concrete floors, b = 40 m ; Max. static head = 2.4 m ;

HFL u/s = 103 m ; RL of down stream floor = 100 m

Sol. Given,

Discharge @ high flood level, Q= 1000 m^3/s ; Weir length, L = 255m ; Silt factor, f = 1.1

$\therefore$ Discharge per unit weir length, $q = \frac{Q}{L} = 3.92$ m^3/s

By Lacey equation, the normal depth of scour is

$$R = 1.35\left(\frac{q^2}{f}\right)^{\frac{1}{3}} = 1.35\left[\frac{(3.92)^2}{1.1}\right]^{\frac{1}{3}} = 3.25 \text{ m}$$

Where, q = discharge intensity ; f = silt factor

Assuming down stream cut off to be 1.5 R below the down stream water level

1.5 R = 1.5 × 3.25 = 4.877 m

Now, u/s water level = HFL at u/s= 103 m

H = Maximum static head causing seepage = 2.4 m

$\therefore$ D/s water level = HFL @ u/s – H = 103 – 2.4 = 100.6m

Hence, RL of bottom of d/s cut off = 100.6 – 4.877 = 95.723m

RL of d/s floor = 100m

$\therefore$ Depth of d/s cut off, d = 100 – 95.723 = 4.277m

Exit gradient at d/s end for a vertical cut off depth d and floor length b

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

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

and $$\alpha = \frac{b}{d}$$

$\therefore$ $$\alpha = \frac{40}{4.277} = 9.352$$

$$\lambda = \frac{1+\sqrt{1+(9.352)^2}}{2} = 5.203$$

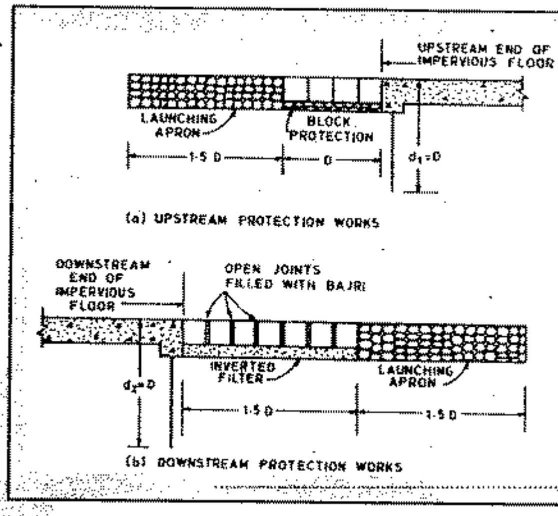
$\therefore$ $$G_E = \frac{2.4}{4.277} \times \frac{1}{3.14 \times \sqrt{5.203}} = 0.07835 = \frac{1}{12.76} < \frac{1}{6} \qquad \text{OK}$$

DESIGN OF PROTECTION WORKS AT THE U/S AND D/S ENDS OF THE IMPERVIOUS FLOOR

Protection works are provided at both the u/s and the d/s ends of the impervious floor in order to further safeguard the impervious floor against the failure due to piping
These protection works consist of

(1) Inverted filter; (2) Block protection (3) Launching apron or pervious apron

(1) Inverted filter



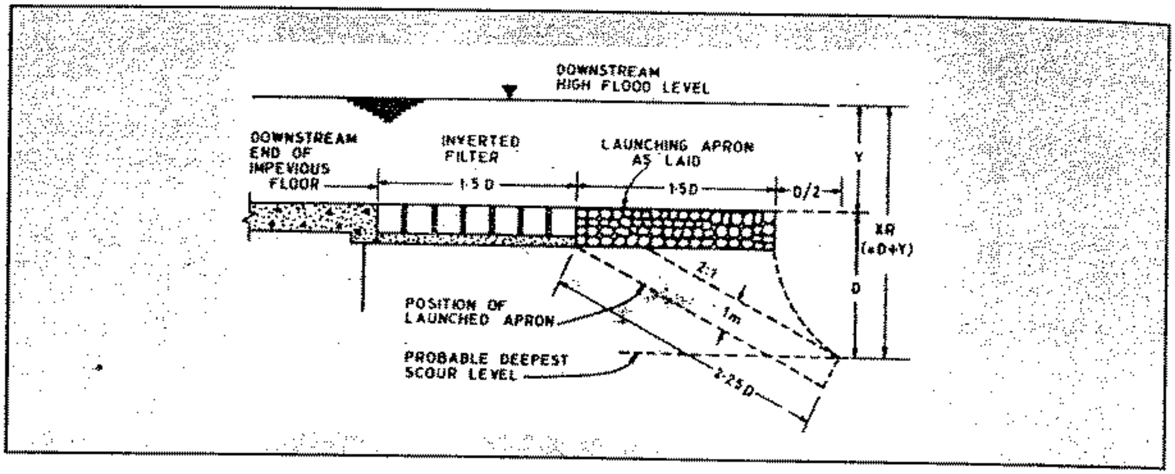
- An inverted filter consists of layers of materials of increasing permeability from bottom to top.
- An inverted filter reduces the possibility of piping as it allows free flow of seepage water through itself without allowing the foundation soils to be lifted upwards.
- The thickness of the inverted filter varies from 0.5 to 1.25 m. To prevent the filter material from dislocation by surface flow they are loaded with large size stone or concrete block.
- The blocks are usually 0.9 to 1.2 m thick and are placed with open joints filled with bajri (i.e., river sand) or filter material.
- The length of this inverted filter depends on the depth of scour D below the river bed or the impervious floor and it usually varies from 1.5 D to 2 D.
- The depth of inverted filter is kept equal to the depth of d/s launching apron.
- An inverted filter is provided immediately at the d/s end of the impervious floor

(2) Block protection

- A block protection consists of 0.6 to 1.0 m thick stone or concrete blocks laid on 0.4 to 0.6 m thick loosely packed stone.
- A block protection is provided immediately at the u/s end of the impervious floor.
- The length of the block protection is usually equal to scour depth below the river bed or the impervious floor at the u/s end.

(3) Launching apron

- A launching apron is an apron of loosely packed stones.
- The function of a launching apron is to protect the impervious floor and the pile from the scour holes progressing towards the floor and the pile.
- The protection is provided by a launching apron by forming a protective covering of stones over a certain slope below the bed of the river at which the apron is originally laid to the bottom of the deepest scour likely to occur.
- As shown in figure, when scour occurs, the new position attained by this apron is called *launched position* and the apron is then known as *launched apron*.



- The design of a launching apron depends on

(a) depth of scour (b) velocity of flow and (c) slope of the launched apron.

The velocity of flow affects the size of the stone because the stone used in the launching apron should be large enough so that it is not washed away during maximum flood.

RETROGRESSION OF D/S LEVELS

- The first effect produced by the construction of a weir across a river, is that the d/s bed of the river goes on eroding, consequently causing progressive lowering of the d/s levels.
- The progressive lowering of the d/s levels is called as *Retrogression of d/s levels* or *simply retrogression.*
- The basic cause for retrogression is the variation in the silt carrying capacity of the channel.
- As soon as a weir is constructed, the water starts ponding on its u/s side, causing the water surface slope to flatten for some distance behind the weir.
- This reduces the silt carrying capacity of the river in this reach, and consequently the silt deposition starts, and this leads to formation of *shoals* and islands on the u/s side.
- This clearer water posses over the weir and picks up sediment from its d/s bed, so as to fulfil the increased demand of the silt carrying capacity of the channel in the d/s, i.e. the sediment deficit caused on the u/s is made up by eroding extra sediment from the d/s.
- This leads to progressive lowering of the d/s bed levels.
- The process continues for few years till the river starts to region it original slope in the u/s portion by extending the afflux more and more u/s.
- The stage is gradually reached, when u/s pond absorbs no more silt.
- As off-taking channel takes comparatively silt free water, the sediment will go d/s, while the discharge going down will be below normal, and hence, the sediment taken shall be more than the hence, this will result consequently resulting in sediment deposition on the d/s river bed and long range recovery of d/s bed levels.
- In the design of weirs or barrages, we consider a retrogression of 0.5 m at high flood stage, and a higher retrogression varying linearly upto 2.0 m at lower discharges.

OBJECTIVE QUESTIONS

1. Consider the following statements related to undersluices provided in diversion weirs on permeable foundations :
   1. They are fully gate-controlled and have crest at the same level as the weir crest when no silt excluders are provided.
   2. They scour the silt deposited on the river bed in the pockets u/s of the canal head regulator.
   3. It is not necessary to provide end pile line on the d/s end of the undersluice floor.
   4. The discharge capacity of the undersluice is 10-15% of the maximum flood or two times the maximum discharge of the offtaking canal or maximum winter discharge, whichever is the highest.

   Which of these statements is/are correct?

   (a) 1 (b) 2 and 4 only
   (c) 2, 3 and 4 (d) 3 and 4 only

2. Match **List-I** (Theory) with **List-II** (Propounded by) and select the correct answer using the codes given below the lists:

List-I	List-II
A. Exit gradient	**1.** G. Lacey
B. Alluvial canal	**2.** L.K. Sherman
C. Unit hydrograph	**3.** A.N. Khosla
D. Boundary layer	**4.** C. Inglis
	5. T.V. Karman
	6. L. Prandtl

Codes:

	A	B	C	D
(a)	1	3	2	6
(b)	6	2	3	5
(c)	3	1	2	6
(d)	3	1	4	2

3. In a river, silt excluder and silt ejector are constructed
   (a) at a location after the head regulator and at the head of the canal, respectively
   (b) at the head of the canal and at a location after the head regulator, respectively
   (c) at the same location
   (d) at specific locations depending upon diverse factors and their locations do not follow a set pattern

4. The following parameters relate to the design of weirs on permeable foundations:

   1. Scour depth 2. Exit gradient
   3. Uplift pressure 4. Unbalanced head

Design of the d/s end pile of the weir depends upon

(a) 1 and 2 (b) 1 and 4 (c) 2 and 3 (d) 3 and 4

5. While considering weir designs on permeable soils, the correction for mutual interference of sheet piles is NOT applicable on an intermediate pile if the outer pile
   (a) goes deeper than the intermediate pile and is farther from the intermediate pile by more than twice its own length.
   (b) goes only just as deep as the intermediate pile and is within a distance of one and a half times its own length.
   (c) does not go as deep as the intermediate pile, no matter what the horizontal distance between them is.
   (d) is safe against deleterious exit gradient.

6. Consider the following statements:
   1. In designing a hydraulic structure in permeable foundation by Khosla's theory, the slope correction is applicable to piles located at the ends of the sloped floor only in a structure with one or more sloped floors.
   2. Khosla's theory can be correctly applied in alluvium of finite depth.
   3. Length of flow has lesser effect on exit gradient than the depth of piles.
   4. Intercepts between hydraulic gradient line and free water surface on the glacis and horizontal floor d/s is the unbalanced uplift in a structure.

   Which of these statements related to the design of hydraulic structure are correct?

   (a) 1, 2 and 3 (b) 1, 3 and 4 (c) 2, 3 and 4 (d) 1, 2 and 4

7. In a barrage on pervious foundation, sheet piles are provided both u/s and d/s of the barrage to reduce uplift pressure and to prevent piping. Which one of the following statements is true in this regard?
   (a) Compared to d/s sheet pile, the u/s sheet pile is more effective in reducing uplift and piping
   (b) Compared to u/s sheet pile, the d/s sheet pile is more effective in reducing uplift and piping
   (c) D/s sheet pile is more effective in reducing uplift while the u/s sheet pile is more effective in reducing piping
   (d) U/s sheet pile is more effective in reducing uplift while the d/s sheet pile is more effective in reducing piping

8. Consider the following statements:

   The d/s impervious floor of concrete for a barrage has ruptured. This can be due to
   1. insufficient length of u/s impervious floor
   2. insufficient length of d/s impervious floor
   3. insufficient depth of d/s pile
   4. choking of inverted filter

   Which of these statements are correct?

   (a) 1 and 2 (b) 1 and 4 (c) 2, 3 and 4 (d) 1, 2, 3 and 4 **3**

9. At a certain point in the floor of weir, the uplift pressure head due to seepage is 4.5 m. If the relative density of concrete is 2.5, the minimum thickness of floor required at this point to counteract the uplift pressure is

   (a) 1 m (b) 2 m (c) 3 m (d) 4 m

10. Which one of the following is the purpose of providing the d/s sheet pile in a barrage?
   (a) To control failure due to piping by high value of exit gradient
   (b) To control failure due to scour
   (c) To stop failure due to sliding
   (d) To stop failure due to uplift pressure

11. Consider the following statements:
   1. Giving equal weightages to horizontal and vertical creeps for design of weir foundations is one of the drawbacks of Kennedy's theory.
   2. Khosla's theory of design of foundations for weirs is based on potential theory.
   3. Piping problem can be reduced by increasing the length of floor.
   4. In Lane's weighted creep theory, horizontal creep is given less weightage compared to vertical creep.

   Which of these statements is/are correct?

   (a) 1 (b) 2, 3 and 4 (c) 2 and 4 only (d) 3 and 4 only

12. Consider the following statements:

   The function of a launching apron at the end of d/s impervious apron of a weir is
   1. to protect the d/s pile from the scour holes progressing in the u/s direction.
   2. when the scour holes are formed, to provide for the stones of the falling apron to settle down in the holes and cover them.
   3. to provide relief to the uplift pressure at the d/s end.
   4. to provide extra length for proper formation of hydraulic jump.

   Which of these statements is/are correct?

   (a) 1, 2, 3 and 4 (b) 3 only (c) 2 only (d) 1 and 2 only

13. While designing a hydraulic structure, the piezometric head at bottom of the floor is computed as 10 m. The datum is 3 m below floor bottom. The assumed standing water depth above the floor is 2 m. The specific gravity of the floor material is 2.5. The floor thickness should be

   (a) 2.00 m (b) 3.33 m (c) 4.40 m (d) 6.00 m

14. Uplift pressure at points E and D (figure A) of a straight horizontal floor of negligible thickness with a sheet pile at d/s end are 28% and 20%, respectively. If the sheet pile is at u/s end of the floor (figure-B), the uplift pressures at points D_1 and C_1 are

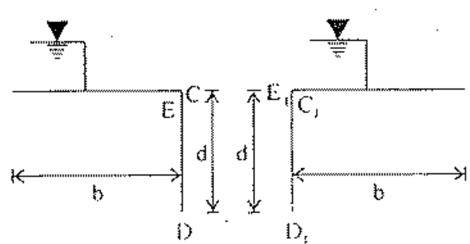


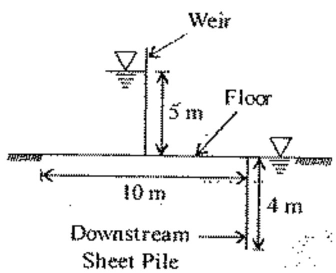
Figure-A Figure-B

   (a) 68% and 60% respectively (b) 80 % and 72% respectively
   (c) 88% and 70% respectively (d) 100% and zero respectively

15. A launching apron is to be designed at d/s of a weir for discharge intensity of 6.5 $m^3/s/m$. For the design of launching aprons the scour depth is taken two times of Lacey scour depth. The silt factor of the bed material is unity. If the tailwater depth is 4.4m, the length of launching apron in the launched position is

(a) $\sqrt{5}$ m (b) 4.7 m (c) 5 m (d) $5\sqrt{5}$ m

16. A weir on a permeable foundation with d/s sheet pile is shown in the figure below. The exit gradient as per Khosla's method is



(a) 1 in 6.0 (b) 1 in 5.0 (c) 1 in 3.4 (d) 1 in 2.5

17. In a barrage project, a divide wall is provided to :

(a) separate the lower crest 'undersluice side' from the higher crest 'weir side'
(b) separate the higher crest 'undersluice side' from the lower crest 'weir side'
(c) keep the cross currents away from the barriage body
(d) server none of the above purposes.

18. A canal headworks has nothing to do with a :

(a) weir (b) guide bank (c) head regulator (d) safety ladder.

19. Head sluices are the gate controlled openings, in :

(a) the entire length of the barrage
(b) the under sluice length of the barrage
(c) the regulator of the main off taking canal
(d) none of them

20. The silt exclusion device, constructed on the bed of the canal, taking off from a headwork, is called :

(a) silt excluder (b) silt ejector (c) both (a) and (b) (d) none of them

21. The tunnel openings provided in front of a canal head regulator at a Diversion headworks

(a) discharge sedimented water into the canal
(b) discharge sediment load into the under sluices, from where it ejects out to the d/s river
(c) discharge clear water into the canal
(d) none of them

22. Canal headworks in the upper rocky reaches of the rivers are uncommon, because :

(a) more drops are required on the resulting canal system
(b) costly head regulator is required
(c) more cross drainage works are required on the resulting canal network
(d) all of the above

23. The hydraulic jump that develops usually in barrages and canal head regulators, is of the type

(a) strong jump (b) steady jump
(c) oscillating and weak jump (d) undular jump

24. The formation of a hydraulic jump on a sloping glacis, as compared to that on a horizontal floor, is always :

(a) more definite and more efficient (b) more definite and less efficient
(c) less definite and more efficient (d) less definite and less efficient.

25. While designing the d/s floor thickness of a weir, the ordinate of the uplift pressure at a point is 2.8 m. If the relative density of concrete is taken to be 2.4, then the minimum thickness of the floor to be provided for resisting uplift pressure, without accounting safety factor, is :

(a) 1.16 m (b) 2 m (c) 0.8 m (d) none of them

26. Point out the incorrect statement in relation to the design of weirs and barrages :

(a) Bligh's safe hydraulic gradient is the same as the Khosla's safe exit gradient
(b) the first streamline below a barrage section follows the bottom profile of the section
(c) equipotential lines are the lines joining the points of equal residual seepage head
(d) none of them.

27. Retrogression is :

(a) the back water effect of a weir
(b) the raising of the river bed u/s of the weir, during initial years of its construction
(c) the lowering of the river bed d/s of the weir, during initial years of its construction
(d) none of them.

28. Critical exit gradient, as enunciated in Khosla's theory of design of weirs and barrages on pervious foundations, is :

(a) the left out pressure in the seeping water at the d/s end point, where it emerges out on the river bed
(b) the rate of loss of pressure of the seeping water at the d/s emerging point, which is just enough to lift the soil grains at that point
(c) the actual pressure gradient of the seepage, at the down stream emerging point
(d) none of them

29. The back water effect of a weir is best called :

(a) retrogression (b) afflux (c) back water curve (d) none of them

30. A breast wall is usually provided :

(a) in the weir section (b) in the under sluice section
(c) in the main canal section (d) in the head regulator section

31. Just d/s of the pucca concrete floor of a barrage section, an inverted filter covered by cement concrete blocks is laid in a length of about 1.5 to 2 D, where D is :

(a) the Lacey's normal scour depth
(b) 1.5 times the Lacey's normal scour depth
(c) 1.5 times the Lacey's scour depth minus the d/s water depth
(d) none of them

32. The safety against any possible scour, on u/s or d/s side of the pucca floor of a hydraulic structure, is usually ensured by laying :

(a) inverted filter (b) toe wall (c) rock toe (d) stone apron.

33. For the head regulator, the most severe condition of uplift pressure on the floor occurs when

(a) the flow in the river is at flood level and canal is running at full supply depth
(b) the canal runs dry and the river flow is at high flood level
(c) the canal runs at full supply depth and the river flow is at pond level
(d) the canal runs dry and the river flow is at pond level.

ANSWERS

1. (b)	2. (c)	3. (b)	4. (a)	5. (a)	6. (d)
7. (d)	8. (a)	9. (c)	10. (a)	11. (b)	12. (c)
13. (a)	14. (b)	15. (c)	16. (c)	17. (a)	18. (d)
19. (c)	20. (b)	21. (b)	22. (d)	23. (c)	24. (b)
25. (b)	26. (a)	27. (c)	28. (b)	29. (b)	30. (d)
31. (c)	32. (d)	33. (b)			

River Engineering

CHAPTER 8

River Engineering

INTRODUCTION

Rivers take off from mountains, flow from the mountainous plain terrains, and finally join the oceans. They are formed along more or less defined channels, drain away the land water obtained by precipitation and snow melting in high altitudes, and discharge the unutilised waters back into the sea, thus completing the hydrological cycle. The rivers not only carry this huge amount of water but also carry a tremendous amount of silt and sediment which is washed down from the catchment area or gets eroded from their beds and banks. So, it very important to know the behaviour of the river, control and train the river in order to stabilize the river channel.

TYPES OF RIVERS AND THEIR CHARACTERISTICS

(1) Classification of Rivers on the Basis of the Topography of the River Basin

Depending upon the topography of the basin, the river reaches can be classified as:

(i) Rivers in hills (Upper reaches) (ii) Rivers in alluvial flood plains (Lower reaches)

(iii) Tidal rivers

(i) Rivers in Hills

The rivers generally take off from the mountains and flow through the hilly regions before traversing the plains. These upper reaches of the rivers may be termed as *Rivers in Hills*. They can be further sub-divided into:

(a) Incised or Rocky River stage (b) Boulder River stage

(a) Rocky Stage

- In this type, the flow channel is generally formed by the process of degradation.
- The sediment transported in this reach is different from the river bed material, since most of it comes from the catchment due to denundation and soil erosion.
- These river-reaches are highly steep with swift flow.
- The beds and banks of such rivers are less susceptible to erosion.

(b) Boulder River Stage

- The river bed in these reaches consists of a mixture of *boulders, gravels* and *alluvial sand-deposits* created by itself.

- In the boulder stage, *the river flows through wide shallow beds and interlaced channels and develops a straighter course.*
- During a flood, the boulders and gravels are transported d/s, but as the flood subsides, the material gets deposited in heaps.
- The water, then unable to shift these heaps, go round them, and the channel often wanders in new directions, often attacking the banks and consequently widening the bed.

(ii) Rivers in Alluvial Flood Plains

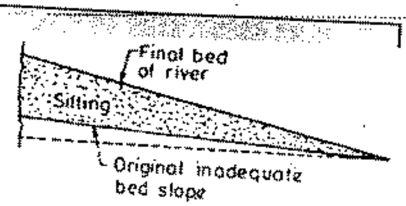
- The chief characteristics of these river reaches is the zig-zag fashion in which they flow. This zig-zag fashion is called **meandering**.
- They meander freely from one bank to another and carry sediment which is similar to bed material.
- Material gets eroded constantly from the outer edge of the bend and gets deposited either on the inner edge of the successive bends or between two successive bends to form a bar
- When once a straight moving river slightly deviates from its axis, the unbalance created goes on multiplying with constant erosion from the outer edge and deposition on the inner edge. If unchecked, the process continues, resulting in the formation of large meanders.

Rivers in Flood Plains can be Further Classified as:

(a) *Aggrading;* (b) *Degrading;* (c) *Stable;* (d) *Braided;* and (e) *Deltaic*

(a) Aggrading Type

An aggrading river is a silting river. Such a river increases its bed slope, which is called *building up of slope.* The silting may be due to various reasons,



Aggrading River

i) Heavy sediment load

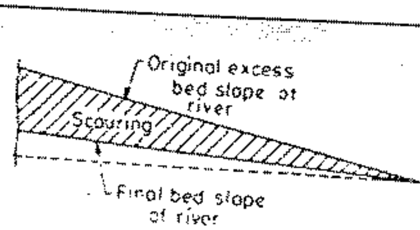
ii) Construction of as a dam or a weir;

iii) Sudden intrusion of sediment from a tributary.

This type of river usually has straight and wide reaches with shoals in the middle.

(b) Degrading Type

If the river bed is constantly getting scoured to reduce and dissipate available excess land slope then the river is known as a *degrading river.* It may be found either above a *cutoff* or below a dam or a weir or a barrage. etc.



(c) Stable Type

- River which does not change its alignment, slope & its regime significantly is called a stable river.
- Changes such as silting or scouring or advancement of delta into the sea may take place, but they are negligible and may fail to produce any change in the regime of the channel, except, that the river may shift within its *Khadirs*.

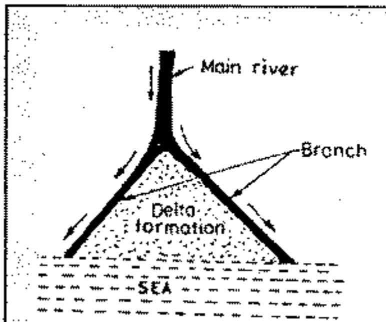
Note : *Khadirs are the extreme limits within which a river is known to have wandered*

(d) Braided Rivers

- When a river flow in two or more channels around alluvial islands, it is known as a braided river.
- The braided pattern in a river develops after local deposition of coarser material, which can not be transported under prevailing conditions of flow, and which subsequently grows into an island consisting of coarse as well as fine material.

(e) Deltaic Rivers

- A river before it joins the sea, gets divided into branches, thus forming a Δ shaped delta.
- As the river approaches the sea, its velocity is reduced, and consequently the channel gets silted and water level rises resulting is spills and eventual formations of new channels.
- These branches multiply in their number as the river approaches the sea.
- Delta is a stage, rather than a type of a river.



Delta formation.

(iii) Tidal Rivers

- The tail reaches of the rivers adjoining the oceans are affected by the tides in the ocean.
- The ocean water enters the river during the flood tide and goes out into the ocean during the ebb tide.
- The river undergoes periodical rise and fall in its water level, depending upon the nature of the tide.
- The distance upto which the tidal effect is experienced, depends upon various factors, such as the shape and configuration of the river, the tidal range etc.

(2) Classification of Rivers on the Basis of Flood Hydrographs

The rivers may be classified on the basis of stage and nature of their flood hydrograph, into the following two types:

(i) Flashy rivers; and (ii) Virgin rivers

(i) Flashy Rivers

If the flood rise and flood fall in a river is sudden, then it is called a *flashy river*. In flashy rivers, the flood flows, occur suddenly and rise and fall of water level is very quick.

(ii) Virgin Rivers

In arid zones, a river water may completely dry before it joins another river or the ocean. Such a river is called a *virgin river*. After flowing for a certain distance from its source, the water of such a river disappears due to high percolation or due to excessive evaporation.

e.g.,: Rivers in the States of Kutch and Rajasthan in India.

INDIAN RIVERS AND THEIR CLASSIFICATION

Indian rivers can be broadly classified into:

(1) Himalayan Rivers

- These rivers take off from Himalayas and flow through alluvial plains.
- They derive their water from rains during monsoon and winter, and from the melting snow during summer.
- These rivers are, almost perennial and can give dependable yields throughout the year.
- These rivers carry huge sediment
- The important Himalayan rivers are:

  Indus, Jhelum, Chenab, Ravi, Beas, Sutlej, Ganga, Kosi, Brahmaputra, etc.

(2) Non-Himalayan Rivers

- They are non-perennial rivers.
- They receive their major water supply only in rainy seasons
- They draw water from ground water as base flow for the rest of the year.
- They flow in Central and South India and takeoff from Aravalli, Vindhya and Satpura mountain ranges.

  ***e.g.,**: Chambal, Mahanadi, Cauvery, Godavari, Tapti, Narmada.*
- These rivers are much more stable than the Himalayan rivers, and pose lesser problems, as they flow through non-alluvial soils.

BEHAVIOUR OF RIVERS

The chief factor which is responsible for moulding the behaviour of rivers is the silt and sediment that flows in the river. The sediment carried by the river poses numerous problems, such as: *increasing of flood levels, silting of reservoirs, silting channels, meandering of rivers, splitting up of a river into a number of interlaced channels, etc.*

Special situation in which behaviour of river is analysed.

(1) Straight Reaches

In a straight reach of a river, the river cross-section is in the shape of a trough, with high velocity flow in the middle of the section. Since the velocity is higher in the middle, the water surface level will be lower in the middle and higher at the edges. Due to the existence of this transverse gradient from sides towards the centre, transverse rotary currents get developed. Straight reaches are very few in alluvial rivers.

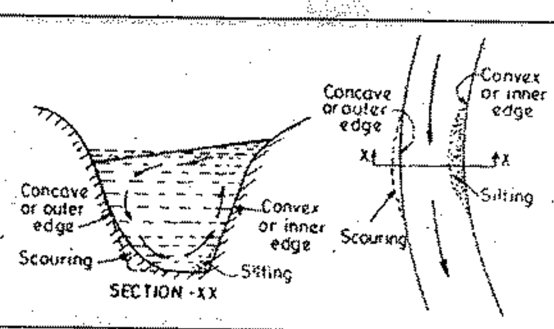
(2) Bends

- Every alluvial river tends to develop bends, which are characterised by scouring on the concave side and silting on the convex side. The silting and scouring in a bend may continue due to the action of the centrifugal force.
- When the flow moves round a bend, a centrifugal force is exerted upon the water, which results in the formation of transverse slope of water surface from the convex edge to the concave edge, creating greater pressure near the convex edge.

- To keep its own level, water tends to move from the convex-side towards the concave side. However, the topmost water surface movement is prevented by the centrifugal force.
- Moreover, towards the bottom, the velocities are much less than towards the top, and enough centrifugal force is not available to counteract the tendency of water at the top to move inwards. Hence, the water dives in, from the top at the concave end, and moves at the bottom towards the convex end.
- These rotary currents cause the erosion of concave edge and deposition on convex edge, forming shoals on the convex sides. When once a bend is formed, the flow tends to make the curvature larger and larger.

(3) Meanders

- When once a river deviates from its axial path and a curvature is developed the process moves d/s by building up shoals on the convex side by means of secondary currents. The formation of shoals on the convex side, results in further shifting of the outer bank by erosion on the concave side.
- Formation of successive bends of reverse order may lead to the formation of a complete S curve called *meander*.
- When consecutive curves of reverse order connected with short straight reaches called crossings are developed in a river reach, the river is stated to be a meandering river.



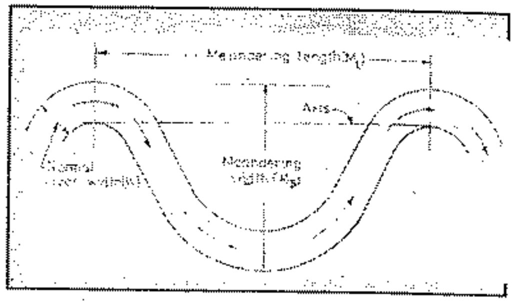
Causes of Meandering

- The latest and widely accepted theory behind meandering is based upon the extra turbulence generated by the excess of river sediment during floods. During floods, the river carries tremendous amount of silt charge. It has been established that when the silt charge is in excess of the quantity required for stability, the river starts building up its slope by depositing the silt on its bed i.e. the river reach becomes an aggrading type.
- The increase in river slope tends to increase the width of the channel, if the banks are not resistant. The banks are attacked by water and in the process, one bank is likely to be attacked slightly more than the other, causing a slight deviation of flow. This slight deviation from uniform axial flow, helps in moving more and more flow towards one bank than towards the other. The process continues causing more and more flow towards the former bank and forming shoals along the latter, thus increasing the curvature of flow, and finally, producing meanders.

  The concave bank goes on eroding and the convex bank goes on silting.

  There are four variables, which govern the meandering process.
- They are: *(a) Valley slope, (b) Silt grade and silt charge, (c) Discharge (d) Bed and side materials and their susceptibility to erosion.*

Meander Parameters



(i) ***Meander Length (M_L):*** It is the axial length of one meander, i.e. the tangential distance between the corresponding points of a meander.

(ii) ***Meander Belt (M_B):*** Distance between the outer edges of clockwise and anti-clockwise loops of the meander.

(iii) ***Meander Ratio:*** Ratio of meander belt to meander length, i.e. M_B/M_L.

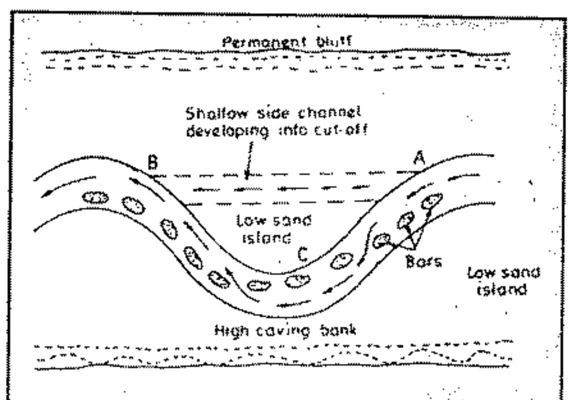
(iv) ***Tortuosity:*** Ratio of the length along the channel (*i.e.* actual length) to the direct axial length of the river reach.

(v) ***Crossings or Cross-overs:*** The short straight reaches of the river, connecting two consecutive clockwise and anti-clockwise loops, are called *crossings or cross-overs.*

(4) Cut off

- In an excessively meandering river, a particular bend may sometimes be abandoned by the formation of a straighter and a shorter channel.
- The process, whereby, this chord channel is developed or the chord channel itself, is termed as *cut-off.*
- A meander increases the river length but a cut-off reduces the river length.
- A cut-off is a natural phenomenon for counterbalancing the otherwise ever increasing length of the river course due to the development of meanders.

Development of a cut-off



- A meandering river flowing along the curved path has a shallow side channel besides its main curvilinear path. This might be the remains of its old course or may be created by floods spilling over the river bank.
- During high floods, excessive deepening of the pools occur, and is supplemented by the growth of bars at the inflections.
- Both these factors tend the water to flow more and more towards the side channel.
- When the flow starts reducing in the main channel, more and more silting occurs in the main channel, which further increases the flow in the side channel.
- The process continues and finally, a time may come when the entire water starts flowing from the full developed chord channel and the curvilinear path gets silted up.

Cut-off Ratio

- Ratio of the length of the bend to that of the chord *i.e.* ACB/AB is called the cut-off ratio.
- This ratio varies depending upon the characteristics of the river at site, such as the discharge, the river flood stage, surface fall, bed material and its suitability for the growth of protective grass and weeds, etc.

RIVER TRAINING AND ITS OBJECTIVES

- By the term *river training* we mean various measures adopted on a river to stabilize the river channel along a certain alignment with a certain cross section.
- These measures are required to be adopted because river in alluvial plains frequently alter their courses and cause damage to land and property adjacent to their banks.

The main objectives of river training are as follows :

(1) Provide a safe passage to flood discharges without overflowing to the bank for protection of cultivated or inhabited area.

(2) Prevent outflanking of structures like a bridge, weir or aqueduct constructed across the river.

