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IRRIGATION ENGINEERING



BY JASPAL SIR

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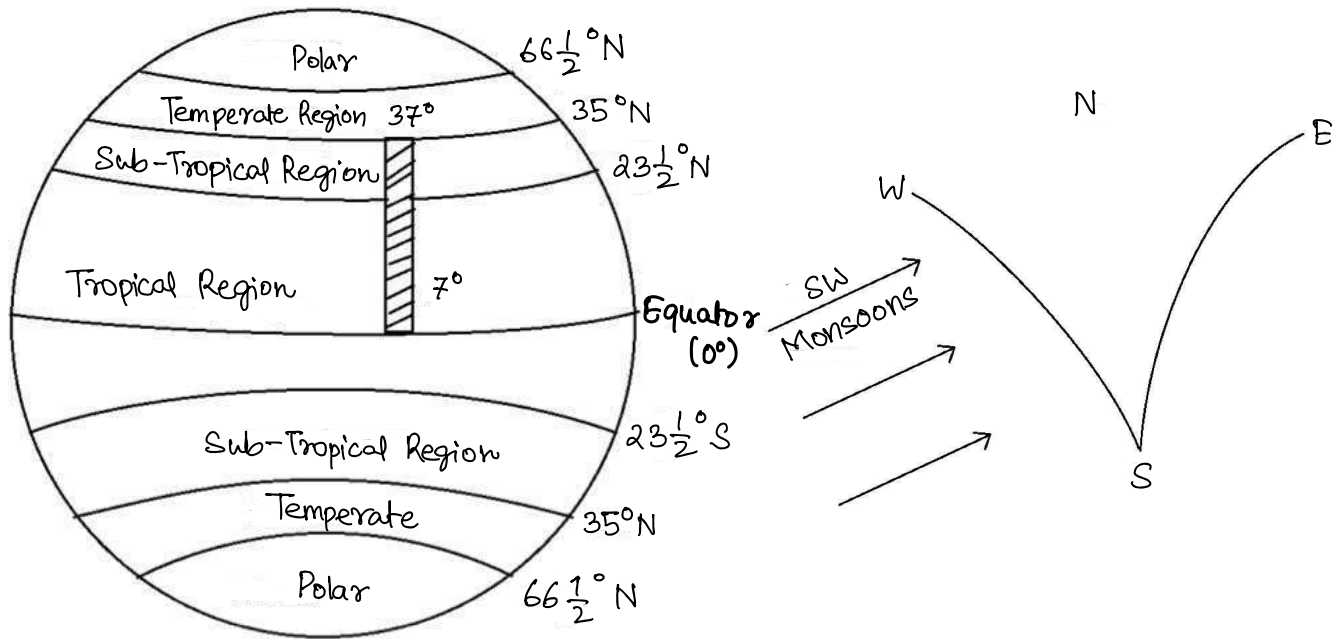
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INTRODUCTION TO IRRIGATION



Q. Why Irrigation is Required in India.

Ans:



- Every Crop Requires certain amount of water at regular interval throughout the growth period at desired time so that the crop may attain maturity.
- If in an area natural rainfall is both timely and sufficient then irrigation facilities are not required to develop.
- In a tropical country like India rainfall is neither sufficient nor timely therefore irrigation facilities are required to be developed.
- Water requirement of crop may not be fulfilled by natural rainfall, in such case also irrigation facilities are required to be developed, so that adequate amount of water may be supplied to the crops required for achieving max. cultivation.

RICE

(Kharif Monsoon)
(June - Sept)

Sowing (June) Cultivation (Sept)

WHEAT

(Rabi Winter)
(Nov - March)

Sowing (Nov) Cultivation (March)

Hence, irrigation may be defined as artificial application of water at desired interval for specific duration throughout the entire growth period of crop so as to attain full maturity.

Note: Classification of area based on deficiency of rainfall.

ARID (Dry)

- These are the areas where for cultivation of any type of crop irrigation is must.

Semiarid {Semi-dry}

- These are the areas where cultivation of inferior crops can be done without irrigation.

Ex $\rightarrow$ Thar, Kacchh
Leh and Ladakh

Ex $\rightarrow$ Telangana, Bundelkhand,
Uttaranchal.

Note: Inferior Crops: These are those crops which can be cultivated in inferior condition. Here inferior conditions corresponds to

- (a) Poor Quality of Soil
- (b) less Availability of water

Example - Bajra, Jowar

- These inferior crop shows low yield and high suicide.



•• ADVANTAGES OF IRRIGATION

- (i) obtaining maximum yield since by developing irrigation facilities we can supply adequate amount of water which is required by the crop max. yield is obtained.

$$\text{Yield} : \text{Yield} = \frac{\text{Cultivation}}{\text{Area}} = \frac{(\text{tonnes})}{\text{Hectares}} \quad \{1 \text{ hec} = 10^4 \text{ m}^2\}$$

- (ii) Elimination of mixed cropping

- Growing of 2 or more crops together in the same field in the same time is called mixed cropping

- Mixed cropping has following drawbacks

(a) Max yield cannot be obtained from entire area, at any given time quantity is reduced.

(b) Therefore irrigation helps in eliminating mixed cropping, because adequate amount of water can be supplied to crop that is required for its max yield.

- (iii) Improving Domestic water supply $\rightarrow$ Development of irrigation facilities in an area helps in increasing/ Supporting the water supply in nearby villages and towns where other sources of water is not available or there is scarcity of water.

- (iv) Generation of Hydro - Electric power $\rightarrow$ Cheaper power generation can be obtained from water development project primarily design for irrigation.

- (v) Facility of Communication $\rightarrow$ Irrigation channels are primarily designed for embankments and inspection road which connect as the mode of communication also.

- (vi) Afforestation $\rightarrow$ Trees are generally grown along the banks of the channel which increases the proportion of green cover and also helps in reducing the soil erosion.

- (vii) optimum Benefits.

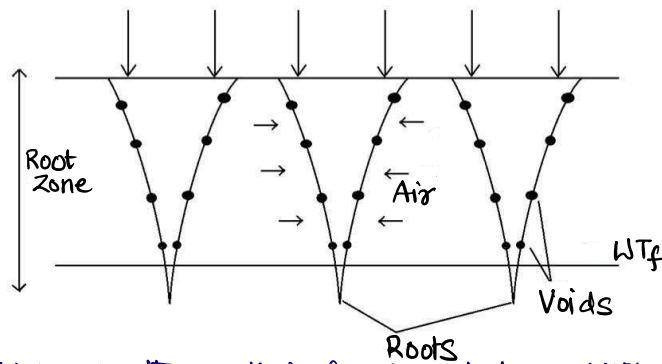
- (viii) General prosperity.

•• DISADVANTAGES OF IRRIGATION

- (i) Irrigation may lead to creation of wet climate condition, which results in ambient growth of mosquitoes and micro-organisms that increases the risk of water-borne disease.

- (ii) Intensive irrigation may lead to waterlogging if water is applied over a agricultural land or field than certain fraction get lost due to seepage and ultimately joins ground water.

- If the process continues for few successive years, then ground water table rises and if it reaches upto root zone, which will result in choking of pores present in the roots, that are responsible for Aeration, hence growth of crop reduce.



iii) Since Indian soils are deficient in nitrogen, it is supplemented by addition of fertilizer (Urea, NH_2 , CONH_2) which along with water seeps up to ground water level, thereby increases concentration of NO_3^- in H.

- If this ground water is used for domestic use, it may lead to methemoglobinemia.

• TYPES OF IRRIGATION

Irrigation is broadly classified into 2 types :-

- i) Surface Irrigation
- ii) Sub-Surface Irrigation



i) SURFACE IRRIGATION: In this method water is applied over the surface and agricultural field remaining in wet condition.

- It is further classified as :-

- a) Flow Irrigation
- b) Lift Irrigation.

a) FLOW IRRIGATION: When the water is available at higher level and it is supplied to the lower level by the action of gravity then it is termed as flow irrigation.

b) LIFT IRRIGATION: If the water is lifted by some mechanical or manual mean and then supplied for irrigation, it is termed as lift irrigation.

- Use of tube well, open wells for supplying irrigation water falls in this category.

- Flow Irrigation is further classified as -

- A) Perennial irrigation
- B) Flood irrigation.

A) PERENNIAL IRRIGATION: In this system of irrigation constant and continuous water supply is assured to the crops in accordance with the requirement of the crop throughout the crop period in this system, water is supplied through canal distribution system, taking off from a reservoir and weir.

This Perennial Irrigation is further classified as :-

- i) Direct Irrigation
- ii) Storage Irrigation

- When irrigation is done by diverting the river runoff into the main canal by using diversion headwork across the river it is termed as direct irrigation.

- If the dam is constructed across a river to store the water during Monsoon, so as to use in dry period is termed as Storage Irrigation.

B) FLOOD IRRIGATION: This type of irrigation is also termed as inundation irrigation.

- In this method soil is kept submerged and thoroughly flooded with water so as to cause saturation of the land.
- The moisture soaked by soil, is further used by the crops over the crop period for growth.
- It is usually adopted in areas near rivers in alluvial soil.
- This system of irrigation is also called uncontrolled irrigation.

② SUB-SURFACE IRRIGATION: In this method water does not wet the soil surface as it is being applied in root zone below the ground surface.

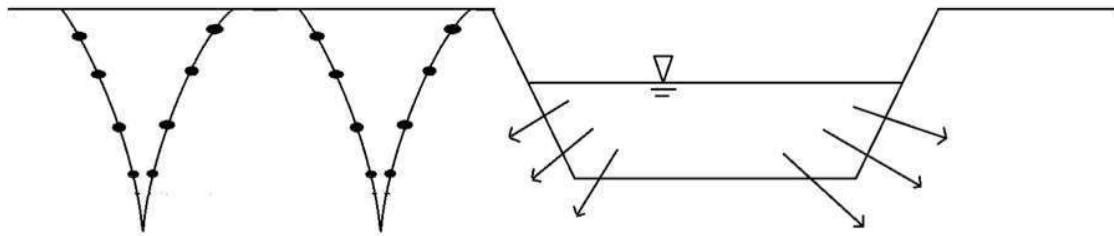
- In this case, roots of plants utilize the water by capillarity.

It is further classified into :-

① Natural

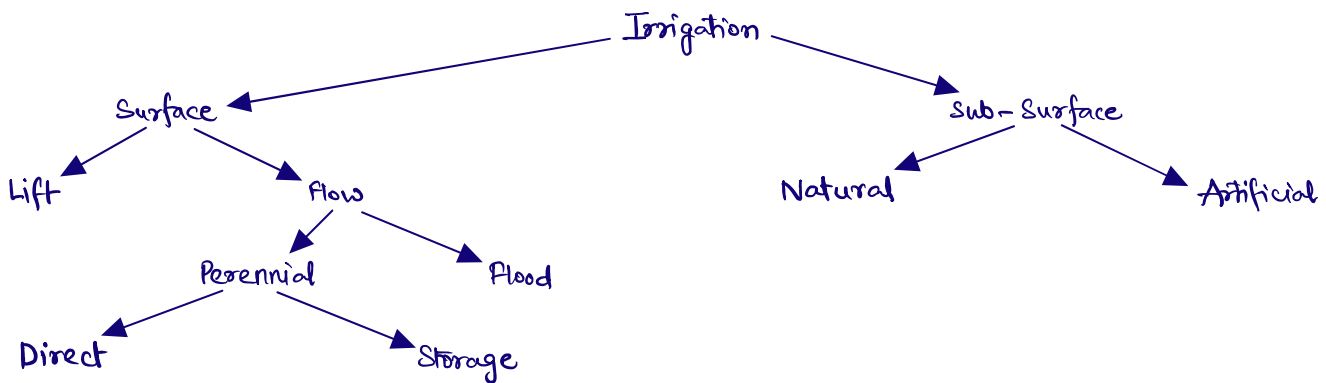
leakage water from channel goes underground and during passage through the subsoil it may irrigate the crops by capillarity.

② Artificial



In this method (Artificial) water is supplied artificially direct into the root zone with the help of network of pipe and during its passage through the subsoil it may irrigate the crops by capillarity.

- Since in this case, cost is very high, it is not used in India.

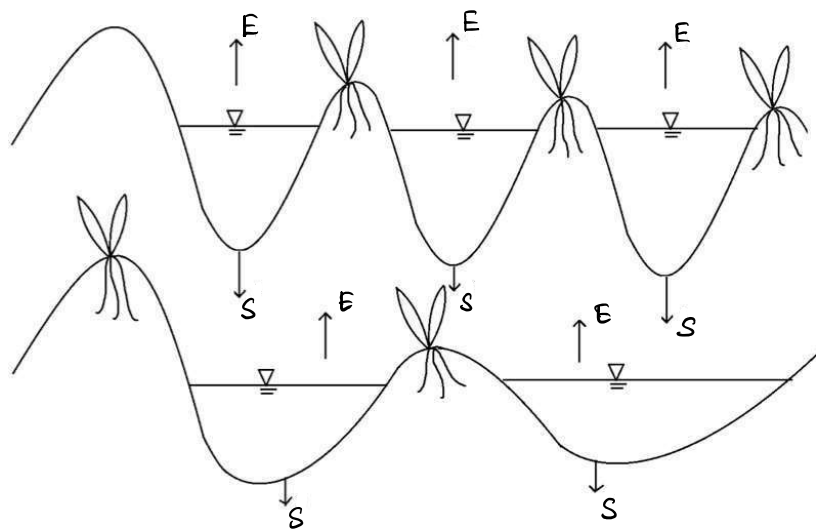


•• METHOD OF WATER DISTRIBUTION

① FREE FLOODING: It is also termed as ordinary or wild flooding.

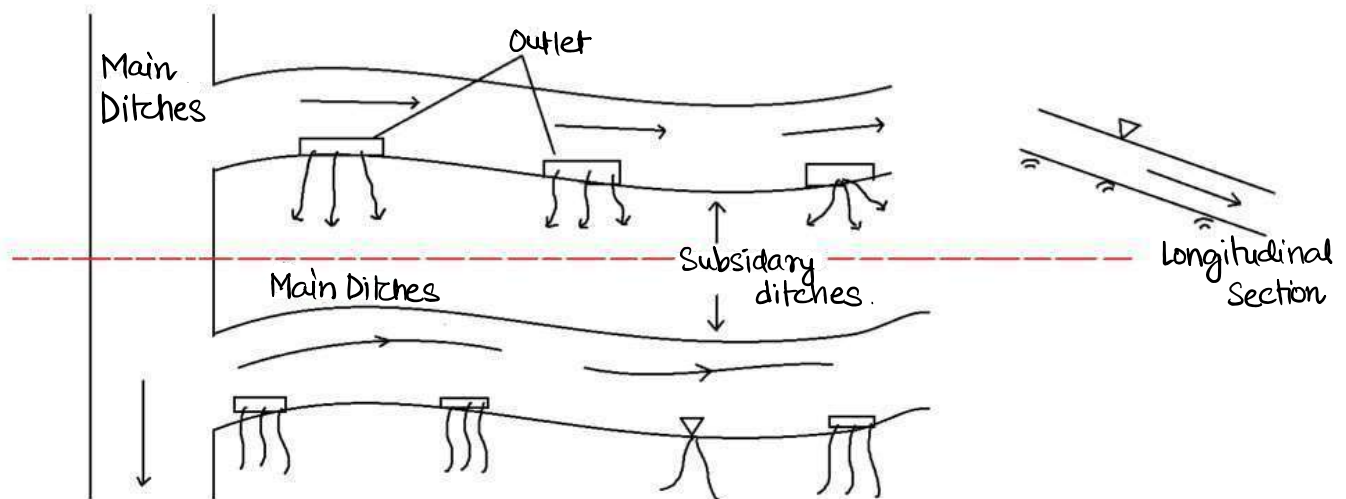
- In this method ditches are excavated into the field and they may be either on contour side or up and down the slope.
- Water from these ditches flows across the field.
- After the water leaves the ditches no attempt is made to control, hence it flows freely.
- Land preparation is low, labour requirement is adequate.
- It is adequate and water application efficiency is low.
- It is suitable for close growing crops as losses in this case are comparatively less or where land is steep.
- The lateral or subsidiary ditches in this case are spaced about 20 to 50 m apart depending upon the slope, texture of soil and type of crop.
- It is suitable for rolling terrain where other methods are not suitable.





— Close Growing Crop

— Far Growing Crop



② **BORDER FLOODING**: In this method, land is divided into no. of strips, separated by low levees [Note: levees are small embankment, which don't permit the flow of water across it] that are termed as Borders.

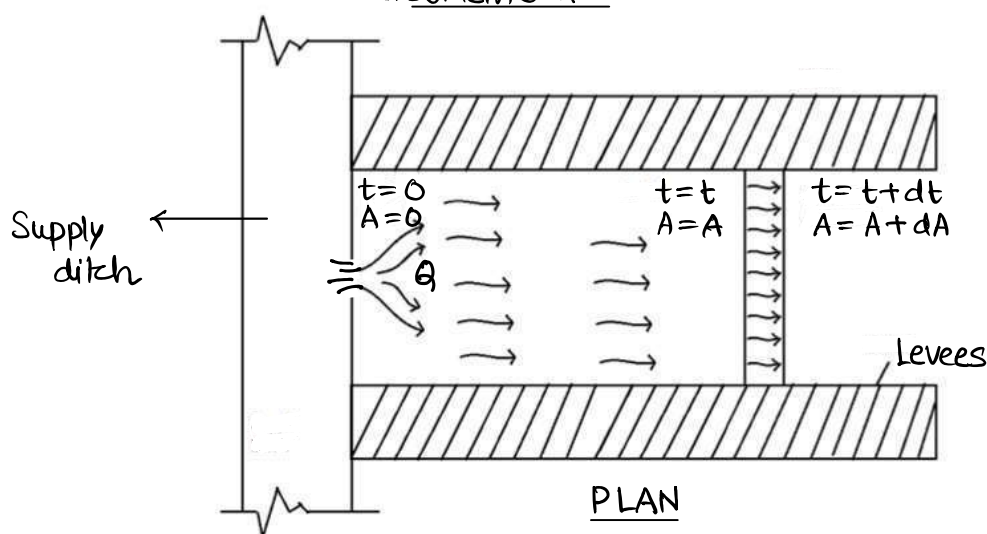
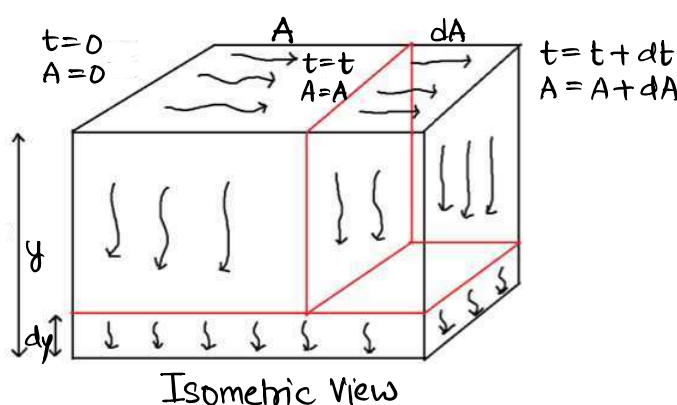
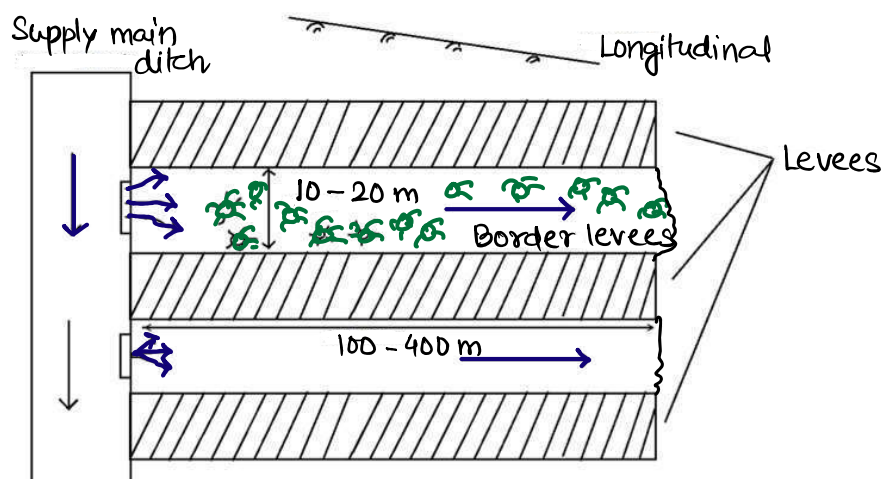
- The area Confined in each Strip is of the order of 10-20 m in width and 100-400 m in length.
- To prevent water from Concentrating on either Side of the Border, the land is levelled Perpendicular to flow.
- Water is made to flow from Supply channel at regular interval.

Note: The Size of Supply ditch depends upon type of Soil, Coarse Grained Soil with high infiltration will require high discharge rate to Cover the entire field therefore, Size of Supply ditch should be comparatively bigger.

- for fine grained Soil, where infiltration rate is low smaller ditches are required to avoid losses due to runoff at lower Reaches.

This method is very Popular among the other methods which are available.

- Short and narrow Strips are found to be more Efficient.
- Entry of Water into Strips generally Controlled by placing a gate in Supply ditches.



A relationship between the discharge through the Supply ditch, the average depth of water flowing over the strip (y), the rate of infiltration of Soil (f), the area of the land irrigated (A) and the approximate time required to cover the given area is given by -

$$t = 2.303 \cdot \frac{y}{f} \cdot \log_{10} \left\{ \frac{A}{A - fA} \right\}$$

Volume of Soil getting irrigate in the time dt .

$$\begin{aligned} &= dA(y - dy) + (A + dA) dy \\ &= dA \cdot y - dA \cdot dy + A \cdot dy + dA \cdot dy \\ &= dA \cdot y + A \cdot dy \quad \text{--- (i)} \end{aligned}$$

Volume of water Supplied to the field in time dt .

$$= Q \cdot dt \quad \text{--- (ii)}$$

Assuming losses = 0

Volume of water Stored = Volume Supplied.



$$dA \cdot y + A \cdot dy = Q \cdot dt$$

If rate of infiltration is f , then depth of water dy infiltrated in time $dt = f \cdot dt$

$$dA \cdot y + A \cdot f \cdot dt = Q \cdot dt$$

$$y \cdot dA = Q \cdot dt - A \cdot f \cdot dt$$

$$y \cdot dA = (Q - A \cdot f) \cdot dt$$

$$\frac{y \cdot dA}{Q - A \cdot f} = \int_0^t dt$$

$$y \int_{A=0}^A \frac{dA}{(Q - A \cdot f)} = \int_{t=0}^t dt$$

$$y \left| \frac{\ln(Q - A \cdot f)}{-f} \right|_0^A = [t]_0^t$$

$$-\frac{y}{f} \left| \ln(Q - A \cdot f) - \ln Q \right| = t$$

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

$$\frac{t \cdot f}{2.303 \cdot y} = \log_{10} \left(\frac{Q}{Q - A \cdot f} \right)$$

$$\text{let } \frac{t \cdot f}{2.303 \cdot y} = x \text{ then, } \Rightarrow x = \log_{10} \left(\frac{Q}{Q - A \cdot f} \right)$$

$$\Rightarrow 10^x = \frac{Q}{Q - A \cdot f}$$

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

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

$$A = \frac{Q}{f} \left\{ \frac{10^x - 1}{10^x} \right\}$$

Max. Value which can be attained by $\frac{10^x - 1}{10^x} = 1$

Hence, max Area of field that can be irrigated as given by $A_{\max} = \frac{Q}{f}$

it can also be concluded that after this max. area is irrigated, losses will take place.

- The discharge per Unit Area of the border strip (Q/A) should be varied accordingly to the infiltration capacity of the field (f), or else excess water would be lost or Entire area would not be irrigated.

$$\frac{Q}{A} = f \left\{ \frac{10^x}{10^x - 1} \right\}$$

Q. Find the time required to irrigate a strip of land whose Area is 0.04 hectare from a tube well having a discharge of 0.02 m³/sec. The infiltration Capacity of the soil can be taken as 5 cm/hr and avg. depth of flow in the field is 10 cm. Also compute the max area which can be irrigated for the given discharge.

$$\text{Ans: } t = 2.303 \cdot \frac{y}{f} \log_{10} \left(\frac{Q}{Q - A \cdot f} \right)$$

$$= 2.303 \times \frac{10}{5} \log_{10} \left(\frac{0.02}{0.02 - \frac{0.04 \times 10^4 \times 5 \times 10^{-2}}{60 \times 60}} \right)$$

$$t = 0.64 \text{ hr} = 39 \text{ min}$$

$$(ii) A_{\max} = \frac{Q}{f} = \frac{0.02}{5 \times 10^{-2}} \times 3600 \times 10^{-4}$$

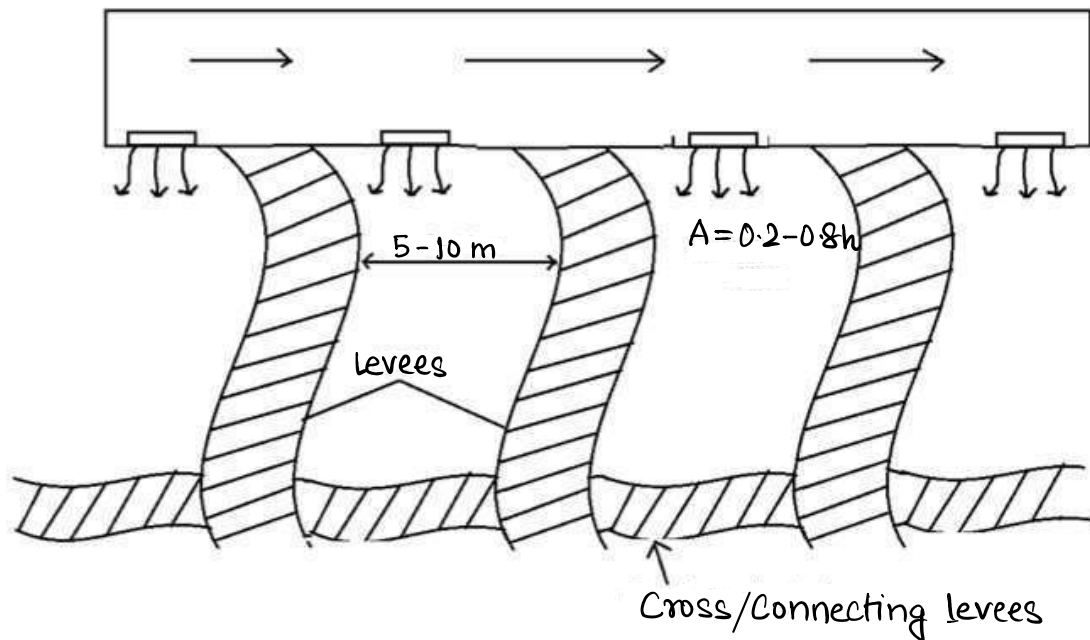
$$= 0.144 \text{ hac.}$$



Note: 1 Cumec = m^3/sec
1 cusec = $\text{ft}^3/\text{sec} = 0.028 \text{ Cumec}$

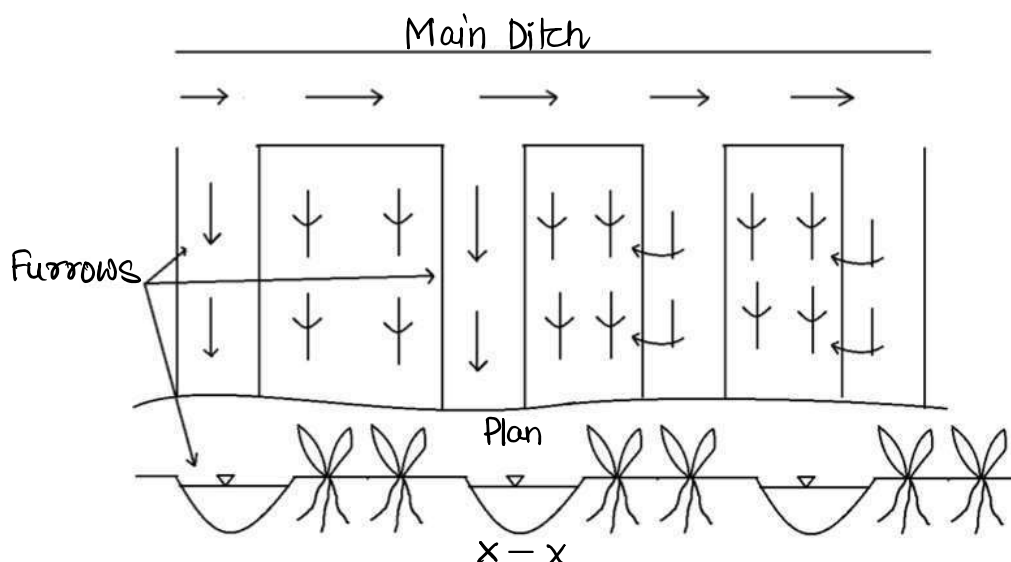
③ CHECK FLOODING: It is similar to Ordinary flooding Except that water is Controlled by Surrounding the check area with low and flat levees.

- levees are generally Constructed along the Contours having vertical interval of 5-10 m.
- These levees are Connected with Cross levees that Confines an area of approximately 0.2-0.8 hac Supply ditch.



④ FURROW IRRIGATION: In this method Only $\frac{1}{5}^{\text{th}}$ to $\frac{1}{2}$ of the land Surface is wetted by water Unlike Previous method in which the Entire land Surface was flooded.

- It helps in reducing Evaporation losses, less Pudding of Soil and Permits the Cultivation Soon after the irrigation.
- Furrows are Narrow field ditches Excavated between the row of crop and Irrigation water is filled in them.
- Spacing of furrows is determined by the spacing of crop.
- Furrows may vary from 8-30 cm in depth and may 400 mtr long.
- Excessive long furrows may results in excess percolation near the upper end and water near the tail end.