(3) Protect the banks from erosion and improve the alignment by stabilizing the river channel.

(4) Deflect the river away from the bank which it might be attacking.

(5) Provide minimum depth of flow and a good course for navigation purposes.

(6) Ensure effective disposal of sediment load.

Classification of River Training

Depending upon the purpose for which a river training programme is undertaken, the river training works may be classified into the following three categories:

(i) High water training (or Training for discharge). (ii) Low water training (or Training for depth).
(iii) Mean water training (or Training for sediment).

(i) High Water Training

- High water training is undertaken with the primary purpose of flood control.
- It aims at providing sufficient river cross-section for the safe passage of maximum flood, and is concerned with making the adjoining area flood proof, by construction of dykes or levees, etc.

(ii) Low Water Training

- Low water training is undertaken with the primary purpose of providing sufficient water depth for navigation during lower water periods.
- It may be accomplished by concentrating and enhancing the flow in the desired channel by closing other channels.

(iii) Mean water training

- Mean water training aims at efficient disposal of suspended load and bed load, and thus, to preserve the channel in good shape.
- The mean water training is the most important type and forms the basis on which the former two are planned.

METHODS FOR RIVER TRAINING

The various methods adopted for river training are as follows :

(1) Marginal embankments/levees (2) Guide banks or Guide bunds

(3) Groynes or spurs (4) Pitching of banks and provision of launching aprons.

(5) Pitched islands (6) Miscellaneous methods

(1) Marginal Embankments

- Marginal embankments are earthen embankment constructed parallel to the river to protect the area on one side of it from flooding.
- It is also termed as bund, dyke as levees.
- Construction of levees on one or both side of a river to contain the floods within the leveed portion is the oldest and most common method of flood control.
- Marginal embankments are designed on the some principles as an earth dam.
- They may be constructed on both sides of the river or only on one side for some suitable length of the river where the river is passing through towns or cities on some important place.
- If the marginal embankments are likely to come in contact with high velocity flow then the waterside of the embankements should be provided with pitching protection.
- Launching apron may also be provided if the embankment is close to the main river channel.
- A levee or dyke is mainly used for flood protection by controlling the river and not by training the river.

The effect of confining the flood waters of a river between marginal embankments or levees is:

(a) To increase the rate at which the flood wave travels down the stream,

(b) To increase the water surface elevation at flood,

(c) To increase the maximum discharge at all points d/s due to reduction in river storage.

(d) To reduce the water surface slope of the stream on the u/s of the leveed portion, and

(e) To increase the velocity and the scouring action through the leveed section.

The following causes may lead to failure of levees, either singly or due to a combination of two or more factors:

(a) Overtopping

(b) River current may erode the river-side slope,

(c) Banks may cave in,

(d) Infiltration through the foundation,

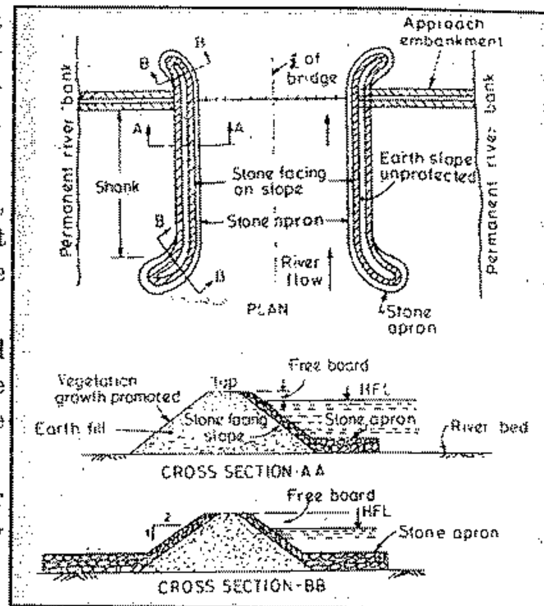
(e) Infiltration through the embankment,

(2) Guide Bank

- The guide banks are the earthen embankments provided to confine the flood water of alluvial rivers within reasonable waterway and provide a straight non-tortuous approach towards a structure constucted across the river.
- They prevent the river from changing its course and outflanking the work.
- By providing the guide banks an artifical gorge section is created for the river in which it can flow without causing abnormal velocities or scours, and hence the length of the work to be constructed is considerably reduced.
- If a weir, barrage or a bridge is constructed across a river, the river width is reduced and trained in such a fashion, as to ensure not only a safe and expeditious disposal of flood water but also to ensure a permanent reasonable width of the waterway for the river flow.
- It is uneconomical to span the entire width of the river and to expose the structure to vagaries of attack and deep scour. Hence, weir or a barrage or a bridge etc. is extended in a smaller width of

the river, and river water is trained to flow almost axially through this trough without out-flanking the structure. The river is normally trained for this purpose with the help of a pair of guide banks.

- The guide banks are generally provided in pairs, symmetrical in plan and may either be kept parallel or may diverge slightly up-stream of the works.
- Before the water enters into the trough formed between these two guide banks, the flow may have to be partially controlled and directed with the help of marginal bunds or by groynes or both.
- The portion of the river between the normal river banks and the guide banks is closed by ordinary embankments.



Guide bank may be classified according to their layout u/s of the work as

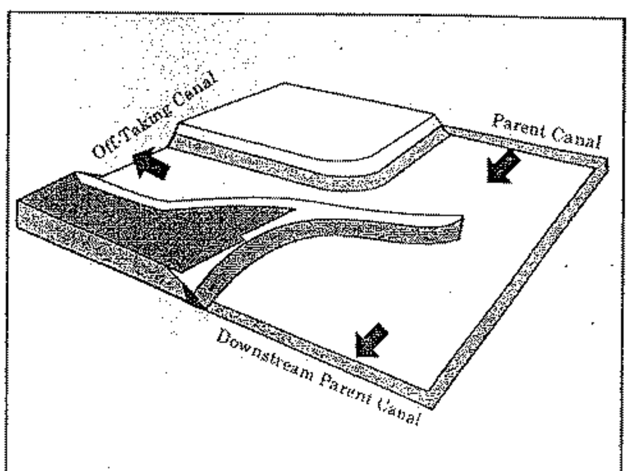
(i) Divergent guide banks (ii) Convergent guide banks, and (iii) Parallel guide banks.

Divergent guide banks tend to attract the flow towards them and are suitable where the river current has been oblique to the structure or where the work is located at one edge of *Khadir*.

Convergent guide banks have the disadvantage of excessive attack and heavy scour at the u/s head and shoaling all along the bank thus rendering the end bays ineffective.

- Due to these drawbacks, divergent and convergent guide banks with excessive splay are rarely used.
- In most of the cases parallel guide banks or slightly convergent guide banks with a splay of 1 in 20 or 1 in 40 are used unless local conditions necessitate the other types.

(3) Groynes or Spurs



- Groynes are the structures constructed transverse to the river flow extending from the bank into the river.
- Groynes may be aligned either perpendicular to the bank or at an angle pointing u/s or d/s.
- These are also known as spur dikes or transverse dikes, and constitute the most widely used river training works. They are constructed in order to protect the back from which they are extended, by deflecting the current away from the bank. Water is not able to take sharp embayment, hence the back gets protected for certain distance u/s and d/s of the groyne.
- The nose of the groyne is subjected to tremendous action of water and has to be heavily protected by pitching.

Functions of Groynes

(i) Training the river along a desired course by attracting, deflecting or repelling the flow in the river.

(ii) Creating a slack flow with the objective of silting up the area in the vicinity.

(iii) Protecting the river bank by keeping the flow away from it.

(iv) Contracting a wide river channel, usually for the improvement of depth for navigation.

Type of Groynes

Groynes may be classified according to different considerations as indicated below.

(i) According to methods and materials of construction. (ii) According to height of groyne.

(iii) According to function served. (iv) Special types

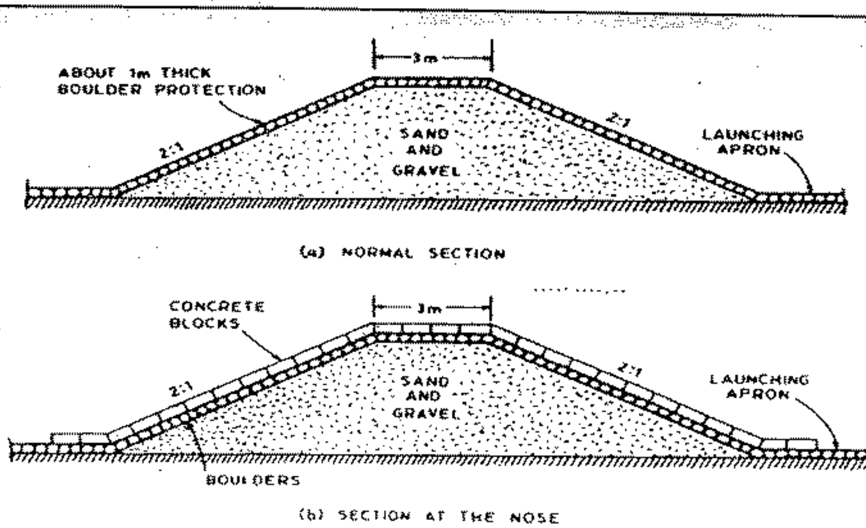
(i) Classification according to methods and materials of construction

According to methods and materials of construction the groynes may be classified as

(a) Impermeable groynes (b) Permeable groynes

(a) *Impermeable groynes*

- Impermeable or solid groynes do not permit appreciable flow of water through them.
- These groynes consist of either rockfill, or a core of sand, or sand and gravel, protected on the sides and top by a strong armour of stone pitching or concrete blocks.
- The section of groyne has side slope varying from 2:1 to 3:1 depending on the material used for its construction and front or head slope varying from 3:1 to 5:1.
- It needs protection since the head of the groyne is subjected to severe attack by the stream.
- These are called impermeable groynes because they do not allow significant flow through them.



Cross-sections of impermeable groynes.

(b) *Permeable groynes*

- The permeable groynes permit restricted flow of water through them.

- These groynes obstruct the flow and slacken it to cause deposition of sediment carried by the river.
- The permeable groynes are best suited for rivers which carry considerable amount of sediment in suspension.
- As the sediment accumulates between the groynes, the bank becomes more or less permanent so that there is no need to use any other material for its protection.

The common types of permeable groynes are: (i) Tree groynes (ii) Pile groynes.

(i) Tree groynes

- A tree groyne consists of a thick wire rope 25 mm diameter firmly anchored at one end to the bank and tied at the other end to a concrete block.
- The length of the wire rope is kept equal to the length of the groyne with some allowance being given for the sag due to the weight of the trees.
- Entire leafy trees with abundant branches are then fixed to the wire rope starting from the bank and proceeding outwards to the free end.
- For fixing the trees a hole is drilled through the stem of each tree, in which an iron ring is inserted and attached either directly or by another piece of wire rope to the main rope.

(ii) Pile groynes

- This type of groynes may be constructed out of timber or R.C.C. piles or sheet piles.
- In pile groynes, the piles constitute the main verticals which are driven down 6 to 9 m inside the river bed, 2.4 to 3 m apart and in at least 2 or 3 similar rows.
- The rows of the piles are not more than 1.2 to 1.8 m apart. Between the main piles there can be two intermediate piles embedded at least 1.2 m below the bed.
- Each row is closely inter-twined either by brushwood branches going in and around each pile, or by horizontal railing.
- The u/s row is braced to the d/s row by transverse and diagonals.
- Every other main pile of the rear row is strutted, the struts being embedded a minimum of 2.4 m below the bed.
- Between two rows, the space is filled with alternate layers of 1.8 m thick brushwood branches weighted by 0.6 m thick stones or sand bags.

Advantages of permeable groynes

The various advantages of permeable groynes are as indicated below:

(1) They require only temporary construction and hence they are relatively cheap.

(2) Small quantity of stone is required for the construction of permeable groynes and hence they are specially suitable where stone is scarce.

(3) Permeable groynes are more effective then solid or impermeable groynes in the regulation of river courses or in the protection of banks especially in a silt-laden river.

Disadvantages of permeable groynes

The permeable groynes are not strong enough to resist shocks and pressure from debris, floating ice and logs. They are, therefore, unsuitable for upper reaches or river.

(ii) Classification according to height of groynes

According to height of groynes these may be classified as :

(a) *Non-submerged groynes* (b) *Submerged groynes*

For training of river and protection of bank generally non-submerged groynes are used but in some cases submerged groynes, are also used.

e.g., In the case of deep river where depths are considerable submerged groynes may be used.

(iii) Classification according to function served

According to function served by the groynes these may be classified as

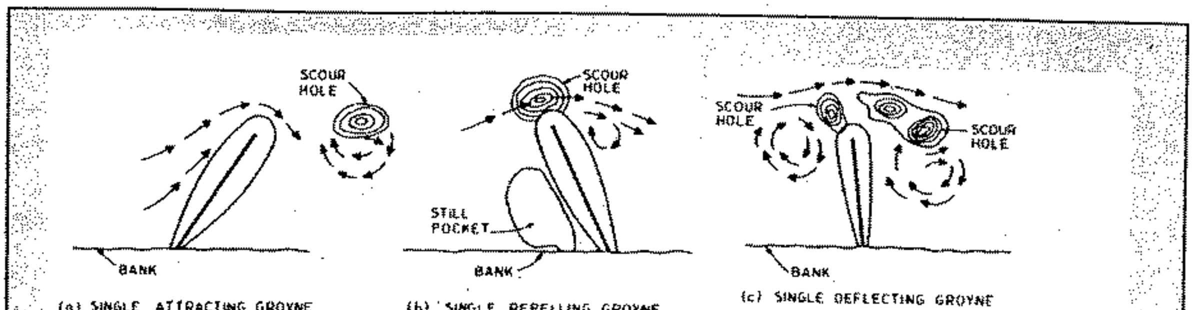
(a) Attracting groynes (b) Repelling groynes (c) Deflecting groynes (d) Sedimenting groynes

(a) Attracting groynes

- A groyne pointing d/s tends to attract the river flow towards the bank on which it is provided and hence it is called an attracting groyne.
- Such a groyne causes scour hole to form closer to the bank than the groyne inclined at right angles to the bank or that inclined slightly u/s.
- The angle of inclination of an attracting groyne with the bank may be in the range of 30° to 60°.
- In the case of an attacting groyne, the main attack of the stream is on its u/s face and therefore it needs better protection as compared to the d/s face.
- The attracting groynes safeguard the opposite bank against the attack of the current as they attract the current towards the bank adjacent to them.
- The attracting groynes are not useful for bank protection and may sometimes even endanger the adjacent banks since silting between successive groynes is absent.
- Attracting groynes are not commonly used because of above factor.

(b) Repelling groynes

- A groyne pointing u/s tends to repel the river flow away from the bank on which it is provided and hence it is called a repelling groyne.
- The angle of inclination of repelling groynes with the bank varies from 60° to 80°, or with a line perpendicular to the bank varies from 10° to 30°.
- On the u/s side of a repelling groyne a still water pocket is formed and the suspended sediment carried by the river gets deposited in the pocket.
- The head of a repelling groyne causes disturbances in the flow at its nose.
- Heavy scour occurs at the nose and slightly d/s of it due to eddy formation.
- The head of a repelling groyne needs a very strong protection because it is subjected to direct attack of a swirling current.
- Moreover, a large stagnant pool is created on the u/s side which is able to protect a greater lenth of the back than an attracting groyne.



Attracting, repelling and deflecting groynes.

Note : As compared to attracting groynes, repelling groynes are more effective and do not cause any trouble. As such repelling groynes are commonly used for the purpose of river training and bank protection.

(c) Deflecting groynes

- A groyne perpendicular to the bank or pointing slightly u/s & having a relatively short length tends to only deflect the flow without repelling it and hence it is called a deflecting groyne.
- A deflecting groyne, gives only local protection.

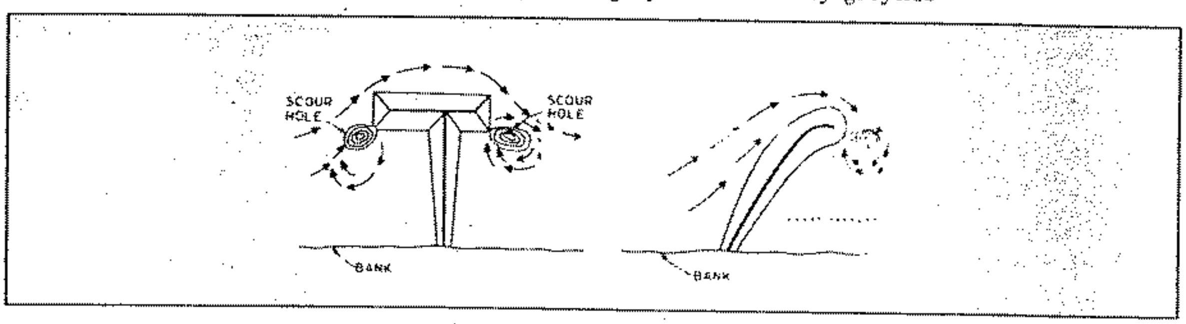
(d) Sedimenting groynes

- A groyne which dampens the velocity of flow and thus causes deposition of sediment carried by the river without repelling or deflecting the flow, is called a sedimenting groyne.
- Permeable groynes are usually classified as sedimenting groynes.
- These groynes are best suited for rivers carrying considerable amount of sediment in suspension.

Special Types of Groynes

Some of the special type of groynes that have also been used are

(i) Denehy's T-headed groynes (ii) Hockey head groynes or Hockey groynes



(i) Denehy's T-headed groynes

- Denehy's groynes or T-headed groynes were developed in India by Denehy and were first used by him in 1880 for the purpose of river training and bank protection at the Okhla Headworks, Delhi on the Yamuna River.
- It is an ordinary groyne provided with an extra cross groyne at the head giving it a T-shape.
- Cross groyne protects the main groyne on the same principle as main groyne protects the bank.
- A greater length of the cross groyne projects on the u/s side and a smaller length on the d/s side on the main groyne.

- In the case of a T-headed groyne, scour holes are formed on both ends of the cross groyne.
- These groynes are usually spaced 800 m apart with the T-head on a regular curved or straight line to train the river channel along a required course.

(ii) Hockey head groynes or Hockey groynes

- These groynes are provided with a curved head such that, at the lower end their shape is like a hockey stick.
- The hockey head groynes behave more like attracting groynes and hence they are not likely to be useful for bank protection or repelling of the current away form the bank.

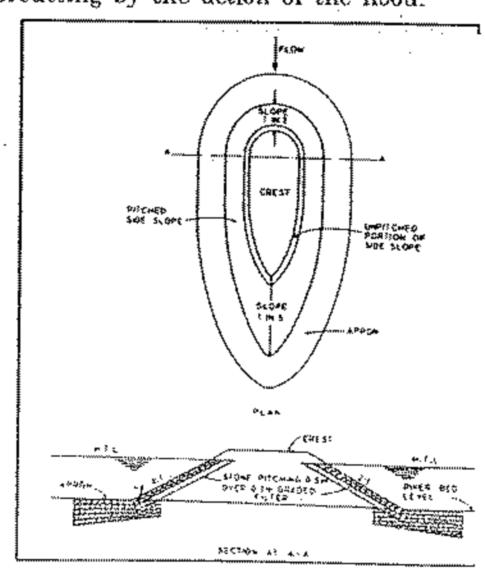
(4) Bank Pitching and Launching Aprons

Bank protection work may be classified as

(i) Direct and (ii) Indirect.

- **Direct protection** includes works done on the bank itself.
- **Indirect protection** includes works constructed not directly on the banks but in front of them for reducing the erosive action of the current.
- The pitching of a bank and the provision of a launching apron are the direct methods of bank protection while groynes deflecting the river current and cutoffs altering its course are the indirect methods of the bank protection.
- The materials used for the pitching of banks are stone, concrete blocks, bamboos and brush wood, natural growing grasses etc.
- Stone is the most commonly used material and most easily available material locally.
- Bricks & concrete blocks are usually too costly to be used for bank pitching except in small reaches for protection of a town or an engineering work. Pitching by brush wood or bamboos is usually temporary.
- Bank to be protected should be cut to a stable slope that varies from 1 : 1 to 2 : 1 before pitching is provided.
- A launching apron of loose stone is provided at the toe of the bank to ensure safety against the damage to the toe of a well protected bank from undercutting by the action of the flood.

(5) Pitched Islands



- A pitched island is an artificially created island in the river bed and is protected by stone pitching on all sides.
- A launching or falling apron is provided around the island to protect the toe of its side slopes.
- The pitched island causes redistribution of velocity and tractive force.
- The tractive force near a pitched island begins to increase rapidly after the construction of the island, with the result that deep scour begins to form round the island and gradually draws the main current towards itself and ultimately holds it permanently.

- The pitched island may thus be utilised to attract the current towards itself to reduce undue concentration on the river banks or to maintain the approach to some engineering work in the desired direction.

(6) Miscellaneous Methods

(i) Sills

Sills (also called *submerged sills or submerged dykes)* are used to counteract the tendency to cause excessive scour and large depth in any part of the river cross-section.

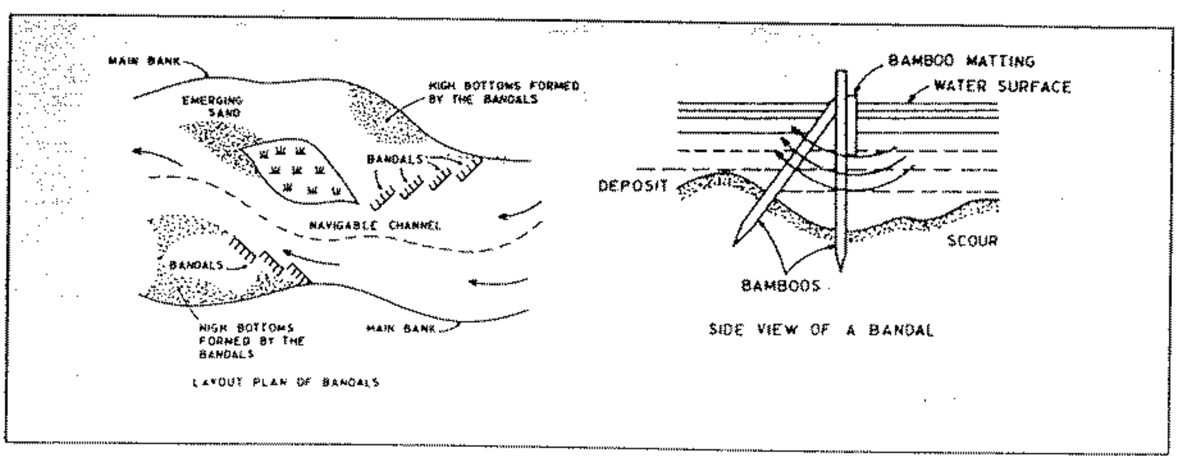
- This situation may arise at a sharp curve or adjacent to natural or artificial length of hard inerodible bank in a river.
- In such cases, sills are placed across the scoured portion of the bed with their top level at or a little below the designed bed level, which is desired to be achieved after correction of the abnormal scours.
- They must be spaced close enough to ensure their functioning properly to stop scour.

(ii) Closing dykes

- A Closing dyke is an obstruction put across a branch of a river which is desired to be closed so that the river flow may be diverted to single channel. The dykes may be either *solid or permeable.*
- A ***solid dyke*** must be designed to resist the pressure of water on its u/s face and to be safe against the possiblity of undermining by seepage.
- A ***permeable dyke*** retards the flow and causes a gradual silting up of the branch above itself.

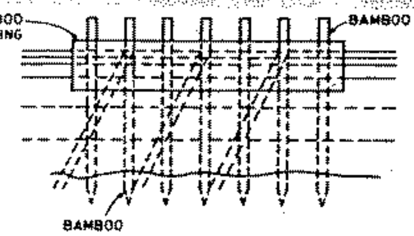
(iii) Bandalling

- Bandalling is a method of river training which is generally used to confine the low water flow in a single channel for maintaining required depth for navigation.
- The work of bandalling usually commences after the flood season as soon as the water levels begin to lower.



LAYOUT PLAN OF BANDALS

SIDE VIEW OF A BANDAL



Layout plan and other views of bandals.

- A bandal consists of a framework of bamboos driven into the river bed, set 0.6 m apart by means of horizontal ties and supported by struts at every 1.2 m.
- To this bamboo framework, bamboo matting are tied with coir ropes at water levels.
- The bamboo used on the framework are generally 3 to 6 m in length and the matting is 0.9 m wide strengthened at the edges by strips of split bamboo.
- Bandals are placed at an angle of 30° to 40° inclined d/s. They check the flow and cause sand to be deposited parallel to and behind the bandals.
- A channel confined between the bandals is formed with sand banks on either side and the whole discharge of the river is directed through this channel.
- Deepening of the channel is generally achieved in two or three weeks after the construction of the bandals. The depth thus achieved subsists until the next flood season without requiring any maintenance.

OBJECTIVE QUESTIONS

1. When a river starts meandering, the sediment carrying capacity
   (a) first decreases and ultimately increases
   (b) first increases and ultimately decreases
   (c) remains unaffected as the plan shape changes continuously
   (d) changes erratically all the time leaving permanent braids

2. In curved reach of a meandering river, both deposition of sediments and erosion of bank occur. Which one of the following statements is true in this regard?
   (a) Deposition of sediments occurs in the inner bank while the outer bank is subjected to erosion.
   (b) Deposition of sediments occurs in the outer bank while the inner bank is subjected to erosion.
   (c) In the direction of flow, the outer bank undergoes erosion first and the sediment is later deposited on the outer bank d/s.
   (d) In the direction of flow, the inner bank undergoes erosion first and the sediments are subsequently deposited on the inner bank d/s.

3. Which of the following are the purpose of a groyne as a river training structure?
   1. It contracts a river channel to improve its depth.
   2. It protects the river bank.
   3. It does not allow silt to deposit in the vicinity.
   4. It trains the flow along a certain course.

   Select the correct answer using the codes given below:

   (a) 1 and 2 (b) 1, 2 and 4 (c) 1 and 3 (d) 2, 3 and 4

4. Denehy's groyne is a special type of groyne which is

   (a) pointing u/s (b) pointing d/s (c) Hockey type (d) T-headed

5. Groynes are adopted for river bank protection works. When it is placed inclined d/s in the direction of flow in the river, it is designated as which one of the following?
   (a) Repelling groyne (b) Attracting groyne
   (c) Neither repelling nor attracting groyne (d) Fixed groyne

6. The flood plain of a river carries a discharge of 2000 m^3/s. What are the values of the meander length and dominant flow width?

   (a) 160 m, 48 m (b) 180 m, 42 m (c) 200 m, 38 m (d) 220 m, 36 m

7. Under which one of the following categories is the river Ganga classified in the reach through UP and Bihar ?

   (a) Straight river (b) Meandering river (c) Braided river (d) Deltaic river

8. Heavy scour at the head and shank of guide banks can lead to undermining of the stone pitching and consequent failure of the guide bank. This situation is avoided by providing

   (a) spurs (b) vertical cutoffs (c) marginal bunds (d) launching apron

9. Consider the following statements:

1. High water training is undertaken to protect against damage due to floods.
2. Low water training is undertaken to provide sufficient depth for navigation.
3. Mean water training is undertaken to provide efficient disposal of sediment load.

Which of these statements is/are correct?

(a) 1 and 2 only (b) 1, 2 and 3 (c) 2 and 3 only (d) 2 only

10. The 'meander length' for an alluvial river is :

(a) the total channel length along its looped course
(b) the total channel length minus the direct straight length
(c) the axial length of one meander
(d) the looped length of one meander
(e) none of the above.

11. The 'meander belt' for an alluvial river is :

(a) total river width between embankments
(b) width b/w the outer edges of fully developed meander loop, measured perpendicular to river axis
(c) the same as meander width
(d) both (b) and (c).

12. In a meandering river, the ratio of 'arcual channel length' to direct axial length', is called

(a) tortuosity (b) inverse of tortuosity
(c) cut off ratio (d) none of the above

13. A river reach having tortuosity of 1.2, can be said to have :

(a) 20% tortuosity of meander (b) 80% tortuosity of meander
(c) 120% tortuosity of meander (d) none of the above

14. Meander ratio in an alluvial meandering river is given by :

(a) $\frac{\text{meander length}}{\text{meander width}}$ (b) $\frac{\text{meander width}}{\text{meander length}}$ (c) $\frac{\text{meander width}}{\text{meander length}} \times 100$ (d) none

15. Sinuosity of a meandering river is :

(a) same as tortuosity (b) inverse of tortuosity
(c) log of tortuosity (d) none of them.

16. Aggrading rivers are :

(a) silting rivers (b) scouring rivers
(c) rivefs in regime (d) meandering rivers.

17. The river reach u/s of a newly built dam may behave, as :

(a) aggrading (b) degrading (c) virgin (d) none of them

18. The u/s angle of inclination of a repelling groyne with normal to the bank line, is of the order of :

(a) 5 to 10° (b) 10 to 30° (c) 30 to 50° (d) 70 to 90°

19. Out of the following choices given below, choose impermeable spur(s) :
   (a) an earthen spur protected by stone upron
   (b) an earthen spur, unprotected by stone apron
   (c) a balli spur (d) a tree groyne (e) a and b both (f) c and d both.

20. Permeable spurs are best suitable for rivers, which :
   (a) carry heavy suspended load
   (b) carry large bed load, but light supended load
   (c) need permanent protection to dikes
   (d) need attracting the river current, for providing deeper channel
   (e) flow in upper hilly reaches.

21. Denehey's spur is :
   (a) a hockey shaped earthen spur
   (b) T shaped stone spur, as used in Australia
   (c) T shaped earthen spur, as used in India
   (d) a type of a balli spur, especially developed for Indian rivers.

22. Which one of the following effects produced by a cut off in an alluvial river, is not an advantage to navigation :
   (a) shortened route and elimination of sharp bends
   (b) shortened travel time, particularly at low and moderate river stages
   (c) increased water depth at low river stages
   (d) lowering of the flood stages and flood periods.

23. Guide bank is hydraulic structures across an alluvial river
   (a) are obsolete and are not used in modern structures
   (b) are always used in pairs on both sides of the river
   (c) are useless in meandering streams
   (d) prevent the outflanking of the structure by the changing course of the stream.

ANSWERS

1. (a)	2. (a)	3. (b)	4. (d)	5. (b)	6. (d)
7. (b)	8. (d)	9. (b)	10. (c)	11. (d)	12. (a)
13. (a)	14. (b)	15. (a)	16. (a)	17. (a)	18. (b)
19. (e)	20. (a)	21. (c)	22. (d)	23. (d)	

Cross Drainage Works

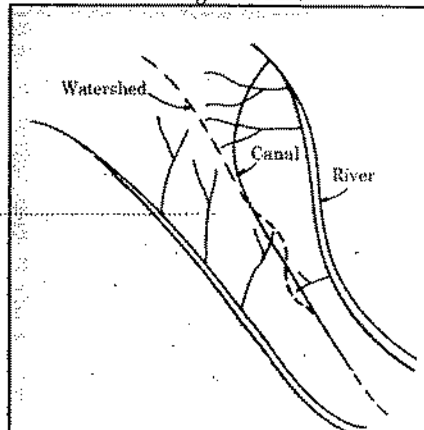
CHAPTER

9

Cross Drainage Works

INTRODUCTION

When a canal takes off from a river, it has to cross some streams or rivers before it can reach the top of the intended watershed for the purpose of irrigating an area. So, crossings are an important requirment on the canal alignment in the form of cross drainage works.



Canal crossing natural drains.

- A cross drainage work is a structure constructed for carrying a canal across a natural drain or river intercepting the canal so as to dispose the drainage water without interrupting the continuous canal supplies.
- These are unavoidable in any type of canal system.
- In order to minimise the number of cross drainage works, the alignment of canals should be generally along the watershed so that we have less number of natural drains.
- A canal taking off from a river has to travel a certain distance before it can mount the watershed. In this reach, the canal intercepts a number of natural drains from the watershed towards the river and it is necessary to carry the canal across these drains. Major CD works are required to be constructed on this reach.

TYPES OF CROSS DRAINAGE WORKS

Based on the relative bed levels , water levels of the canal and the drain and their relative discharge the CD works are of the following types :

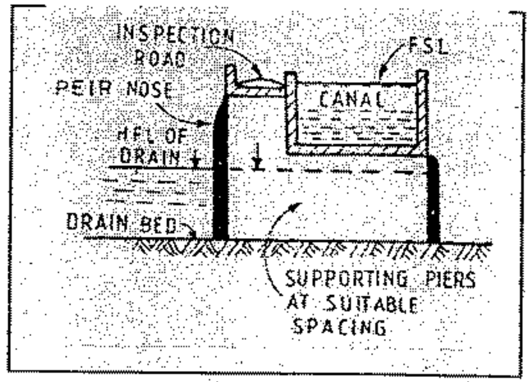
(1) Cross drainage works carrying the canal over the natural drain.

(i) Aqueduct (ii) Syphon adueduct

(i) Aqueduct

- An aqueduct is a hydraulic structure which carries a canal (through a trough or a duct) across and above the drainage similar to a bridge in which instead of road or a railway, a canal is carried over a natural drain.

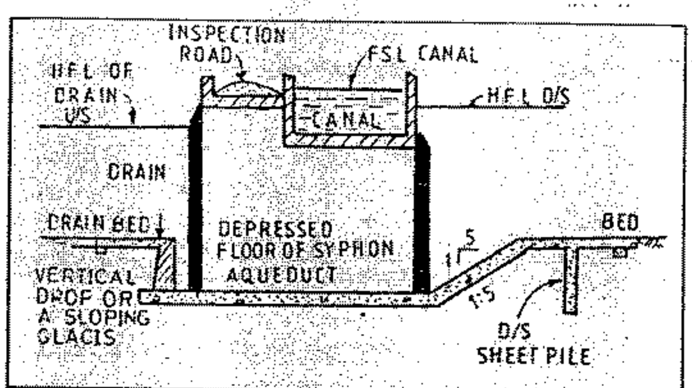
- In the case of an aqueduct, HFL (highest flood level) of the drainage should remain lower than the level of the underside of the canal trough.
- The canal water is taken across the drain in a trough supported on piers.
- The drain water flow under the canal such that there is sufficient headway available between the H.F.L. of the drain and the underside of the canal trough.
- The drain flows at atmospheric pressure under the work.