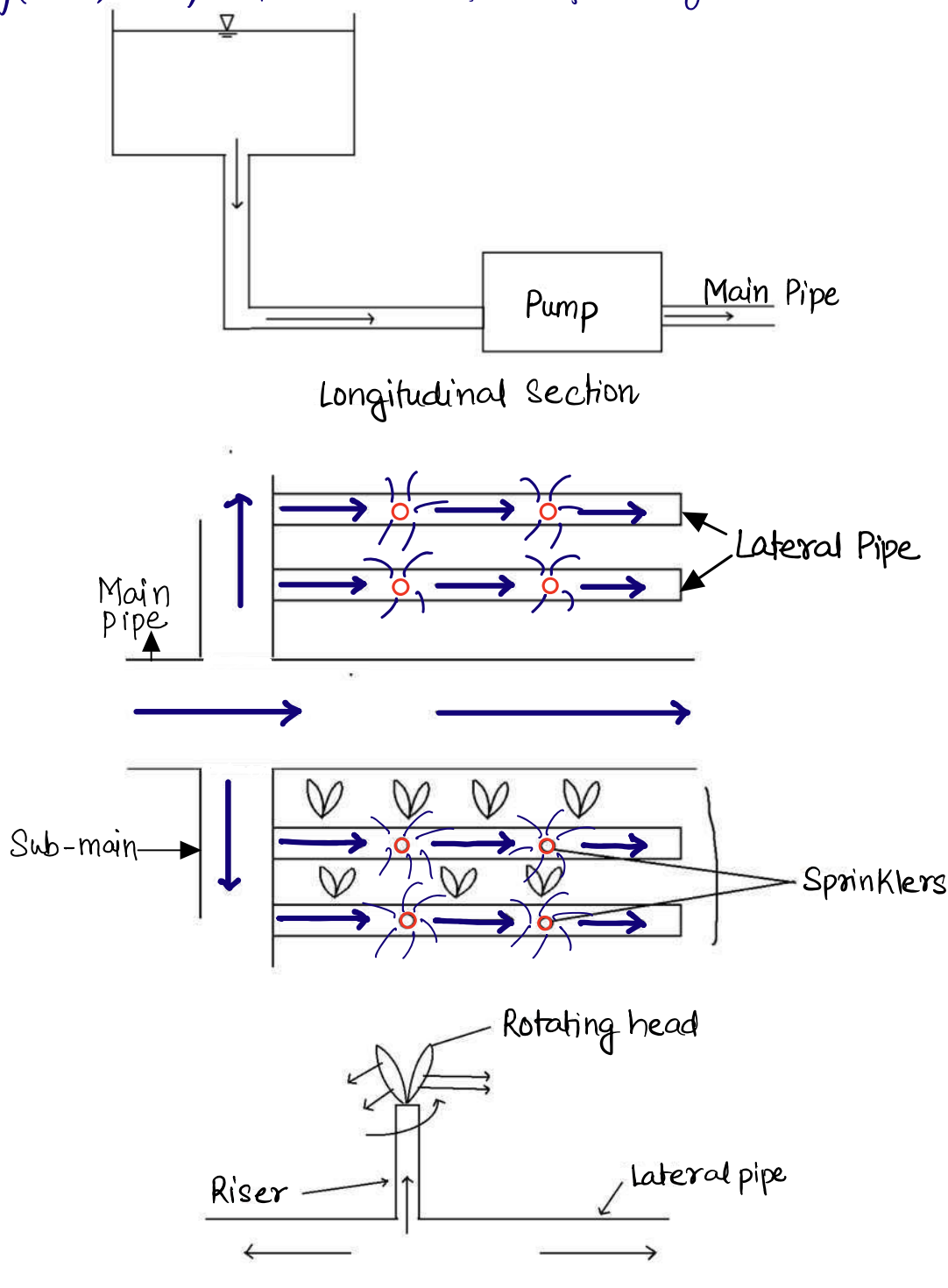


- This can be used only for those crops which can be cultivated along the row.
Ex: $\rightarrow$ Sugarcane, Potato and Maize.

- Deep Furrows are widely used for row crops and small furrows are particularly suitable for relatively irregular topography and close growing crops.
- Water may be diverted into the furrows by opening in the bank or by use of hose pipe.

⑤ SPRINKLER IRRIGATION In this method, a network of pipes and pumps are laid over the field and water is supplied in the form of spray which resembles natural rainfall thereby it is also termed as Artificial Rainfall.

- In this method losses $\{ \text{runoff and seepage} \}$ comparatively less hence it offers very high efficiency (80-85%) thereby it is best method for surface irrigation.



Note: Riser is provided to increase the area of spread.

•• CONDITIONS FAVOURING THE ADOPTION OF SPRINKLER METHOD

- ① When the Land Slope is Steeper and Soil is Easily Erodible.
 - If Soil is Erodible i.e. its Shear Strength is less in such case rate of supply of water is kept less than rate of filtration in order to minimize runoff and avoid the erosion of Soil.
- ② When the Land Soil is Excessively Permeable or highly impermeable.
 - Since in this case rate of supply can be controlled in highly permeable soil it can be increased in low permeable soil it can suitably reduced.
- ③ When the land is irregular.
 - In such case since other surface method of distribution of water is not suitable, sprinkler method is used.
 - This method is preferred if there is deficiency or scarcity of water.
 - However, if land is irregular and water is in excess free flooding is adopted.
- ④ When water table is high
 - Since in this case, rate of supply of water can be controlled, it is reduced to such an extent, to be used only by crops for their growth, avoiding seepage losses which reduced the tendency of water logging.
- ⑤ When the area is such that seasonal water requirement is low such as near coast.
- ⑥ When the water is available with difficulty and is scarce.
- ⑦ When the crop to be grown are such
 - Ⓐ as to required humidity control.
 - Ⓑ crops having shallow roots.
 - Ⓒ crops requiring high and frequent irrigation.

•• TYPES OF SPRINKLER SYSTEM

These system are of following type:-

- ① Permanent System
- ② Semi-Permanent
- ③ Portable System

- In Permanent System, pipes are permanently buried into the soil in such a way that they do not interfere with the farming operation.
- In Semi-Permanent System the main lines are buried in the ground with the laterals are portable / flexible.
- In the Portable System, the main as well as lateral pipes are portable.

•• ADVANTAGES AND DISADVANTAGES

- Seepage losses which occur in earthen channels of surface irrigation are not observed here.
- Land levelling is not required.
- It offers high efficiency (80-85%).
- It is less labour oriented.
- It leaches down the salts and prevents the water logging or salinity of soil.
- Fertilizers can be uniformly applied in this case by mixing it in irrigation water only.

- It prevents the erosion of soil.
- it is suitable for all type of soil and all type of topography.

•• DISADVANTAGES

- Since Power and technical knowledge required is more, this method proves to be costly.
- High wind may divert the pattern of distribution.
- In areas of high temp. and high wind velocity evaporation losses are comparatively more.
- It causes interference with farming operation.
- They are not suited for the crop requiring frequent and large depth of irrigation. Ex → Rice.
- Initial cost of installation is very high.
- A constant water supply is needed for this method.
- Quality of irrigation water required in this case is comparatively better, free from suspended soil so as to avoid clogging of pipe.

⑥ DRIP IRRIGATION/TRICKLE IRRIGATION: It is an artificial subsurface method of irrigation in which water is applied directly into the root zone with the help of pipe and perforation.

In this case evaporation and percolation losses are negligible hence its efficiency is very high { 90-95% }. Since the cost involved in this case is also more/very high, it is not so popular in Indian Subcontinent.

•• QUALITY OF IRRIGATION WATER:

- Quality of irrigation water is represented in terms of following parameters-

① SUSPENDED SOLIDS

The effect of sediment present in irrigation water depends upon the type of irrigated land and type of solid.

- When fine sediments from water having nitrogen, phosphorus, sulphur in it and obtained from erosion of sedimentary rocks, is deposited in sandy soil its fertility is improved.
- However if coarse sediment from water (having deficiencies of N, P, S and obtained from erosion of igneous rock) is deposited in soil it may reduce the fertility of soil.

② TOTAL CONCENTRATION OF SOLUBLE SALTS

- Generally salts of calcium, magnesium, sodium and potassium are present in irrigation water that proves to be harmful for the growth of crop, and if they are excess they lead to-

- Injury to the crops.
- Reduces the osmotic activity.
- Chock the wide in roots hence effect aeration.
- The concentration of salts in water is not of more concern but concentration of salts in soil solution is of more concern, as with passage of time these salts gets deposited in soil and effects the growth of crops.

- Concentration of the salt in soil solution is given by -

$$C_s = \frac{C_a}{Q + R_e - C_u} **$$

C_s = Salinity Concentration of Soil Solution

Q = Quantity of Water Applied.

C_u = Consumptive Use of Water.

R_e = use full / Effective Rain fall



C = Concentration of Salts in Irrigation water.

Note: In Above Expression, Concentration of Salts in rain water is not considered, if it is also considered then,

$$C_s = \frac{C_A + C_R \cdot R_e}{A + R_e - C_u}$$

C_R = Concentration of Salts in Rain water.

- If total Concentration of Salt is more than 700 mg/l, it is harmful for some plant but if its Concentration is more than 2000 mg/l it is harmful for all Crops.

Note: The Salt Concentration is also measured in terms of Electrical Conductivity of water.

• Types of Water and its Suitability for Irrigation is as follows:-

TYPE OF WATER

CONCENTRATIONS OF SALTS

① Low Salinity Water (C_1)

it can be Used for irrigation of almost all types of Crops and for almost all type of Soil except for Soil of extremely low permeability.

100 - 250

② Medium Salinity Water (C_2)

can be Used if moderate amount of leaching is provided and normal Salt tolerant plant can also be grown.

250 - 750

③ High Salinity Water (C_3)

can not be Used on Soil with Restricted drainage

750 - 2250

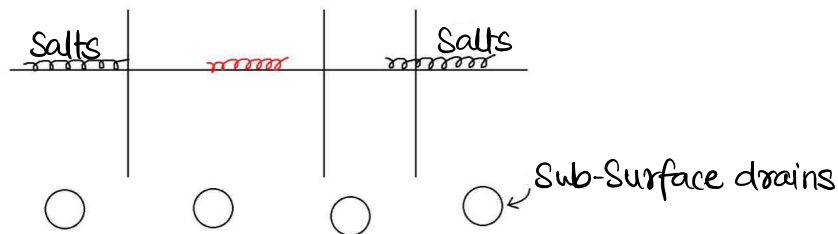
④ Very High Salinity Water (C_4)

it is not Suitable for irrigation.

>2250



Note: LEACHING → It is the process in which water is added to the Soil, so that the Salts Present in Soil may get dissolve in it and can be drained Out with the help of Surface or Sub-surface drain.



(iii) PERCENTAGE OF SODIUM IONS TO OTHER CATION

- Generally, Soil contains Calcium, magnesium and Sodium ions. If the %age of Sodium ions is less than 5% of total Exchangeable Cations it is tolerable.

- If the %age of Sodium ions increases to about 10% following changes induced in Soil

(a) it increases the pH of Soil.

(b) Reduces the Permeability of Soil.

(c) it increases Plasticity of Soil hence, it becomes Sticky when wet and crumbled when dry.

The Proportion of Sodium ions present in the soil is generally measured by a Parameter termed as Sodium Absorption Ratio (SAR).

$$SAR = \frac{Na^+}{\sqrt{\frac{[Ca^{2+}] + [Mg^{2+}]}{2}}}$$



Here Concentration of these cations in terms of meq/l.

• Classification of Irrigation Water and its Uses on the basis of SAR.

TYPE OF WATER

SAR

① LOW SODIUM WATER (S₁)

it can be Used for Irrigation of almost all soils and for all Crops except those which are highly Sensitive to Sodium.

ex: → Stone, fruits, trees

0-10

② MEDIUM SODIUM WATER (S₂)

it is not Suitable for fine textural soil which may required gypsum prior to its Use.

10-18

③ HIGH SODIUM WATER (S₃)

it proves harmful for almost all types of soils and do require good drainage for leaching.

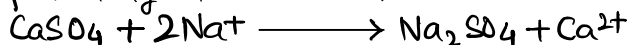
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④ VERY HIGH SODIUM WATER

Not Suitable for irrigation.

>26

Note :- S₂ type of water is treated with Gypsum prior to its Use as it helps in reducing the Concentration of exchangeable Na²⁺ ions.



$$SAR = \frac{[Na^{2+}]}{\sqrt{\frac{[Ca^{2+}] + [Mg^{2+}]}{2}}}$$

•• CLASSIFICATION OF SALINE AND ALKALINE SOIL

Note : Here concentration of Sodium with respect to total exchangeable cations is represented in terms of ESP [Exchangeable Sodium percentage].

Classification	E _c (μmho/cm)	ESP	pH
① Saline Soil (White Alkali/Less Soluble)	>4000	<15	≤8.5
② Alkaline Soil (Non Saline Soil/Black Soil/ Sodic Soil)	<4000	>15	8.5-10

Q. What is the Classification of irrigation water having the following properties

$$\text{Na}^{2+} = 44 \text{ mg/l}$$

$$\text{Ca}^{2+} = 6 \text{ mg/l}$$

$$\text{Mg}^{2+} = 18$$



Electrical Conductivity of this water by di-ionic tester comes to be $350 \mu\text{mho/cm}$ @ 25°C .

Also Comment upon problem which may be faced if this water is used for irrigation and suggest the remedies.

Ans :-

$$\text{SAR} = 44/23.5$$

$$= \frac{\sqrt{\frac{6}{40} + \frac{18}{24}}}{2} = 2.01$$

$$\text{SAR} = 2.01 < 10 \rightarrow S_1$$

$$\text{EC} = 350 \mu\text{mho/cm} \quad (250 - 750 \mu\text{mho/cm}) \rightarrow C_2$$

Hence, this water is $C_2 - S_1$.

ii) If water is used for irrigation it would effect Osmotic balance of plant, would reduce the aeration capacity of roots which leads to injury of plant.

iii) To use this water for irrigation prior leaching must be carried out.

iv) BI-CARBONATES CONCENTRATION

- High Concentration of Bi-Carbonates of Calcium and magnesium may results in Precipitation of Ca^{2+} and Mg^{2+} , thereby increasing the Relative proportion of Sodium ions and causing Sodium hazards.

$$\text{SAR} = \frac{[\text{Na}^{2+}]}{\sqrt{\frac{\text{Ca}^{2+} + \text{Mg}^{2+}}{2}}}$$

v) CONCENTRATION OF TOXIC ELEMENTS

- Toxic elements like boron, Selenium are present in irrigation water [Soap water consist of Excess of Boron].
- Conc. of Boron above 0.3 mg/l may prove to be toxic for certain plants and its Conc. above 4 mg/l is toxic for all the plants.

PREVIOUS YEAR QUESTIONS

ESE

Q. In Drip Irrigation System, Which One of the following emitters is not Based On definition by American Society of Agricultural Engineers (ASAE)?

- (a) Emitter
- (b) Pulsating Emitter
- (c) Long Path emitter
- (d) Multi-outlet emitter.

[IES 2020]

Ans: (b) Pulsating Emitter.

— Emitters are small micro-Irrigation dispensing devices designed to dissipate pressure and discharge of small uniform or trickle of water at a constant rate (as per ASAE definition). In Pulsating emitter, discharge will not be constant.

Q. Consider the following data for Irrigation water.

	Concentration	Milli-equivalent per litre
1	Na^+	24
2	Ca^{++}	3.6
3	Mg^{++}	2

The Sodium Absorption ratio (SAR) is nearly

- (a) 13.1
- (b) 14.3
- (c) 15.5
- (d) 16.7

[IES 2019]

Ans:

$$\text{SAR} = \frac{[\text{Na}^{2+}]}{\sqrt{\frac{[\text{Ca}^{2+}] + [\text{Mg}^{2+}]}{2}}}$$

$$= \frac{24}{\sqrt{\frac{3.6 + 2}{2}}}$$

$$\text{SAR} = 14.34$$

Option (b) is correct.

GATE

- Q. 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.

[2004]

Ans : (a)

- The Conditions favouring the use of Sprinkler Irrigation method are —
- i. When the land Topography is irregular.
 - ii. When the land gradient is Steeper and Soil is easily erodible.
 - iii. When the Soil is excessively Permeable.
 - iv. When Water table is high.
 - v. When water availability is difficult as Scarce.

SSC — JE

Q. Which of the following process includes Spreading of External material On the Soils to increase infiltration and reduce evaporation?