- Generally an inspection road is provided along with the trough.

(ii) Syphon aqueduct

- A syphon aqueduct is a cross drainage structure similar to an aqueduct except that the streambed is depressed locally where it passes under the trough of the canal and the barrels discharges the stream flow under pressure.
- A syphon aqueduct is constructed where the water surface level of the drain at high flood is higher than the canal bed.



- In the case of a sypon aqueduct, the bed of the drain is usually depressed and provided with pucca floor (i.e. concrete or masonry floor) thus forming barrels between the piers to pass the drain water.
- The horizontal floor of the barrels is provided with slopes at its ends to join the drain bed on either side.
- If the drop is of the order of 1m only, instead of slope a vertical drop may be provided on the u/s side. The drain water flows under pressure through the barrels which act as inverted syphons and hence this cross drainage work is known as syphon aqueduct,

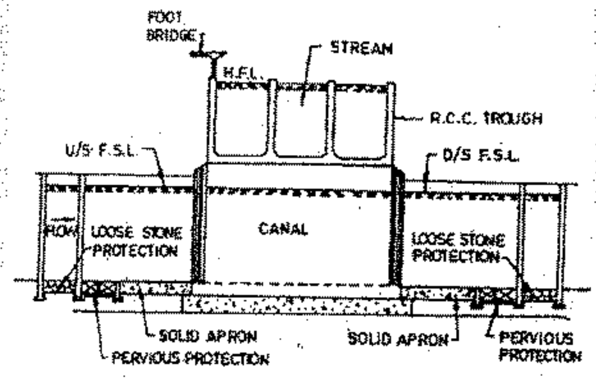
(2) Cross drainage works carrying the natural drain over the canal

(i) Super passage (ii) Syphon (or canal syphon)

(i) Super passage

A super passage is also similar to a bridge in which the natural drain is carried over the canal.

- A super passage is reverse of an aqueduct.
- A super passage is constructed where the bed of the drain is well above the canal F.S.L.
- In this case, the drain water is taken across the canal in a trough supported on piers.
- There is sufficient headway available between the full supply level of the canal and the underside of the drain trough.
- The canal flows at atmospheric pressure under the work.

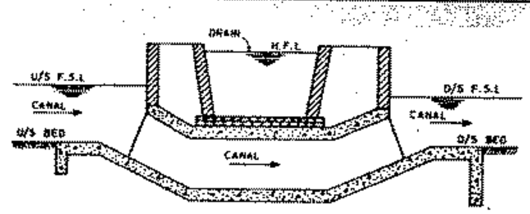


A Typical Cross-Section of a Superpassage

- In this case it is not possible to provide an inspection road along the canal.

(ii) Syphon

- A syphon is similar to a syphon aqueduct with the difference that in the case of a syphon the canal water is carried through the barrels under the drain.
- A syphon is constructed where the full supply level of the canal is higher than the bed of the drain.
- The barrels in this case also act as *inverted syphons* through which the canal water flows under pressure.



(3) Cross drainage works admitting the drain water into the canal

In this type of cross drainage works, the canal water and the drain water are allowed to intermingle with each other. This may be achieved by the following two types of the cross drainage works.

(i) Level crossing (ii) Inlet and outlet.

(i) Level crossing

A level crossing is a cross drainge work in which the drainage and the canal meet each other at approximately the same level. A level crossing is generally provided when a large canal and huge drainage (may be a stream or a river) approach each other at almost the same level.

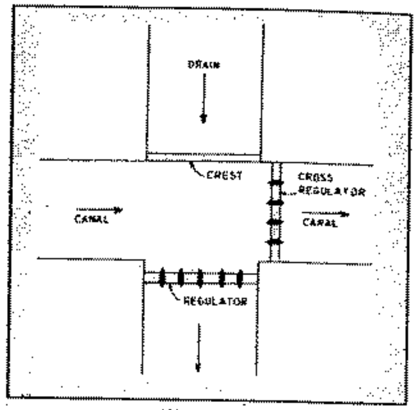
A level crossing consists of:

(a) a crest with its top at the F.S.L. of the canal across the drain at its u/s junction with the canal

(b) a regulator with quick falling shutters across the drain at its d/s junction with the canal

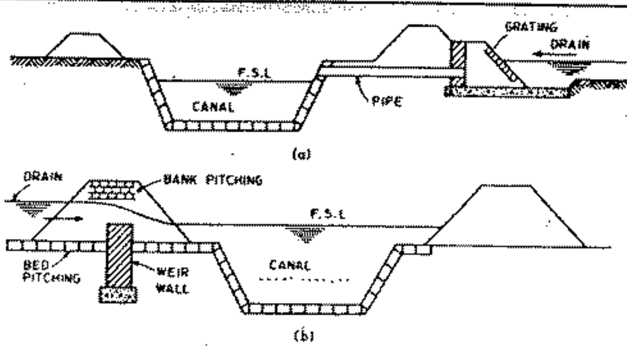
(c) a cross regulator across the canal at its d/s junction with the drain.

- Such an arrangement is adopted when both the canal and the drainage carry considerable discharge, the latter during the high flood season when syphoning either the canal or the stream proves to be extremely costly or else the head loss through the syphon barrels is very high.
- Arrangement is practically similar to that provided on a canal head works.
- In this arrangement, the perennial discharge is used advantageously in order to increase the canal supplies.
- During dry season, regulator of the drain is generally kept closed and the outgoing canal regulator is kept fully open.



(ii) Inlet and outlet

- An inlet is an open cut or a pipe which is provided in a canal bank suitably protected by pitching to pass the drain water into the canal.
- The bed and sides of the canal are also pitched for a certain distance u/s and d/s of the inlet.
- This arrangment is provided only where the silt load of the drainage is small.
- An inlet may be provided for a small drain coming across a canal if the bed level of the drain is slightly higher or lower than the canal F.S.L.
- It is not necessary that the no. of inlets and outlets should be the same.
- There may be one outlet for two or three inlets.



Inlet (a) pipe type ; (b) open cut type.

- Inlet is a non regulating structure
- Outlet is another open cut in the canal bank with bed and sides of the cut properly pitched.
- The escaping water from the outlet is taken away by a lead channel to some nearby drain or outfall on the d/s side of the surface outlet.

Note: This type of CD work are inferior to aqueduct or super passage but they are cheaper.

SELECTION OF A SUITABLE TYPE OF CROSS-DRAINAGE WORK

The relative bed levels, water levels, and discharge of the canal and the drain are the primary factors on which the type of cross drainage work depends. In ideal cases, our choices will be the following:

(i) Aqueduct will be the first choice, if the bed level of the canal is significantly above the HFL of the drains.

(ii) Superpassage will be the first choice if the bed level of the drain is significantly above the HFL of the canal.

(iii) On similar basis, we can make choices for canal syphon and syphon aqueduct according to their definition

In actual field, we will not have such ideal conditions and our choice would depend upon various other factors, such as :

(i) Suitable canal alignment

(iii) Permissible head loss in canal

(ii) Position of W.T. and availability of dewatering equipment.

(iv) Suitability of soil for embankment.

(v) Nature of available foundation.

- By changing the alignment in a proper way, the bed levels of the canal and the drainage can be changed and manipulated in order to provide best solution while designing a CD works.

e.g., If headway between HFL of drain and bed of the canal bed is insufficient (although canal bed is higher), we normally adopt syphon aqueduct. But if above conditions are not favourable for the construction of a syphon aqueduct, canal alignment will be changed in such a way that the crossing is shifted to the d/s where drainage bed is low and we get sufficient headway, so that we can construct an aqueduct in place of syphon aqueduct.

- Superpassage should be avoided whenever possible as it is inferior to an aqueduct.
- A syphon aqueduct (unless large drop in drainage bed is required) is superior to a syphon.
- A Level crossing can't be avoided when a large canal crosses a large torrent at equal bed levels
- An inlet is adopted when a small drain crosses the canal with its bed level equal to canal FSL or a bit higher than it.

METHODS USED FOR DESIGNING THE CHANNEL TRANSITIONS

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

(i) Mitra's method of design of transitions (when water depth remains constant).

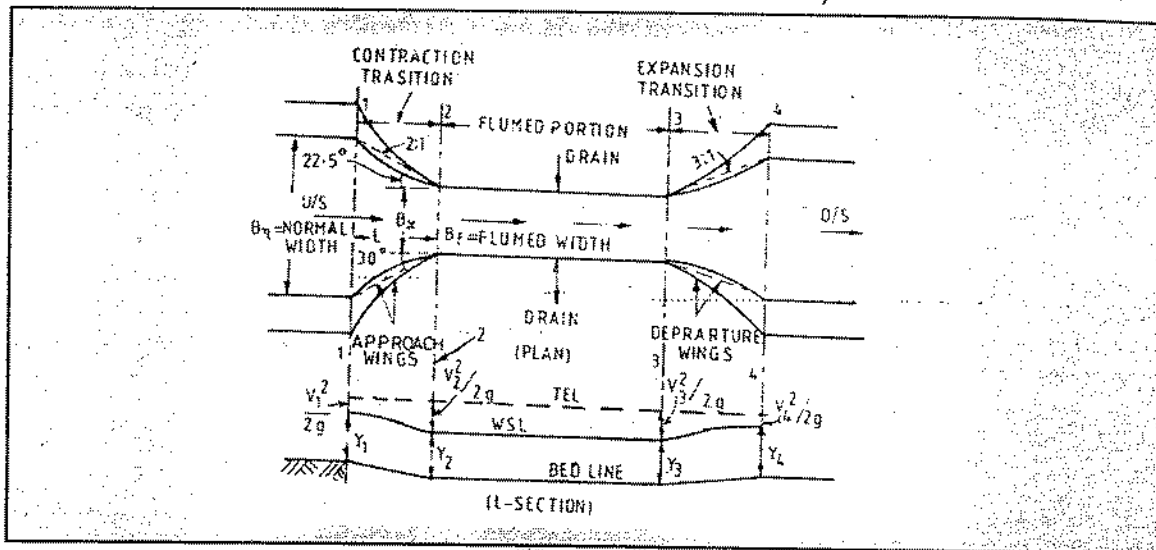
(ii) Chaturvedi's method of design of transitions (when water depth remains constant).

(i) Mitra's Hyperbolic Transition when water depth remains constant

Shri A.C. Mitra, Chief Engineer, U.P. Irrigation Deptt. (Retd.), has proposed a hyperbolic transition for the design of channel transitions. According to him, the channel width at any section X–X, at a distance x from the flumed section is given by

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

where, B_n = Bed width of normal channel section ; B_f = Bed width of flumed channel section
B_x = Bed width at any distance x from flumed section ; L_f = Length of transition



(ii) Chaturvedi's Semi-Cuhical parabolic Transition when water deptb remains constant

Prof. R.S. Chaturvedi, in 1963, on the basis of his experiments had proposed the following equation for the design of channel transitions when water depth remains constant.

$$\boxed{x = \frac{L_f B_c^{3/2}}{B_c^{3/2} - B_f^{3/2}}\left[1 - \left(\frac{B_f}{B_x}\right)^{3/2}\right]}$$

Choosing various convenient values of B_x; the corresponding distance x can be computed easily

Example 1

Design a transition using Mitra's hyperbolic transition given by $B_x = \frac{B_c B_f L_f}{L_f B_c - (B_c - B_f)_x}$ and compare the results using Chaturvedi's semi-cubical parabolic transition given by

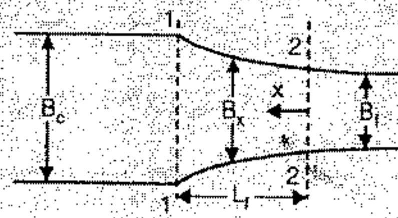
$$x = \frac{L_f B_c^{3/2}}{B_c^{3/2} - B_f^{3/2}}\left[1 - \left(\frac{B_f}{B_x}\right)^{3.2}\right]$$

Given nominal bed width = 25 m, width of flumed section = 10.0 m and total length of transition = 15.0 m.

Sol. The design of transition as per Mitra's hyperbolic transition equation is given as:

$$B_x = \frac{B_c B_f L_f}{L_f B_c - (B_c - B_f)_x}$$

where B_c is nominal bed width of the normal channel section, B_f is the width of flumed section, B_x is the bed width at any distance x from the flumed section and L_f is the total length of transition. Here B_c = 25m; B_f = 10.0 m; L_f = 15m



$$\therefore \quad B_x = \frac{25\times10\times15}{15\times25-(25-10)x}$$

$$\Rightarrow \quad B_x = \frac{3750}{375-15x}$$

$$\Rightarrow \quad B_x = \frac{250}{25-x}$$

For various values of x lying between 0 and 15m, values of B_x are worked out and tabulated below

x(in m)	0	3	6	9	12	15
$B_x = \frac{250}{25-x}$	10	11.36	13.16	15.63	19.23	25

As per Chaturvedi's semi-cubical parabolic transition equation various distances from the flumed section can be calculated by choosing various convenient values of B_x i.e.

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

B_x as per Mitra's transition equations	10	11.36	13.16	15.63	19.23	25
Corresponding value as per Chaturvedi's transition equation	0	3.5	6.78	9.80	12.55	15

OBJECTIVE QUESTIONS

1. The following data pertain to a natural drain crossing an irrigation canal:

Item	Canal data	Drainage data
Flow (m^3/s)	5	500
Bed level (m)	120	116
Depth of flow (m)	0.8	10

Which one of the following types of cross-drainage should be recommended in this case?

(a) Aqueduct (b) Syphon aqueduct (c) Syphon (d) Super-passage

2. The worst condition of uplift on the floor of a syphon aqueduct occurs when there is

(a) high flood flow in the drainage with canal dry
(b) full supply flow in the canal with drainage dry
(c) high flood flow in the drainage with canal running full
(d) Water is at drainage bed and canal is dry

3. Match **List-I** (Relative position of canal and drainage channel) with **List-II** (Type of cross drainage work) and select the correct answer using the code given below the lists:

List-I	List-II
A. Canal taken above the drainage channel at its grade	**1.** Canal siphon
B. Drainage channel taken above the canal at its bed slope	**2.** Drainage siphon
C. Canal taken below the drainage channel	**3.** Aqueduct
D. Drainage channel taken below the canal	**4.** Super passage

Codes:

	A	B	C	D
(a)	2	4	1	3
(b)	3	1	4	2
(c)	2	1	4	3
(d)	3	4	1	2

4. What type of cross drainage work is provided when the canal runs below the drain, with FSL of canal well below the bed of the drain?

(a) Aqueduct (b) Super passage (c) Level crossing (d) Siphon aqueduct

5. Consider the following statements:
An aqueduct is a cross drainage work in which

1. a canal is carried over the drainage channel.
2. a drainage channel is carried over the canal.
3. both drainage channel and canal are at the same level.

Which of these statements is/are correct?

(a) 1 only (b) 1 and 2 only (c) 2 and 3 only (d) 1, 2 and 3

6. An irrigation canal flowing freely above a drainage, which in turn is flowing under pressure, is specifically, called a:

(a) canal siphon (b) canal aqueduct (c) siphon aqueduct (d) super passage.

7. The crossing arrangement, preferably made at the junction of a huge canal and a river stream carrying short lived-high flood discharge at almost equal bed levels, is a :

(a) super passage (b) aqueduct (c) level crossing (d) canal siphon.

8. The drainage water is sometimes allowed to join the canal water to augment canal supplies, through a hydraulic structure, called a :

(a) canal outlet (b) canal inlet (c) module. (d) level crossing

9. The DBL of a canal and the HFL of a drain at their crossing point are, respectively, 216 m and 214 m. The adopted cross-drainage work here, will be a :

(a) super passage (b siphon (c) aqueduct (d) siphon aqueduct

10. Pinpoint the correct statement in relation to the relative merits of the two cross drainage works:

(a) a super passage is preferred to an aqueduct
(b) a canal syphon is preferred to a super passage
(c) a super passage is preferred to a canal syphon
(d) a siphon aqueduct is inferior to a canal siphon, and is seldom used.

11. A cross-drainage work is called a siphon, when it carries the canal water :

(a) below the drainage under pressure (b) below drainage at atmospheric pressure
(c) above drainage at atmospheric pressure (d) none of the above

12. In an aqueduct provided with a pucca bottom floor, the uplift will occur:

(a) on the roof slab (b) on the bottom floor
(c) on both the roof slab as well as the bottom floor
(d) no where, since the flow is free in the canal as well as in the drainage channel.

13. Point out the incorrect statement :

(a) aqueducts and super passages are usually not provided with pucca bottom floors
(b) aqueducts syphons and canal syphons are usually provided with pucca bottom floors
(c) in a level crossing, a cross regulator is provided on the canal below the crossing
(d) canals or drainage channels are usually flumed to affect economy at the sites of crossings, where in the contraction transitions are not to be steeper than $22\frac{1}{2}^{\circ}$ and the expansion transitions not to be steeper than 30°.
(e) none of the above.

14. The following data is available at the proposed site of a canal crossing:

Item	Drain	Canal
B.L.(m)	252.2	248.0
FSL / HFL(m)	253.2	253.0
Discharge(cumecs)	2	400

The most appropriate and economical cross-drainage work at the above site will be:

(a) an aqueduct (b) a super passage (c) a syphon aqueduct (d) a syphon.

15. Select the correct statement concerning the relative merits of two cross drainage works, when a choice is to be made.

(a) Compared to an aqueduct, a superpassage is superior and is thus preferable
(b) Compared to a superpassage, a canal syphon is superior and is preferred
(c) A syphon aqueduct is inferior to a canal syphon and is seldom used
(d) The superpassage is superior to a canal syphon and is preferred in general.

16. In an aqueduct,

(a) there is no uplift problem
(b) the condition of high flood flow in the drainage and no water in the canal causes maximum uplift on the canal
(c) when the canal is at FSL and there is no water in the drainage, the floor of the drainage experiences maximum uplift
(d) When the canal is at FSl and the drainage is at HFL maximum uplift conditions on the canal floor prevail.

17. A level crossing type of cross drainage work consists of

(a) one regulator only (b) two weirs only
(c) one weir and two regulators (d) two weirs and two regulators.

ANSWERS

1. (b)	**2. (a)**	**3. (d)**	**4. (b)**	**5. (a)**	**6. (c)**
7. (c)	**8. (b)**	**9. (c)**	**10. (c)**	**11. (a)**	**12. (b)**
13. (c)	**14. (d)**	**15. (d)**	**16. (a)**	**17. (c)**	

Dams and Reservoirs

CHAPTER

10

Dams and Reservoirs

INTRODUCTION

A dam is a barrier constructed across a river in order to create a reservoir for impounding water or to provide the facility of diverting water from the river or to retain debris flowing in the river along with water.

CLASSIFICATION OF DAMS

may be classified in different ways on the basis of their function, hydraulic design, material of construction, structural design and size.

(1) Classification based on function

(i) Storage Dam or Impounding Dam

A storage dam is constructed to create a reservoir to store water during the period when the flow in the river is in excess of the demand; and utilising it later on during the period when the demand exceeds the flow in the river. The water stored in the reservoir so created may be used for various purposes such as *irrigation, hydroelectric power generation, water supply etc.*

(ii) Detention Dam

A detention dam is constructed to temporarily detain all or part of the flood waters of a river and to gradually release the stored water at controlled rates so that the entire region on the d/s side of the dam may be safeguarded against the possible damage due to floods.

(iii) Diversion Dam

A diversion dam is constructed for the purpose of diverting part or all of the water from a river into a conduit or a channel. For the purpose of diversion of water from a river into an irrigation canal a weir is constructed across a river which is known as *diversion weir.*

(iv) Coffer Dam

A coffer dam is a temporary dam constructed to exclude water from a specific area. Such a dam is invariably constructed on the u/s side of the site where actual dam is to be constructed so that the site for the constructional work is rendered dry. It also behaves as a diversion dam.

(v) Debris Dam

A debris dam is constructed to catch & retain debris/silt flowing along with water in the river.

(2) Classification Based on Hydraulic Design

(i) Overflow Dam or Overfall Dam

An overflow dam is a type of dam constructed with a crest to permit the overflow of surplus water which cannot be retained in the reservoir. Usually dams are not designed as overflow dams for their entire length. Only some dams few metres in height have their entire length designed for overflow, which are mainly used for diversion of water lie weirs.

(ii) Non-overflow Dam

A non-overflow dam is a type of dam for which water is not allowed to flow over its crest.

Note : In most of the cases, a part of the length of dam is designed as an overflow dam, while the rest is designed as non-overflow dam.

(3) Classification Based on Material of Construction

(i) Rigid Dam

A rigid dam is a type of dam which is constructed with rigid material such as masonry, concrete, steel or timber. Earlier stone masonry was commonly used for the construction of dams, but now a days it is almost totally replaced by concrete.

Note : Bhakra dam (a concrete dam) & Rana Pratapsagar dam (a stone masonry dam), are rigid dams in India.

(ii) Non-rigid Dam

A non-rigid dam is a type of dam which is constructed with non-rigid material such as earth, rockfill etc. ***e.g.***, *earth dam, rockfill dam and rockfill composite dam*.

- An earth dam is constructed with gravel, sand, silt and clay.
- A rockfill dam consists of fragmental rock material supporting a water tight membrane on the u/s face.
- A rockfill composite dam consists of a rockfill on the d/s side and an earth fill on the u/s side.

Note: In most of the cases an earth dam is provided with a concrete or stone masonry overflow or spillway section. Such as a dam is known as composite dam.

(4) Classification Based on Structural Behavior

(i) Gravity Dam

A gravity dam is a masonry or concrete dam which resists the forces exerted upon it by its own weight. Its cross-section is approximately triangular in shape. If a gravity dam is straight in plan it is known as straight gravity dam, while if it is curved in plan it is known as curved gravity dam. A curved gravity dam resists the forces exerted upon it both by gravity action and arch action. Further a gravity dam is also classified as solid gravity dam and hollow gravity dam.

Note: *Most of the gravity dams constructed in India are straight solid gravity dams and e.g., Bhakra dam*

(ii) Arch Dam

An arch dam is a curved masonry or concrete dam, convex u/s, which resists the forces exerted upon it, mainly by arch action. The best or most economical central angle in an arch dam is the one whose value is equal to 133° – 34' at mid height in constant radius dam, in constant angle dam → 133° – 34'.

Simple arch dams can be divided into three types (depending upon the shape) : *(i) Constant radius arch dams, (ii) Variable radius arch dams ; and (iii) Constant angle arch dams.*

e.g., Idduki dam is an example of arch dam in India.

(iii) Buttress Dam

A buttress dam consists of a water retaining sloping membrane or deck on the u/s which are generally in the form of equally spaced triangular reinforced concrete slab. In some cases the u/s slab is replaced by multiple arches supported on buttresses or by flaring the u/s edge of the buttresses to span the distance between the buttresses.

(iv) Embankment Dam

It is a *non-rigid dam*, which resists the forces exerted upon it mainly by its shear strength.

FACTORS GOVERING SELECTION OF TYPE OF DAM

(1) In general, topography or valley shape is the first choice for selection of type of dam The shapes of valley normally found in nature may be broadly classified in three categories:.
   (i) A narrow V-shaped valley with sound rock in abutments (suited for arch dam).
   (ii) A narrow or moderately wide U-shaped valley with sound rock foundation (best suited for gravity dam or buttress dam).
   (iii) A wide valley with foundation soil material to a considerable depth (suited for embankment dam).

(2) Foundation conditions at the dam site need to be thoroughly investigated as all the force acting on dam including its weight are transmitted to the foundation. Foundation conditions depend upon the geological character and thickness of the strata, their inclination, permeability, and relation to underlying strata, existing faults and fissures. The common types of foundations generally encountered are:
   (i) Good rocky Foundation (ii) Gravel and Coarse Sand foundations
   (iii) Silt and Fine Sand Foundations (iv) Clay Foundation (v) Nonuniform Foundations

(3) The choice of the type of dam depends on the types of construction materials that may be

available in sufficient quantity at or near the dam site. A dam constructed with locally available materials will be the most economical due to considerable reduction in the transportation costs

(4) The choice of the type of dam is affected by size, type and location of a spillway. Thus if a large spillway is required to be provided, then generally spillway and dam are combined into one structure. In that case, a concrete dam with overflow and non-overflow sections may be adopted. On the other hand, if small spillway is required, then even in narrow dam sites the choice may be in favour of earthfill or rockfill dams.

(5) The section of the type of dam, its dimensions and location of spillway and other appurtenances should be such that there are no adverse effects on the environment and as far as possible maximum protection is provided for the environment.

e.g., a particular river might be having scenic or recreational quality for most of its length, which will be spoiled completely if a high dam is constructed and a big reservoir is created in which considerable land will be submerged.

(6) If a dam is to be constructed in an area that is subjected to earthquake shocks then the section of the type of dam should have the ability to resist the earthquake shocks without damage.

e.g., Earthfill and concrete gravity dams are the best suited types of dams in this respect.

(7) The overall cost of construction and cost of maintenance is an important factor in the choice of the type of dam. The cost of construction of dam is affected by the availability and price of construction materials and labour, while the cost of maintenance mainly depends on the nature of the construction materials.

SELECTION OF SITE FOR A DAM

The selection of a suitable site for the construction of a dam depends on various factors which are briefly described below :

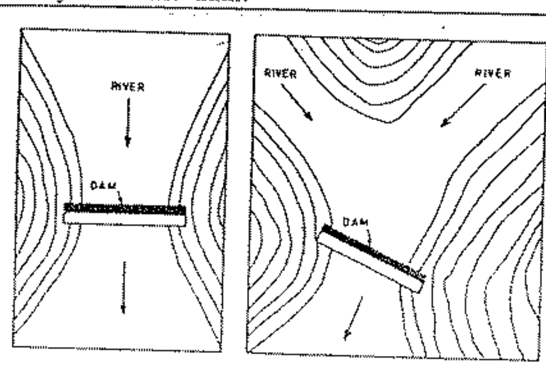
(i) Suitable foundations should be available at the dam site.

(ii) The length of the dam should be as small as possible from economic point of view and for a given height it should store large volume of water.

(iii) The river valley at the dam site should be as narrow as possible and it should open out u/s to create a reservoir with as far as possible large storage capacity.

(iv) The dam should be located on high ground as far as possible compared to the river basin in order to reduce the cost and facilitate drainage of the dam section.

(v) A suitable site for the spillway should be available in the vicinity of the dam if the spillway is to be located separately from the dam.



(a) NARROW VALLEY (b) CONFLUENCE

(vi) The value of the property and land submerged in the reservoir created by the proposed dam should be as low as possible.

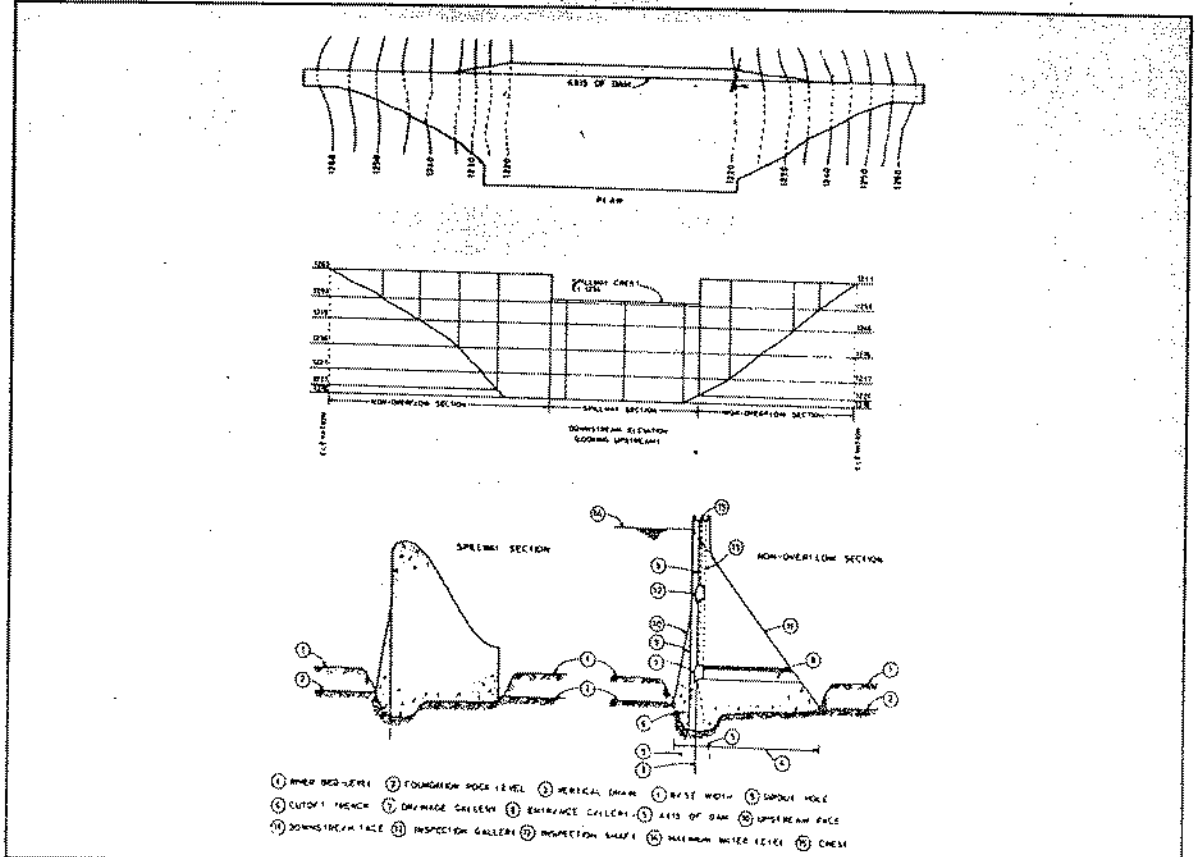
(vii) The dam site should be such that the reservoir would not silt up soon. For this if any of the tributaries of the river is transporting relatively large quantity of sediment, then the dam site may be selected on the u/s of the confluence of this tributary with the river.

(viii) It is preferable to select a dam site which is already connected or can be conveniently connected to a nearby rail or road in order to facilite transportation of men, material, machinery and various other essential items to the dam site.

Note: Dam is often located on the d/s of the confluence of two rivers, so that advantage of both the valley to provide larger storage capacity is available.

GRAVITY DAM

- A gravity dam is a solid masonry or concrete structure with an approximately triangular cross-section, so that the external forces exerted on it are resisted by its own weight. It is also called a *solid gravity dam.*
- It is mostly straight in plan but in some cases it may be slightly curved as well.
- A sound rock foundation is an essential requirement for the construction of a gravity dam.
- The dam consists of two sections viz., non-overflow section and overflow (or spillway) section



Layout of a gravity dam

- The axis of a dam is taken as the reference line which is defined separately in the plan and in the cross-section of the dam.
- In plan, the *axis of dam* is defined as the horizontal trace of the u/s edge of the top of the dam and it is also called the *base line of dam*.
- Axis or base line of dam may be straight, slightly curved u/s or a combination of end curves and central straight line to take the best advantage of topographic conditions at the site.
- In the cross-section of the dam, the vertical line passing through the u/s edge of the top of the dam is considered as the axis of the dam.
- The *length of dam* is the length measured along the axis of the dam at the top of the dam from one abutment to the other abutment.
- The *maximum base width* of the dam is the horizontal distance between the outer points of the heel and toe of the cross-section of the dam.
- The *maximum height of dam* or *structural height of dam* is the vertical distance between the lowest point in the foundation and the top of the dam.

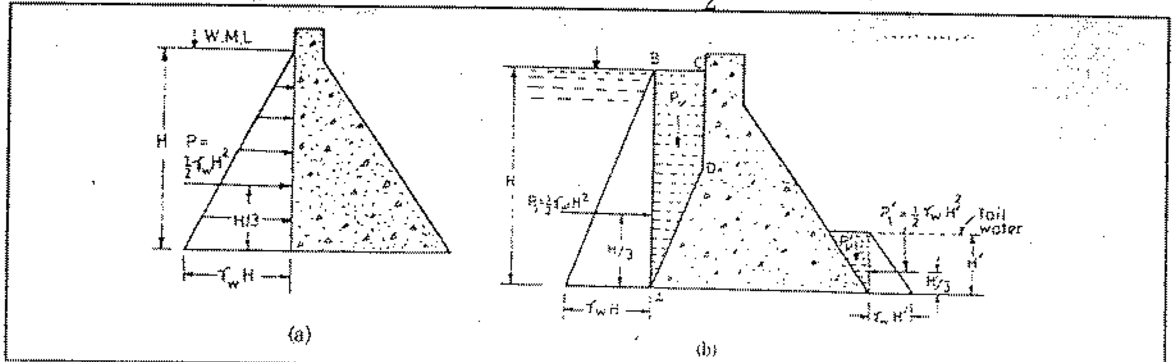
FORCES ACTING ON GRAVITY DAM

The various external forces acting on a gravity dam may be :

(1) Water Pressure (2) Uplift Pressure (3) Pressure due to earthquake forces (4) Silt Pressure (5) Wave Pressure (6) Ice Pressure (7) Weight of the dam

(1) Water Pressure

- Water pressure (p) is the major external force acting on a dam. The horizontal water pressure, exerted by the weight of the water stored on the u/s side on the dam can be estimated from rules of *hydrostatic pressure distribution*.
- When the u/s face is vertical, the intensity is zero at the water surface and equal to $\gamma_w H$ at the base where, γ_w = unit weight of water and H = depth of water.
- The resultant force due to this external water, $P = \frac{1}{2}\gamma_w H^2$, acting at H/3 from base.



(a) (b)

- When the u/s face is partly vertical and partly inclined, the resulting water force can be resolved into horizontal component (P_h) and vertical component (P_v).
- If there is tail water on the d/s side, it will have horizontal and vertical components.

(2) Uplift Pressure

- Water seeping through the pores, cracks and fissures of the foundation material, exert an uplift pressure on the base of the dam.
- An uplift force reduces the downward weight of the body of the dam and hence destabilises the dam against the dam stability.
- According to USBR recommendations, the uplift pressure intensities at the heel and the toe should be taken equal to their respective hydrostatic pressures and joined by straight line in between as explained in figure below.

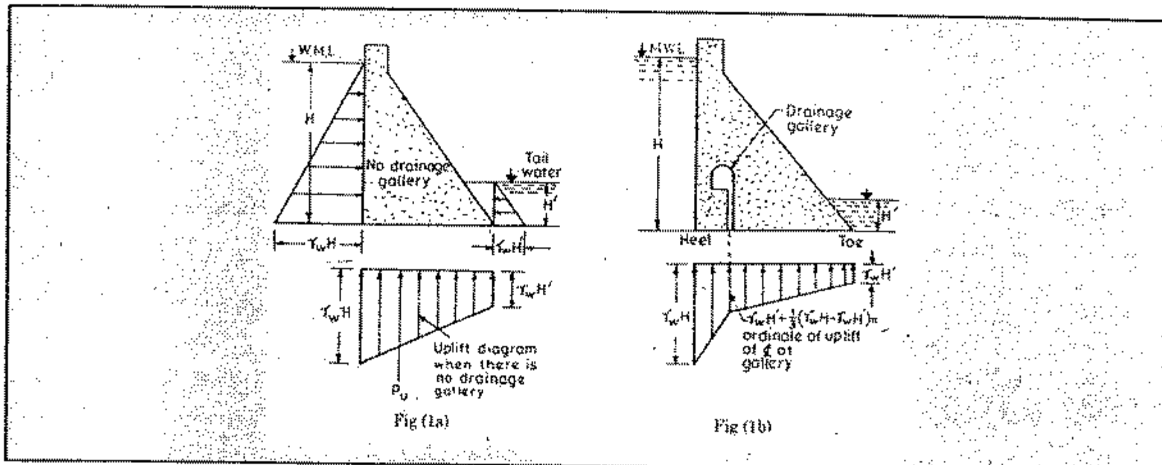


Fig (1a) Fig (1b)

- Recommended uplift at the face of the gallery (when drainage galleries are provided to relieve the uplift) is equal to the hydrostatic pressure at toe (γ_w.H') plus $\frac{1}{3}$rd the difference of the hydrostatic pressures at the heel and the toe
- It is assumed that the uplift pressures are not affected by the earthquake force.

(3) Earthquake Forces

- The effect of an earthquake is equivalent to imparting an acceleration to the foundations of the dam in the direction in which the wave is travelling.
- Earthquake wave may move in any direction.
- Both horizontal acceleration (α_h) and vertical acceleration (α_v) are induced by an earthquake. The values of these acceleration are generally expressed as percentage of the acceleration due to gravity (g), i.e., α = 0.1 g or 0.2 g, etc.
- In India, the entire country has been divided into five seismic zones depending upon the severity of the earthquakes.
- Zone V is the most serious zones. (Now zone i and ii have been merged such that we have now only iv zones.
- For areas not subjected to extreme earthquakes, α_h = 0.1g and α_v = 0.05g.
- These forces may be neglected in areas of no or very less earthquakes.
- In extremely seismic regions and in conservative design, even a value upto 0.3 g may sometimes be adopted.

(i) Effect of vertical acceleration (a_v)

- When the vertical acceleration is acting in the upward direction, the effective weight of the dam will increase and hence, the stress developed will increase.
- When the vertical acceleration is acting downward, the foundation shall try to move downward away from the dam body and thus the effective weight and the stability of the dam will decrease. This is the worst case for designs.

Such acceleration will exert an inertia force $\frac{W}{g}\alpha_v$ (W is total wt. of the dam)

$\therefore$ The net effective weight of the dam $= W-\frac{W}{g}.\alpha_v$.

If $\alpha_v = k_v.g$

[where k_v is the fraction of gravity adopted for vertical acceleration, such as 0.1 or 0.2, etc.].

Then, the net effective weight of the dam $= W - \frac{W}{g}.k_v \, . \, g = W\,[1 - k_v]$

Note : *Vertical acceleration reduces the unit weight of the dam material and that of water to (1 - k_v) times their original unit weights.*

(ii) Effects of horizontal acceleration (a_h)

Horizontal acceleration may cause the following two forces:

(a) Hydrodynamic pressure (b) Horizontal inertia force.

(a) Hydrodynamic pressures

Horizontal acceleration acting towards the reservoir causes a momentary increase in the water pressure as the foundation & dam accelerates towards the reservoir and the water resists the movement owing to its inertia. The extra pressure exerted by this process is known as hydrodynamic pressure.

According to Von-Karman, the amount of this hydrodynamic force (P_e) is given by

$P_e = 0.555 \, . \, k_h \, \gamma_w . \, H^2$ (acts at the height of $\frac{4H}{3\pi}$ above the base)

Moment of this force about base, $\boxed{M_e = P_e\left(\frac{4H}{3\pi}\right) = 0.424 \, P_e \, . \, H}$

Note : *Zanger has given certain big formulas for evaluating the amount of this force and its position, etc. on the vertical as well as on an inclined faces. For average ordinary purposes, the Von-Karman equation is sufficient.*

(4) Silt Pressure

Silt gets deposited against the u/s face of the dam. If h is the height of silt deposited, then the force exerted by this silt in addition to external water pressure, can be represented by Rankine's formula as : $\boxed{P_{silt} = \frac{1}{2}.\gamma_{sub}.h^2K_a}$ (it acts at $\frac{h}{3}$ from base) where, $K_a = \frac{1-\sin\phi}{1+\sin\phi}$ = coefficient of active earth pressure of silt

(ϕ = angle of internal friction of soil, and cohesion is neglected).

γ_{sub} = submerged unit weight of silt material ; h = height of silt deposited.

Note : If the u/s face is inclined, the vertical weight of the silt supported on the slope also acts as vertical force.

(5) Wave Pressure

Waves are generated on the surface of the reservoir due to blowing winds, which causes a pressure towards the d/s side. Wave pressure depends upon the wave height.

Wave height may be given by the equation,,

$$h_w = 0.032\sqrt{VF} + 0.763 - 0.271\,(F)^{3/4} \text{ for } F < 32 \text{ km,} \quad \text{----(i)}$$

$$h_w = 0.032\sqrt{VF} \text{ for } F > 32 \text{ km} \quad \text{----(ii)}$$

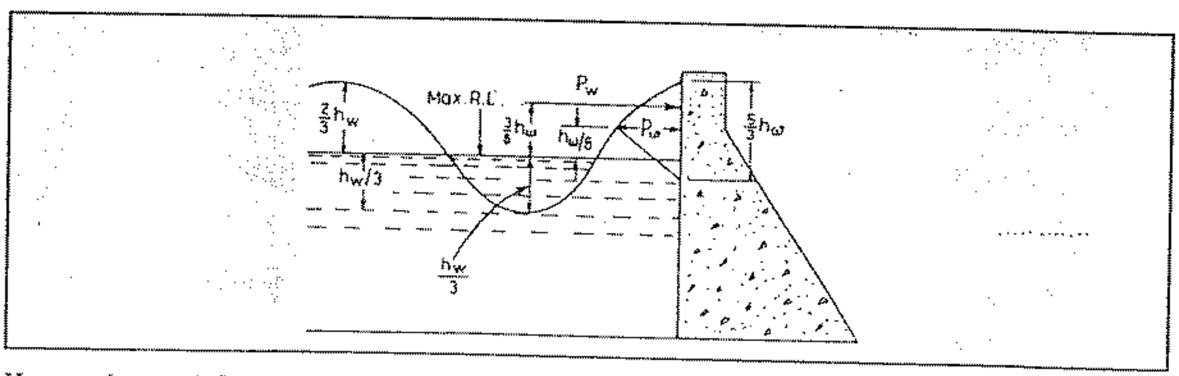
where h_w = height of water from top of crest to bottom of trough (in metres).

V = wind velocity (in km/hr) ; F = Fetch or straight length of water expense (in km)

The maximum pressure intensity due to wave action may by given by

$$p_w = 2.4\,\gamma_w\,.h_w \qquad \text{(acts at } \frac{h_w}{2} \text{ metres above the still water surface).}$$

The pressure distribution may be assumed to be triangular, of height $\frac{5h_w}{3}$, as shown in figure.



Hence, the total force due to wave action,

$$P_w = \frac{1}{2}\ (2.4\ \gamma_w.h_w).\frac{5}{3}.h_w = 2\ .\ \gamma_w\ .h_w^2 = 2 \times 9.81\,h_w^2 = 19.62\,h_w^2 \ \text{kN/m}$$

This force acts at a distance $\frac{3}{8}h_w$ above the reservoir surface.

(6) Ice Pressure

In cold countries, ice gets formed on the water surface of the reservoir. The dam face has to resist the thrust exerted by the expanding or melting ice. This force acts linearly along the length of the dam and at the reservoir level.

(7) Weight of the dam

- The weight of the dam body and its foundation is the major resisting force.

The design of a gravity dam should be checked for two cases,

(i) When reservoir is full; and (ii) When reservoir is empty.

(i) Reservoir full case

- Here, the major forces acting are : *weight of the dam, external water pressure, uplift pressure,* and *earthquake forces* in serious seismic zones.
- The minor forces are : *silt pressure, ice pressure* and *wave pressure.*
- A situation will never arise when all the forces are taken together.

USBR has classified the normal load combinations and extreme load combinations.

(a) *Normal Load Combinations*

- Water pressure upto normal *pool level, normal uplift, silt pressure* and *ice pressure.* This class of loading is taken when ice force is serious.
- Water pressure upto *normal pool level, normal uplift, earthquake forces,* and *silt pressure.*
- Water pressure upto maximum *reservoir level (maximum pool level), normal uplift,* and *silt pressure.*

(b) *Extreme Load Combinations*

Water pressure due to *maximum pool level, extreme uplift pressure* without any reduction due to drainage and silt pressure.

(ii) Reservoir empty case

- Empty reservoir without earthquake forces are computed for determining bending diagrams, etc. for reinforcement design, for grouting studies or other purposes.
- Empty reservoir with a horizontal earthquake force produced towards the u/s has to be checked for non-development of tension at toe.

MODES OF FAILURE AND CRITERIA FOR STRUCTURAL STABILITY OF GRAVITY DAMS

A gravity dam may fail in the following ways :

(1) By overturning about the toe. (2) By crushing.

(3) By development of tension . (4) By shear failure called sliding.

Note: The failure may occur at the foundation plane (i.e. at the base of the dam) or at any other plane at higher level.

(1) Over-turning

- If the resultant of all the force acting on a dam at any of its sections passes outside, the dam shall rotate and overturn about the toe. But actually, dam fails much earlier by compression.
- The ratio of the righting moments about toe (anti clockwise) to the over turning moments about to (clockwise) is called the factor of safety against overturning.
- Its value, generally varies between 2 to 3.

(2) Compression or Crushing

A dam may fail by the failure of its materials, i.e., the compressive stresses produced may exceed the allowable stresses, and the dam-material may get crushed.

The vertical direct stress distribution at the base is given by the equation.

$$p = \text{Direct stress} + \text{Bending stress.}$$

$$\therefore \quad p_{max/min} = \frac{\sum V}{B} \pm \frac{M}{I} y = \frac{\sum V}{B} \pm \frac{\sum V.e}{B^2/6} = \frac{\sum V}{B}\left[1 \pm \frac{6e}{B}\right]$$

or

$$\boxed{p_{max/min} = \frac{\sum V}{B}\left[1 \pm \frac{6e}{B}\right]}$$

where, e = eccentricity of the resultant force from the centre of the base.

$\sum V$ = Total vertical force ; B = Base width.

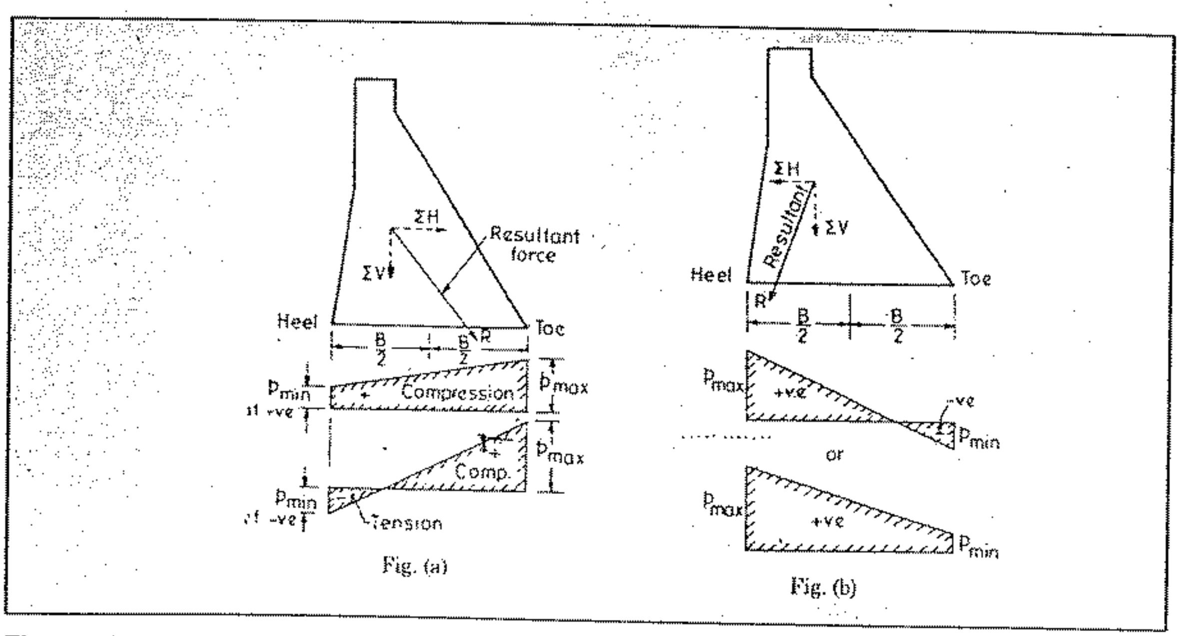


Fig. (a)

Fig. (b)

The maximum stress i.e., p_{max}, will be produced on the end which is nearer to the resultant, as shown in figure (a) and (b).

If p_{min} comes out to be negative, it means that tension shall be produced at the appropriate end.

If p_{max} exceeds the allowable compressive stress of dam material [generally taken as 3000 kN/m^2 for concrete], the dam may crush and fail by crushing.

(3) Tension

- Masonry and concrete gravity dams are usually designed in such a way that no tension is developed anywhere, because these materials cannot withstand sustained tensile stresses. If subjected to such stresses, these materials may finally crack.
- However, for achieving economy, certain amount of tension may be permitted under worst loading condition. This may be permitted because of the fact that such worst loading conditions shall occur only momentarily for a little time.
- The maximum permissible tensile stress for high concrete gravity dams under worst loading condition may be taken as 500 kN/m^2.
- In order to ensure that no tension is developed anywhere, we must ensure that p_{min} is at the most equal to zero.

Since $$p_{\substack{max\\min}} = \frac{\sum V}{B}\left[1 \pm \frac{6e}{B}\right]$$

$$p_{min} = \frac{\sum V}{B}\left[1 - \frac{6e}{B}\right]$$

If $$p_{min} = 0, \text{ then } \frac{\sum V}{B}\left[1 - \frac{6e}{B}\right] = 0$$

or $$1 - \frac{6e}{B} = 0 \quad \text{or} \quad e = \frac{B}{6}$$

Hence, maximum value of eccentricity that can be permitted on either side of the centre is equal to $\frac{B}{6}$. Hence, the middle third rule should be followed.

(iv) Sliding

- Sliding will occur when the net horizontal force ($\sum H$) above any plane in the dam or at the base of the dam exceeds the frictional resistance developed at that level.
- The friction developed between two surface = $\mu \sum V$

  where, $\sum V$ = algebraic sum of all the vertical forces whether upward or downward

  μ = coefficient of friction between the two surfaces.
- For no sliding to takes place, external horizontal forces must be less than the shear resistance

  i.e., $$\sum H < \mu \sum V. \quad \Rightarrow \quad \frac{\mu \sum V}{\sum H} > 1$$

  $\therefore$ F.S.S. (Factor of safety against Sliding) = $\frac{\mu \sum V}{\sum H}$.

✣ *In low dam, the safety against sliding should be checked only for friction, but in high dams, for economical designs, the shear strength of the joint, which is an additional shear resistance, must also be considered. If this shear resistance of the joint is also considered, then the equation for factor of safety against sliding which is measured by shear friction factor (S.F.F) becomes.*

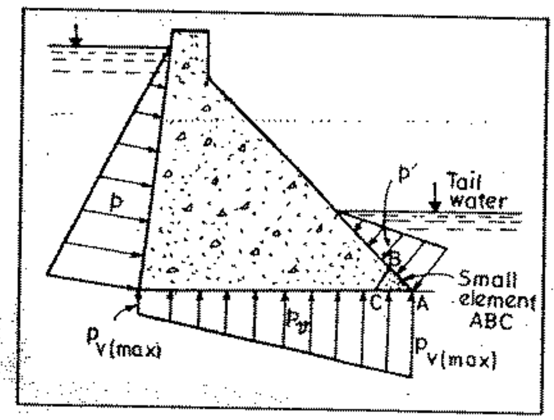
$$\boxed{\text{S.F.F.} = \frac{\mu \sum V + B.q}{\sum H}}$$

where, B = width of the dam at the joint ; q = Average shear of the joint which varies from about 1400 kN/m² for poor rocks to about 4000 kN/m² for good rocks. The value of μ generally varies from 0.65 to 0.75.

Note: During the construction of a dam horizontal joints have to be left. The shear strength of ,these joints should be made as good as possible by ensuring better bond between the two surface. For small dams, where quality control is less, this shear strength of the joint is not taken into account at all, while determining the shear friction factor or factor of safety against sliding.

PRINCIPAL AND SHEAR STRESSES

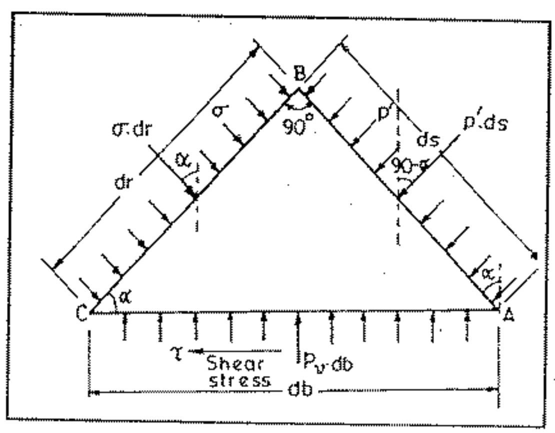
- The vertical stress intensity, p_{max} or p_{min} is not the maximum direct stress produced anywhere in the dam. The maximum normal stress will be the major principal stress that will be generated on the major principal plane.
- When the reservoir is full, the vertical direct stress is maximum at the toe as the resultant is nearer to the toe,
- Consider a small element ABC near the toe of the dam. The element is so small that the stress intensities may be assumed to be uniform on its faces.



Let the d/s face of the dam be inclined at an angle α to the vertical. This face of the dam will act as a principal plane (because the water pressure p' acts at right angles to the face, and also there is no shear stress acting on this plane). Since the principal planes are at right angles to each other. The plane BC drawn at right angles to the face AB will be the second principal plane. Let the stress acting on this plane be σ.

Principle stress

Let *ds, dr* and *db* be the lengths of AB, BC and CA respectively. p' is the intensity of water pressure on face AB and p_v is the intensity of vertical pressure on face AC, and σ is the intensity of normal stress (principal stress) on face BC. Considering unit length of the dam, the forces acting on the faces AB, BC and CA are p' *ds*, σ *dr* and p_v. *db* respectively.



Resolving all the forces in the vertical direction, we get

$$p'.ds.\sin\alpha + \sigma \,.\, dr \,.\, \cos\alpha = p_v \,.\, db,$$

Now $\dfrac{ds}{db} = \sin\alpha,$ or $ds = db \,.\, \sin\alpha$

$\dfrac{dr}{db} = \cos\alpha,$ or $dr = db \,.\, \cos\alpha,$

$$\therefore \quad p'.(db.\sin\alpha).\sin\alpha + \sigma.(db \,.\, \cos\alpha)\cos\alpha = p_v \,.\, db$$

or $\quad p' \,.\, \sin^2\alpha + \sigma \,.\, \cos^2\alpha = p_v. \quad$ or $\quad \sigma = \dfrac{p_v - p'.\sin^2\alpha}{\cos^2\alpha}$

or $$\boxed{\sigma = p_v.\sec^2\alpha - p'\tan^2\alpha} \quad \text{... (i)}$$

For σ to be maximum, p' should be zero, i.e. when there is no tail water; then in such a case

$$\sigma = p_v.\sec^2\alpha \quad \text{... (ii)}$$

Since $\sec^2\alpha$ is always more than 1, it follows, that σ will be more than p_v. This value of normal stress (maximum stress produced anywhere in the body of the dam) should not be allowed to exceed the maximum allowable compressive stress of dam material.

✢ *If the hydrodynamic pressure (p_e') exerted by the tail water during an earthquake moving towards the reservoir is also considered, than the net pressure on the face AB will be $(p' - p_e')$, because the effect of this earthquake will be to reduce the tail water pressure.*

The principal stress (σ) can then be given by

or $$\boxed{\sigma_{\text{at toe}} = p_v.\sec^2\alpha - (p' - p_e')\tan^2\alpha} \quad \text{... (iii)}$$

Shear stress

A shear stress τ will act on the face CA on which the vertical stress is acting. Resolving all the forces in the horizontal direction, we get.

$$\sigma.dr\sin\alpha - p'.ds.\cos\alpha = \tau_0.db$$

or $$\sigma.(db.\cos\alpha)\sin\alpha - p'(db.\sin\alpha)\cos\alpha = \tau_0.db$$

or $$\sigma.\sin\alpha\cos\alpha - p'\sin\alpha\cos\alpha = \tau_0$$

or $$\tau_0 = (\sigma - p')\sin\alpha\cos\alpha$$

Substituting the value of σ from equation (i), we get

$$\tau_0 = \left[p_v\sec^2\alpha - p'\tan^2\alpha - p'\right]\sin\alpha\cos\alpha$$

$$\tau_0 = [p_v\sec^2\alpha - p'(1 + \tan^2\alpha)]\sin\alpha\cos\alpha = \left[(p_v - p')\sec^2\alpha\right]\sin\alpha\cos\alpha$$

or $$\tau = (p_v - p')\sec^2\alpha.\sin\alpha.\cos\alpha$$

or $$\boxed{\tau = (p_v - p')\tan\alpha} \quad \text{... (iv)}$$

Neglecting tail water, shear stress is given by $\tau_0 = p_v.\tan\alpha$ ... (v)

✢ *If the effect of hydrodynamic pressure produced by an earthquake moving towards the reservoir, is also considered, the equation for shear stress on a horizontal plane near the toe becomes,*

$$\tau_0 = [p_v - (p' - p_e')]\tan\alpha$$

STABILITY ANALYSIS

- The stability of gravity dam can be approximately analysed by *two dimensional gravity method* and can be precisely analysed by *three dimensional methods* (such as slab analogy method, trial load twist method, or by experimental studies on models).

GRAVITY METHOD OR 2D STABILITY ANALYSIS

- The preliminary analysis can be made easily by isolating a typical cross-section of the dam of a unit width. This section is assumed to behave independently of the adjoining section.
- The dam is considered to be composed of a number of cantilevers of which is 1 m thick and each of which acts independent of the other.
- No loads are transferred to the abutments by beam action.
- The foundation and the dam behave as a single unit; the joint being perfect.
- The materials in the foundation and body of the dam are isotropic and homogeneous.
- The stresses developed in the foundation and body of the dam are within elastic limits.
- No movements of the foundations are caused due to transference of loads
- Small openings made in the body of the dam do not affect the general distribution of stresses and they only produce local effects as per *St. Venant's principal.*

Two dimensional analysis can be carried out either

(1) Analytically (2) Graphically.

(1) Analytical Method

The stability of the dam can be analysed in the following steps :

(a) Consider unit length of the dam.

(b) Calculate the magnitude and directions of all the vertical forces acting on the dam and their algebraic sum

(c) Similarly, calculate all the horizontal forces and their algebraic sum, i.e., $\sum H$.

(d) Determine the lever arm of all these forces about the toe.

(e) Determine the moments of all these forces about the toe and find out the algebraic sum of all those moments

(f) Find the location of the resultant force by determining its distance from the toe, $\bar{x} = \dfrac{\sum M}{\sum V}$.

(g) Find out the eccentricity (e) of the resultant (R) using $e = \dfrac{B}{2} - \bar{x}$. It must be less than B/6 in order to ensure that no tension is developed anywhere in the dam.

(h) Determine the vertical stresses at the toe and heel using equation, *i.e.*, $p_v = \dfrac{\sum V}{B}\left[1 \pm \dfrac{6e}{B}\right]$

(i) Determine the maximum normal stresses, *i.e.*, principal stresses at the toe and the heel using equations (iii) to (v). They should not exceed the maximum allowable values. The crushing strength of concrete varies between 1500 to 3000 kN/m² depending upon its grade M15 to M30.

(j) Determine the factor of safety against overturning as equal to

FOS $= \dfrac{\sum \textit{Stabilising moment}(+)}{\sum \textit{Disturbing moment}(-)}$; (+ve for anti-clockwise moment and –ve for clockwise moment).

(k) Determine the factor of safety against sliding, using sliding factor.

$$\text{Sliding factor} = \frac{\mu \sum V}{\sum H}.$$

$$\text{Shear friction factor (S.F.F.)} = \frac{\mu \sum V + bq}{\sum H}.$$

Note : (i) Sometimes stresses are found by ignoring uplift. (ii) Sliding factor must be greater than unity and S.F.F. must be greater than 3 to 5. The analysis should be carried out for reservoir full case as well as for reservoir empty case.

ELEMENTARY PROFILE OF A GRAVITY DAM

- The elementary profile of a dam, subjected only to the external water pressure on the u/s side, will be a right-angled triangle having zero width at the water level and a base width (B) at bottom.
- In other words, the shape of such a profile is similar to the shape of the hydrostatic pressure distribution.
- When the *reservoir is empty*, the only single force acting on it is the self weight (W) of the dam and it acts at a distance B/3 from the heel.
- This is the maximum possible stabilising moment about the toe without causing tension at toe.

$$p_{max/min} = \frac{\sum V}{B}\left[1 \pm \frac{6e}{B}\right]$$

Here, $\sum V = W$, and $e = \dfrac{B}{6}$.

$$\therefore \quad p_{max/min} = \frac{W}{B}\left[1 \pm \frac{6.B}{B.6}\right]$$

or $\quad p_{max} = \dfrac{2W}{B}$ and $\quad p_{min} = 0.$

Hence, the maximum vertical stress equal to $\dfrac{2W}{B}$ will act at the heel ($\because$ the resultant is nearer the heel) and the vertical stress at toe will be zero.

When the *reservoir is full*, the base width is governed by:

(i) The resultant of all the forces, i.e. P, W and U passes through the outer most middle third point (i.e. lower middle third point)

(ii) The dam is safe in sliding.

For the 1st condition to be satisfied, we proceed as follows

Taking moment of all the forces about the lower middle third point (i.e. the point through which resultant is passing), we get

$$W\left(\frac{B}{3}\right)-U\left(\frac{B}{3}\right)-P\frac{H}{3}=R\times 0$$

or $$(W-U)\frac{B}{3}-P\frac{H}{3}=0 \quad \text{... (i)}$$

But, $$W=\frac{1}{2}B\times H\times 1\times S_C\times \gamma_w$$

where, ... S_C = Sp. gravity of the material of the dam.

γ_w = unit wt. of water = 9.81 kN/m³.

Let the uplift at the heel be $C.\gamma_w.H.$, where C is a constant which according to U.S.B.R. recommendation is taken equal to 1.0 in calculation and will be equal to zero when no uplift is considered.

$$\therefore \quad U=\left(\frac{1}{2}C.\gamma_w.H\right)B \quad \text{and} \quad P=\frac{1}{2}\gamma_w.H.H=\frac{\gamma_w H^2}{2}$$

$\therefore$ Equation (i) becomes

$$\left[\frac{1}{2}B.H.S_c.\gamma_w-\frac{1}{2}C.\gamma_w.H.B.\right]\frac{B}{3}-\frac{\gamma_w H^2}{2}.\frac{H}{3}=0$$

or $$\frac{B}{3}\times\frac{1}{2}B.H.\gamma_w.[S_c-C]=\frac{\gamma_w H^3}{6}$$

or $$B^2(S_c-C)=H^2$$

$$\boxed{B=\frac{H}{\sqrt{S_c-C}}} \quad \text{.... (ii)}$$

Note: If B is taken equal to or greater than $\frac{H}{\sqrt{S_c-C}}$; no tension will be developed at the heel with full reservoir.

when $C = 1$; $$\boxed{B=\frac{H}{\sqrt{S_c-1}}} \quad \text{... (iii)}$$

If uplift is not considered, $$\boxed{B=\frac{H}{\sqrt{S_c}}} \quad (\because \quad C=0) \quad \text{... (iv)}$$

For the 2nd condition to be satisfied, we proceed as follows :

The frictional resistance $\mu\Sigma V$ or $\mu(W-U)$ should be equal to or more than the horizontal forces $\Sigma H = P$.

or $\mu(W-U) \ge P$ or $\mu\left(\frac{1}{2}BH.S_c.\gamma_w - \frac{1}{2}C.\gamma_w.H.B\right) \ge \frac{\gamma_w H^2}{2}$

or $\mu(S_c - C)\frac{1}{2}.B.H.\gamma_w \ge \frac{\gamma_w H^2}{2}$

$\mu(S_c - C)B \ge H$

$$\boxed{B \ge \frac{H}{\mu(S_c - C)}} \quad \text{... (v)}$$

If $C = 1$; $\boxed{B = \frac{H}{\mu(S_c - 1)}}$ ... (vi)

If $C = 0$, (no uplift is considered) then, $\boxed{B \ge \frac{H}{\mu S_c}}$ ... (vii)

$\therefore$ For all practical purposes, the base width may be taken as $\frac{H}{\sqrt{S_c}}$

The vertical stress distribution when reservoir is full is given as : $p_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$

where, $\Sigma V = W - U = \left(\frac{1}{2}B.H.1.S_c.\gamma_w - \frac{1}{2}C.\gamma_w.H.B\right) = \frac{1}{2}B.\gamma_w.H.[S_c - C]$; $e = \frac{B}{6}$.

$$p_{max/min} = \frac{\frac{1}{2}.B.\gamma_w.H(S_c - C)}{B}\left[1 \pm \frac{6B}{6B}\right]$$

Maximum stress will occur at toe, because the resultant is near the toe.

$\therefore$ p_{max} at toe $= \frac{1}{2}\gamma_w.H(S_c - C)2.0 = \gamma_w H(S_c - C)$

$$\boxed{p_v \text{ at toe} = \gamma_w H(S_c - C)} \quad \text{...(viii)}$$

p_{min} at heel $= 0$.

The principal stress near the toe (σ) which is the maximum normal stress in the dam, is given

$$\sigma = p_v \sec^2 a - p' \tan^2 a$$

when there is no tail water *i.e.*, $p' = 0$; $\sigma = p_v \sec^2 a$

σ at toe, with full reservoir in elementary profile

$$= \gamma_w H(S_c - C)\sec^2\alpha = \gamma_w H(S_c - C)[1 + \tan^2\alpha] = \gamma_w H(S_c - C)\left[1 + \frac{B^2}{H^2}\right]$$

But $B = \frac{H}{\sqrt{S_c - C}}$ or $\frac{B^2}{H^2} = \frac{1}{S_c - C}$

$\therefore$ $\boxed{\sigma = \gamma_w H(S_c - C + 1)}$ ... (ix) when $\quad C = 1, \quad S_C = 2.4$

$$\sigma = \gamma_w H\left(\frac{2.4-1+1}{2.4-1}\right) = \frac{2.4}{1.4}\gamma_w H = 1.71\gamma_w H.$$

The shear stress τ_0 at a horizontal plane near the toe is given by the equation (vii) as :

$$\tau_0 = (p_v - p') \tan\alpha$$

If $\quad p' = 0; \quad \tau_0 = p_v \tan\alpha$

But $\quad p_v = \gamma_w H\ (S_c - C) \quad$ from Equation (viii)

$\therefore \quad \tau_0 = \gamma_w H\ (S_c - C) \tan\alpha$

or $$\tau_0 = \gamma_w . H(S_c - C)\frac{B}{H} = \gamma_w H(S_c - C)\frac{1}{\sqrt{S_c - C}}$$

or $$\boxed{\tau_0 = \gamma_w H\sqrt{S_c - C}} \quad \text{... (x)}$$

HIGH AND LOW GRAVITY DAMS

The principal stress calculated for an elementary profile is given by Equation (xvi), i.e., $\sigma = \gamma_w H\ (S_c - C + 1)$. Value of principal stress varies only with H (as all other factors are fixed).

To avoid dam failure by crushing, the value of σ should be less than or at the most equal to the maximum allowable compressive stress of dam material. If f represents the allowable stress of the dam material, then the maximum height (H_{max}) which can be obtained in an elementary profile, without exceeding the allowable compressive stress of the dam material, is given as :

$$f = \gamma_w H(S_c - C + 1)$$

$$\boxed{H = f/\gamma_w(S_c - C + 1)} \quad \text{... (i)}$$

When C = 0 (i.e., when uplift is neglected), the lowest value of H will be obtained. Hence, for determining the limiting height and to be on a safer side, uplift is neglected.

Max. possible height, $$\boxed{H_{max} = \frac{f}{\gamma_w(S_c + 1)}} \quad \text{... (ii)}$$

Hence, if the height of a dam having an elementary profile of a triangle, is more than that given by the equation (ii), the max. compressive stress generated will exceed the allowable value.

Hence, a *low gravity dam* is the one whose height is less than that given by Equation (ii). If the height of the dam is more than this, it is known as a *high gravity dam*.

EARTHEN DAMS

- Earthen dams are the most ancient type of embankments. They can be built with the natural materials with very less processing and use of primitive equipment.
- Modern developments in earth moving equipments have made the cost of carriage and dumping of the dam materials very less compared to ancient days.

CAUSES OF FAILURE OF EARTHEN DAMS

The various causes of failure of earth dams may be grouped into the following three categories

(1) Hydraulic failures (2) Seepage failures (3) Structural failures.