- (a) Mulching
- (b) Paleo Irrigation
- (c) Ploughing
- (d) Tillage

[2017]

Ans :

(a) Mulching

— It is the process of applying or Spreading material to the Surface of Soil.

— It helps in Conservation of Soil Moisture, improving fertility and health of the Soil, reducing weed growth increases infiltration and reduces evaporation.

Q. Method of Applying water directly to the root Zone of the plant is called _____.

- (a) Check Flooding
- (b) Drip Method.
- (c) Furrow Method
- (d) Sprinkler Method

[2017]

Ans :

(b) Drip Method

— In this method, water and fertilizer is slowly and directly applied to the rootzone of the plants, in order to minimize the losses due to evaporation and Percolation.

Q. Which of the following Statement is Correct for Sprinkler Irrigation method ?

- Ⓐ It is Used for the rice and Jute.
- Ⓑ It is Used for the Soil has Very low infiltration rate.
- Ⓒ It is best Suitable for Very light Soil.
- Ⓓ It Requires borders and field channels.

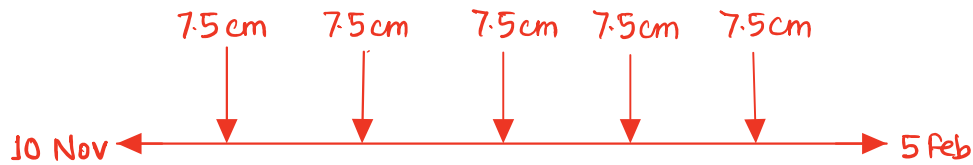
[2017]

Ans :

- Ⓒ It is best Suitable for Very light Soil.
— As deep Percolation is avoided. } Light Soil : These Soil drain quickly after rain and watering.

WATER REQUIREMENT OF CROP

- Every crop requires a certain quantity of water after a fixed interval throughout its period of growth.
- Hence, water requirement of crop signifies the total quantity and the way in which crop requires water throughout its period of growth.



Here water requirement is total 37.5 cm at regular interval of 2-3 weeks.

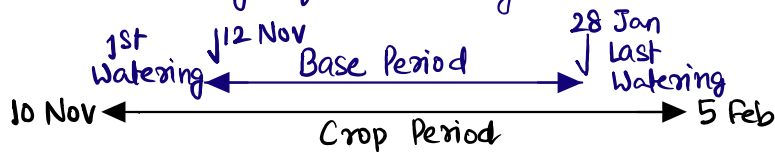
• SIGNIFICANT TERMS USED FOR WATER REQUIREMENT

- ① CROP PERIOD: The time elapsed from the instant of its sowing to the instant of the harvesting of crop is termed as CROP PERIOD.

Crop Period = 3 Months



- ② BASE PERIOD (B): The time between the first watering of a crop at the time of its sowing to its last watering before harvesting is called BASE PERIOD.



- Base Period is less than Crop Period.
- Crop Period is slightly more than base Period but for all practical purpose it is taken same.
- it is also termed as Growth Period.

- ③ DELTA OF CROP (Δ): Every crop requires a certain amount of water after a certain fixed interval of time throughout its growth period.

- The depth of water required every time generally varies 5-10 cm, depending upon type of crop, type of soil and type of climate.
- This water is required at an interval of (termed as frequency) 6-15 days.
- Hence, the total quantity (can be expressed in terms of depth or volume) of water required by the crop for its full growth (maturity) is termed as Delta.
- Average approximate value of Delta of different crops.

<u>CROP</u>	<u>DELTA ON FIELD (cm)</u>
Sugar Cane	120
Rice	120
Fruits	60
Vegetables	45
Wheat	40
Barley	30
Peas	15

Q. If Rice Requires 11.5 cm depth of water at an avg. interval of about 11 days and Crop Period for rice is 121 days where as if wheat requires 7cm of water after every 25 days and the Base Period of wheat is 150 days. Find the ratio of Δ of rice to that of wheat.

Ans:→ For Rice

Frequency of irrigation is known as no. of times irrigation is done = $\frac{121}{11} = 11$

Total depth of water required by rice = $11 \times 11.5 = 126.5$ cm.

for wheat No. of Times irrigation is done = $\frac{150}{25} = 6$

Total depth of water required by wheat (Δ) = $6 \times 7 = 42$ cm.

$$\frac{\Delta_{\text{rice}}}{\Delta_{\text{wheat}}} = \frac{126.5}{42} = 3.02.$$



④ DUTY OF CROP: It is defined as the number of hectare of land area irrigated for given crop for its full growth by supply of $1 \text{ m}^3/\text{sec}$ of water continuously during the entire base Period (B) of the crop.

- Volume of Water applied to the crop during the Base Period of B days @ $1 \text{ m}^3/\text{sec}$ -

$$V = 1 \times B \times 24 \times 60 \times 60 \\ = 86400 \times B \text{ m}^3$$

Assuming all types of losses to be Zero

$$V_{\text{applied}} = V_{\text{stored/required}}$$

$86400 B = \text{Area irrigated (D)} \times \text{depth of water required for full growth of crop } (\Delta)$

$$86400 B = D \cdot \Delta$$

$$D = \frac{86400 B}{\Delta} \quad (\text{m}^3/\text{cumec})$$

$$D = \frac{8.64 B}{\Delta} \quad (\text{hec}/\text{cumec})$$

$$D = \frac{8.64 B}{\Delta} \quad (\text{m})$$

$$D = \frac{864 B}{\Delta} \quad (\text{cm})$$

• Average approx value of duty for Indian Crops

CROP
Sugarcane
Rice
Wheat
Fodder

DUTY (hec/cumec)

730
775
1800
2000

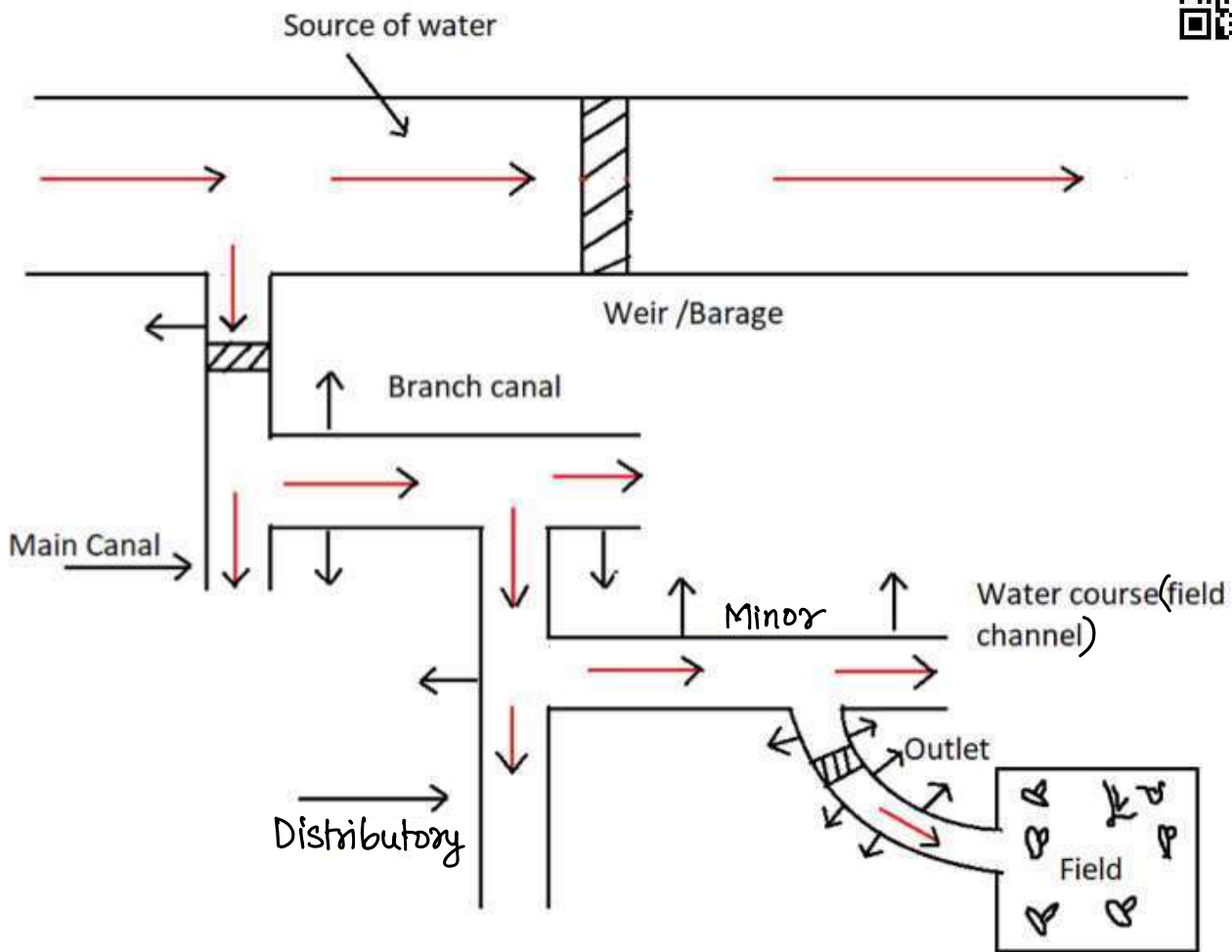
Q. Find the delta for a crop when its duty 1728 hec/cumec on the field, the Base Period of this crop is 115 days.

Ans:→ $\Delta = \frac{864 \cdot B}{D} \text{ cm}$

$$= \frac{864 \times 115}{1728}$$

$$= 57.5 \text{ cm.}$$

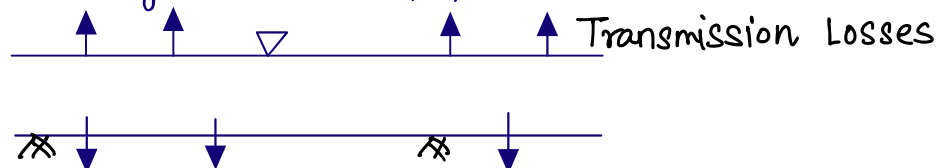
•• DUTY AT VARIOUS PLACES



In Irrigation System water from Source flows into the main canal, from main Canal into the Branch Canal, from Branch canal into the distributary, from distributary to minor, from minor into the water Course or field channel finally into the field.

- During this Passage of water from one point to another point in Irrigation System, loss of the water due to Evaporation and Percolation takes place, which is termed as transmission or Conveyance loss.

- GROSS Quantity — Duty at the head of Main Canal
- LATERAL Quantity — Duty at the head of Branch.
- OUTLET Quantity — Duty at the Outlet Course.
- NET Quantity — Duty at the head of field.



As We Move Ahead in Irrigation System from head of Main Canal to the head of field, length of flow of water in these Channel reduces, which in turn reduces the Transmission losses.

→ Hence, the availability of water for irrigation increases thereby for same delta more area can be irrigated hence Duty increases.

→ Increasing Order → Main < Branch < Minor < Field.

- Note:**
- ① Duty of Water, therefore Varies from One place to Another place and increases as we move D/S from head of the main Canal towards the head of Branch or Water Course.
 - ② Duty at the head of Water Course or Outlet of Minors is termed as Outlet discharge factor.
The Outlet point is the point where the Responsibility of Irrigation department finishes and it becomes the responsibility of Individual farmer to Carry the Water to his field beyond this point.
 - ③ In case of flow/direct irrigation, duty is always expressed as area/discharge.
Eg: hec/cumec is termed as Flow duty or duty.
but in case water is stored i.e. Storage Irrigation duty is expressed in terms of area per Unit Volume of Water available in Reservoir for Irrigation ($\text{hec/m} - \text{m}^3$). This type of duty is termed as Quantity Duty or storage duty.

•• FACTORS AFFECTING DUTY

- ① **TYPE OF CROP:** A Crop Requiring more water will have less Coverage of area for Same Quantity of Water being Supplied as Compared to the Crop requiring less Water.
- ② **TYPE OF SOIL:** If the Permeability of Soil Under Irrigation is high, water loss due to Percolation will be more and hence duty will be less and Vice-versa.
- ③ **USEFUL RAINFALL:** In some of the Cases, if Useful rainfall is more hence Certain demand of Water of Crop is fulfilled by it, thereby more irrigation water is available which provides higher duty.
- ④ **CLIMATE AND SEASON:** As Duty includes the water lost in Evaporation and Percolation these losses will Vary in Different Climatic Condition, diff. Season and different days of the Particular Season. hence, More is the loss due to high temperature (Evaporation loss) Lesser in the duty.
- ⑤ **EFFICIENCY OF IRRIGATION:** If higher is the Efficiency of irrigation lesser is the loss of Water More is the availability of water for irrigation, that results in higher Duty.

•• SIGNIFICANCE OF DUTY:

- ① If area of the field to be irrigated is known and duty of Water for Various crops is also known then the rate at which Water needs to be Supplied (Q) can be Calculated as -

$$Q = \frac{A}{D} \frac{\text{hec}}{\text{hec/cumec}}$$

- ② If Discharge required by the Crop is known and duty of Water for Various Crop is also known then area of field which can be irrigated can be Calculated as -

$$(A)_{\text{hec}} = Q_{\text{cumec}} \times \text{Duty} (\text{hec/cumec})$$

•• METHODS FOR IMPROVING THE DUTY OF WATER

- ① Alkaline Soil should be properly leached.



- 
- II Use of Improved method of Irrigation.
 - III Land to be used must be levelled as far as Possible.
 - IV Field should be properly ploughed before application of Water.
 - V Adoption of rotation of Crops.
 - VI The Source of Irrigation water must be selected within the Prescribed limits.
 - VII highly efficient method of Application of Water must be adopted. {drip irrigation, Sprinkler Irrigation}.
 - VIII By lining of Canal.
 - IX Free Flooding must be Avoided.

•• AGRICULTURAL OR CROPPING SEASON IN INDIA

- i Kharif Season [June - Sept] / Summer
eg: → Rice, Cotton, Jawar, Bajra
- ii Rabi Season [Nov - March] / Winter
eg: → Wheat, gram
- iii Zaid Season [April - June]
eg: → Fruits, Vegetables.
- iv Perennial
eg: → Sugarcane

Note: When a Crop requires water for its Crop Season and also for some time in the beginning of the next Crop Seasoning, Provisions has to be made for this additional water/ overlap, it is termed as 'Overlap Allowance'.

•• CROP RATIO/AREA RATIO/KHARIF RABI RATIO

- The Area to be irrigated for Rabi Crop is generally more than that of Kharif Crop as delta (Δ) for Kharif Crop is more.
- This ratio of Proposed Area of Kharif Season to that of rabi Season to be irrigated is called Kharif rabi ratio.
- Its value is generally 0.5 {Ratio of $\frac{AK}{AR} = 0.5$ }.