(1) Hydraulic failures

About 40% of earth dam fails due to these causes. The failure may occur due to the following reasons:

(i) By overtopping : The water may overtop the dam, if the design flood is underestimated or if spillway is of insufficient capacity or if the spillway gates are not properly operated. Overtopping may also be due to insufficient free board and settlement of foundation and dam.

(ii) Erosion of u/s face : The waves developed near the top water surface due to the winds, may notch out the soil from the u/s face and may even, sometimes, cause the slip of the u/s slope.

(iii) Cracking due to frost action : Frost in the upper portion of the dam may cause heaving and cracking of the soil with dangerous seepage and consequent failure.

(iv) Erosion of d/s face by gully formation : Heavy rains falling directly over the d/s face and the erosive action of the moving water, may lead to the formation of gullies on the d/s face, ultimately leading to the dam failure.

(v) Erosion of the d/s toe : The d/s toe of the earth dam may get eroded due to two reasons, (i) erosion due to cross currents from spillway buckets (ii) erosion due to tailwater depth.

(2) Seepage Failures

Controlled seepage or limited uniform seepage is unavoidable in earth dams, but ordinarily, it does not produce any harm. However, uncontrolled or concentrated seepage through the dam body or through its foundation may lead to piping or sloughing and the subsequent failure of the dam.

- *Piping* is the progressive erosion and subsequent removal of the soil grains from within the body of the dam or the foundation of the dam.
- *Sloughing* is the progressive removal of soil from the wet d/s face. In other words, *sloughing* may be defined as the falling away of the soil mass of an earth dam.

(i) Piping through foundations : Sometimes when highly permeable cavities or fissures or strata of coarse sand of gravel are present in the foundation of the dam water may start seeping at a huge rate through them. This concentrated flow at a high gradient, may erode the soil. This leads to increased flow of water and soil, ultimately creating hollows below the foundation. The dam may sink down into the hollow so formed, causing its failure

(ii) Piping through the dam body : When the concentrated flow channels get developed in the body of the dam, soil may be removed in the same manner as in above case leading to the formation of hollows in the dam body, and subsequent subsidence of the dam.

Note: *These flow channels may develop due to faulty construction, insufficient compaction, cracks developed in embankment due to foundation settlement, shrinkage cracks, animal burrows, etc.*

(iii) Sloughing : Sloughing may take place when under the full reservoir most of the d/s portion of the dam becomes saturated and continuously remains in the same state due to which the soil mass in the d/s portion of the dam gets softened. The extensive saturation of the d/s portion of the dam may be either due to choking of filter toe drain or due to the presence of highly pervious layer in the body of the dam.

The sloughing begins when a small amount of softened soil mass at the d/s toe is eroded and produces a small slump or miniature slide. It leaves a relatively steep face which being saturated slumps again, forming a slightly higher and more unstable face. This process is continued until the remaining portion of the dam becomes so thin that it is not able to withstand the water pressure and complete failure of the dam takes place.

(3) Structural failures

The structural failure includes the failure of earth dams due to the following causes.

(*i*) Sliding of u/s and d/s slopes. (ii) Liquefaction slides

(iii) Damage caused by burrowing animals. (iv) Damage caused by water soluble materials.

(v) Damage caused by earthquake.

(i) Sliding of u/s and d/s slopes : Sliding of u/s and d/s slopes is one of the frequent causes of failure which occurs in earth dams when along any potential sliding surface the forces tending to cause sliding of the soil mass becomes greater than the forces resisting sliding.

The critical conditions during which the sliding of u/s and d/s slopes may be caused are as follows :

(a) Sliding of u/s of slope during sudden drawdown.

(b) Sliding of d/s slope during steady seepage.

(c) Sliding of u/s or d/s slope (or both) during construction.

(ii) Liquefaction slides : In flow or liquefaction slides, the saturated soil mass move under gravity like a heavy viscous fluid. These slides may occur in the foundation of an earth dam if the foundation consists of fine sand and silt or some types of clays. Similar slides may also occur in the lower portion of an earth dam consisting of loosely compacted granular soil mass. These slides are caused when a large part of the weight of the overlying soil mass is carried momentarily by the pore water of the saturated soil and very little intergranular pressure exists within the soil mass so that the shear strength of the soil mass is reduced almost to that of the liquid and it flows downward. Hence these slides are termed as *flow slides* or *liquefaction slides.*

(iii) Damage caused by burrowing animals : The burrowing animals dig holes in earth dams to make their homes through which free passage of water from the u/s to the d/s face of the dam may be developed which may ultimately lead to the failure due to piping. However, burrowing animals may cause failure of only small earth dams.

(iv) Damage caused by water soluble materials : The leaching of natural deposits of water soluble materials (such as gypsum, iron oxide etc.,) from abutments and foundations may result in creating cavities and flow channels which may cause excessive settlement of the dam. Further, the soluble material leached from the natural soil may get deposited in the toe filters and the drains and thus tend to plug them .

The soil mass of the dam may also contain water soluble materials. However, no serious damage may be caused to the dam if only small quantities of soluble materials are present in the soil mass.

(v) Damage caused due to earthquake : Some of the damages which may be caused to earth dams due to earthquake are as follows.

(i) Longitudinal cracks at the top of the dam.

(ii) Liquefaction of loose and saturated soil mass in the lower portion of the dam

(iii) Cracking of central core wall leading to leakage and piping failure.

(iv) Generating large waves on the water surface in the reservoir thus causing overtopping.

(v) Shear failure at the base of the dam.

MEASURES TO CONTROL SEEPAGE THROUGH EARTH DAMS AND THEIR FOUNDATIONS

The water seepage through the body of an earth dam and through its foundation may have the following adverse effects.

(i) Loss of water (ii) Dislodging of the soil due to erosive forces leading to piping..

(iii) Reduction of slope stability which may result in the failure of the dam due to sliding of slopes.

(iv) Local sloughing resulting in the failure of the dam.

In order to prevent the adverse effect of the water seeping through the dam and its foundation, it is essential to adopt certain measures to control the seepage.
The various seepage control measures usually adopted are of the following two types:

(1) Measures adopted for reducing the quantity of seepage.

(2) Measures adopted for safe drainage of the seeping water.

(1) Measures adopted for reducing the quantity of seepage

Different measures adopted to reduce the quantity of seepage through the dam and through the foundation are :

(i) In dam

The measures adopted to reduce the quantity of seepage through the dam is the provision of a core of impervious soil, a core wall of concrete or masonry within the body of dam. Out of these core of impervious soil also known as impervious zone is commonly used.

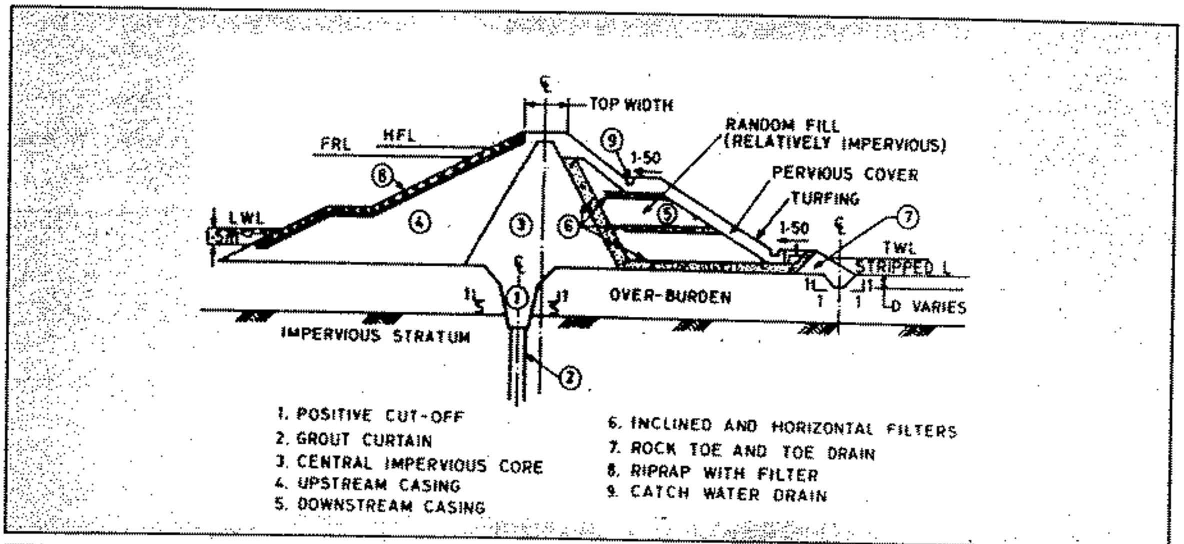
Impervious core

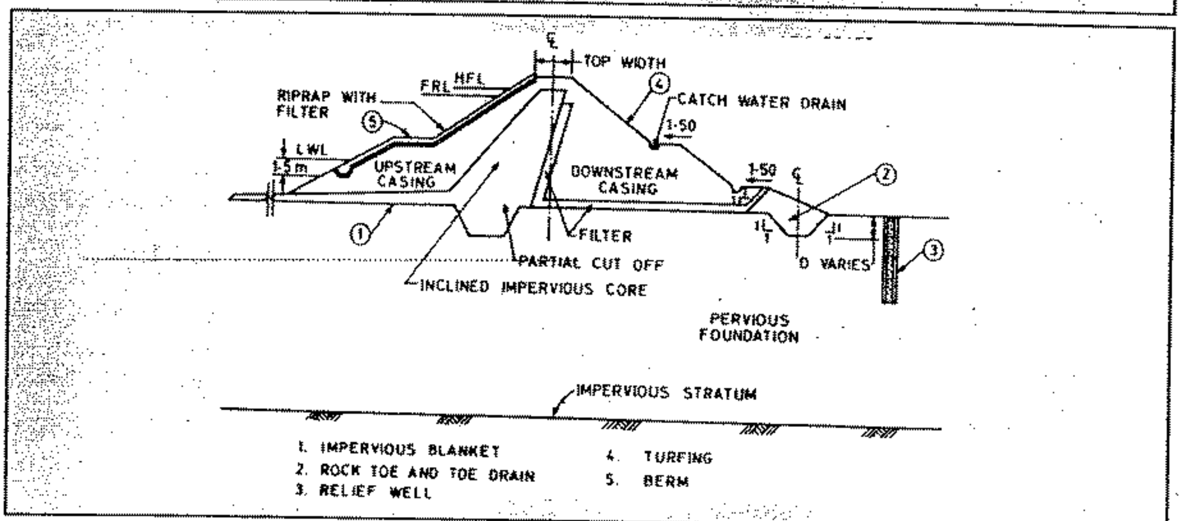
Impervious core forms a barrier within the body of the dam for seeping water.

These can be classified in following two types

(i) Central vertical cores (ii) U/S inclined cores

- The type of core to be provided depends on the availability of material, topography of site foundation condition, etc.





- The main advantage of ***central vertical core*** is that is provides higher pressures at the contact between the core and the foundation thus reducing the possibility of leakage and piping.
- For a given quantity of impervious soil, the thickness of a vertical core is slightly greater than the thickness of an inclined core.
- The main adv. of the ***u/s inclined core*** is that the d/s portion of the dam can be constructed first and the core placed later.

(ii) In foundation

If an impervious stratum such as rock, clay, etc., is available close to the bed of the river then no specific measure is required for reducing the seepage through the foundation and only grouting and surface treatment may be carried out if necessary. However, if the foundation consists of alluvial deposits of pervious sands and gravels with an impervious stratum at a greater depth below the bed of the river then the following measures may be adopted to reduce the seepage of water through the foundation.

(a) Cutoff. (b) U/S impervious blanket. (c) D/S berm

(a) Cutoff

- A cutoff is a vertical impermeable barrier provided within the pervious foundation of an earth dam to reduce the seepage of water through the foundation.
- It usually extends down from the base of the impervious core provided within the body of the dam. On the basis of the length of the cutoffs, these may be classified as follows

  (i) Full cutoff *(ii) Partial cutoff*

Full cutoff

- A full cutoff is a vertical barrier provided for the entire depth of the pervious foundation below the earth dam.
- It joins the base of the impervious core of the dam with the impervious stratum in the foundation.
- Such a cutoff is provided where the impervious stratum is available in the foundation at a reasonable depth below the base of the dam. The various type of full cutoff usually adopted are as follows:

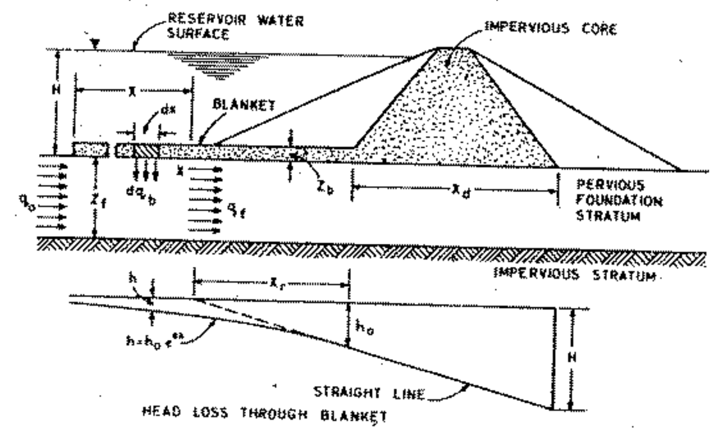
(i) Positive cutoff trench (ii) Concrete cutoff wall; (iii) Grout curtain.
(iv) Steel sheet pile. (v) Slurry trench cutoff.

Partial cutoff

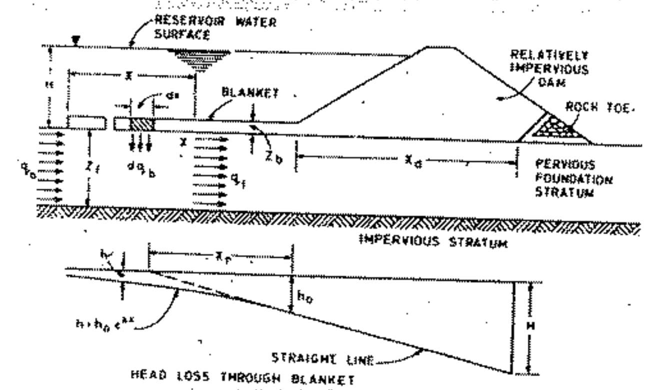
- In many cases, the impervious stream may exist at such a large depth below the bed of the river that it would be extremely expensive to provide full cutoff extending upto the impervious stratum. In such case, a partial cutoff may be provided.
- A partial cutoff is a vertical barrier which extends down from the impervious core of the dam into the underlying pervious strata but does not reach the impervious stratum.
- A partial cutoff would be effective in reducing the quantity of seepage through the foundation if the horizontal permeability of the foundation soil is much more than the vertical permeability and the permeability decreases with depth i.e., the top strata are more pervious than the lower ones.
- Partial cutoffs may be effective in alluvial deposits which are usually stratified.
- At sites where the foundation soil is homogeneous and isotropic, a partial cutoff is rather ineffective in reducing the quantity of seepage through the foundation.

(b) U/S impervious blanket

- A horizontal blanket of impervious soil is provided on river bed on the u/s side to reduce the quantity of seepage through the pervious foundation under an earth dam.
- At some of the dam sites, a natural impervious blanket is available if there exists thin layer of impervious soil of large areal extent at the bed of river overlying a thick layer of previous soil below.
- The impervious blanket increases the length of the path of seepage under the dam and thus reduce the velocity and quantity of seepage.
- The impervious blanket should be connected to the impervious core of the dam.



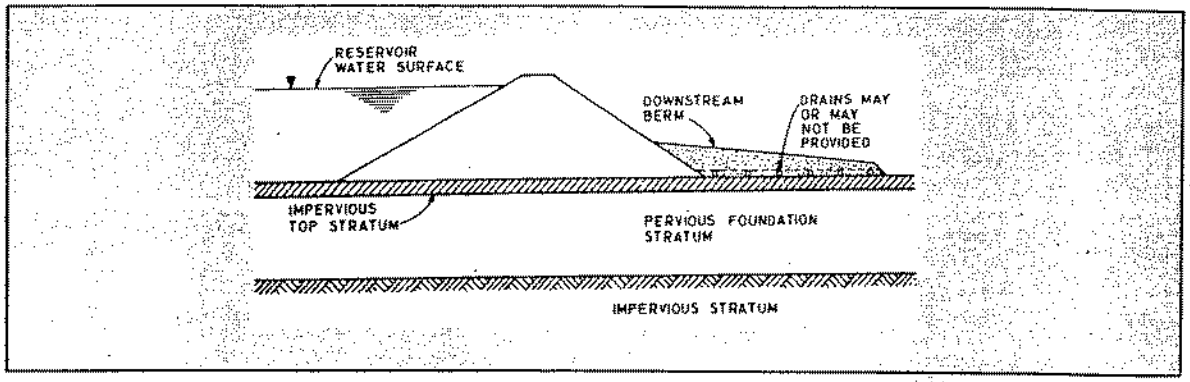
(a)



- The soil used for impervious blanket should have far less permeability then that of the foundation soil.
- The necessary thickness and length of the blanket depend on the permeability of the soil of the blanket, thickness of the pervious foundation, and the maximum depth of water in the reservoir.

(c) D/S berm

- A d/s or landside berm which extends beyond the d/s toe of the dam is provided to reduce the quantity of seepage by increasing the seepage path.
- It provides some protection against sloughing of the d/s slope as a result of seepage.
- A d/s berm may be provided so that the weight of the berm plus that of the top impervious layer will be able to resist the excessive underseepage pressures if a relatively thin impervious layer (at any dam site) exists at the surface overlying a pervious stratum underneath.
- A d/s berm is also known as a d/s loading berm or simply a loading berm.



(2) Measures adopted for safe drainage of the seeping water

The various measures that are adopted for safe drainage of water seeping through the dam and through the foundation are as follows.

(i) In dam

The water seeping through the dam will be drained off if the outer zones on the u/s and d/s sides are highly pervious. However, for *homogeneous dams* and for *zoned dams* where the shells are not adequately pervious different types of drainage arrangements may be provided as indicated below.

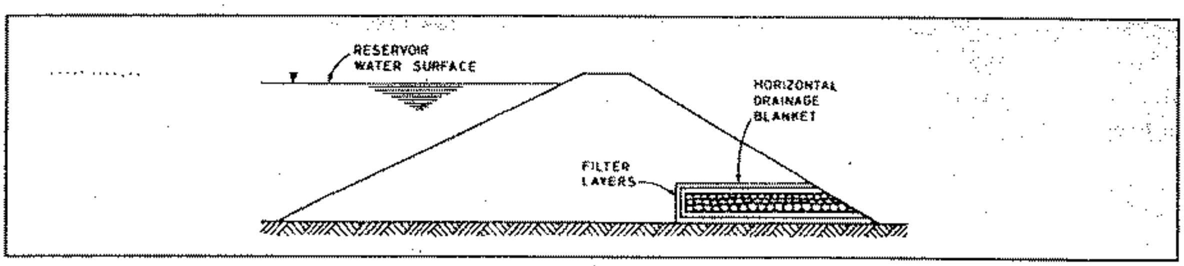
Drainage of the d/s portion

In the d/s portion of the dam, the following different types of drainage arrangements may be provided.

(a) Horizontal drainage blanket. (b) Strip drain (c) Rock toe (d) Chimney drain.

(a) Horizontal drainage blanket

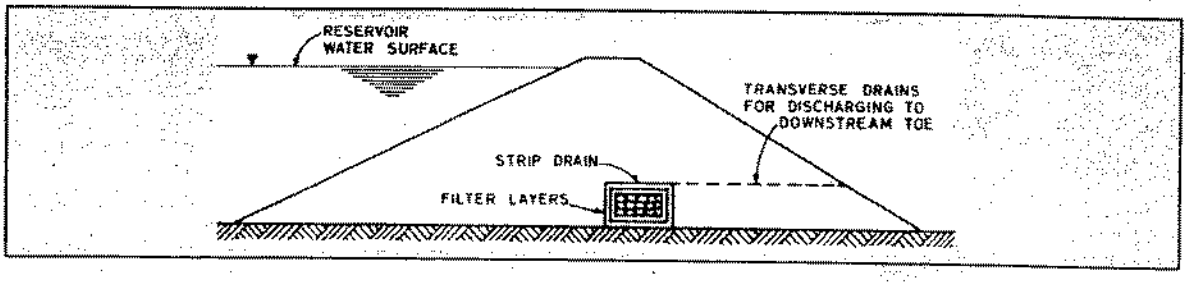
- It is provided at the base of the dam which extends from the d/s toe of the dam in the u/s direction.
- It is widely used for dams of low to moderate heights.
- The drainage blanket should be pervious enough to drain off effectively and fulfill filter criteria



- The main advantage here is that in most of the cases it will keep the seepage line farther away from the d/s slope and permit design of a dam section with slightly steeper or more stable d/s slope.
- The main disadv. here is that it does'nt intercept the stratification in the dam. So, if there is a relatively pervious layer higher up in the dam the water may seep through it and discharge on the face of d/s slope which may result in surface sloughing.

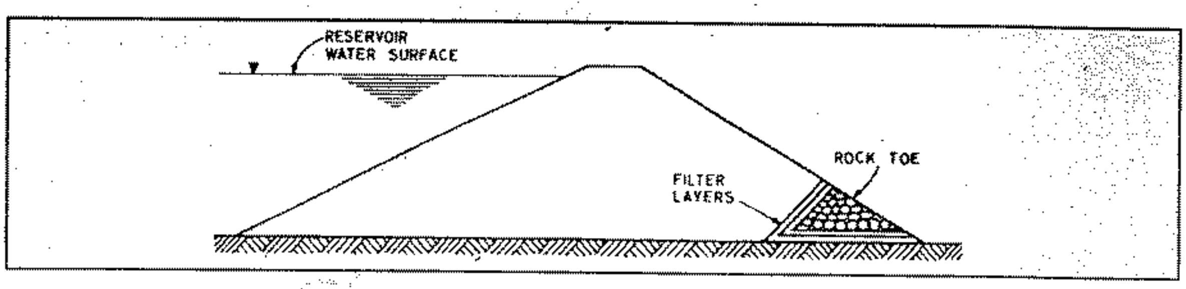
(b) Strip drain

- If there is a lack of pervious material, instead of a continuous horizontal drainage blanket a strip drain may be provided.
- A strip drain is a narrow drain, provided inside the dam parallel to the axis of the dam.



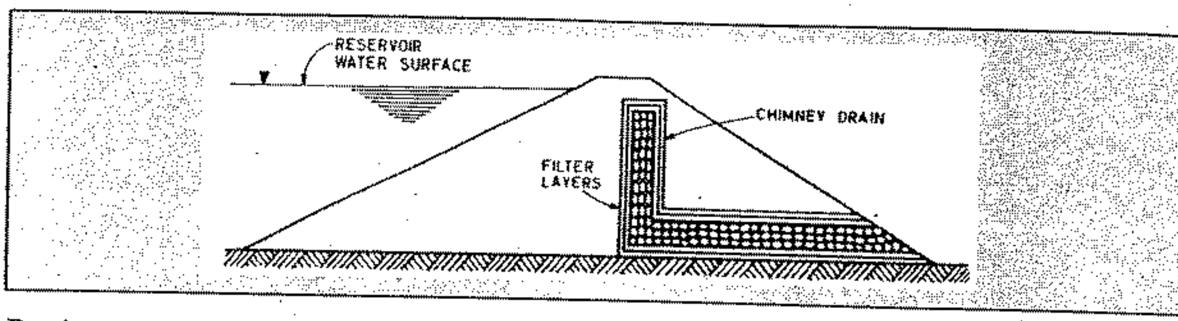
(c) Rock toe

- A rock toe is provided at the d/s end of an earth dam.
- A rock toe consists of stones of size varying from 150 mm to 200mm
- A filter is required to be provided between the rock toe and the soil mass in the dam as well as in the foundation, if is pervious.
- The rock toe is also suitable for the dams of low to moderate heights.
- The height of the rock toe is generally kept between 1/3rd to 1/4th of the height of the dam.
- A rock toe also improves the stability of the d/s slope of the dam.
- The disadv. of a rock toe is that it does not extend far enough into the dam and the seepage line is usually very close to the d/s slope.



(d) Chimney drain

- A chimney drain extends vertically up into the dam and intercepts all layers of the dam in the seepage zone.
- A chimney drain prevents the emergence of seeping water on the d/s slope even if some layers of higher permeability exist in the dam.
- A chimney drain helps in reducing pore pressure both during construction and during sudden drawndown.
- Chimney drains may be vertical or slanting u/s to follow the core of the dam



Drainage of u/s portion

For the drainage of the u/s portion of the dam, a horizontal drainage blanket or a chimney drain may be provided.

Note : *A continuous horizontal drainage blanket is preferred to a strip drain, because due to choking of an individual transverse drain a significant length of the dam would become undrained.*

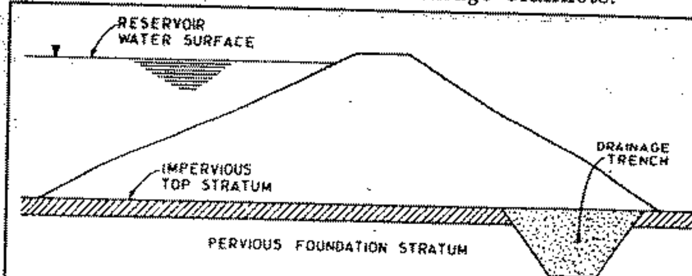
(ii) In foundation

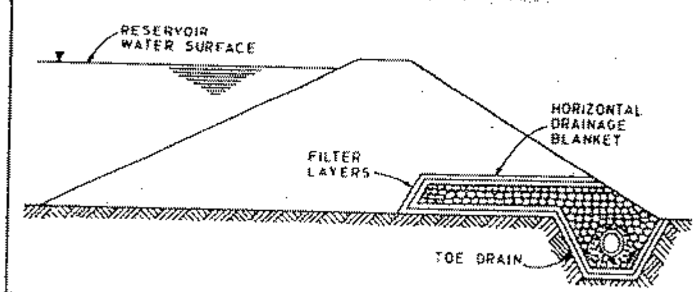
In general, for the safe drainage of water seeping through the foundation, most of the measures adopted for the drainage of water seeping through the dam will serve also for the foundation. Following measures are adopted for the safe drainage of water seeping through the foundation.

(a) Toe drains and drainage trenches (b) Relief wells (c) Vertical sand drains

(a) Toe drains and drainage trenches

- Both the *toe drain* and the *drainage trenches* are installed along the d/s toes of dam.
- The toe drains are commonly installed together with horizontal drainage blankets.
- These drains collect the seepage from the horizontal drainage blanket and lead it to an outfall pipe which discharges into the spillway stilling basin or into the river channel below the dam.
- The *toe drains* are usually made of vitrified clay
- The drain pipes are placed in trenches at a sufficient depth below the ground surface to ensure effective interception of the seepage flow.
- *Drainage trenches* are the open cut trench drains (without drain pipes) backfilled with layer of material in accordance with the filter criteria.



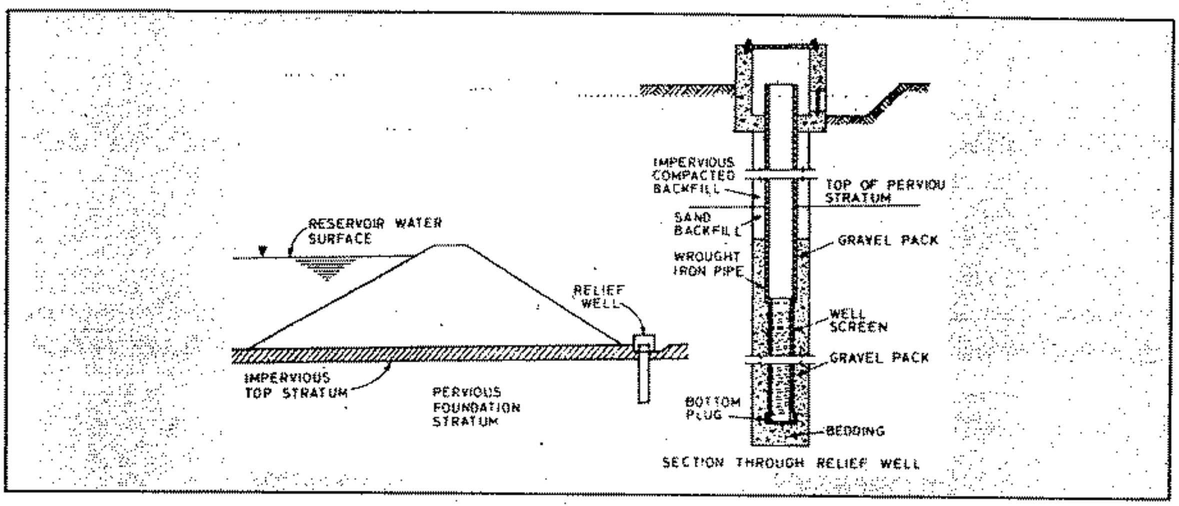


- The drainage trenches can be used to control the underseepage where a thin impervious top stratum overlies a shallow pervious stratum so that the trench can be built to penetrate the pervious stratum substantially.

- A drainage trench will usually be not effective if the underlying pervious stratum is deep and stratified. This is so because a drainage trench of any reasonable depth will attract only a small portion of the underseepage and most of the underseepage will bypass the trench.

(b) Relief wells

- Relief wells are provided at or near the d/s toe of the dam to collect water seeping through the foundation and to reduce the pore pressures in the foundation.
- They also minimise the risk of piping failure through the foundation. They are quite effective for deep pervious foundation particularly where the foundation are stratified and where permeability increases with depth.



(c) Vertical sand drains

- The vertical sand drains consist of vertical holes drilled in the foundation all along the base of the dam and filled with clean, coarse sand of high permeability.
- Path of drainage is considerably reduced by providing these drains which results in keeping the pore pressure low and accelerates the consolidation of the foundation soil.
- Quite effective for dams on soft clay foundations.
- Also act as relief wells in addition to accelerated consolidation of the foundation soil.

DIVERSION PROBLEM IN DAMS CONSTRUCTION

The diversion of river water can be accomplished in either of the following two ways.

(i) **Provision of a Diversion Tunnel:** If geological and topographical conditions are favourable, a diversion tunnel or a diversion open channel may be constructed to carry the entire flow around the dam site. The area in which construction work has to take place, is closed by cofferdams.

(ii) **By constructing the dam in two stages:** The dam is sometimes constructed in two stages. In such case flow is diverted and confined to one side of the channel by constructing a semicircular type of a coffer-dam. The construction work can be taken up in the water free zone. When the work on the lower portion of the dam on half of its length in one side of the channel gets completed, the remaining half width of the channel is closed by a coffer-dam.

CONSTRUCTION OF GALLERIES IN GRAVITY DAMS

Galleries are the horizontal or sloping openings or passages left in the body of the dam. They may run either parallel or normal to the axis of dam and are provided at various elevations.

Functions and Types of Galleries in Dams

(i) Foundation Gallery

A gallery provided in a dam may serve one or multiple purposes. ***e.g.***, a gallery provided near the rock foundations drains off the water which percolates through the foundations. This gallery is called a *foundation gallery* or a *drainage gallery*. It runs in longitudinal direction and is quite near to the u/s face of the dam.

(ii) Inspection Galleries

The water which seeps through the body of the dam is collected by means of a system of galleries provided at various elevations and interconnected by vertical shafts, etc. They provide access to the interior of the dam and are, therefore, called *Inspection galleries*.

Their main functions are summarised below:

(a) Intercept and drain off the water seeping through the dam body.

(b) Provide access to interior of the dam for controlling the behaviour of the dam.

(c) Provide enough space for carrying pipes, etc. during artificial cooling of concrete.

(d) Provide access for grouting contraction joints when this can't be done from the face.

(e) Provide access to all the outlets and spillway gates, valves, etc.

(f) Provide space for drilling and grouting of the foundations.

IMPORTANT TERMS CONNECTED WITH HYDROPOWER

(1) Normal Water Level (N.W.L.) : The highest elevation of water level that can be maintained in the reservoir without any spillway discharge, either with a gated or a non-gated spillway, is known as Normal water level.

(2) Minimum Water Level (M.W.L) : The elevation of water level which produces minimum net head on the power units (i.e. 65% of design head H) is known as minimum water level.

(3) Weighted Average Level (W.A.L.) : The level above and below which equal amounts of power are developed during an average year (i.e. 50% units between N.W.L. and W.A.L., and 50% units between W.A.L. and M.W.L.) is called weighted average level.

(4) Load Factor (L.F.) : Load factor is defined as the ratio of the average load over a certain period of time to the peak load during the same period. We may have different load factors depending upon a period chosen, (i.e. daily, monthly or annual).

$$\therefore \quad \boxed{\text{Load factor} = \frac{\text{Average load over a certain period}}{\text{Peak load during that period}}}$$

An annual load factor may be defined as the ratio of the actual energy consumed during that year to the peak demand assumed to continue for one year.

$$\therefore \quad \boxed{\text{Annual load factor} = \frac{\text{Total yearly electrical units (kWh) produced}}{(\text{Max. power demand in kW}) \times 365 \times 24}}$$

(5) Capacity Factor or Plant Factor : It may be defined as the ratio of average output of the plant for a given period of time to the plant capacity.

$$\text{Capacity factor} = \frac{\text{Average Load (over a given period of time)}}{\text{Plant capacity}}$$

In other words, the capacity factor is the ratio of the energy actually produced by the plant in any given period to the energy it would be capable of producing at its full capacity during that period. *e.g.*, if a plant with a capacity of 10,000 kW were to produce 40,000 kWh when operating for 100 hours, then

$$\text{Capacity factor} = \frac{40{,}000/100}{10{,}000} = 0.4 \text{ or } 40\%$$

Note : The capacity factor and load factor would become identical, if the peak load is equal to the plant capacity

(6) Utilisation Factor or Plant Use Factor

$$\text{Utilisation factor (U.F.)} = \frac{\text{Water actually utilised for power production}}{\text{Water available in the river}}$$

It the water head is assumed to be constant, then the utilisation factor would be equivalent to

$$\text{U.F} = \frac{\text{Max. power utilised}}{\text{Max. power available}}$$

The value of utilisation factor usually varies from 0.4 to 0.9 for a hydel plant, depending upon the plant capacity, load factor and storage.

(7) Firm Power : The net amount of power which is continuously available from a plant without any break on firm or on guaranteed basis is known as firm power. This power should be available under the most adverse hydraulic conditions. The consumers can always be sure of getting this power.

(8) Power Factor : It is defined as

$$\text{Power factor} = \frac{\text{Actual power in kilowatts (kW)}}{\text{Apparent power in kilo volt-amperes (KVA)}}$$

The power factor can never be greater than unity. Its value depends upon the relationship between the inductance and resistance in the load. A load with very little inductance such as lighting bulbs, will have a power factor close to unity.

Example 1

A masonry dam 10 m high is trapezoidal in section with a top width of 1 m and bottom width of 8.25 m. The face exposed to water has a batter of 1 : 10. Calculate

(i) Factor of safety against overtuming (ii) Factor of safety against sliding
(iii) Shear friction factor.

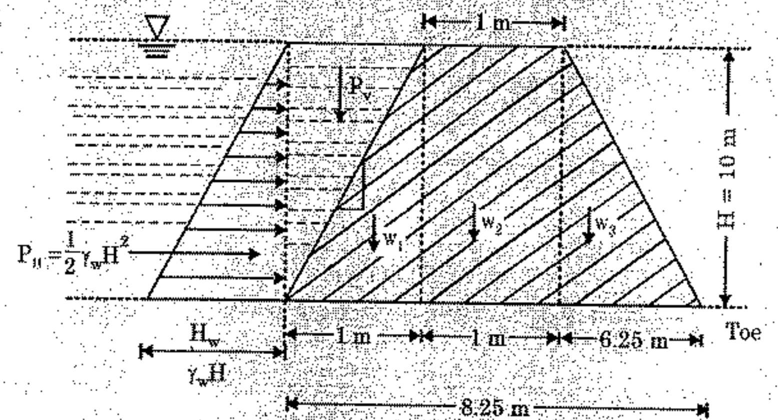
Assume coefficient of friction as 0.75, unit weight of masonry as 2240 kg/m^3.

Permissible shear stress of joint = 14 kg/cm^2. Based on the result given your remarks.

Neglect uplift pressure.

Sol. Consider the unit length of masonry dam

Let W represents the self weight of the dam. The dam section has been divided into three parts to separately calculate the weights of different parts of the section. Let, P_H be the borizontal force due to water and P_v be the vertical force due to triangular block of water.



Given, Height of the masonry dam, H = 10 m ; Top width of the masonry dam, T = 1 m

Bottom of the masonry dam, B = 8.25 m ; Batter of exposed face = 1 : 10

Coefficient of friction, $\mu = 0.75$; Unit weight of masonry = 2240 kg/m³

Permissible shear stress at the joint, $q = 14 \text{ kg/cm}^2 = 14 \times 10^4 \times 9.81 \text{ N/m}^2 = 1.373 \times 10^6 \text{ N/m}^2$

We calculate the required parameters through table (neglecting uplift forces)

Forces	Force calculation	F_v (kN)	F_H (kN)	Lever (m)	M_R (kN-m)	M_0 (kN-m)
W_1	$\left(\frac{1}{2}\times1\times10\right)\times1\times2240\times9.81$	109872	–	$\frac{1}{3}\times1+7.25=7.583$	833159	–
W_2	$(1\times10)\times1\times2240\times9.81$	219744	–	$0.5+6.25=6.75$	1483272	–
W_3	$\left(\frac{1}{2}\times6.25\times10\right)\times1\times2240\times9.81$	686700	–	$\frac{2}{3}\times6.25=4.17$	2863539	–
P_v	$\frac{1}{2}\times1\times10\times1000\times9.81$	49050	–	$8.25-\frac{1}{3}\times1=7.92$	481180	–
P_H	$(-)\frac{1}{2}\times1000\times10^2\times1\times9.81$	–	490500	$\frac{10}{3}=3.33$	–	(–)1633365
		$\sum F_V$ =1065366	$\sum F_H$ = –490500		$\sum M_R$ =5568450	$\sum M_o$ = –1633365

$\therefore$ Net moment $= \Sigma M_R - \Sigma M_0 = 5568450 - 1633365 = 3935085$ kN-m

Net vertical force $= \Sigma F_v = 1065366$ kN

$\therefore$ Distance of resultant from the toe, $\bar{x} = \dfrac{\text{Net moment}}{\text{Net vertical force}} = \dfrac{3935085}{1065366} = 3.694$ m

(i) Factor of safety against overturning$(FOS)_0 = \dfrac{\Sigma M_R}{\Sigma M_0} = \dfrac{5568450}{1633365} = 3.409 > 1.5$ **OK**

(ii) Factor of safety against sliding

$$(FOS)_s = \frac{\mu \Sigma F_v}{\Sigma F_H} = \frac{0.75 \times 1065366}{490500} = 1.63 > 1 \textbf{ OK}$$

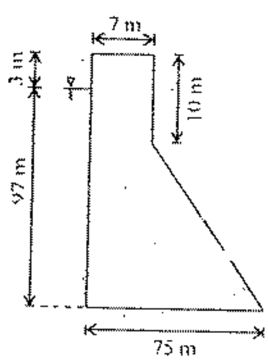
(iii) Shear friction factor $= \dfrac{\mu \Sigma F_v + (B \times 1) \times q}{\Sigma F_H} = \dfrac{0.75 \times 1065366 + 8.25 \times 1.373 \times 10^6}{490500}$

$$= 24.72 > 3$$

So, dam is safe in all the given conditions

Example 2

The following figure gives profile of a gravity dam with reservoir level as shown. If the coefficient of friction is 0.75, is the dam safe against sliding? Take weight density of concrete = 2.4 tonnes/m^3.



Sol.

Given, weight density of concrete = 2.4 tonnes/m^3 = 2400 × 9.81 = 23.544 kN/m^3

Coefficient of friction, $\mu = 0.75$

We know that, weight density of water = 1000 kg/m^3 = 1000 × 9.81 = 9.810 kN/m^3

$W_1 = (7 \times 100 \times 1) \times 23.544 = 16480.800$; $W_2 = \left(\dfrac{1}{2} \times 68 \times 90\right) \times 1 \times 23.544 = 72044.64$kN

$$U = \frac{1}{2}\times(\gamma_w H_1)\times(75\times1)) = \frac{1}{2}\times9.81\times97\times75 = 35683.88 \text{ kN}$$

$$\Sigma F_v = W_1 + W_2 - U = 52841.56 \text{ kN}$$

Horizontal force due to water on the vertical face,

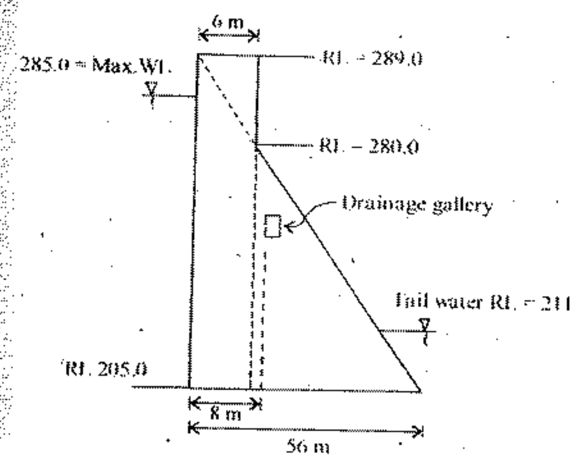
$$P_H = \frac{1}{2}\times\gamma_w\times H_1\times(H_1\times1) = 0.5\times9.81\times97^2 = 46151.14 \text{ kN}$$

$\therefore$ Factor of safety against sliding $= \dfrac{\mu\Sigma F_v}{\Sigma F_H} = \dfrac{0.75\times52815.6}{46151.14} = 0.859 < 1$ **Not OK**

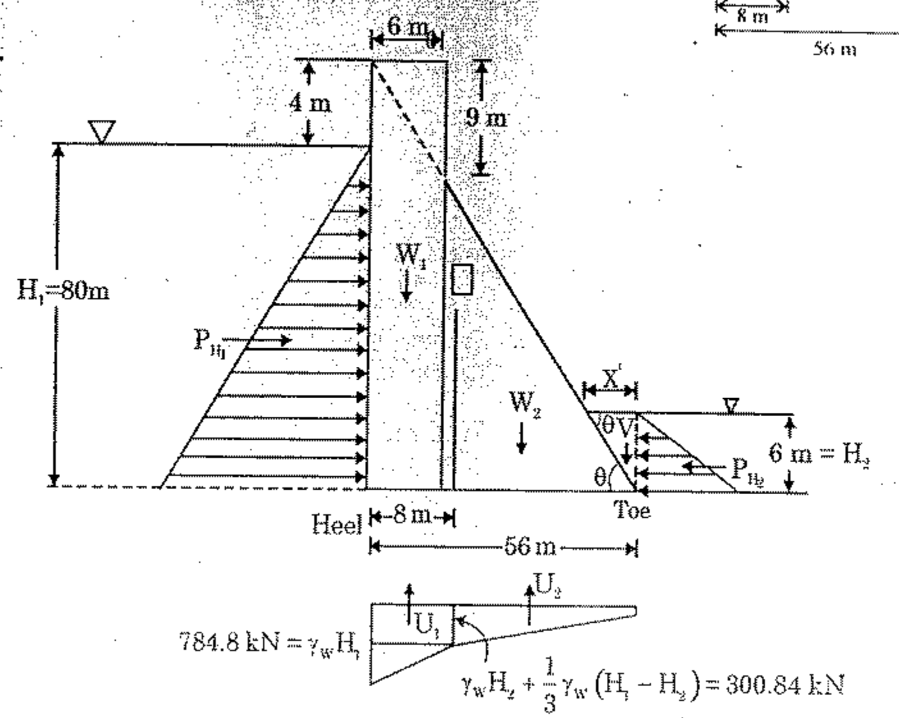
Hence, the dam is not safe against sliding due to more uplift.

Example 3

The adjoining figure below shows the section of a gravity dam (non-overflow portion) built of concrete. Calculate the maximum vertical stresses at the heel toe of the dam. Assume weight of concrete 23.5 kN/m^3. Neglect earthquake effects.



Sol.



Given, Weight of concrete = 23.5 kN/m^3

Also, we have to not take into account earthquake effects

Force	Force Calculation	F_v(kN)	F_H(kN)	LA from toe	M_R	M_0
W_1	$(6\times4)\times1\times23.5$	11844	–	$50+\frac{6}{2}=53$	627732	
W_2	$\left(\frac{1}{2}\times50\times75\right)\times1\times23.5$	44062.5	–	$50\times\frac{2}{3}=33.33$	1468750	
V	$\left(\frac{1}{2}\times4\times6\right)\times1\times9.81$	117.72	–	$\frac{1}{3}\times4=1.33$	156.96	
U_1	$(-)\frac{1}{2}\times8\times(784.8+300.84)\times1$	(–)4342.56	–	52.594		228392.6
U_2	$(-)\frac{1}{2}\times(48\times1)\times(300.84+58.86)$	(–)8632	–	29.382		253648.93
P_{H_1}	$(-)\frac{1}{2}\times(80\times1)\times784.8$		(–)31392	$\frac{1}{3}\times80=26.67$	–	837120
P_{H_2}	$\frac{1}{2}\times(6\times1)\times58.86$		176.58	$\frac{1}{3}\times6=2$	353.16	
Σ		**43048.86**	**31215.42**		**2096992**	**1319161.53**

$$\text{CG of } U_1 \text{ from toe} = 48+\frac{(300.84\times8)\times\frac{8}{2}+\frac{1}{2}\times(483.96\times8)\times\frac{2}{3}\times8}{(300.84\times8)+\frac{1}{2}\times483.96\times8} = 48+4.594 = 52.594$$

$$\text{CG of } U_2 \text{ from toe} = \frac{(58.86\times48)\times24+\frac{1}{2}\times\{(300.84-58.86)\times48\}\times\frac{2}{3}\times48}{(58.86\times48)+\left[\frac{1}{2}\times(300.84-58.86)\times48\right]} = \frac{253647.36}{8632.8} = 29.382$$

Location of resultant force from toe

$$\bar{x} = \frac{\Sigma M_R-\Sigma M_0}{\Sigma F_v} = \frac{2096992-1319161.33}{43048.86} = 18.069$$

Eccentricity (e) of the resultant force $= \frac{B}{2}-\bar{x} = \frac{56}{2}-18.069 = 9.931$

$$\therefore \quad \frac{B}{6} = \frac{56}{6} = 9.33 \quad e > \frac{B}{6}$$

As, resultant is nearer to the toe

Hence, tension will be developed at heel

$$\sigma = \frac{\Sigma F_v}{(B\times 1)}\left[1\pm\frac{6e}{B}\right] = \frac{43048.86}{(56\times 1)}\left(1\pm\frac{6\times 9.33}{56}\right)$$

$$\sigma_{toe} = \frac{43048.86}{56}(1+1.064) = 1586.66 \text{ kN/m}^2$$

$$\sigma_{heel} = \frac{43048.86}{56}(1-1.064) = -49.2 \text{ kN/m}^2$$

Example 4

Find the width of elementary gravity dam whose height is 100m. Specific gravity of dam material 2.2. and seepage coefficient at the base C = 0.8.

Sol. Height of elementary dam, H = 100 m ; Specific gravity of dam material, $S_c = 2.2$

Seepage coefficient at the base of the dam, C = 0.8

Hence, the width of elementary gravity dam, $B = \frac{H}{\sqrt{S_c - C}} = \frac{100}{\sqrt{2.2-0.8}} = 84.52$ m

Example 5

The load on a hydel plant varies from a minimum of 10,000 kW to a maximum of 35,000 kW. Two turbo-generators of capacities 22,000 kW each have been installed. Calculate :
(a) plant factor (b) load factor (c) utilisation factor

Sol. (a) Plant factor (i.e. capacity factor)

$$= \frac{\text{Energy actually produced in time t}}{\text{Max. energy that can be produced in time t}} = \frac{(10000+35000)/2}{44000\times t}\times t = \frac{22500}{44000}$$

= 0.511, i.e. 51.1%. **Ans.**

(b) Load factor $= \frac{\text{Avg. load over certain period}}{\text{Peak load during that period}} = \frac{(10000+35000)/2}{35000} = \frac{22500}{35000}$

= 0.643 i.e. 64.3% **Ans.**

(c) Utilisation factor $= \frac{\text{Max. power utilised}}{\text{Max. power available}} = \frac{35000}{44000}$ = 0.795, i.e. 79.5%. **Ans.**

OBJECTIVE QUESTIONS

1. The best design of an arch dam is when
   (a) all horizontal water loads are transferred horizontally to the abutments
   (b) the dam is safe against sliding at various levels
   (c) the load is divided between the arches and cantilevers and the deflections at the conjugal points being equal
   (d) the deflections of the cantilevers are equal at different points

2. A constant angle arch dam when compared to a constant radius arch dam utilizes concrete quantity of about
   (a) 33% (b) 43% (c) 73% (d) 143%

3. **Assertion (A):** Cut-offs are provided on the d/s base of hydraulic structures.

   Reason (R): Hydraulic structures fail by overturning about the d/s base due to excessive erosion at the base caused by overflowing water.

4. Consider the following statements:
   The function of a cut-off in an earth dam is
   1. to reduce uplift pressures on the dam
   2. to prevent undermining of foundation
   3. to reduce loss of stored water
   4. to support the dam

   Which of these statements are correct?
   (a) 1 and 3 (b) 2 and 4 (c) 2 and 3 (d) 3 and 4

5. A check dam is a
   (a) flood control structure (b) soil conservation structure
   (c) river training structure (d) water storage structure

6. The base width of a solid gravity dam is 25 m. The material of the dam has a specific gravity of 2.56 and the dam is designed as an elementary profile ignoring uplift. What is the approximate allowable height of the dam?
   (a) 64 m (b) 40 m (c) 164 m (d) 80 m

7. What is the critical combination of vertical and horizontal earthquake accelerations to be considered for checking the stability of a gravity dam in reservoir full condition ?
   (a) Vertically upward & horizontally d/s (b) Vertically upward & horizontally u/s
   (c) Vertically downward & horizontally u/s (d) Vertically downward & horizontally d/s

8. The wave height, in metres, generated on the surface of a reservoir, having a fetch length F = 30 km, due to wind blowing on the surface of the reservoir at a velocity of 30 km/hr, is
   (a) 0.26 m (b) 0.96 m (c) 0.52 m (d) 1.2 m

9. The maximum height of a low gravity dam of elementary profile made of concrete of relative density 2.5 and safe allowable stress of foundation material 3.87 MPa without considering uplift force is about

(a) 113 m (b) 217 m (c) 279 m (d) 325 m

10. **Assertion (A):** A seepage passing through the body of an earth dam affects the weight of dam.

Reason (R): The specific weight of submerged soil is not dependent on the porosity of soil.

11. The base width of an elementary profile of a gravity dam of height H is b. The specific gravity of the material of the dam is G and uplift pressure coefficient is K. The correct relationship for no tension at the heel is given by

(a) $\frac{b}{H} = \frac{1}{\sqrt{G-K}}$ (b) $\frac{b}{H} = \sqrt{G-K}$ (c) $\frac{b}{H} = \frac{1}{G-K}$ (d) $\frac{b}{H} = \frac{1}{K\sqrt{G-K}}$

12. Which of the following categories best describes the Hirakud reservoir ?

(a) Reservoir for irrigation and power
(b) Reservoir for flood control, power and irrigation
(c) Reservoir for irrigation and water supply
(d) Reservoir for recreation and fishery

13. The most economical method of soil conservation is to

(a) construct check dams (b) construct contour bunds
(c) drain the soil (d) afforest the area

14. The construction of impounding reservoir is required when

(a) average annual flow in the stream is lower than average demand
(b) the rate flow in the stream, in dry season is more than the demand
(c) the rate of flow in the stream, in dry season is less than the demand
(d) the rate of flow in the stream is equal to the demand

15. Which one of the following sets is used to control the seepage through the foundations of an earth dam?

(a) Chimney drain, u/s blanket and cut-off trench
(b) Cut-off sheet piles, u/s blanket and cut-off trench
(c) U/s blanket, cut-off sheet piles and chimney drain
(d) Relief wells, u/s blanket and chimney drain

16. The vertical component of the earthquake wave, which produces adverse effect on the stability of a dam, is, when it is acting in:

(a) upward direction (b) downward direction
(c) both (a) and (b) (d) none of the above

17. The vertical downward earthquake acceleration $\alpha_v = 0.1g$, acting on a gravity dam, will :

(a) increase the resisting weight of the dam by 10%
(b) decrease the resisting weight of the dam by 10%
(c) increase the uplift by 10%
(d) none of the above

18. A gravity dam is subjected to hydro dynamic pressure, caused by :

(a) the rising waters of the reservoir when a flood wave enters into it
(b) the rising waves in the reservoir due to high winds
(c) the increase in water pressure, momentarily caused by the horizontal earthquake, acting towards, the reservoir.
(d) the increase in water pressure, momentarily caused by the horizontal earthquake, acting towards the dam.

19. Development of tensile stresses in a concrete or masonry gravity dam are usually not allowed, because it may lead to development of tension cracks, ultimately causing failure of the structure, by :

(a) excessive seepage
(b) excessive tensile stresses
(c) excessive compressive stresses
(d) none of the above.

20. The bottom portion of a concrete or a masonry gravity dam is usually stepped, in order to

(a) increase the overturning resistance of the dam
(b) increase the shear strength at the base of the dam
(c) decrease the shear stress at the base of the dam
(d) none of the above

21. The governing compressive stress in a concrete gravity dam, which should not be allowed to exceed the permissible value of about 3000 kN/m^2, while analyzing full reservoir case, is

(a) the vertical maximum stress at the toe
(b) the major principal stress at the toe
(c) the shear stress at the toe
(d) none of the above.

22. An elementary triangular concrete gravity dam, supporting 60 m height of reservoir water and full uplift, should have a minimum base width equal to :

(a) 36 m (b) 39 m (c) 51 m (d) 61 m

23. Shear key is several times provided between the bottom of a masonry or concrete gravity dam and its foundation, to increase the frictional resistance of the dam against sliding. This key is usually provided :

(a) near toe
(b) near the heel
(c) near the individual seams in the bed rock
(d) none of the above.

24. During the construction of an earthen dam by hydraulic fill method, development of pore pressures become important in the :

(a) central impervious core
(b) pervious other shell
(c) both (a) and (b)
(d) none of the above.

25. When the reservoir is full, the slope which is most likely to slide, is :

(a) the u/s slope (b) the d/s slope
(c) both (a) and (b) (d) none of them.

26. The only provision among the following, which can help control the seepage through the body of an earthen dam and, thus, to keep the phreatic line well within the dam width, is

(a) u/s impervious cutoff (b) drain trench along the d/s toe
(c) relief wells (d) chimney drain.

27. The minimum power, which a hydro-power plant can generate throughout the year, is called:

(a) power plant capacity (b) power plant load
(c) firm power (d) water power

28. 400 cumecs of water is being released from a dam storage to meet the d/s demand, through the turbines of the connected hydro-plant. The effective head of water acting on the turbines is 50 m. The efficiency of the hydroplant is 0.8. The electrical power generated from this plant, would be :

(a) 1,56, 800 MW (b) 156, 8 MkW (c) 156, 8 MW (d) none of these

29. If the peak load for a poswer plant equals the plant capacity, then the ratio of the capacity factor to load factor will be :

(a) 1 (b) 0 (c) < 1 (d) > 1
(e) – 1 (f) none of these

30. If the peak load on a power plant having a capacity of 100 MW is 70 MW during a given week, and the energy produced is 58,80,000 kwh, the capacity factor for the plant for the week, will be :

(a) 35% (b) 50% (c) 70% (d) none of these

31. The load factor for the data given in the above question will be:

(a) 0.35 (b) 0.50 (c) 0.70 (d) none of these

32. The utilisation factor for the data given in Q. No. 30, will be :

(a) 0.35 (b) 0.50 (c) 0.70 (d) none of these

33. The most economical central angle of the arch rings in an arch dam, is :

(a) $123° - 34'$ (b) $133° - 34'$ (c) $143° - 34'$ (d) $153° - 34'$

34. In gravity dam design, the horizontal *silt* and *water pressure* is assumed as equivalent to that of a fluid with a mass density, in kg/m^3, of

(a) 1925 (b) 360 (c) 1000 (d) 1360

35. In gravity dam design, the vertical *silt* and *water pressure* is determined by assuming that the silt and water together have a density, in kg/m^3, of

(a) 2650 (b) 1360 (c) 1925 (d) 1000

36. In a gravity dam, frequently, a grout curtain is provided

(a) near the toe to reduce the exit gradient
(b) in the middle of the base to reduce seepage
(c) near the heel to reduce uplift and seepage
(d) at the middle-third of the base nearest to the toe to reduce the reaction pressure.

37. If the uplift pressure is neglected, the base width of an elementary profile of a gravity dam of height H, having relative density of the dam material = G, and coefficient of friction = μ is

(a) $H/(G+1)$ (b) $H/(G-1)^{1/2}$

(c) large of $H/\sqrt{G}$ and $H/\mu G$ (d) smaller of $H/\mu\sqrt{G}$ and H/G

38. The limiting height of a low gravity dam of elementary profile having full uplift condition is

(a) $f_c/(\gamma G)$ (b) $f_c/(\gamma\sqrt{G})$ (c) $f_c/[(G-1)\gamma]$ (d) $f_c/[\gamma\sqrt{G-1}]$

where f_c = allowable stress for the foundation material, G = relative density of dam material and, γ = unit weight of water

39. For an elementary gravity dam profile, made up of concrete of relative density = 2.5 the limiting height of a low dam without considering the uplift was found to be h_0 and with consideration of full uplift it was h_1.

Then h_1/h_0 will have a value of

(a) 0.71 (b) 1.40 (c) 1.00 (d) 1.18

40. Provision of a filter and a rock toe in an earthen dam is done

(a) to prevent the piping action in the dam section
(b) to collect and drain out the seeping water
(c) to reduce the seepage quantity
(d) to have the resultant in the middle third of the base.

ANSWERS

1. (c)	2. (b)	3. (b)	4. (c)	5. (d)	6. (b)
7. (d)	8. (b)	9. (a)	10. (b)	11. (a)	12. (b)
13. (d)	14. (c)	15. (b)	16. (b)	17. (b)	18. (c)
19. (c)	20. (b)	21. (b)	22. (d)	23. (b)	24. (a)
25. (b)	26. (d)	27. (c)	28. (c)	29. (a)	30. (a)
31. (b)	32. (c)	33. (b)	34. (d)	35. (c)	36. (c)
37. (c)	38. (a)	39. (b)	40. (b)		

Spillway, Energy Dissipators & Spillway Gates

CHAPTER

11

Spillways, Energy Dissipators & Spillway Gates

INTRODUCTION

- A spillway is a waterway provided to dispose off surplus flood waters from a reservoir after it has been filled to its maximum capacity.
- Spillways are provided for almost all the dams and these acts as safety valves for the dams.
- A spillway may be located either within the body of the dam or at one end of the dam or entirely away from the dam in a saddle as an independent structure.
- It is essential to provide a spillway of sufficient capacity so that the surplus flood water is discharged keeping the water level in the reservoir below some predetermined maximum level and no damage is done to the dam.

ESSENTIAL REQUIREMENTS OF A SPILLWAY

The essential requirements of a spillway are as follows.

(1) The spillway must have sufficient capacity.

(2) It must be hydraulically and structurally adequate.

(3) It must be so located that it provides safe disposal of water *i.e.*, spillway discharges should not erode or undermine the d/s toe of the dam.

(4) The bonding surfaces of the spillway must be erosion resistant to withstand the high scouring velocities created by the drop from the reservoir surface to tail water.

(5) Some devices for dissipation of energy on the d/s side of the spillway will be required.

SPILLWAY CAPACITY

The required capacity of a spillway (i.e. the maximum outflow rate through the spillway) may be determined by flood routing which requires following data.

(1) *Inflow hydrograph* (plot of rate of inflow v/s time).

(2) *Reservoir capacity curve* (plot of reservoir storage v/s reservoir water surface elevation).

(3) *Discharge curve* (plot of rate of outflow through spillway v/s reservoir water surface elevation).

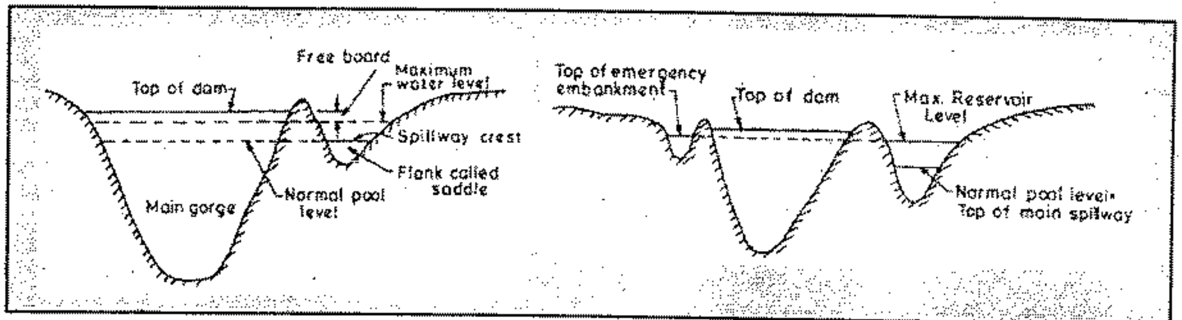
The required capacity of a spillway depends on the following factors.

(i) The *inflow flood.*

(ii) The *available storage capacity.*

(iii) The *discharge capacity of other outlet works.*

(iv) Provision of *gates* in a spillway.



Note: (i) By flood routing, corresponding to a particular inflow hydrograph the maximum outflow rate and maximum rise in the water surface may be determined. (ii) The required discharging capacity of the spillway should be as closely estimated as possible. The underestimation will lead to overtopping of the main dam while overestimation will lead to waste investment due to costly constructions.

TYPES OF SPILLWAYS

Classification based on the prominent features pertaining to the various components of tbe spillway

According to the prominent features pertaining to the various components of the spillways such as control structures, discharge channel etc., the spillway may be classified in the following types.

(i) Straight drop spillway (or free overfall spillway)

(ii) Ogee spillway (or overflow spillway)

(iii) Chute spillway (Trough spillway)

(iv) Side channel spillway

(v) Shaft spillway (or Morning glory spillway)

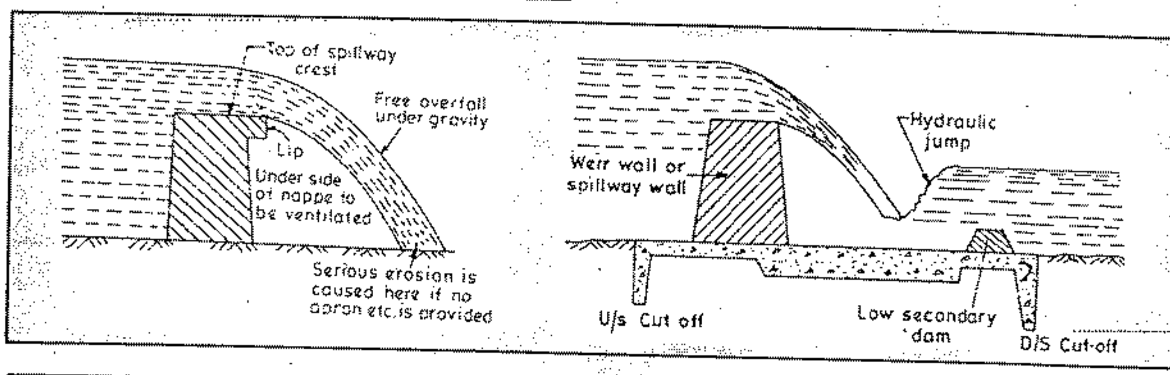
(vi) Siphon spillway.

STRAIGHT DROP SPILLWAY

- A straight drop spillway is type of a spillway for which the control structure is a low height narrow crested weir having its d/s face vertical or nearly vertical.
- The overflowing water may be discharged as in the case of a sharp crested weir or it may be supported along the narrow section of the crest. In either case, the water flowing over the crest of this spillway drops as a free jet clearly away from the d/s face of the spillway.
- The underside of the nappe is ventilated sufficient to prevent pulsating fluctuating jet.
- If no artificial protection is provided on the d/s side of the overfall section, the falling jet will cause the scouring of the streambed and will form a deep plunge pool. As such in order to protect the stream bed from scouring, an artificial pool may be created by constructing a low auxiliary dam d/s of the main structure or by excavating a basin which is then

provided with a concrete apron.

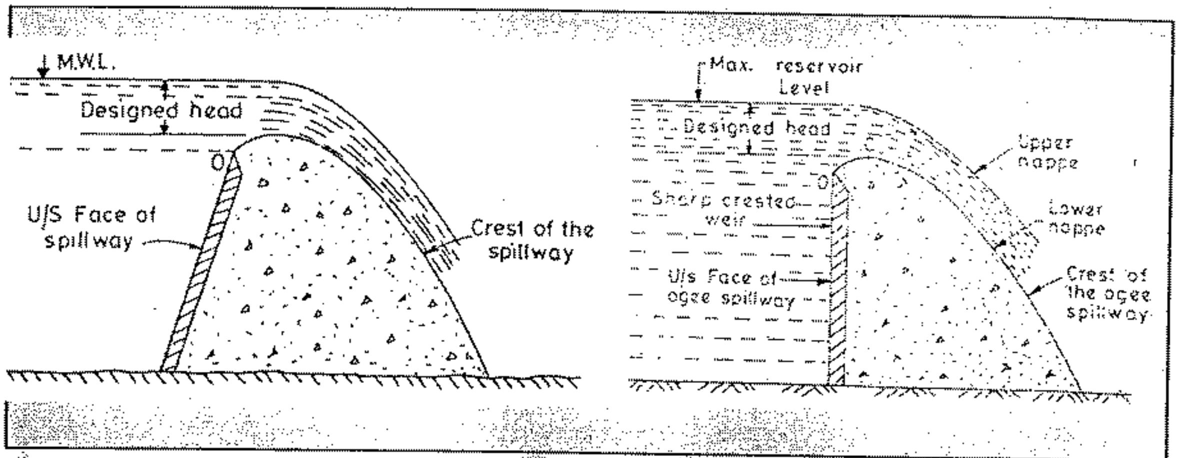
- If tailwater depths are sufficient, a hydraulic jump will form when the jet falls freely from the crest, in which case a sufficiently long flat apron may be provided.
- Moreover, floor block and an end sill may be provided.
- The free overfall spillway is most commonly used for low earth dams (or earthen bunds).
- This type of spillway is also suitable for thin arch dams and for other dams which have nearly vertical d/s face and would permit free fall of water.
- This type of spillway is not suitable for high drops on yielding foundations, because the apron will be subjected to large impact forces at the point of impingement of the jet.
- Ordinarily, free overfall spillways are used where the hydraulic drops from head pool to tailwater are not in excess of about 6 m.



OGEE SPILLWAY

- Ogee spillway is an improvement upon the free overfall spillway and is widely used with concrete, masonry, arch and buttress dams.
- Such a spillway can be easily used on valleys where the width of the river is sufficient to provide the required crest length and the river bed below can be protected from scour at moderate costs.
- The profile of this spillway is made in accordance with the shape of the lower nappe of a free falling jet over a ventilated sharp crested weir.
- The shape of the lower nappe of freely falling jet over a sharp crested weir can be determined by the principle of projectile.
- It generally rises slightly (to point C) as it originates from the crest (O) of a sharp crested weir and then falls to make a parabolic form.
- The space b/w sharp crested weir & lower nappe is filled with concrete/masonry & the weir so formed has a profile similar to an 'ogee' and hence called an ogee spillway.
- Normally, the u/s face of the spillway is kept vertical and the crest shape conforms to the lower nappe of a vertical sharp crested weir under maximum head. But if the u/s face of the spillway is kept sloping, the crest shape should also conform to the lower nappe that would be obtained for an inclined sharp crested weir.
- The crest of the Ogee spillway can be made to conform only to one particular nappe that would be obtained at one particular head. This head is called the *designed head*.

- But in practice, the actual head of water on the spillway crest, called the operating head, may be less or more then the designed head.
- If the head of water over the spillway is more than the designed head, cavitation may occur.
- On the other hand, if the head of water over the spillway is less than the designed head, the falling jet would adhere to the crest of the ogee spillway, creating positive hydrostatic pressures and thereby reducing the discharge coefficient of the weir.