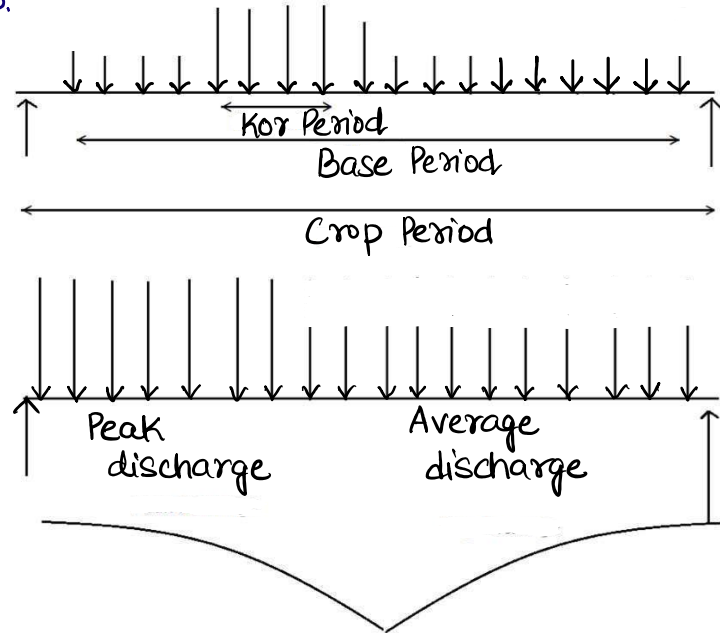
•• PALEO IRRIGATION

- In Initial Stages before the Crop is Sown, the field is very dry, which is generally found at the time of Sowing of rabi {As of hot September Month when the Soil may be too dry to be Sown Easily}.
- In Such Case, Soil is moistured with water so as to help in the Sowing of the Crop, it is termed as PALEO IRRIGATION.

• PUDDING:- To add Water so that the Evaporation losses becomes less.

•• KOR WATERING

- The first watering which is given to a crop when the crop is few cm high is called Kor watering.
- It is usually the max. single watering followed by the watering at usual interval as required by the crop.



The Optimum depth of Kor Watering for diff. crops are different.

for ex: → For Rice : 19 cm
For Wheat : 13.5 cm

- The Period (Limited) for which Kor Watering is done is termed as Kor-Period.
- If this Kor Watering is not received by the plant at desired time period, the plant suffers significant loss.
- for Rice Kor Period : 2-4 weeks
- for Wheat Kor Period : 3-8 weeks

•• CASH CROP

- A crop which has to be encashed in the market for processing, as it can not be consumed directly by the cultivator is termed as Cash Crops.

Ex: → Jute, Sugarcane

- All the Non-food crops are included in this category.
- The food crops are excluded from this list.

Ex: → Wheat, Rice, Maize etc.



- CROP ROTATION: when the same crop is grown again and again in the same field over the period of time, the fertility of land gets reduced as soil becomes deficient in specific number of nutrients or have/accumulate excess of diff. type of nutrients.

- In order to enhance the fertility of the soil and to make the soil regain its original strength, it is necessary to either allow the land to lie/fallow or grow some other types of crop which complement the previous crop.

- This method of growing diff. crops in rotation one after the another on same field is termed as Crop Rotation.

- For Example, following crops are suggested to be grown in Rotation.

① Rice - Gram

② Cotton - Jowar - Gram

③ Wheat - Jowar - Gram



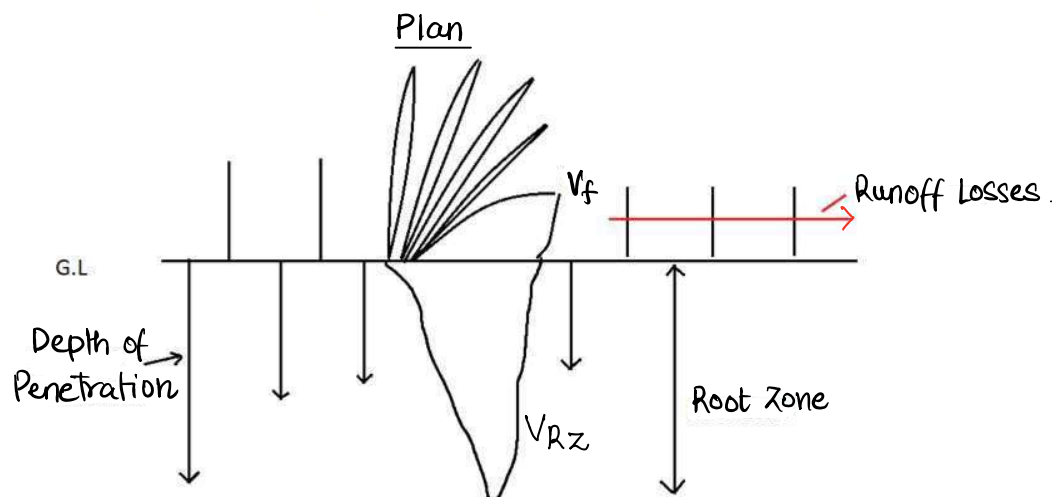
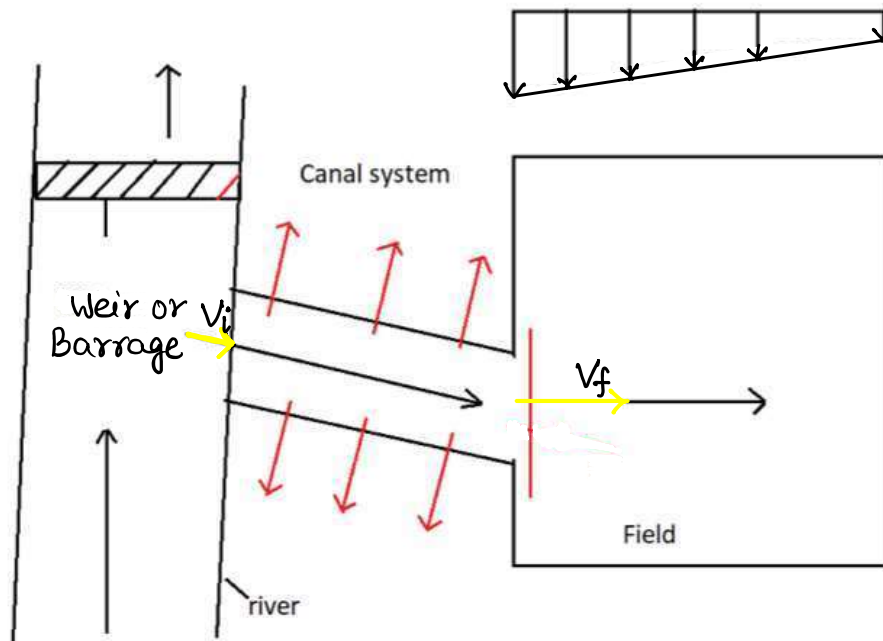
•• IRRIGATION EFFICIENCY :- Efficiency of any system depends on output and input of Parameter (Water) involved.

- Here, it is the ratio of Output Water to Input Water.

- This difference of Output and Input Water is termed as Losses.

- Hence, Higher are losses of water, Lesser is the efficiency.

$$\eta \propto \frac{1}{\text{loss of } H_2O}$$



① EFFICIENCY OF WATER CONVEYANCE: It is the ratio of Water delivered into the field from Outlet point to the Water Entering into the Main channel, at its Starting point.

$$\eta_c = \frac{V_f}{V_i} \times 100$$

This efficiency takes into account of Conveyance losses.

② EFFICIENCY OF WATER APPLICATION: It is the ratio of the Quantity of water stored into the root zone of crop, to the Quantity of water actually delivered into the field.

$$\eta_a = \frac{V_{RZ}}{V_f} \times 100$$



- V_{RZ} is the Quantity of water stored in Root Zone.
- This Efficiency Represents losses taking place in the form of Run off losses.

③ EFFICIENCY OF WATER STORAGE: It is the ratio of water stored in the rootzone during irrigation to the water Required in root zone prior to irrigation.

$$\eta_s = \frac{V_{RZ}}{V_{RRZ}} \times 100$$

- V_{RRZ} - It is the Quantity of water Required for the irrigation.
- It represents the extent of Chocking of Voids in root zone.

④ EFFICIENCY OF WATER USED: It is defined as the ratio of water beneficially Used by the crop for its growth (including leaching water Requirement) to the Quantity of water delivered.

$$\eta_u = \frac{V_u}{V_f} \times 100$$

V_u - Volume of water beneficially Used for Crop Growth.

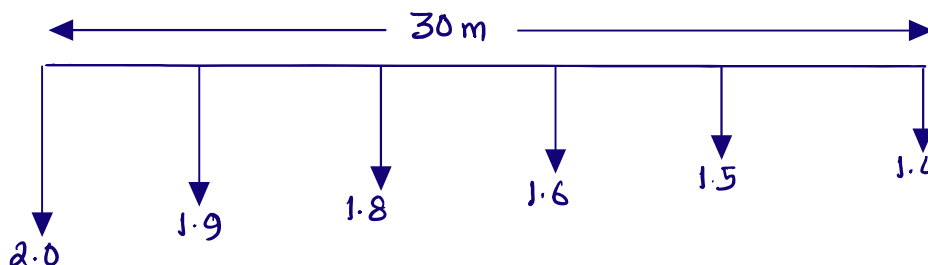
⑤ EFFICIENCY OF WATER DISTRIBUTION/UNIFORM COEFF.: This Efficiency Represents the Extent to which the water has been distributed or Penetrated to a Uniform depth throughout the field.

$$\eta_d = \left\{ 1 - \frac{d}{D} \right\} \times 100$$

- It is given by

D → Mean depth of water stored during irrigation.

d → Avg. of absolute value of deviation of depth of storage from Mean.



$$D = \frac{2 + 1.9 + 1.8 + 1.6 + 1.5 + 1.4}{6} = 1.7 \text{ cm}$$

$$d = \frac{|2 - 1.7| + |1.9 - 1.7| + |1.8 - 1.7| + |1.6 - 1.7| + |1.5 - 1.7| + |1.4 - 1.7|}{6}$$

$$d = 0.2 \text{ cm}$$

$$\Rightarrow \eta_d = \left(1 - \frac{d}{D}\right) \times 100$$

$$= \left(1 - \frac{0.2}{1.7}\right) \times 100$$

$$\eta_d = 88.23 \%$$



Q. A stream of 150 lit/sec was diverted from a Canal and 120 lit/sec were delivered to the field. An area of 1.5 hectare was irrigated in 8 hrs, the effective depth of root zone is 1.7 m. The runoff loss in the field was 400 cum. The depth of water Penetration varied from 1.7 cm at the head of field to at the tail of field. Available Moisture holding Capacity of Soil is 20 cm per meter depth of the Soil. It is required to find all the irrigation efficiencies. Assuming irrigation was started at a Moisture Extraction level of 40% of Available moisture and leaching requirement is 200 m³.

Ans: $\Rightarrow$ ① $\eta_c = \frac{120}{150} \times 100 = 80\%$

② $\eta_a = \frac{V_{RZ}}{V_f} \times 100 = \frac{3056}{3456} = 88.42\%$

Volume of Water Applied in field @ 8 hr (V_f) = $120 \times 8 \times 60 \times 60$
 $= 3456 \text{ m}^3$

Runoff loss in the field = 400 m³
 Volume of Water Stored in root zone (V_{RZ}) = $3456 - 400$
 $= 3056 \text{ m}^3$

$\therefore \eta_a = \frac{3056}{3456} \times 100$
 $= 88.42\%$

③ $\eta_s = \frac{V_{RZ}}{V_{RRZ}}$

Moisture holding Capacity of the Soil = 20×1.7
 $= 34 \text{ cm}$

Moisture holding Capacity before Irrigation = 34×0.6
 $= 20.4 \text{ cm}$

Additional Water required in root Zone before Irrigation.
 $V_{RRZ} = 1.5 \times 20.4 \times 10^{-2} \times 10^4$ (1 hec = 10⁴ m)
 $V_{RRZ} = 3060$

$\eta_s = \frac{3056}{3060} \times 100 = 99.86\%$

④ $\eta_d = \left\{1 - \frac{d}{D}\right\} \times 100$

$D = \frac{1.7 + 1}{2} = 1.35 \text{ cm}$

$d = \frac{|1.7 - 1.35| + |1.35 - 1|}{2}$
 $= 0.35$

$$\eta_d = \left\{ 1 - \frac{0.35}{1.35} \right\} \times 100 = 74.07\%$$

$$\textcircled{v} \eta_w = \frac{V_v}{V_f}$$

$$= \frac{3056 + 200}{3456} \times 100 = 94.21\%$$



•• EVAPOTRANSPIRATION OR CONSUMPTIVE USE (C_u)

- Consumptive Use may be defined as amount of Water Used in Evapotranspiration from an area Under Vegetation plus the Water Used by the Plant in their metabolic Process for building Up of new tissues.
- Since the Quantity of Water required by the Plant for its metabolic Activity is Very less it is neglected and Consumptive Use (C_u) is taken equal to evapotranspiration.
- Evapotranspiration is the total amount of Water Used by the plant in transpiration of the Plant and Evaporation from adjacent Soil or any other water body. {in Case plant grows in water}.
- The Values of Evapotranspiration or Consumptive Use of Crop may be diff. for diff. Crops or it may be diff for Same Crop at diff time and place.
- The transpiration is defined as the process in which the water of a living plant during Photosynthesis through its leaves, to enter the atmosphere for its growth by the process of Photosynthesis.
- Since, Transpiration occurs during day - light hours its 95 % extent is Observed during photosynthesis.
- On the other hand, Evaporation is continuous throughout the day and night.
- Transpiration losses not Only depends On available Sun light but also upon the available moisture.
- Diff. Plant will have diff amount of Water and hence their Water Consuming property will be diff, thereby in Order to Standardized, it is reported in terms of transpiration Ratio.

• TRANSPIRATION RATIO :- It is defined as the ratio of mass of Water transpired by the plant to the mass of dry matter produced after its full Growth.

$$T_R = \frac{\text{Total mass of Water transpired by plant}}{\text{Mass of dry matter Produced.}}$$

- T_R for Wheat is 300-600 (avg. 450) and for Rice 600-800 (Avg. 700).

Note: Transpiration in Lab is generally measured Using 'PHYTOMETER'.

- This is most widely used method which consist of a closed and water tight tank containing sufficient soil to nourish the plant.
- Water is applied to it artificially over the growth period of plant and transpiration is measured as follows: $\rightarrow$

$$T = M_1 + M - M_2$$

T = Transpiration

M_1 = Initial mass of the instrument.

M = Total mass of water added during its full growth.

M_2 = final mass of the instrument.



- The results obtained from this method does not match with those of actual, as exact field conditions can not be simulated, Moreover Screened shelters are observed to give 10% lower results.

Note: When sufficient moisture is freely available to completely meet the needs of plant for evaporation and transpiration is termed as Potential Evapotranspiration (PET). However, the actual amount of water used for this purpose is termed as actual Evapotranspiration. PET depends upon climatic factors whereas AET depends upon type of soil and crop.

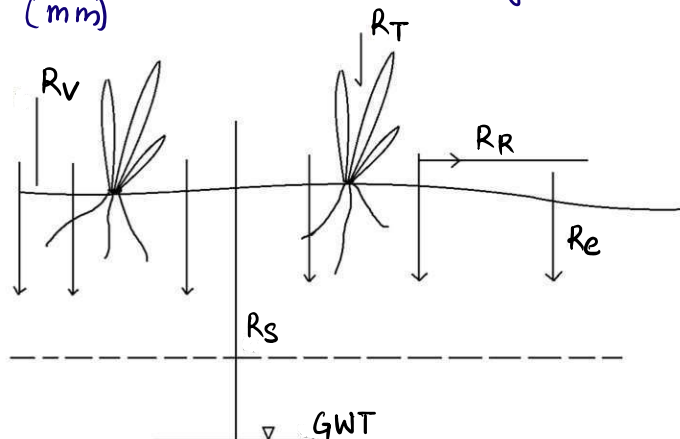
- PET in field can be measured using device termed as 'Lysimeter'.