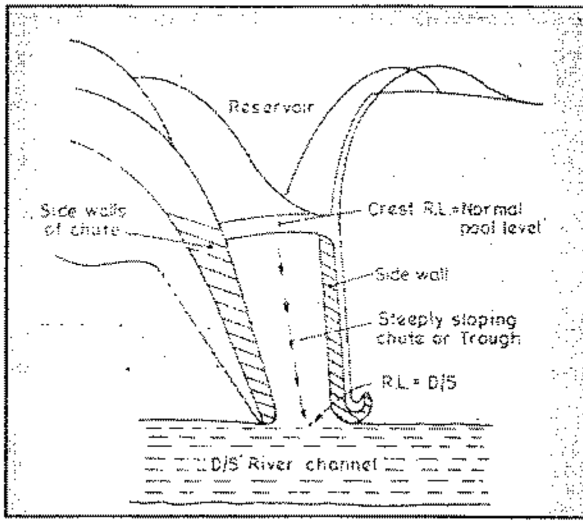
CHUTE SPILLWAY

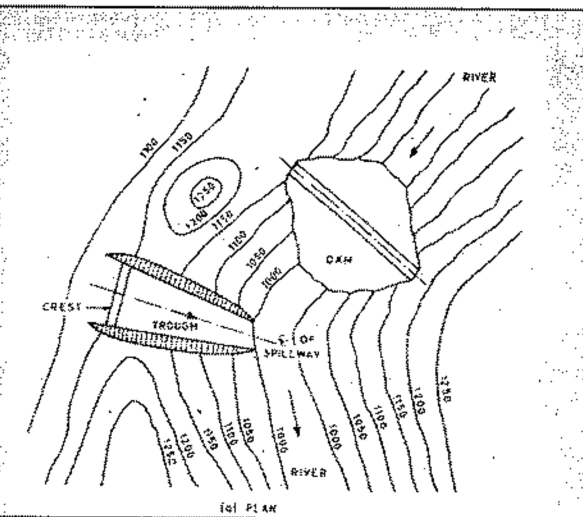
- A spillway whose discharge is conveyed from reservoir to d/s level through an open channel placed either along a dam abutment or through a saddle may be called as chute spillway.
- This type of spillway may be having chute type discharge channel controlled by an *overflow crest, a gated orifice, a side channel crest* or some other control device.
- The Chute Spillway is the simplest type of a spillway which can be easily provided independently and at low costs.
- It is lighter and adaptable to any type of foundations; and hence provided easily on earth and rockfill dams.
- In a chute spillway, the water flows at right angles to the weir crest after spilling over it
- A chute spillway is sometimes known as a **waste weir**. If it is constructed in continuation to the dam at one end, it may be called a **flank weir**.
- If it is constructed in a natural saddle in a bank of the river separated from the main dam by a high ridge, it is called a **saddle weir**.
- If the slope of the chute can conform to available topography, the excavations shall be minimum. But the slope of the chute must be high enough, and should at least be able to maintain super critical flow, to avoid unstable flow conditions.

  Chute Slope : The water spilling over the control structure (i.e., ogee weir), then flows through the chute channel. The minimum slope of the chute is governed by the condition that supercritical flow must be maintained. The slope of the chute is kept just sufficient to meet this flow requirement from the crest for as long a distance as possible without any filling.

Side Walls : The side walls of the chute should be of such a height that water does not spill over them.

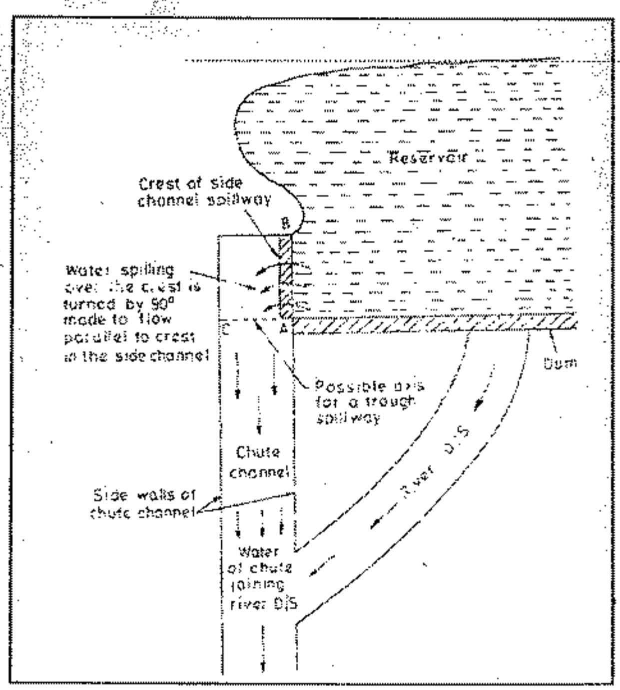
Approach Channel : An approach channel, trapezoidal in shape with side slopes 1 : 1, may be constructed so as to lead the reservoir water up to the control structure.





SIDE CHANNEL SPILLWAY

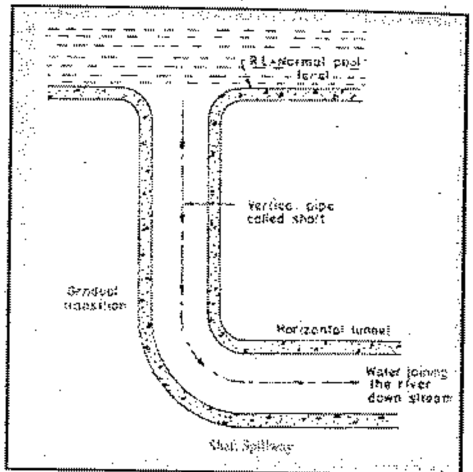
- The side channel spillway is a type of spillway in which the control weir is placed along the side of and approximately parallel to the upper portion of the spillway discharge channel.
- In a side channel spillway, the flow of water after spilling over the crest is turned by 90° such that it flow parallel to the weir crest.
- This type of spillway is provided in narrow valleys where there are no side flanks of sufficient width to accommodate a chute spillway.
- Analysis of flow in the side channel, is made by the application of the momentum principle in the direction of flow. The water entering the side channel has no momentum in the direction in which it has to move.



- The slope of the side channel should be sufficient to overcome friction losses as well as to provide acceleration in the direction of flow against the mass of incoming water.

Note : Many other spillways may be constructed somewhere in between the chute spillway and the side channel spillway. In such cases, the direction of water after passing over the crest is changed somewhere between 0° and 90°.

SHAFT SPILLWAY



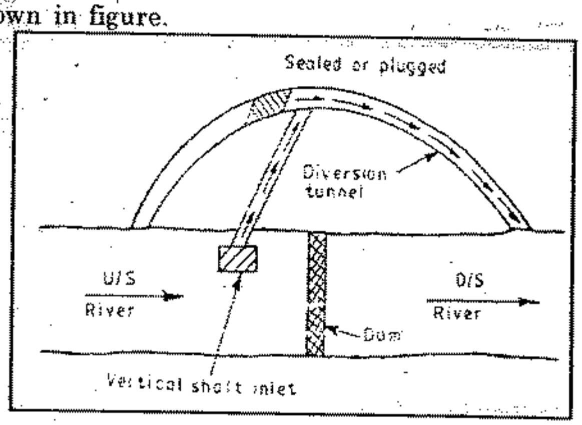
Shaft Spillway

- A shaft spillway is a type of spillway in which the water enters from the reservoir into a vertical shaft which conveys this water into a horizontal tunnel from where this water is discharged into a d/s river channel.
- It consists of:

  a) An *overflow control weir*

  b) A *vertical transition*

  c) A closed *discharge channel.*
- Sometimes, the vertical shaft may by excavated through some natural rocky island existing on the u/s of the river near the dam.
- Sometimes, *artificial shafts may be constructed.*
- The horizontal tunnel or the conduit may be taken either through the body of the dam (as may be done in concrete gravity dams) or below the foundations (as may be done in earthen dams). The diversion tunnels constructed for diversion of the river, may sometimes be planned and used for shaft spillways, as shown in figure.



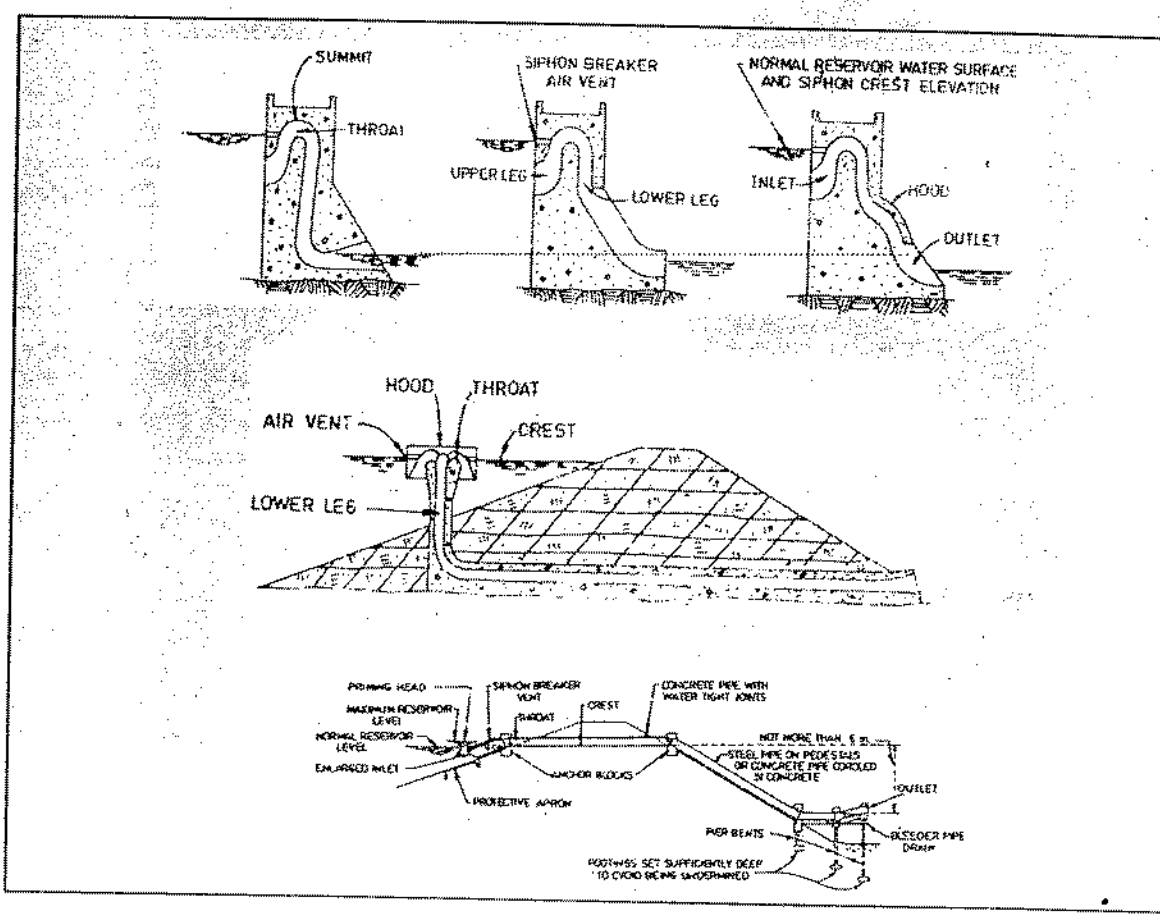
- It is also called as '*Morning glory*' in case the inlet is tunnel shaped.
- Its discharge characteristics may vary with the range of head.
- A shaft spillway may be adopted when the possibility of an over-flow spillway and a trough spillway has been ruled out because of non-availability of space due to topography.
- If some suitable rock spur is available near the reservoir on the u/s, shaft spillway may become economical and the first choice.
- Since, a shaft spillway is surrounded by water on all sides, it must be connected to the dam or the hill side by a bridge.

SIPHON SPILLWAY

- A siphon spillway is a closed conduit system formed in the shape of U, positioned so that the inside of the load of the upper passage way is at normal reservoir storage level.
- The initial discharges of the spillway are similar to flow over a weir as the reservoir level rises over normal. But as soon as the air in the level over the crest has been exhausted, siphonic action takes place.
- Suction effect maintains a continuous flow due to the gravity pull of water in the down leg of the siphon.
- Most siphon spillways are composed of five components:

  (i) *Inlet* (ii) An *upper leg* (iii) A *throat or control section* (iv) A *lower leg* (v) An *outlet*

- An siphon-breaker air vent is provided in order to stop the operation when the reservoir water surface is drawndown to normal level.
- Inlet is generally placed well below the normal reservoir water surface to prevent entry of ice and drift and to avoid the formation of vertices and drawdowns which might break the siphonic action.
- Upper leg is formed as a bending convergent transition to join the inlet to a vertical throat section.
- Lower leg is placed on an adverse slope to provide a more positive priming action by forming a flow certain which seals across the leg.
- The lower leg can be terminated so as to discharge vertically or along the face of a concrete dam or it may be provided with a lower bend and diverging outlet tube to release the flow in a horizontal direction.
- The outlet flow can be free discharging or submerged depending on the tailwater conditions and on arrangement of the lower leg.



Advantages of Siphon Spillway

The following are the advantages of a siphon spillway over the other types of spillways.

(i) It is able to pass full capacity discharges with narrow limits of head water rise.

(ii) It is automatic in action without involving any mechanical devices or moving parts.

(iii) Its discharge per unit length is more because it has a higher operating head.

(iv) There is practically no maintenance cost and it is leak proof.

(v) The cost of acquisition of areas which will get submerged between maximum water level and full reservoir level is minimum in this case.

(vi) The height of the dam above the crest of the spillway is also reduced significantly.

(vii) It is useful in sucking up sediment from the bed of the reservoir when it is in full action during floods.

Limitations of Siphon Spillway

The following are the limitation of a siphon spillway.

(i) It is unable to pass ice and debris.

(ii) There is a possibility of clogging of the siphon duct and siphon breaker vents with debris or leaves.

(iii) As a result of erratic make-and-break action of the siphon sudden surges and stoppages of ouflow may occur which may cause considerable fluctuations in the water level in the river on the d/s side.

(iv) If a single siphon is provided there is a possibility of outflows from the reservoir being more than the inflows.

(v) It requires strong foundation to resist vibrations which are usually quite severe in this case.

ENERGY DISSIPATORS

The water flowing over the spillway when floods are anticipated acquires a lot of kinetic energy by the time it reaches near the toe of the spillway due to conversion of potential energy into kinetic energy. If arrangements are not made to dissipate this huge kinetic energy of water large scale scouring will take place on the d/s side near the toe of the dam and away from it. These arrangements are known as energy dissipators.

In general, the kinetic energy of this super-critical flow can be dissipated in two ways:

(1) By converting the super critical flow into sub-critical flow by hydraulic jump.

(2) By directing the flow of water into air and then making it fall away from the toe of the structure.

Hydraulic Jump

- Hydraulic jump is defined as a sudden and turbulent passage of water from supercritical to subcritical state.
- The flow in a hydraulic jump is accompanied by the formation of extremely turbulent rollers and there is considerable dissipation of energy.
- Hydraulic jump is the most suitable and effective means of energy dissipation.
- The amount of energy dissipated in a jump varies with the type of the jump.

  Hydraulic jumps may be classified according to the values of Froude's no. F_1 of the incoming flow as follows:

(i) ***For F_1 = 1, the flow is critical,*** and hence no jump can form.

(ii) ***For F_1 = 1 to 1.7,*** the water surface shows undulations, and the jump is called an ***undular jump***. The energy dissipation in this case is quite low being only about 5%.

(iii) ***For F_1 = 1.7 to 2.5,*** a series of small rollers develop on the surface of the jump, but the d/s water surface remains smooth. The velocity throughout is uniform. The energy dissipation is also less being only about 20%. This jump is called a ***weak jump.***

(iv) ***For F_1 = 2.5 to 4.5,*** the entering jet oscillates back and forth from the bottom to the surface and back again without any periodicity. This jump is called an ***oscillating jump***. The energy dissipation in this case ranges from 20 to 45%.

(v) ***For F_1 = 4.5 to 9, a stable and well balanced jump*** is developed. The action and position of this jump are least sensitive to variation in tail water depth. This jump has the best performance. The energy dissipation ranges from 45 to 70%. This jump is called a **steady jump.**

(vi) ***For F_1 = 9.0 and more,*** the jump action is rough which results in a rough water surface with strong surface waves d/s from the jump. The jump action is however effective since the energy dissipation may reach 85%. This jump is called a ***strong jump.***

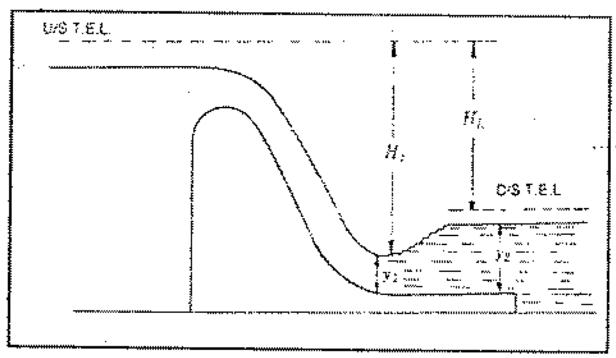
RELATIONSHIP BETWEEN JUMP HEIGHT CURVE AND TAIL WATER CURVE

A hydraulic jump can form in a horizontal rectangular channel, when the following relation is satisfied between the pre-jump depth (y_1) and post jump depth (y_2).

$$\boxed{y_2 = -\frac{y_1}{2} + \sqrt{\frac{y_1^2}{4} + \frac{2q^2}{gy_1}}} \qquad \text{----(i)}$$

where, q is the discharge intensity

For a given discharge intensity over a spillway, the depth y_1 is equal to q/V_1; and V_1 is determined by the drop H_1, being equal to $\sqrt{2gH_1}$.



- Hence, for a given discharge intensity and given height of spillway, y_1 is fixed and thus y_2 is also fixed.
- But the availability of a depth equal to y_2 in the channel on the d/s cannot be guaranteed as it depends upon the tail water level.
- Tail water level depends upon the hydraulic dimensions & slope of the river channel below.
- For different discharge, the tail water depth is found by actual gauge discharge observations and by hydraulic computations. The post jump depths (y_2) for all those discharges, are also computed from equation (i).
- If a graph is now plotted between q and tail water depth, the curve obtained is known as the Tail Water Curve (T.W.C.). Similarly, if a curve is plotted on the same graph, between

q and y_2, the curve obtained is known as the Jump Height Curve (J.H.C.).

- Jump height curve is also called y_2 curve.

There are five possible relations between T.W.C. and y_2 curve:

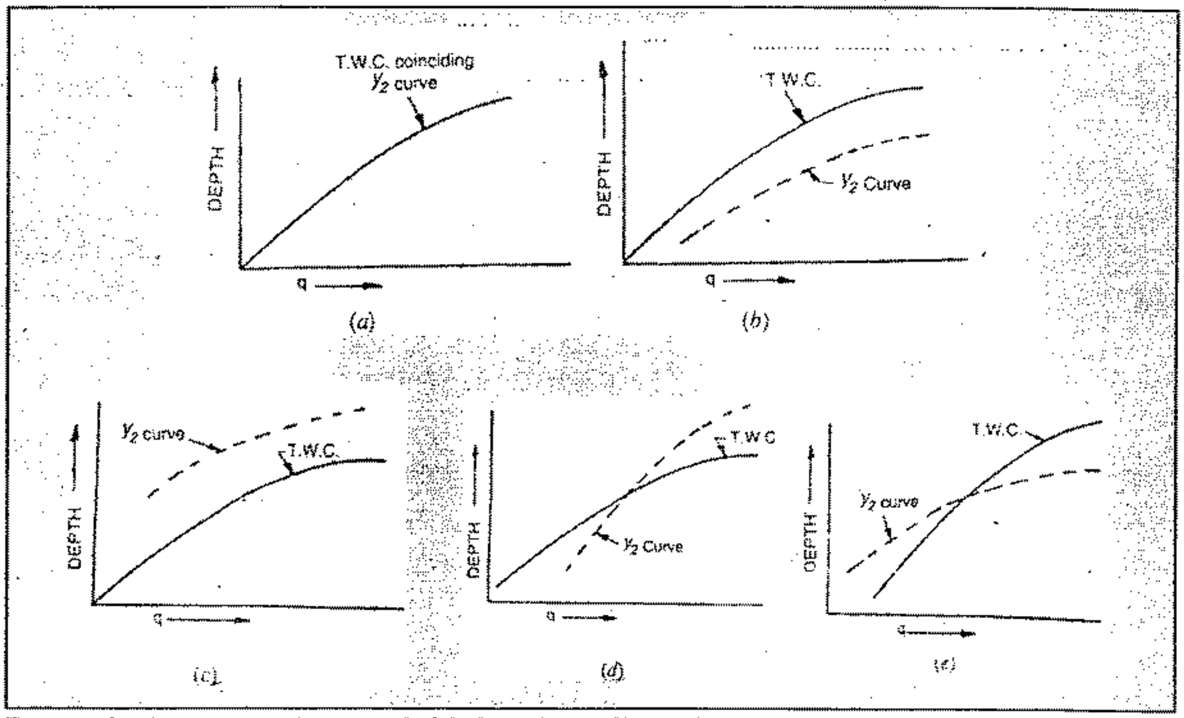
(a) T.W.C. coinciding with y_2 curve at all discharges (fig. a)

(b) T.W.C. lying above the y_2 curve at all discharges (fig. b).

(c) T.W.C. lying below the y_2 curve at all discharges (fig. c)

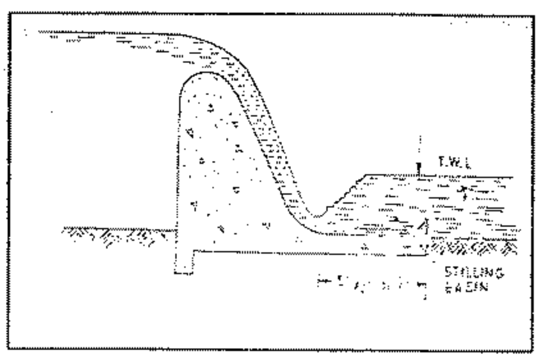
(d) T.W.C. lying above the y_2 curve at smaller discharges and lying below the y_2 curve at larger discharge (fig. d)

(e) T.W.C lying below the y_2 curve at smaller discharges and lying above the y_2 curve at larger discharges (fig. e)



Energy dissipators can be provided below the spillway depending upon the relative positions of TWC and y_2 curve as explained below.

(a) When T.W.C coincides with y_2 curve at all discharge



- This is the most ideal condition for jump formation.
- The hydraulic jump will form at the toe of the spillway at all discharge.
- A simple concrete apron of length 5 $(y_2 - y_1)$ is generally sufficient to provide protection in the region of hydraulic jump,

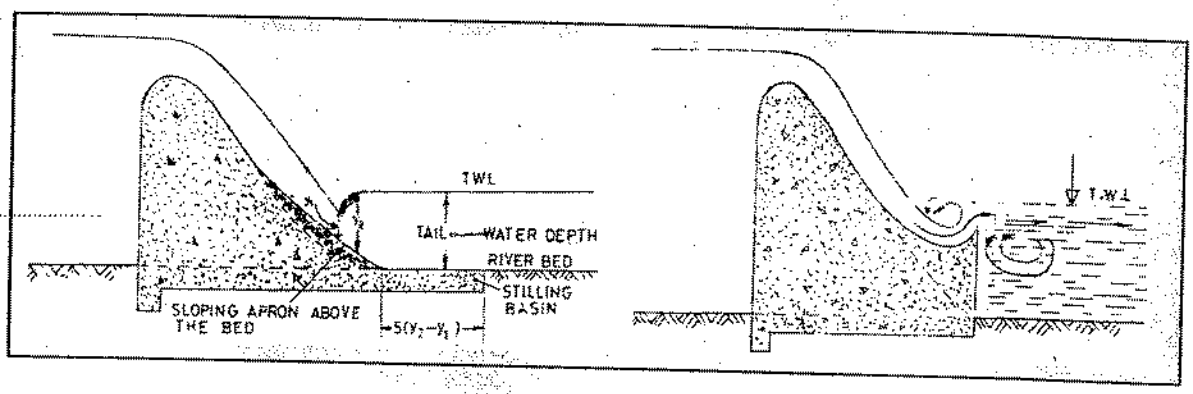
(b) When T.W.C. is lying above the y_2 curve at all discharges

- When y_2 is always below the tail water the jump forming at toe will be drowned by the tail water, and little energy will be dissipated.
- Water continues to flow at high velocity along the channel bottom for a considerable distance.

  Expected solutions are :

  (i) by construction of a sloping apron above the river bed level
- The jump will form on the sloping apron where depth equal to y_2 is available.
- Slope of the apron is made in such a way that proper conditions for a jump will occur somewhere on the apron at all discharges.

  (ii) A provision of a roller bucket type of energy dissipator. It consists of an apron, which is upturned sharply at ends.
- Two main rollers are formed which dissipate the energy due to internal turbulence.

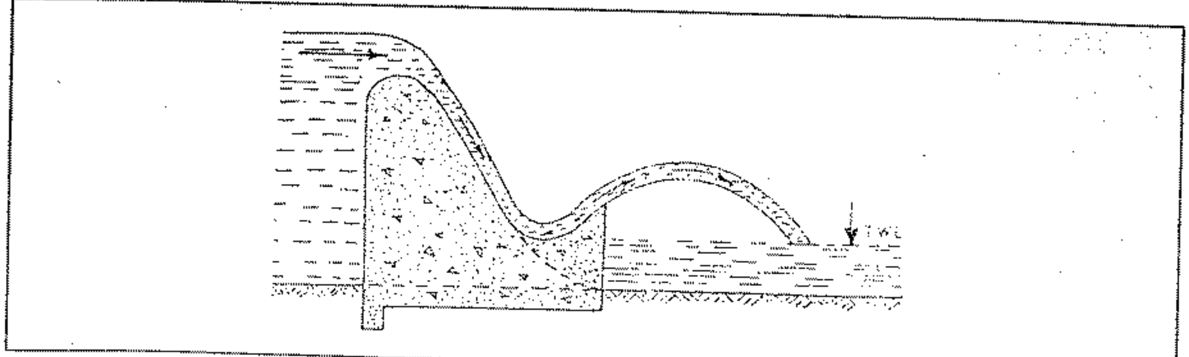


(c) When T.W.C. lies below the y_2 curve at all discharges.

Expected solution are

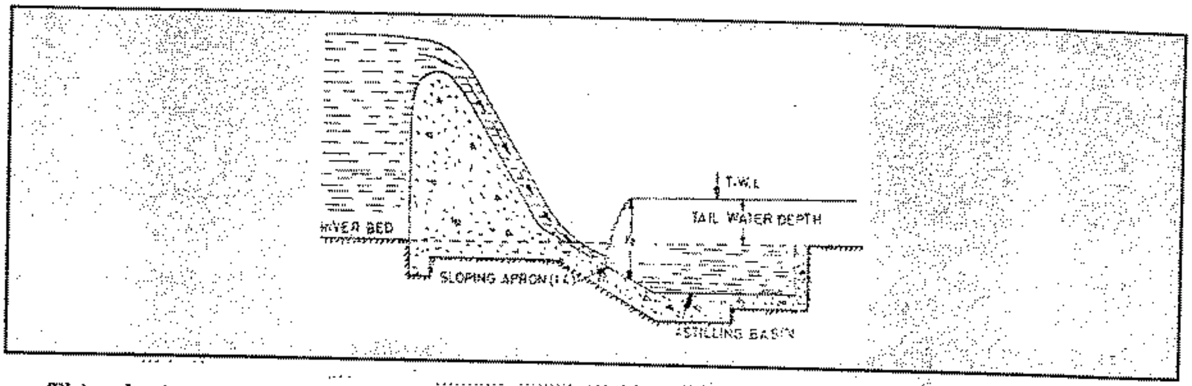
(i) If the tail water is very low, the water may shoot up out of the above bucket, and fall harmlessly into the river at some distance d/s of the bucket.

This bucket is then known as ski jump bucket and can be used for energy dissipation in case (c) : i.e., when the tail water depth is insufficient or low at all discharges.



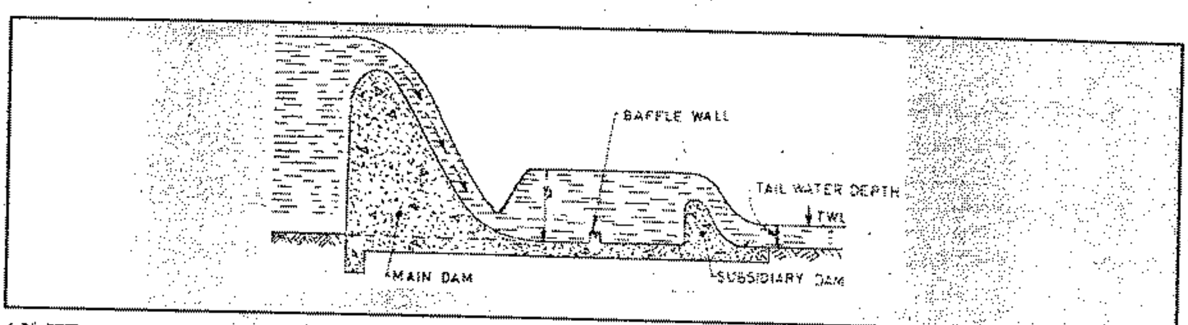
The ski jump bucket type of an energy dissipator requires sound and rocky river bed, because a part of the energy dissipation takes place by impact, although some of the energy is dissipated in air by diffusion and aeration.

(ii) Sloping apron can be provided below the river bed similar to case (b). The required depth y_2 which is greater than tail water depth can thus be made available by letting the jump form on this sloping apron.



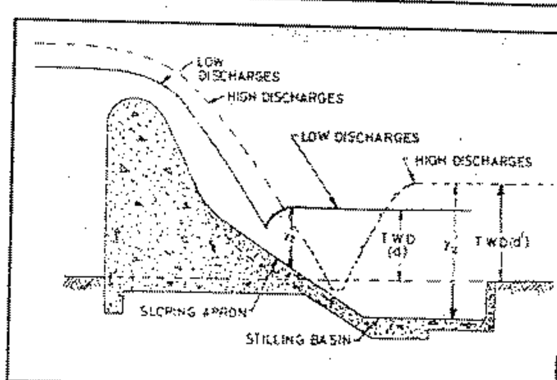
This sloping apron and the horizontal cistern of length 5 ($y_2 - y_1$) shall be entirely in cutting.

(iii) The third solution to this problem may be the construction of a subsidiary dam below the main dam, so as to increase the tail water depth and cause a jump to form at the toe of the main dam. If the tail water deficiency is small, a baffle wall or a row of friction blocks may be provided so as to dissipate the residual energy.



(d) When T.W.C. lies above the y_2 curve at low discharges and lies below the y_2 curve at high discharges

- In this case, at low discharges, the jump will be drowned and at high discharges, tail water depth is insufficient.
- The solution to the problem lies in providing a sloping apron partly above and partly below the river bed as the horizontal apron and end sill should also be provided.



At low discharges, the jump will form on the apron above the river bed, where the available depth is equal to the required depth and less than the T.W. depth. Similarly, at high discharges, the jump will form on the apron below the river bed, required for jump formation .

(e) When tail water depth is insufficient at low discharges and is greater at high discharges.

This case is the reverse of case (d) and the same arrangement which was made in case (d) will serve the purpose. At low discharges, the jump will form on the apron below the bed; and at high discharges, the jump will form on the apron at a point above the bed.

STILLING BASINS

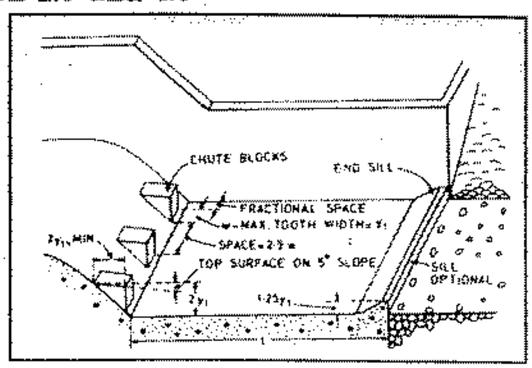
- A stilling basin is defined as a structure in which a hydraulic jump used for energy dissipation is confined partly or entirely.
- Stilling basins are usually provided with certain accessories like chute blocks, baffle blocks, and end sills to reduce the length of the jump and thus to reduce the length and the cost of the stilling basin.
- The selection of a suitable type of stilling basin is dependent on the characteristics of the hydraulic jump which in turn are dependent on the Froude number F_1 of the incoming flow

U.S.B.R. STILLING BASINS

(1) Stilling basins for Froude number between 1.7 to 2.5

- For this case only a horizontal apron needs to be provided.
- As the flow in this case does not have much turbulence usually no accessories (like special stilling basins, baffles and other dissipating devices) are required to be provided.
- The apron should be sufficiently long to contain the entire jump over it.
- The length of the apron should be equal to the length of the jump which in this case is found to be equal to about $5y_2$, where y_2 is the sequent depth.

(2) Stilling basins for Froude number between 2.5 and 4.5



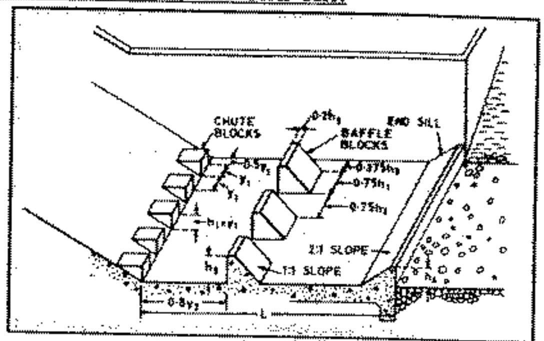
- For this range of Froude number, Type I stilling basin has been found to be effective for dissipating the energy of the flow.
- The basin is provided with chute locks and the end sill is optional.
- In this case due to oscillating jump being developed, wave action is produced which cannot be entirely dampened.
- *Auxiliary wave dampeners or wave suppressors* must sometimes be employed to provide smooth surface flow d/s.
- However, in order to suppress the wave action the floor of the basin should be so set that the tail water depth in the basin is 10% greater than the sequent depth y_2.
- Stilling basins to accomodate these flows are the least effective the providing satisfactory dissipation, since the attendant wave action ordinarily cannot be controlled easily by the usual basin devices

(3) Stilling basins for Froude numbers higher than 4.5

For basins where the Froude no. is higher than 4.5, a true hydraulic jump will form.

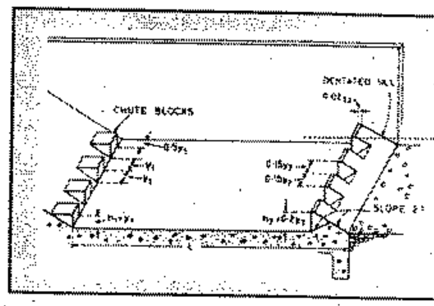
For this case depending upon the velocity of the incoming flow, two types of stilling basins have been developed as indicated below.

(a) When the velocity of the incoming flow is less than 15 m/s, Type II stilling basin may be adopted. This basin utilizes chute blocks, baffle blocks and an end sill.



- By providing the *baffle blocks* the length of the stilling basin is considerably reduced because the dissipation of energy is accomplished by the hydraulic jump as well as by the impinging action of the incoming flow against these blocks.
- However, the baffle blocks will be subjected to large impact forces due to the impingement of the incoming flow. Moreover on the d/s face of the baffle blocks usually suction or negative pressures will be developed which will further increase the force exerted on these blocks. Hence the baffle blocks should be properly anchored at the base. Further the floor of the basin will also be subjected to additional load due to the dynamic force created against the u/s face of the baffle blocks, which should be considered in the design of the floor of the basin.

(b) When the velocity of the incoming flow exceeds 15 m/s, Type III stilling basin may be adopted. In this basin only chute blocks are provided and instead of a solid end sill a dentated end sill is provided.



In this basin baffle blocks are not provided because (i) due to high velocities of the incoming flows these blocks will be subjected to excessively large impact force

(ii) There is a possibility of cavitation along the d/s face of these blocks and the adjacent floor of the basin due to large negative presure being developed in this region.

However, due to baffle blocks being eliminated in this case the dissipation of energy is accomplished primarily by hydraulic jump action and hence the length of the basin will be greater than that indicated for the Type II basin.

INDIAN STANDARDS STILLING BASINS

The stilling basins recommended according to Indian standards may be classified into the following two types:

(1) Stilling basins with horizontal apron (or floor) (2) Stilling basins with sloping apron (or floor)

(1) Stilling basins with horizontal apron

- The stilling basins with horizontal apron (or floor) may be provided when the jump height curve coincides with the tail water rating curve; or the former is slightly above or below the latter.
- In this case, the requisite depth for the development of the jump may be obtained on an apron near or at the ground level.
- *The stilling basins with horizontal apron* may be classified into the following two types.

(i) Indian standards stilling basin Type I (ii) Indian standards stilling basin Type II

(i) Indian standards stilling basin Type I

- It may be provided when the Froude number F_1 of the incoming flow is less than 4.5.
- This case is generally encountered on weirs and barrages.
- This stilling basin is provided with chute blocks,baffle blocks and a dentated end sill.
- The size, spacing and location of the chute blocks for this basin are exactly same as those for Type I stilling basin of U.S.B.R.
- The height and top thickness of the basin blocks for this stilling basin are same as those for Type II stilling basin of U.S.B.R.
- The width and spacing of the basin blocks for this case is equal to the height of the blocks.
- Further the size of the dentated end sill for this stilling basin is exactly same as the one for Type III stilling basin of of U.S.B.R.

(ii) Indian standards stilling basin Type II

- It may be provided when the Froude number F_1 of the incoming flow is greater than 4.5. This case is a general feature for medium and high dams.
- This stilling basin is also provided with *chute blocks, baffle blocks and a dentated end sill.*
- When the velocity of flow at the location of the basin blocks exceeds 15 m/s, no basin blocks are provided and in that case the floor of the basin should be kept at a depth equal to the sequent depth y_2 below the tail water level.
- The size, spacing and location of the chute blocks for this stilling basin are exactly same as those for Type II or Type III stilling basins of U.S.B.R.
- The size, spacing and location of the basin blocks if provided for this stilling basin are exactly same as those for Type II stilling basin of U.S.B.R.
- Also the dentated end sill provided for this stilling basin is exactly same as Type III stilling basin of U.S.B.R.

(2) Stilling basin with sloping apron

- When the tail water depth is too large as compared to the sequent depth y_2, a drowned jump will develop which is undesirable. In such a case a stilling basin with a sloping apron may be provided.
- The stilling basins with sloping apron (or floor) may be further classified into the following two types:

  (i) Indian standards stilling basin Type III (ii) Indian standards stilling basin Type IV.

(i) **Indian standards stilling basin III** is recommended for the case where tail water rating curve is about the jump height curve at all discharges It is usually provided with a sloping apron (or floor) for its entire length.

(ii) **Indian standards stilling basin Type IV** is suitable for the case where the tail water depth at maximum discharge exceeds the sequent y_2 considerably but is equal to or slightly greater than y_2 at lower discharges. It is provided with a partly sloping and partly horizontal apron. For both these stilling basins except a solid or dentated end sill, no other accessories are provided

OUTLET WORKS

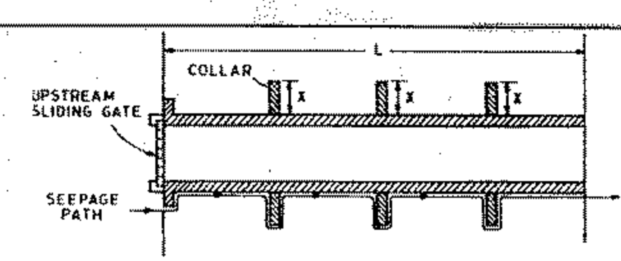
- Outlet works are provided to release water impounded by a dam, as and when needed for various purposes.
- These may be required for several purposes such as feeding water to the turbines for power generation, supplying water to irrigation channels, or for a combination of multipurpose requirements.
- Sometimes the outlet works of a dam may be used instead of a service spillway in conjunction with an auxiliary or secondary spillway. The outlet works may also act as flood regulator, to release waters temporarily stored in flood control storage space or to evacuate storage in anticipation of flood inflows.

The various components of outlet works are:

(1) Sluiceways or waterways (2) Control devices. (3) Intake structures.

(1) Sluiceways or waterways

- A sluiceway (also known as outlet) is a pipe or tunnel that passes through a dam or the hillside at either end of the dam.
- Sluiceways for concrete dams generally pass through the dam, while in the case of earth or rock-fill dams these are preferably placed outside the limits of the dam. However, if a sluiceway must pass through as earth dam, projecting collars should be provided as shown in figure.



- The inlet of the sluiceway must be at minimum reservoir level. However large dams may have sluiceways at different levels.
- The sluiceways may be circular or rectangular in shape. Further the sluiceways should have bell-mouth entrance.

(2) Control devices

- In order to regulate the flow of water through the sluiceways various control devices such as gates and valves are used. The control devices may be classified according to their location as follows

  (i) Entrance gates. (ii) Interior gate valves.
- Most of the sluice ways are provided with some type of gate at their entrance. For low heads upto 15 m sliding gates may be used at the entrance of the sluice ways, but for higher heads roller gates are used.
- *Interior gate valves* are located d/s from the conduit entrance. Standard gate valves of the type used in water distribution system may be used. for interior valves in circular conduits

up to 1.25 m in diameter under heads of less than 100m. The interior gate valves may be used to regulate the flow for heads less than 25 m, but for greater heads they are ordinarily used only in the fully open or fully closed position. Large interior gate valves of different types such as rectangular sliding gate valve, butterfly valve, needle valve etc., have been developed which may be used for conduits of large diameters and under high heads.

(3) Intake Structures

- An intake structure forms an entrance into the outlet works and it accommodates the control devices at the entrance.
- It also supports necessary auxiliary appurtenances such as trash racks, fish screen etc. An intake structure may either be submerged or extended as a tower to some height above the maximum water surface.

Trash racks

A trash rack is a structure which is provided at the entrance to intakes and sluiceways to prevent entrance of debris.

Example 1

At an energy dissipator structure below a low spilllway, the discharge is $19 m^2/sec$ and the energy loss is 1m at the hydraulic jump forming therein. Determine the depths of flow at both ends of the jump.

Sol. Given, $q = 19\ m^2/s$; Energy loss, $H_L = 1m$

Energy loss for hydraulic jump, $H_L = \dfrac{(y_2-y_1)^3}{4y_1y_2}$

and $\dfrac{2q^2}{g} = y_1y_2(y_1+y_2)$

$$y_1y_2(y_1+y_2) = \frac{2\times(19)^2}{g}$$

$$\Rightarrow \quad y_1y_2(y_1+y_2) = 73.6 \quad \text{... (i)}$$

$$1 = \frac{(y_2-y_1)^3}{4y_1y_2} \quad \text{...(ii)}$$

Let $\dfrac{y_2}{y_1} = x \quad \Rightarrow \quad y_2 = xy_1$

Putting the value of y_2 in equation (i)

$$y_1^2x(y_1+xy_1) = 73.6 \Rightarrow xy_1^3(1+x) = 73.6$$

Similarly putting in (ii) , we get

$$\frac{(y_1x-y_1)^3}{4y_1 \cdot y_1x} = 1 \quad \Rightarrow \quad \frac{y_1^3(x-1)^3}{4y_1^2x} = 1$$

$$\Rightarrow \quad 4x = y_1(x-1)^3 \Rightarrow y_1 = \frac{4x}{(x-1)^3}$$

Substituting the value of y_1 in equation (ii), we get

$$x\left[\frac{4x}{(x-1)^3}\right]^3(1+x) = 73.6$$

$\therefore$ By hit and trial, we get $\quad x = 2.8,\ y_1 = \frac{4\times 2.8}{(2.8-1)^3} = \frac{4\times 2.8}{1.8^3} = 1.92$ m

and $\quad y_2 = y_1 x = 1.92 \times 2.8 = 5.377$m

OBJECTIVE QUESTIONS

1. Consider the following statements:

Assertion (A): When friction blocks are provided in a stilling basin to localize and stabilize the jump formation, the d/s depth in the jump is less than that without the friction blocks.

Reason (R): The discharge per unit width at the section where the blocks are located is increased.

Of these statements

(a) both A and R are true and R is the correct explanation of A
(b) both A and R are true but R is not a correct explanation of A
(c) A is true but R is false
(d) A is false but R is true

2. Match **List-I** (Main provision) with **List-II** (Surpassing arrangement) and select the correct answer using the codes below the lists:

List-I	List-II
A. Minor irrigation work	1. Saddle spillway
B. Medium irrigation project in interior area	2. Syphon spillway
C. Earth dam across main river	3. Ogee spillway
D. Masonry dam on good rock	4. Surplus weir

Codes:

	A	B	C	D
(a)	4	2	1	3
(b)	4	2	3	1
(c)	2	4	3	1
(d)	2	4	1	3

3. Match **List-I** (Energy dissipation) with **List-II** (Water level and slope condition) and select the correct answer using codes given below the lists:

List-I

A. Roller bucket
B. Ski-jump bucket
C. Standing wave basin with depressed floor
D. Standing wave basin with raised floor

List-II

1. TWL (tail channel water level) is slightly above JWL (jump height water level) and the slope of the channel is mild
2. TWL is considerably above JWL and the slope of the channel is mild
3. TWL is slightly below JWL and the slope of the channel is mild
4. TWL is considerably less than JWL and the slope of the channel is steep

Codes:

	A	B	C	D
(a)	4	2	1	3
(b)	2	4	1	3
(c)	2	4	3	1
(d)	4	2	3	1

4. The ideal condition for energy dissipation in the design of spillways is the one when the tail water rating curve

(a) lies above jump rating curve at all discharges

(b) coincides with the jump rating curve at all discharges

(c) lies below jump rating curve at all discharges

(d) lies either above or below the jump rating curve depending upon discharge

5. Consider the following statements:

Assertion (A): The USBR type-II stilling basin length requirement is less than that in type-III basin for similar design conditions.

Reason (R): Energy dissipation is primarily accomplished by hydraulic jump in USBR type-II stilling basin.

Of these statements

(a) both A and R are true and R is the correct explanation of A

(b) both A and R are true but R is not a correct explanation of A

(c) A is true but R is false

(d) A is false but R is true

6. A ski-jump bucket is generally used as an energy dissipator when the tail water

(a) is greater than 1.1 times the required conjugate depth for the formation of hydraulic jump; and the river bed rock is 'good'

(b) depth is lesser than the depth required for the jump formation; and the bed of the river channel is composed of 'sound' rock

(c) depth is equal to the depth required for the jump formation, and the river bed rock is 'good'

(d) depth is 1.3 times the required for the jump formation and the river bed is composed of 'weak' rock

7. Which one of the following equations represents the d/s profile of Ogee spillway with vertical u/s face? (x, y) are the coordinates of the point on the d/s profile with origin at the crest of the spillway and H_d is the design head.

(a) $\frac{y}{H_d} = -0.5\left(\frac{x}{H_d}\right)^{1.85}$

(b) $\frac{y}{H_d} = -0.5\left(\frac{x}{H_d}\right)^{1/1.85}$

(c) $\frac{y}{H_d} = -2.0\left(\frac{x}{H_d}\right)^{1.85}$

(d) $\frac{y}{H_d} = -2.0\left(\frac{x}{H_d}\right)^{1/1.85}$

8. Water emerges from an ogee spillway with velocity = 13.72 m/s and depth = 0.3 m at its toe. The tail water depth required to form a hydraulic jump at the toe is
   (a) 6.48 m (b) 5.24 m
   (c) 3.24 m (d) 2.24 m

9. For a saddle siphon, the maximum operative head is 6.25 m. The width and height of the throat of the siphon are 4 m and 2 m respectively. The coefficient of discharge is 0.90. How many units are required to pass a flood of 300 cumec? (Take $g = 10\ m/s^2$)
   (a) One (b) Two
   (c) Three (d) Four

10 The 'safety valve' of a dam is its :
   (a) drainage gallery (b) inspection gallery
   (c) spillway (d) outlet sluices.

11 Which one of the following spillways is least suited to earthen dams?
   (a) ogee spillway (b) side channel spillway
   (c) chute spillway (d) shaft spillway.

12 The spillway, which can be adopted with ease on gravity as well as earthen dams, is :
   (a) ogee spillway (b) chute spillway
   (c) both ogee as well as chute spillway (d) none of the these.

13 Hydraulic jump is widely used for dissipation of energy in :
   (a) ogee spillways (b) trough spillways
   (c) side channel spillways (d) all of these.

14 If the operating head on an ogee spillway is more than the design head, then:
   (a) the pressure on the crest will be zero
   (b) the pressure on the crest will be negative, causing cavitation
   (c) the pressure on the crest will be positive
   (d) the discharge coefficient of the spillway will be reduced.

15 In the functioning of an ogee spillway, the operating head :
   (a) frequently exceeds the design head
   (b) rarely exceeds the design head
   (c) never exceed the design head
   (d) has no connection with the design head.

16 The portion of a chute spillway, which is known as its control structure, is:
   (a) low ogee weir
   (b) chute channel
   (c) approach channel leading the water from the reservoir to the ogee weir.
   (d) stilling basin at its bottom.

17 The spillway, which can be called as an 'overflow spillway', is essentially:
   (a) an ogee spillway (b) shaft spillway

(c) chute spillway (d) syphon spillway

18 The spillway, which may be sometimes be called a 'waste weir', is :

(a) an ogee spillway (b) a trough spillway

(c) a shaft spillway (d) all of the above

19 A shaft spillway is located :

(a) inside the body of a gravity dam (b) inside the u/s reservoir

(c) inside the d/s reservoir (d) on side flanks of the main dam.

20 The energy dissipation at the toe of a spillway is affected basically by the use of hydraulic jump, in:

(a) a roller bucket

(b) a ski-jump bucket

(c) a sloping apron taken below the d/s river bed

(d) non of the above

21 A troublesome and oscillating hydraulic jump is usually formed in flows, involving the incoming Froude number in the range of 2.5 to 4.5, which normally is met in cases of :

(a) weirs and barrages (b) overflow spillways of dams

(c) both (a) and (b) (d) none of them

22 A very steady and stable hydraulic jump is usually formed in flows, involving the approaching Froude number in the range of 4.5 to 9.0, which normally is met in case of :

(a) weirs and barrages (b) overflow spillways of dams

(c) both (a) and (b) (d) none of them

23 In case of very high overflow spillway, the approaching Froude number of the flow at the toe of the spillway may exceed 9.0, bringing the hydraulic jump to be in the category of a 'strong jump'. In such a case, the energy dissipation arrangement preferably made at the toe of the spillway, may consists of :

(a) bucket type dissipator (b) USBR stilling basin

(c) Indian standard stilling basin (d) none of the above.

24 Standard USBR stilling basin II is useful for energy dissipation at the bottom of an overflow structure, if the approaching Froude number is :

(a) less than 4.5 (b) more than 4.5

25 The spillway gate, which when lowered, can not be seen from a distance, is of the type :

(a) sliding gate (b) roller gate

(c) tainter gate (d) USBR drum gate.

26 Bar screens, used to cover dam outlets to prevent entry of debris or ice into the sluiceway conduits, are called :

(a) gate controlled ports (b) projecting collars

(c) trash racks (d) none of these.

27. Designing of the ogee spillway profile to conform to the shape of the nappe of a sharp crested weir makes

(a) the pressure on the spillway crest always positive

(b) the crest free from separation at all heads
(c) the pressure on the crest zero at the design head only
(d) the pressure on the spillway always zero.

28. Identify the component that is not used in a shaft spillway:
(a) radial piers (b) radial gates
(c) a bridge around the spillway (d) a tunnel.

29. A trash rack is not needed at entrance to a
(a) morning glory spillway (b) siphon spillway
(c) high head-gate installation (d) drum gate installation.

30. Identify the *incorrect* statement :
In a slotted-roller bucket
(a) the ground roller is less violent than in a solid-roller bucket
(b) the ground roller is more violent than in a solid-roller bucket
(c) the height of the boil is less than in a solid-roller bucket
(d) the abrasion of the bucket is considerably less than in solid-roller bucket.

31. Currently, the most commonly used form of vertical lift gates on a spillway crest is
(a) stoney gate (b) sliding gate
(c) fixed wheel gate (d) sliding gate

ANSWERS

1. (c)	2. (a)	3. (c)	4. (b)	5. (c)	6. (b)	7. (a)
8. (c)	9. (b)	10. (c)	11. (a)	12. (b)	13. (d)	14. (b)
15. (b)	16. (a)	17. (a)	18. (b)	19. (b)	20. (c)	21. (a)
22. (b)	23. (a)	24. (b)	25. (d)	26. (c)	27. (c)	28. (b)
29. (d)	30. (d)	31. (c)				

Practice Set

PRACTICE SET

IRRIGATION AND METHODS OF IRRIGATION

Q.1 Describe the chemical constituents which affect the suitability of water for irrigation.

Q.2 List the different methods of application of irrigation water to farm land. In a system which relies on supply of water through field channels, indicate methods of achieving high efficiency of water use.

Q.3 Briefly describe the basic methods of applying irrigation water to fields describing the situations when each method is useful.

Q.4 Describe the irrigation method which is considered ideal from the point of view of efficient water application.

Q.5 (i) Write advantages of sprinkler irrigation method.

Q.6 What is a furrow? Give names of some row crops. State the advantages of sprinkler irrigation method over surface irrigation method.

Q.7 Briefly state the various steps needed for planning an irrigation project.

Q.8 How will you estimate the benefits of an irrigation project?

Q.9 Explain the necessity of irrigation in a tropical country like India. What are advantages and disadvantages of assured irrigation?

SOIL-MOISTURE - PLANT RELATIONSHIPS

Q.10 Estimate the depth and frequency of irrigation required for a certain crop, given:

Root zone depth	:	90 cm
Field capacity	:	22%
Wilting point	:	12%
Apparent sp. gr. of soil	:	1.5
Consumptive use	:	22 mm/day
Efficiency of irrigation	:	60%

Assume 50% depletion of available moisture as the indicator to begin application of irrigation water.

Q.11 A loam soil has field capacity of 22% and wilting coefficient of 10%. The density of the soil is 1500 kg/m^3. If the depth of effective root zone is 0.8 m, determine the storage capacity of the soil. Determine the depth of water to be applied at 50% depletion level, assuming the water application efficiency as 75%.

Q.12 *Explain* : effective root zone of crops, soil moisture tension, field capacity and permanent wilting point.

Q.13 *Explain the terms* : (i) Saturation capacity (ii) Field capacity (iii) Moisture equivalent (iv) Wilting point (v) Available moisture (vi) Readily available moisture.

Q.14 *Explain the terms* : (i) Soil moisture tension and (ii) Soil moisture stress.

Q.15 Discuss critically the quality standards required for irrigation water.

Q.16 Write short notes on :

(i) Sodium-Absorption-Ratio (SAR)

(ii) Sodium hazards of irrigation waters.

(iii) Boron conc. in irrigation waters.

WATER REQUIREMENTS OF CROPS & CANAL IRRIGATION

Q.17 For use in Hargreaves method, in respect of Group-D wheat crop, the following K values are suggested:

% of crop growing season	Value of K
0	0.08
5	0.08
10	0.15
15	0.19
20	0.27
25	0.33
30	0.40
35	0.46
40	0.52
45	0.58
50	0.65
55	0.71
60	0.77
65	0.82
70	0.88
75	0.90
80	0.90
85	0.80
90	0.70
95	0.60
100	0.60

Wheat is grown between 1st November and 15th March. The effective rainfall in the individual months is taken as follows: Nov-0.5 cm; Dec-1.6 cm; Jan-3.2 cm; Feb-2.7 cm; March-nil. Percolation losses are nil throughout. Field Irrigation Efficiency is 70% and Gross Irrigation Efficiency is 75%. The mean class A pan evaporation throughout the respective months are: Nov-13 cm; Dec-11 cm; Jan-9 cm; Feb-15 cm; March-10 cm. Compute the gross irrigation requirement for the whole crop period including 7 cm of net pre-sowing requirement at the site.

Q.18 Work out the irrigation schedule based on the soil moisture concept given the following information. Also extract the data on the total depth of irrigation water required and the respective dates of irrigation water supply.

(a) The crop is grown in an appropriate soil with no restrictive layers within the top 1.5 m depth of soil.

(b) Normal root zone depth of the crop is 1.2 m

(c) Bulk density of soil is 1.35

(d) Field capacity is 18% and permanent wilting point is 7%.

(e) Moisture level in the soil is to be maintained at not less than one-third of available retention. Irrigation will then be done over a duration of 2 days at a uniform rate of supply and at a uniform rate of advance to fully and just compensate for the depletion.

(f) No extra water is ever required for leaching.

(g) Sowing is done on 1st November when the soil moisture is left just at field capacity in the entire root zone.

(h) For the crop, at the location, the average evapotranspiration rates are:

1^{st} Nov-30^{th} Nov	: 1.1 mm/day
1^{st} Dec-31^{st} Dec	: 1.7 mm/day
1^{st} Jan-31^{st} Jan	: 2.4 mm/day
1^{st} Feb-28^{th} Feb	: 1.5 mm/day
1^{st} Mar-25^{th} Mar	: 3.5 mm/day

(i) Harvesting is done on or after 26 March

(j) There is expected an effective rainfall of 24 mm during 4^{th} Jan to 19^{th} Jan both days inclusive, with uniform intensity.

(k) By the end of the crop growth season, only the minimum water needed be left unused in the root zone.

Q.19 A farmer wishes to have his own pump set for the following cropping pattern to be followed in five hectares of his land. Calculate the right size of the centrifugal pump he should have, lit/sec.

Season	Crop	Area to be irrigated, ha	Intensity of irrigation, cm	Rotation period, days
RABI	Wheat	2.0	7.5	12
	Cotton	0.4	7.5	20
	Vegetables	0.4	7.5	10
	Mustard	2.2	5.0	40

For each crop, duration of pumping hrs per day is 10.

Q.20 Define duty, delta and base period of a crop and express the relationships connecting them. Explain briefly with the help of a diagram the concept of 'frequency of irrigation'.

WATER LOGGING & RECLAMATION OF SALINE SOILS

Q.21 Briefly discuss the causes of waterlogging in irrigated areas. Describe the procedure for the design of tile drainage for water logged soils.

Q.22 In a subsurface pipe drainage system, it is desired to keep the highest level of the water table at 1.5 m below the ground surface. The depth of impervious layer from land surface is 10.0 m and the depth of the drain below the land surface is 2.0 m. The mean annual rainfall in the area is 96 cm; and the coefficient of permeability is 6×10^{-6} m/s. Design the spacing of the drain pipes.

Q.20 Indicate different methods of control of water-logging. How does each method affect the local water table?

CANAL DESIGN

Q.24 Briefly describe Lacey's theory of transportation of soil in channels.

Design an irrigation channel in alluvial soil according to Lacey's Regime Theory. The following data are given:

Full supply discharge = 15 cumecs.

Lacey's silt factor : 1.0

Channel side slopes = : 1.

Q.25 Write the Rouse's equation for distribution of suspended sediment in the vertical of a channel and label all the terms.

In a wide alluvial channel having suspended sediment load, the depth of flow is 2.8 m. If a suspended load sampler indicated a concentration of 700 ppm at a point 30 cm below water surface, estimate the concentration at a point 10 cm above the bed in the same vertical. The exponent in Rouse's equation can be assumed as 0.4.

Q.26 A distributary is to be designed to irrigate 3600 ha in Rabi and 1400 ha in Kharif. Kor depth (critical stage watering depth) and Kor period (critical stage watering depth) for Rabi and Kharif are 13.5 cm and 4 weeks and 19.00 cm and 2 weeks, respectively. Design the distributary by Lacey's theory, with slit factor 0.85.

Q.27 What is meant by "regime channel"? Why an artificial earthen channel constructed in an alluvium soil has to be designedinregime condition?

Q.28 What is the significance of loss of head in canal design on the alignment of the canal?

Q.29 Water flows at a velocity of 1.2 m/s in a rectangular channel. The bed slope of the channel is 2×10^{-3} and Manning's roughness coefficient is 0.014. Determine the width of the channel and depth of flow for the most efficient channel section.

Q.30 Design a lined canal for the following conditions:

Discharge = 150 cumecs, Slope = 20 cm/km, Side slopes = 1.5 H : 1 V, Limiting velocity = 1.5 m/s, Manning's N = 0.016.

Q.31 List the different types of canal lining in common use. Drawn a neat sketch of a typical cross-section of a canal carrying a discharge of 60 m^3/s and lined with brick in cement mortar. Mark the salient features on the sketch.

Q.32 State the requirements of good canal lining materials

Q.33 Define critical tractive stress and state the main factors on which it depends. A wide unlined channel carrying silt free water has depth of 2.0m. The maximum tractive stress permissible on the bed to prevent scour is 0.20 kg/m^2. What is the maximum slope that can be given to the channel.

CANAL REGULATION WORK

Q.34 Why are canal falls provided in irrigation channels? With the help of sketches, describe and illustrate a trapezoidal notch fall, labelling all the salient features and their functions in flow management and safety.

Q.35 Explain briefly functions and requirements of a good module. How are modules classified? Mention common examples of each type.

Q.36 Describe briefly the main functions of a 'distributary head regulator' and 'cross regulator' mention the salient steps for their design.

Q.37 Mention the various types of canal falls commonly used indicating distinctive features, and describe briefly design principles of any one popular type.

CANAL HEAD WORKS & SEEPAGE THEORY

Q.38 Discuss the role of sheet piles in foundation of weirs on permeable soils. What are the relative advantages and disadvantages vis-a-vis the location of these piles?

Q.39 A homogeneous earth dam is 21.5 m high and has a free board of 1.5 m. A flow net constructed yielded the following results:

Number of potential drops = 12

Number of flow channels = 03

The dam has a horizontal filter of 15 m length on its drawdown end. Calculate the discharge per metre length of the dam, if the coefficient of permeability of the dam material is 2.7×10^{-5} m/s.

Q.40 Discuss in brief various causes of failure of weirs.

Q.41 Explain the design principles of Khosla's theory.

Q.42 What is a launching apron? Where and when it is used? Explain its usefulness.

Q.43 Differentiate between a and weir and barrage. Why barrages are considered to be better than weirs?

Q.44 What is the meant by "safe exit gradient"? Indicate its significance in the design of a barrage on a permeable foundation.

Q.45 Draw a schematic diagram (in plan view) of a barrage on an alluvial river, with provision for overflow and fish passage, for diversion of flow into a main canal, including for sediment extraction. On the diagram, label the several features and the schematic flow paths.

Q.46 In Khosla's theory, how is the exit gradient of a weir in permeable foundation estimated? Using the theory, how can the factor of safety of a weir design against piping be estimated?

Q.47 Enumerate the forces acting on an upright wall breakwater and explain their destructive effects.

Q.48 An irrigation canal takes off from a perennial river. Sketch a typical layout of a diversion headworks. Indicate therein the various components of the headworks.

Q.49 Describe the function and design considerations of under sluices provided in diversion head work.

Q.50 Following particulars are recorded from a barrage:

(i) Maximum reservoir level = 212 m

(ii) Pond level = 211 m

(iii) D/s high flood level in the river = 210 m

(iv) Maximum design flood discharge = 3500 m^3/sec

(v) Crest level of the barrage = 207 m

(vi) Crest level of the head regulator = 208 m

(vii) Coefficient of discharge = 2.10 $m^{1/2}$/sec for barrage

(viii) Coefficient of discharge = 1.50 $m^{1/2}$/sec for head regulator

(viii) River bed level = 205 m

(ix) Design discharge of main canal = 500 m^3/sec

Determine the number of gates required for the barrage and the head regulator if each gate has 10 m clear span. Neglect

(a) End contractions due to piers and abutments

(b) Velocity of approach

If a stilling basin is provided d/s of the barrage for the energy dissipation, find the length and RL of the basin floor. Assume that the length of basin is 5 times the conjugate depth required for hydraulic jump. Neglect losses due to friction.

Q.51 Explain 'Retrogression of d/s levels' consequent to weir construction.

RIVER ENGINEERING

Q.52 What is meant by river training? What are its objectives? What are the in-situ features that help in this context?

Q.53 Describe salient features of meandering of rivers in alluvial plains. Describe suitable methods of training such rivers.

Q.54 Name the different methods of river training. Explain 'guide bank' method of river training.

Q.55 Classify rivers on the basis of variation in discharge and also the plan-form of the river.

Q.56 Explain the purpose of providing groynes. State the factors which effective the selection of type and performance of groyne. Mention about the orientation, desirable length and spacing of groynes in a groyne system.

CROSS DRAINAGE WORKS

Q.57 What are the various types of cross drainage works? What is the purpose of these C.D. works? Describe the use of syphon in cross drainage works.

Q.58 Describe briefly 'uplift pressures on syphon flooring in cross drainage works'.

DAMS AND RESERVOIRS

Q.59 For reducing seepage of water through the body of an earthen dam, what provisions are made? What is piping in earthen dams? How is it prevented?

Q.60 Briefly state the criteria for selection of gravity dam site.

Q.61 A 30 m high homogeneous earth dam with top width of 5 m and side slopes of 3 : 1 and 2:1 on u/s and d/s respectively is analysed for stability of d/s slope on a section drawing to a scale of 1 cm = 3 m. The soil properties are as under:

Cohesion–2000 kg/m^2

Angle of internal friction–20°

Dry weight–1800 kg/m^3

Submerged weight–1200 kg/m^3

The radius of the slip arc is 60 m and the phreatic line lies below the slip arc. The angle subtended by the slip arc at its centre is 60°. The areas of N and T diagrams are 10 cm^2 and 7 cm^2 respectively.

Compute the factor of safety. Is the dam safe? Give reason.

Q.62 Analyse the elementary profile of a gravity dam based on stress criterion for reservoir full condition and evaluate the base width, and the normal and principal stresses at the toe. Hence define the limiting height of dam.

Q.63 Explain the basic causes of failures of earth dams.

Q.64 Describe conditions which would favour selection of a gravity dam.

Q.65 What is the function of a drainage gallery in a concrete gravity dam?

Q.66 Describe the forces acting on a dam and the main causes of failure of a gravity dam.

Q.67 Classify different types of dams. What type of analysis is to be carried out from the point of view of its breaching/breaking?

Q.68 What is an elementary profile of a gravity dam? Derive the expressions for determining base width of such dam based on stress and sliding criterion. Also derive the expressions for normal, principal and shear stresses at the base of the dam.

Q.69 What are the various assumptions made in two dimensional design of gravity dams?

Q.70 Investigate the stability against overturning of a solid gravity dam at the base section. The dam is in the shape of a right angled triangle with the u/s face vertical.

Height of dam = 120 m

Slope of d/s face = 0.75 H : 1V

Height of water = 120 m

Free board = 0

Height of tail water = 0

Unit weight of concrete = 2400 kg/m^3

Uplift intensity factor = 0.5

Allowable coefficient of friction = 0.75

Neglect silt pressure, earthquake forces, wave forces and ice pressure. Assume any suitable data as required.

Q.71 List the various forces to be considered in the stability analysis of a gravity dam. Indicate the expressions to determine their values.

Q.72 The theoretical profile for a concrete gravity dam is a right angled triangle with the water face vertical. Show that for the line of thrust to act within the middle third, the base should be $\frac{H}{\sqrt{S}}$ where H is the vertical height and S is the specific gravity of concrete

Q.73 State the various safety criteria for a gravity dam explaining each one critically. Also elucidate how the dam will fail if any criterion is not satisfied.

SPILLWAYS, ENERGY DISSIPATORS & SPILLWAY GATES

Q.74 Discuss energy dissipation with its purpose. Enumerate the types of energy dissipators.

Q.75 What are the common types of spillways? Explain with neat sketches the syphon type.

Q.76 Enumerate various types of energy dissipation devices which may be recommended below spillway in relation to the relative positions of tail water rating curve and jump height rating curve.

Q.77 Describe the principle used in the development of ogee spillway profile. Describe the discharge characteristics of ogee spillways with vertical u/s face under free flow condition.

Q.78 Write a short note on Radial Gates.