•• EFFECTIVE RAINFALL/USEFUL RAINFALL (R_e)

- A portion of rain falling over a cropped area during growing period of the crop is utilized by the crop to fulfill its Evapotranspiration requirement.
- Thereby it reduces irrigation requirement of the crop.
- The portion of rainfall that is effectively used for the growth of crops is termed as Useful Rainfall (R_e). This value can be calculated by subtracting it from the total rainfall.

- (i) Evaporation.
- (ii) Deep Percolation.
- (iii) Runoff.

Useful rainfall = Total Rainfall $\times$ Avg. Rain factor.
(mm)



$$R_e = R_T - R_v - R_s - R_p$$

- Here Ratio factor (r) signifies the portion of rainfall used by plant for Evapotranspiration.
- This factor (r) will be less for a wide spread rain falling over the 100% area rather than scattered rain falling over less area.

- Useful rainfall can also be found empirically as follows -

$$\textcircled{1} R_e = 0.8.R - 0.25 \text{ mm if } R > 75 \text{ mm/month}$$

$$\textcircled{II} R_e = 0.6.R - 10 \text{ mm, if } R < 75 \text{ mm/month}$$

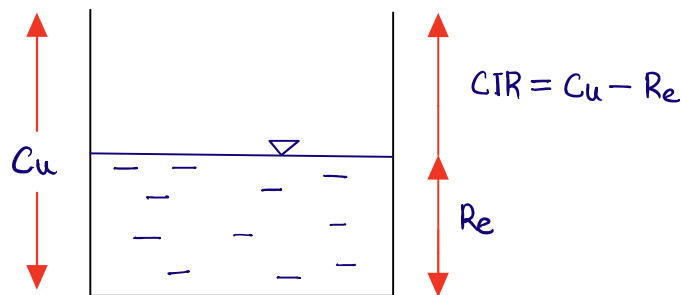
Where, R = total Rainfall in mm/month.



•• CIR (CONSUMPTIVE IRRIGATION REQUIREMENT)

- That portion of Consumptive Use which has to be satisfied through irrigation is called as CIR.

$$CIR = C_u - R_e$$



•• NET IRRIGATION REQUIREMENT {NIR}

- That Portion of Consumptive Use which has to be satisfied by irrigation along with leaching requirement is termed as net Irrigation requirement.

$$NIR = CIR + \text{Leaching requirement.}$$

$$NIR = C_u - R_e + \text{Leaching requirement.}$$

•• FIELD IRRIGATION REQUIREMENT {FIR}

- In addition to net irrigation required, FIR also takes into account surface runoff losses.

$$FIR = NIR + \text{Runoff Loss}$$

•• FACTORS GOVERNING CONSUMPTIVE USE

- HUMIDITY: Higher is the humidity, more is the water available in the voids of air and hence, lesser is the Vaporisation thereby lesser is C_u .

② TYPE OF SOIL: In case of Permeable Soil, water evaporates easily without any obstruction as voids are interconnected hence, C_u will be more.

③ TOPOGRAPHY OF SOIL: In case of Irregular topography as surface area of field exposed to atmosphere is more, more evaporation takes place, hence C_u will be more.

④ BASE PERIOD: If the Base Period for the same crop is increased, then its Consumptive use will also increase.

⑤ MEAN MONTHLY TEMPERATURE: If the average temperature is more, more is the rate of Vaporization hence more is the Consumptive Use of the Crop.

Q. If Consumptive Use and effective rainfall for the Periods are shown for a Jawar sown at Particular location. The Period of growth is from 16th Oct - 2nd Feb. Determine 'NIR' of this Crop. Assuming that water is not required for any other Purpose except that of fulfilling the evapotranspiration need of the Crop.

Time	Consumptive Use C_u (mm)	R_e (effective) Rainfall (mm)	$NIR = C_u - R_e + 0$ $= C_u - R_e + 0$
Oct 16-31	37	31	6
Nov 1-30	84	21	63
Dec 1-31	155	7	148
Jan 1-31	188	2.5	185.5
Feb 1-2	13	1	12
			$\Sigma = 414.5 \text{ mm}$

Ans: $\rightarrow$ Base Period = 110 days

$NIR \rightarrow$ For 1 day = $\frac{414.5}{110}$

= 3.77 mm/day.



•• INDIRECT METHOD TO CALCULATE CONSUMPTIVE USE

- C_u can also be found Empirically by following methods -

① BLANEY-CRIDDLE EQUATION: It states that the monthly Consumptive Use is given by -

$$C_u = \frac{K_p}{40} [1.8t + 32]$$

C_u - Monthly Consumptive Use in cm.

K - Crop factor which is found experimentally and depends upon condition of Particular Area.

t - Mean Monthly temperature ($^{\circ}\text{C}$)

if temperature is in $^{\circ}\text{F}$ then, Consumptive Use is given by,

$$C_u = \frac{K_p t}{40}$$

P - Monthly % age of Annual day light hours that occurs during that Period.

- It is not used in India.

Q. Determine the Volume of Water required to be diverted from the headwork to irrigate an area of 5000 hec, using information given below. Assume 80% as the effective Precipitation to take care of the C_u of the Crop. Assume 50% efficiency of Water Application in field and 75% as the Conveyance efficiency of Canal.

Month	Temp ($^{\circ}$ F)	% of Sunshine Hour	Rainfall R(mm)	Crop factor (K)
June	70	9.9	75	0.8
July	75	10.2	108	0.85
August	73	9.6	130	0.85
Sept	72	8.4	115	0.85
Oct	69	7.8	105	0.65
Nov	55	7.2	25	0.65
Dec	47	6.4	0	0.6
Jan	48	8.6	0	0.6
Feb	54	9.9	0	0.65
March	60	8.8	0	0.70
April	62	8.8	0	0.70
May	68	9.8	0	0.70

Ans: $\rightarrow$

Months

$$C_u = \frac{KPt}{40}$$

6	13.85
7	16.25
8	14.89
9	12.85
10	8.75
11	6.435
12	4.5
1	6.2
2	8.67
3	9.24
4	9.54
5	12.49

$$\text{Total } C_u = 133.673 \text{ cm}$$

$$R_e = 80\% \text{ of } R = 0.80 \times 558 \\ = 44.69 \text{ cm}$$

$$CIR = C_u - R_e = 89 \text{ cm}$$

$$NIR = CIR + \text{Leaching} \\ = 89 + 0 \\ = 89 \text{ cm.}$$

$$FIR = \frac{NIR}{\eta_a} = \frac{89}{0.5} = 178 \text{ cm}$$

$$GIR = \frac{FIR}{\eta_c} = \frac{178}{0.75} = 237.3 \text{ cm}$$

$$\Rightarrow \text{Volume of Water required for 5000 hec area at the head of the Canal} \\ = 237.33 \times 10^{-2} \times 5000 \times 10^4 \times 10^{-6} \\ = 118.665 \text{ M-m}^3.$$



Q. The Monthly Consumptive Use for Paddy are given below. Determine the total Consumptive Use. Also compute what is avg. monthly C_u and Peak monthly C_u .

	Time	C_u (cm)
June	1-30	27
July	1-31	23
August	1-31	23
Sept.	1-30	21
Oct	1-31	25
Nov	1-24	15

Ans:- Total $C_u = 134$

$$- \text{Avg. daily } C_u = \frac{C_u}{\text{Base Period}} = \frac{134}{177} = 0.75 \text{ cm/day.}$$

$$- \text{Avg. Monthly } C_u = 0.75 \times 30 = 22.5 \text{ cm/month.}$$

$$- \text{Peak Monthly Consumptive Use, } C_u = 27 \text{ cm.}$$



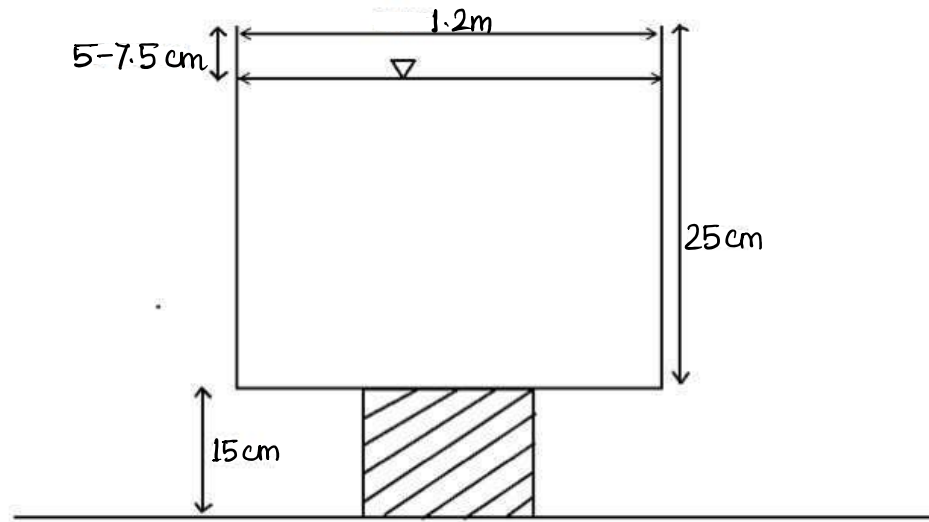
⑪ HARGREAVES CLASS A-PAN EVAPORATION METHOD

— In this method, Evapotranspiration or C_u is related to PAN EVAPORATION by a Constant termed as Consumptive Use Coefficient.

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

$$K = \frac{E_t}{E_p}$$

- Consumptive Use Coeff. K is different for different Crops and is different for some crops at different Places as it depends upon Crop growth location, type of Soil, Stage of growth of Same/Particular Crop.
- The Value of this Constant is available for different Crops in Indian Sub-Continent, hence, this method is Steadily Used here.
- Here the Crops are being divided into 8 groups and the Coeff. ' K ' is fixed accordingly.
- Here Pan-Evaporation can be Experimentally determined by directly measuring the Quantity of water Evaporated from the Standard Class-A Pan.
- This Pan is 1.2 m in dia, 25 cm deep and its bottom is raised by 15 cm above the Ground level.
- The depth of the water in it is Kept in the range of 5 cm - 7.5 cm below the top.
- These values are on the basis of precise experiment.



Class - A PAN

Q. For Use in Hargreaves method, in respect of group D wheat crop, following K values are suggested.

% of Crop growing Session

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	0.80
85	0.70
90	0.60
95	0.60
100	

Wheat is grown between 1 Nov - 15 March, the effective R_e in the individual months are as follows :-

Month	R_e (cm)
Nov	0.5
Dec	1.6
Jan	3.2
Feb	2.7
March	-

Percolation losses are nil throughout the growth period.

FIR = 70 %

$$GIR = 75\%$$

The mean Class A - Pan Evaporation for diff. months are given,

Month	E_p (cm)
Nov	13
Dec	11
Jan	7
Feb	15
March	10

Compute the 'GIR' for the whole Crop Period including 7cm of net Presowing Requirement.

Ans:→	Time	Time upto the mid (t) (days)	% (G) $t/B \times 100$	K	E_p (cm)	$E_T = K E_p$
	Nov	1-30	15	0.16	13	2.08
	Dec	1-31	45	0.44	11	4.84
	Jan	1-31	76	0.72	7	5.04
	Feb	1-28	106	0.90	15	13.5
	March	1-15	127	0.6	10	6.0

t = No. of days upto the mid span of concerned Month.

%G = % of Crop Growing Season.

B = Base Period = 135

$$\text{Total } C_u = 31.46$$

$$R_e = 8$$

$$CIR = C_u - R_e = 31.46 - 8 = 23.46 \text{ cm}$$

$$NIR = CIR + L.R = 23.46 + 0$$

$$= 23.46 \text{ cm.}$$

$$FIR = \frac{NIR}{\eta_a} = \frac{23.46}{0.7} = 33.51 \text{ cm}$$

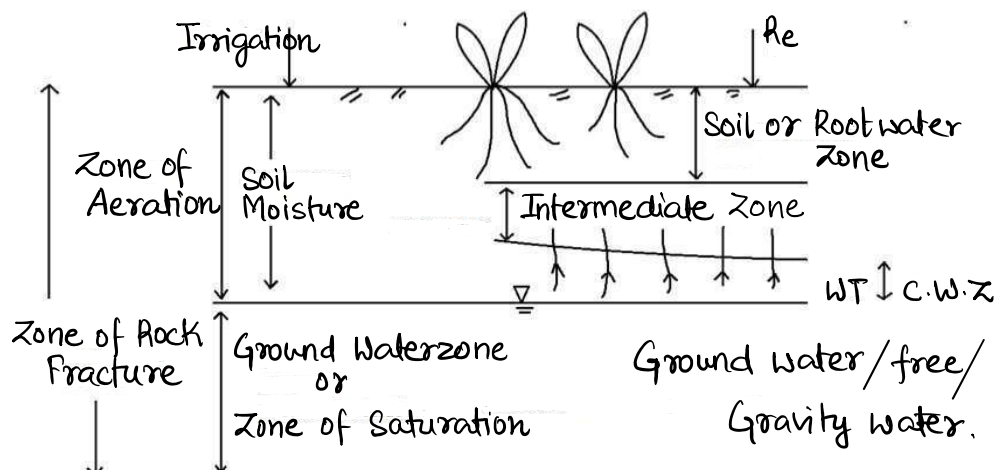
$$GIR = \frac{FIR}{\eta_e} = \frac{33.51}{0.75} = 44.68 \text{ cm}$$

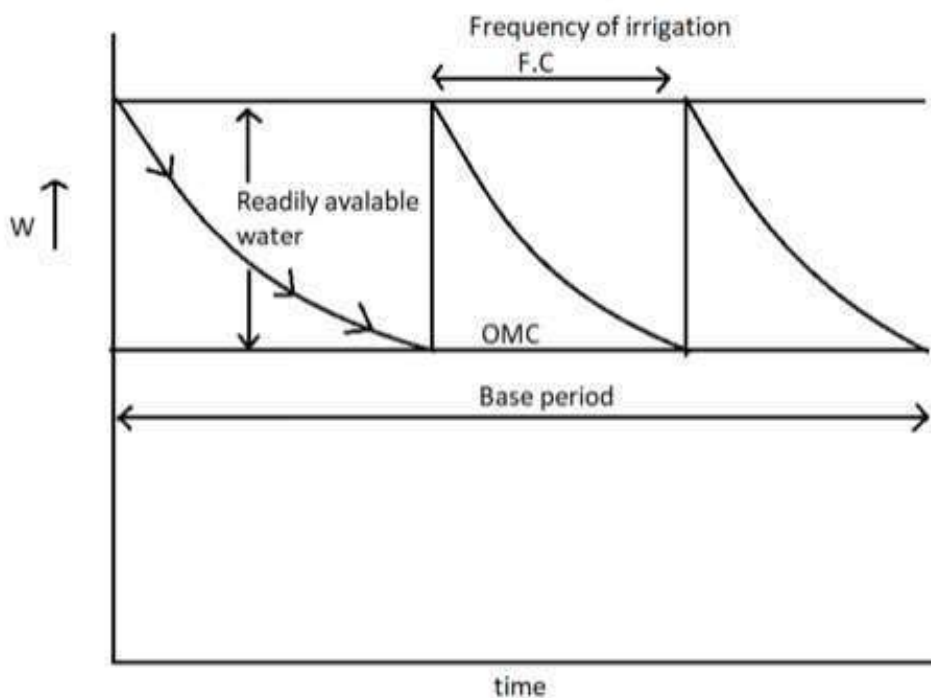
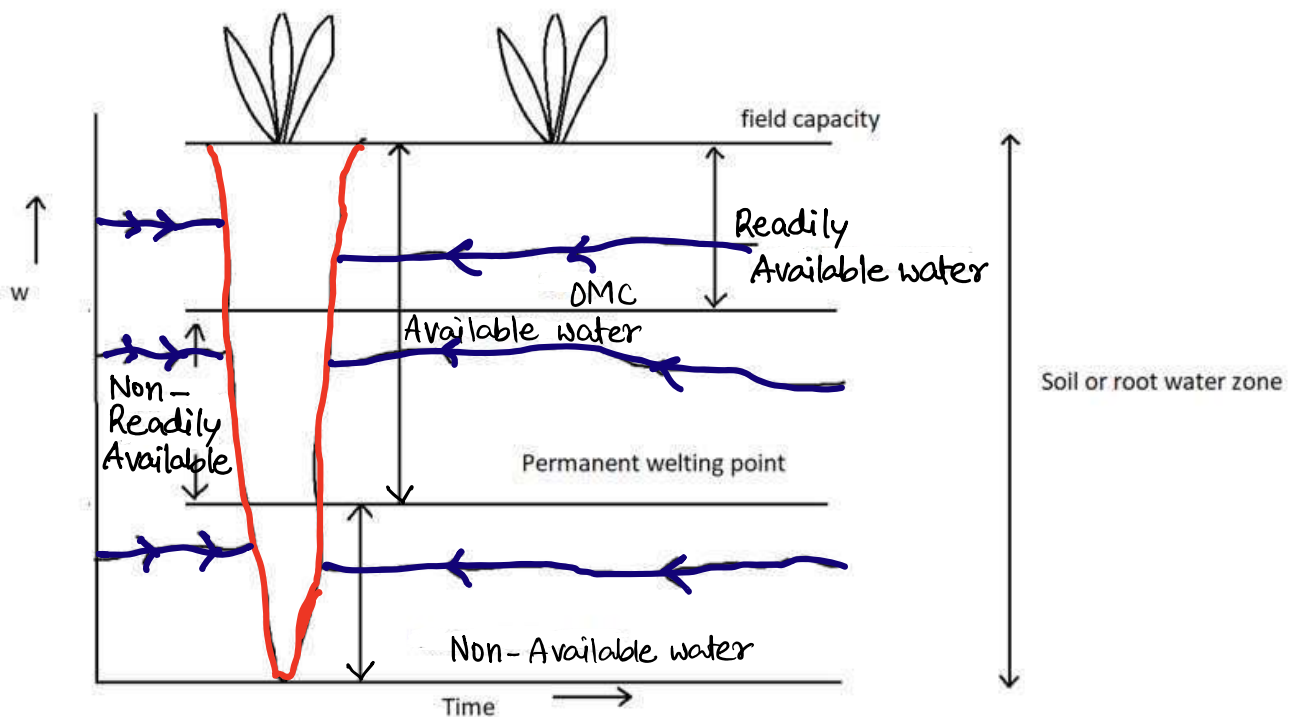
Assuming 7cm of water is available in the field if 7cm of water is to be provided by Canal.

$$= 44.68 + \frac{7}{(0.7 \times 0.75)}$$

$$= 58 \text{ cm}$$

•• SOIL MOISTURE - IRRIGATION RELATIONSHIP





- The Water below the Water table is known as Ground Water and above the Water table is termed as Soil Moisture.
- The Portion of Zone of aeration in which roots of Plants and crops Penetrate to absorb the moisture or nutrients from Soil is termed as Soil/root Water Zone.
- When Water falls Over the ground [either due to irrigation or due to rainfall] a part of it gets absorbed in this root Zone [in Zone of aeration] and the rest flows down to meet Ground Water table Under the action of Gravity.

- FIELD CAPACITY: Immediately after the application of water over the ground a certain amount of water remains on the surface of soil particles by molecular attraction and by chemical bond {Comparatively weaker in adsorption}. This water can not be easily drained under the action of gravity and is termed as field capacity.

- The field capacity is the water content of soil after free drainage has taken place for sufficient period (2-5 days).

- The field capacity water further consists of -

- Available / Capillary water
- Non-Available / hygroscopic water.



①AVAILABLE/CAPILLARY WATER: A part of field capacity of water which is attached to the soil particles by surface tension against gravitational forces and can be used by plant for its growth by capillarity is termed as capillary water.

②NON-AVAILABLE/HYGROSCOPIC WATER: A part of field capacity water that cannot be removed by capillarity of roots/plants and is not available for its growth is termed as hygroscopic water.

- The field capacity water content can be defined as the quantity of water which soil can retain over its surface against gravity to the quantity of dry soil.

$$FC = \frac{\text{Weight of water retained in certain volume of soil against gravity}}{\text{Weight of same volume of dry soil}} \times 100$$

Let area of soil be 1 m^2 and d be the depth of root zone,
Volume of soil in root zone $= d \times 1 = d \text{ m}^3$.

$$\begin{aligned} \text{Dry weight of soil in root zone} &= \text{Volume of soil} \times \text{dry density of soil} \\ &= d \times \gamma_d \end{aligned}$$

$$\begin{aligned} \text{Wt. of water retained in the soil} &= F \cdot \gamma_d \cdot d \\ \text{Volume of water} &= \frac{F \cdot \gamma_d \cdot d}{\gamma_w} \end{aligned}$$

$$\text{depth of water retained in soil upto field capacity} = \frac{F \cdot \gamma_d \cdot d}{\gamma_w} \times \frac{1}{1}$$

It is also termed as total water storage capacity of soil {Expressed in terms of depth}.

Note: Depth of water retained in soil in between two water content is given by $= \frac{(w_1 - w_2) \cdot \gamma_d \cdot d}{\gamma_w}$

$$F = \frac{\text{Wt. of water retained in soil}}{\text{Wt. of dry soil.}}$$

$$F = \frac{\gamma_w \cdot V_w}{\gamma_d \cdot V} = \frac{\gamma_w \cdot V_v}{\gamma_d \cdot V}$$

$$F = \frac{\gamma_w}{\gamma_d} \times \eta$$

- The total field Capacity water can not be Utilized by the Plant, it Can extract Water from the Soil upto an extent, after which it becomes dry, this Water content is termed as Permanent Wilting point.
- It Can be therefore be referred as the Water which is available to the Plant is the difference of FC Water and Permanent wilting point water.
- This is Known as Available moisture, Capillary Water or max. Storage Capacity of Soil.
- The Portion of Available Moisture which is most easily extracted by the plants for its growth is termed as Readily Available Moisture.
- It is approximately 75-80% of Available moisture.
- OMC {OPTIMUM MOISTURE CONTENT}: It is the moisture Content in root Zone below which water can not be easily available or extractable to the Crops for its growth.
- If Water Content falls below OMC during its growth Period, it affects the Productivity of the Crop.
- SOIL MOISTURE DEFICIENCY: Water required to bring Soil Moisture content of a given Soil to its field Capacity is called field Moisture deficiency.
- EQUIVALENT MOISTURE: It is the Water retained by a Saturated Soil after being Centrifuged for 30 min by a force of 100 times more than that of gravity.
- It is approximately equal or slightly less than F.C.
- Hence, objective of Irrigation is that, Irrigation should be Carried Out at Such interval/frequency, So that the moisture Content in the root Zone never falls below OMC.

Q. A Sandy loam Soil hold water at 140 mm/m depth between field Capacity and PWP. The root depth of the Crop is 30 cm and the allowable depletion of water is 25%. The daily water used by the Crop is 5 mm/day (C_u), the area to be irrigated is 60 hec and water can be diverted at 28 l/sec. The Surface irrigation application efficiency is 40%. There is no rainfall and Ground Water contribution. Calculate,

- Allowable depletion depth below irrigation
- Frequency of Irrigation.
- Net Application depth of Water
- Vol. of Water required.
- Time required to irrigate 5 hec farm.

Ans: → ① Moisture holding Capacity of Soil = 140 mm/m
Depth of root Zone = 30 cm



$$\text{Moisture Holding Capacity of root Zone} = 140 \times 0.3 \\ = 42.0 \text{ mm.}$$

$$\text{Permissible depletion} = 0.35 \times 42 \\ = 14.7 \text{ mm.}$$

$$\text{⑩ Frequency of Irrigation} = \frac{14.7}{5} = 2.94 \text{ days} = 2 \text{ days.}$$

$$\text{⑪ Net Water depth to be Applied} = \frac{15}{0.4} = 37.5 \text{ mm for 3 days}$$

$$\text{⑫ Quantity of Water to be applied} = 60 \times 10^4 \times 36.75 \times 10^{-3} \\ = 2.205 \times 10^4 \text{ m}^3 \\ = 2.205 \text{hec-m}$$

$$\text{⑬ Time Required} = \frac{5 \times 10^4 \times 36.75}{28 \times 10^{-3} \times 60 \times 60} \\ = 18 \text{ hr } 13 \text{ min.}$$

Q. After How many days will you supply water to soil in order to ensure sufficient irrigation of the given crop if,

(i) Field Capacity of Soil = 28%

(ii) Permanent Wilting point = 13%

(iii) Dry Density of Soil = 1.3 gm/cc.

(iv) Effective depth of root zone = 70 cm

(v) Daily Consumptive Use of Water for Crop = 12 mm

Assume any other data not given.

$$\text{Ans:} \rightarrow \text{Max. Water Storage Capacity or available Moisture} = 28 - 13 \\ = 15\%$$

$$\text{Assume Readily available Moisture} = 0.8 \text{ of Available Moisture} \\ = 0.8 \times 15 = 12\%$$

$$\text{Depth of Water Stored in root zone between F.C and OMC} = \frac{\gamma_d \cdot d (F_c - OMC)}{\gamma_w} \\ = 1.3 \times 70 \times 0.12 \\ = 10.92 \text{ cm.}$$

$$\text{Frequency of Irrigation} = \frac{10.92}{12} \times 10 = 9.1 \text{ days} = 9 \text{ days.}$$

Q. 800 m³ of Water is applied to a farmer's rice field of 0.6 hec. When the moisture content in soil falls to 40% of the available water between the F.C (36%) of Soil and PWP (15%) of the Soil Crop Combination. Determine the field application efficiency.

The root Zone depth of rice is 60 cm. Assume $n = 0.4$

$$\text{Ans:} \rightarrow \text{Max. Water Storage Capacity or available Moisture} = (36 - 15) = 21\%$$

$$\text{Readily available Moisture for growth} = 21 \times 0.6 = 12.6\%$$

$$\text{Depth of Water Available for ground} = \gamma_d \cdot d (F_c - OMC) / \gamma_w$$



$$F = \frac{\gamma_w}{\gamma_d} \times n \Rightarrow \frac{\gamma_d}{\gamma_w} = \frac{n}{F} \Rightarrow \frac{0.4}{36} = 1.11$$

$$\Rightarrow 1.11 \times \frac{60 \times 12.6}{100} = 8.4 \text{ cm}$$

$$\text{Volume of Water required for growth} = 0.6 \times 10^4 \times 8.4 \times 10^{-1} = 503.5 \text{ m}^3$$

$$\eta_a = \frac{V_{RZ}}{V_f} = \frac{503.5}{800} = 63\%$$

Q. Work out the Irrigation Schedule based on the Soil moisture Concept Using following information. Also find the delta 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 Layer within the top 1.5 m depth of Soil.

(b) Normal Root Zone depth of Crop is 1.2 m.

(c) Bulk density $\Rightarrow \gamma = 1.35$

(d) FC = 18% , PWP = 7%

(e) Moisture level in Soil is to be maintain at not less than $\frac{1}{3}$ of available retention.

(f) Irrigation will then be done over a duration of 2 day at a Uniform Rate of Supply and at Uniform rate of advance to fully and just compensate for the deficiency.

(g) Sowing is done on 1 Nov when the Soil moisture is left just at field capacity in the entire root zone.

(h) No extra water is required for leaching.

(i) For the Crop at the location the average evapotranspiration rates

1 Nov - 30 Nov	1.1 mm/d
1 Dec - 31 Dec	1.7 mm/d
1 Jan - 31 Jan	2.4 mm/d
1 Feb - 28 Feb	1.5 mm/d
1 March - 25 March	3.5 mm/d

(j) Harvesting is done on or after 26 March.

(k) There is expected and effective rainfall of 24 mm during 4 Jan - 19 Jan, both days inclusive, with Uniform intensity.

(l) By the end of the Crop growth Season. only the min. water needed to be left is Used in the root Zone.

Ans $\Rightarrow$ Max. Storage Capacity or available Moisture or available retention = FC - PWP
 $= 18 - 7 = 11\%$

$$\begin{aligned} \text{Depth of Water readily available for growth} &= \frac{\gamma_d \cdot d}{\gamma_w} (FC - OMC) \\ &= 1.35 \times 1.25 \times \frac{2}{3} \times 0.11 \\ &= 11.87 \text{ cm.} \end{aligned}$$

$$\begin{aligned} \text{Water Consumed from first 1 Nov - 20 Nov} &= 1.1 \times 20 \\ &= 22 \text{ cm} \end{aligned}$$



$$1 \text{ Dec} - 31 \text{ Dec} = \frac{1.7 \times 31}{10} = 5.27 \text{ cm.}$$

$$1 \text{ Jan} - 19 \text{ Jan} = 2.4 \times 3 + \left\{ 2.4 - \frac{2.4}{16} \right\} \times 16 = 2.16 \text{ cm.}$$

$$\text{Total Water Consumed upto 19 Jan} = 3.3 + 5.27 + 2.16 = 10.73 \text{ cm.}$$

$$\text{No. of days to Consume left Over water} = \frac{11.87 - 10.73}{2.4 \times 10^{-1}} = 4.75 \text{ days.}$$

Hence, first irrigation will be needed for 4 days from 19 Jan.

$$\begin{aligned} \text{Water required on 24th - 25th Jan} &= 10.73 + 4 \times 2.4 \times 10^{-1} + 2 \times 2.4 \times 10^{-1} \\ &= 12.17 \text{ cm.} \end{aligned}$$

Water Consumed from 26 Jan Onwards -

$$26 \text{ Jan} - 31 \text{ Jan} = \frac{6 \times 2.4}{10} = 1.44 \text{ cm.}$$

$$1 \text{ Feb} - 28 \text{ Feb} = \frac{28 \times 1.5}{10} = 4.2 \text{ cm}$$

$$1 \text{ March} - 25 \text{ March} = \frac{3.5 \times 25}{10} = 8.75 \text{ cm}$$

$$\Sigma = 14.39 \text{ cm} > 11.87 \text{ cm}$$

No. of days in march required to Consume readily available Moisture

$$= \frac{11.87 - 1.44 - 4.2}{3.5 \times 10^{-1}} = 17.6 \text{ days.}$$

Hence, 2nd Irrigation is required on 18 March and the Water applied would be
 $= 14.39 - 11.87 = 2.52 \text{ cm.}$



PREVIOUS YEAR QUESTIONS

ESE

Q. Consider the following data,

Root Zone depth = 2 m

Existing Water Content = 5%

Dry Density of Soil = 15 kN/m^3

Water Applied to the Soil = 500 m^3

Water loss due to Evaporation and deep Percolation = 10%

Area of plot = 1000 m^2

The field Capacity of the Soil will be nearly ;

(a) 16.8 %

(b) 17.7 %

(c) 18.8 %

(d) 19.7 %

[2019]

Ans :

(d) 19.7 %

Water lost due to Evaporation = 10 %

$$\begin{aligned}\text{Remaining Water} &= 90 \% \\ &= 0.9 \times 500 \\ &= 450 \text{ m}^3\end{aligned}$$

$$\begin{aligned}\text{Depth of Water} &= \frac{\text{Volume}}{\text{Area}} \\ &= \frac{450}{1000} = 0.45 \text{ m}\end{aligned}$$

$$\text{Depth of water} = \frac{\gamma_d}{\gamma_w} \times d' \times [\text{F.C.} - \text{Moisture Content}]$$

$$0.45 = \frac{15}{9.81} \times 2 \times [\text{F.C.} - 0.05]$$

$$\text{F.C.} = 19.715 \%$$

Q. The transition region between Unsaturated Zone and Saturated Zone is called

(a) Capillary Fringe

(b) Water table

(c) Vadose water Zone

(d) Confining bed

[2020]

Ans :

(a) Capillary Fringe

— Capillary Fringe treated as a boundary Condition Separating the Water Table (Saturated Zone) from Unsaturated Zone.

Q. Consumptive Use refers to the loss of Water as a result of

- (a) Evaporation and Transpiration
- (b) Crop water Requirement.
- (c) Evaporation and Infiltration.
- (d) Evaporation and Transpiration from Cropped area.

Ans :

(d) Evaporation and Transpiration from Cropped Area.

— Consumptive Use of Water refers to water that is removed from a Watershed by evapotranspiration i.e. evaporation and transpiration.

GATE

Q. The Command Area of a Canal grows Only one crop i.e. Wheat. The base Period of Wheat is 120 days and its total Water Requirement, Δ , is 40 cm. If the Canal discharge is $2 \text{ m}^3/\text{s}$, the area, in hectares, rounded off to the nearest Integer, Which could be Irrigated (neglecting all losses) is _____.

[2019-Shift-I]

Ans :

$$B = 120 \text{ days}$$

$$\Delta = 0.4 \text{ m}$$

$$D \cdot \Delta = 8.64 \cdot B$$

$$\Rightarrow D = 2592 \text{ ha/m}^3/\text{s}$$

$\therefore$ for $2 \text{ m}^3/\text{s}$ water, area will be 5184 ha.

Q. The Intensity of Irrigation for the Kharif season is 50% for an irrigation Project with Culturable Command area of 50,000 ha. The duty for the Kharif Season is 1000 hectare/Cumec. Assuming Transmission loss of 10%, the required discharge (in Cumec, upto two decimal places) at the head of the Canal is _____.

[2018-Shift-II]

Ans :

$$\text{Area} = 50,000 \text{ ha.}$$

$$\begin{aligned} \text{Area Under Cultivation} &= 50\% \text{ of } 50,000 \text{ ha} \\ &= 25000 \text{ ha.} \end{aligned}$$

$$\text{Duty} = \frac{\text{Area}}{\text{Discharge}}$$

$$\Rightarrow \text{Discharge} = \frac{25000 \text{ ha}}{1000 \text{ ha/cumec}}$$

$$= 25 \text{ m}^3/\text{sec}$$

$$\Rightarrow \text{Required Discharge} = \underline{27.5} \text{ m}^3/\text{s}. \quad \{10\% \text{ transmission loss} = 2.5\}$$

Q. The Culturable Command area of a Canal is 10,000 ha. The area grows Only two Crops - rice in the Kharif Season and Wheat in the Rabi Season. The design discharge of the Canal is based on the rice Requirements, Which has an irrigated area of 2500 ha, base period of 150 days and delta of 130 cm. The Maximum Permissible irrigated area (in hac) for Wheat, with a base Period of 120 days and delta of 50 cm is _____, [2017 - Shift - II]

Ans:

For Rice

$$\begin{aligned} \text{CCA} &= 10000 \text{ ha} \\ \text{B.P} &= 150 \text{ days} \\ \Delta &= 130 \text{ cm} = 1.3 \text{ m} \\ D &= \frac{\text{B.P} \times 8.64}{\Delta} \\ &= \frac{150 \times 8.64}{1.3} \\ &= 996.923 \text{ ha/m}^3/\text{s} \end{aligned}$$

Area Irrigated = 2500 ha.

$$\begin{aligned} \text{Design Discharge} &= \frac{2500}{996.923} \\ &= 2.5077 \text{ m}^3/\text{s}. \end{aligned}$$

For Wheat

$$\begin{aligned} \text{B.P} &= 120 \text{ days} \\ \Delta &= 50 \text{ cm} \\ D &= \frac{8.64 \times 120}{0.5} \\ &= 2073.6 \text{ ha/m}^3/\text{s} \end{aligned}$$

Area Irrigated = Duty X Design Discharge

$$\begin{aligned} &= 2073.6 \times 2.5077 \\ &= \underline{5199.97 \text{ ha}}. \end{aligned}$$

SSC - JE

Q. Which One of the following is not a Cause of Water logging?

- (a) Excess Tapping of the Ground water.
- (b) Excess Rainfall.
- (c) Frequent Irrigation.
- (d) High Water table.

Ans:

(a) Excess Tapping of the Ground Water.

Q. Available Moisture is the difference in water content of soil between field capacity and _____ [2017 - Morning]

- (a) Gravitational Water
- (b) Permanent Wilting point.
- (c) Saturation Capacity.
- (d) Ultimate Wilting point.

Ans :

(b) Permanent Wilting point.

Q. Watering done prior to the sowing of the crops is called _____. [2017 - Evening]

- (a) Kor Watering
- (b) Duty
- (c) Delta
- (d) Paleo Irrigation.

Ans :

(d) Paleo Irrigation.

— It is done to prepare the land for sowing and to add sufficient moisture to the soil which would be required for the initial growth of the crop.

CANAL IRRIGATION SYSTEM

The Entire distribution System from the head of the Main Canal to the agricultural field is divided into following 3- System -

- ① Primary distribution System { Main Canal and Branch Canal }
- ② Secondary distribution System { Major distributary and Minor }
- ③ Tertiary distribution System { Water Course and field Channel }



Note: The Above integral Management of Conveying and distributing Water is Called Warabandi System.

- By following this Warabandi System, the turn of different farmers are fixed so that Daily Water is not Supplied to a Single Farmer. Hence, reducing losses and allowing application of Water Under Pressure.

•• CLASSIFICATION OF CANALS

- Classification of canals can be done as follows :-

① ALLUVIAL AND NON-ALLUVIAL CANALS : → The Canals when excavated through alluvial soil are called ALLUVIAL CANALS.

- Canals in such areas use the water from direct irrigation instead of storage irrigation as river in alluvial soil has tendency to change its water course.

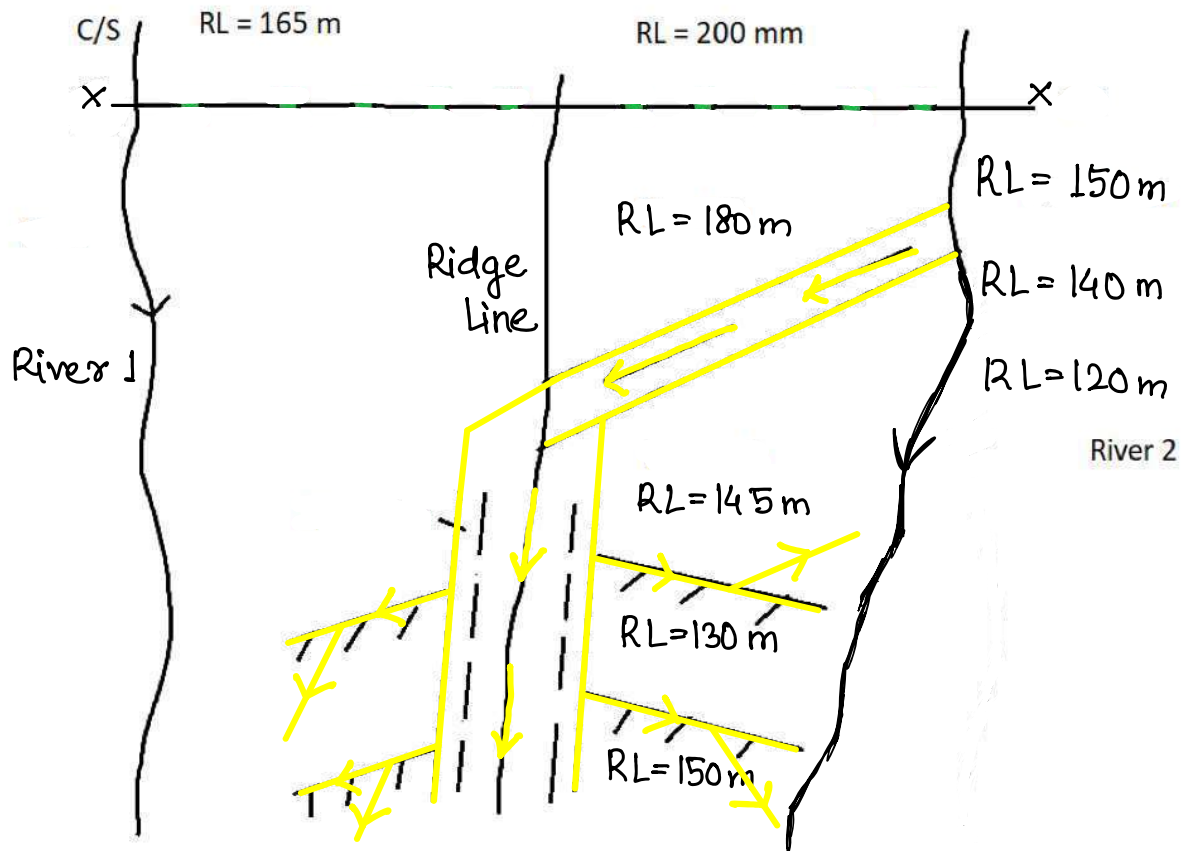
Note: The soils which are formed by continuous deposition of solids from running water (river) through an area is termed as Alluvial Soil.

- Hard foundations are generally not available in this kind of soil.
- Canals excavated through Non-Alluvial soil { formed due to disintegration of rock in mountainous region } are termed as Non-Alluvial Canals.
- Hard foundations are generally available in this case.

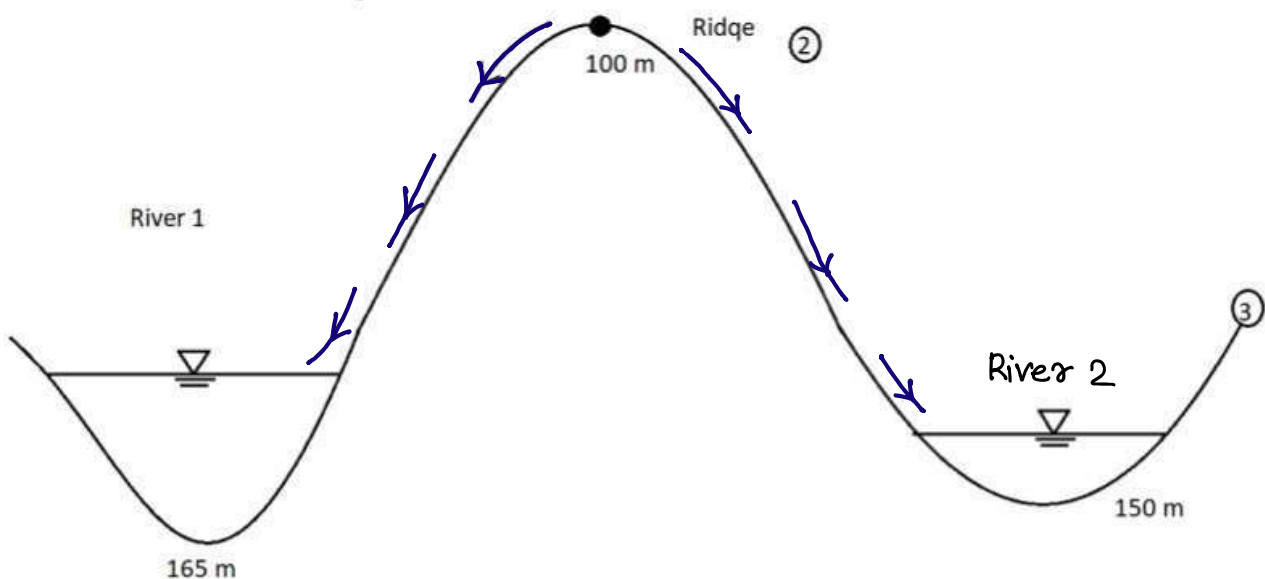


② ON THE BASIS OF ALIGNMENT OF CANAL

① WATERSHED CANAL/RIDGE CANAL



- The Canal which is aligned along any Watershed or ridge line is called watershed Canal or Ridge Canal.
- The Natural limits of the Command area of such irrigation channels would be on either Side of the Channel.

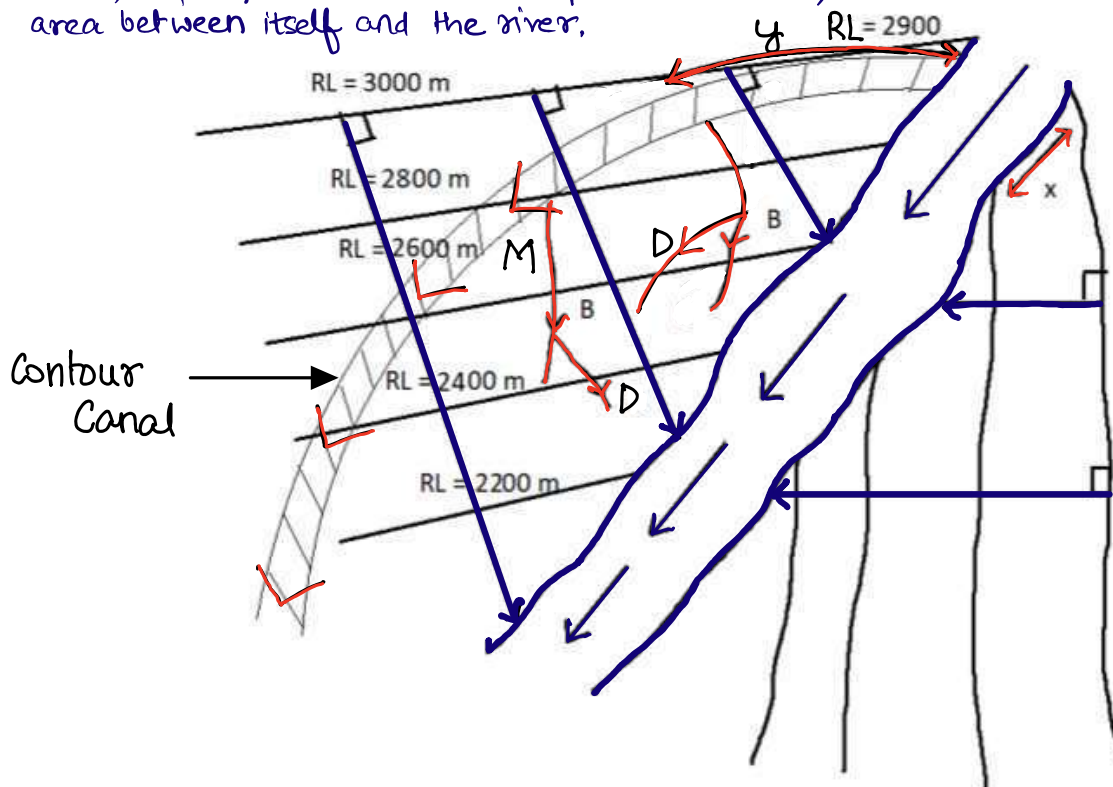


- Aligning a Canal { Main or Branch } along the ridge line ensures gravity irrigation on both Sides of Canal.
- Since drainage are always away from the ridge also drainage can Cross a Canal, hence there is no requirement of Cross-drainage Work for these Canal.

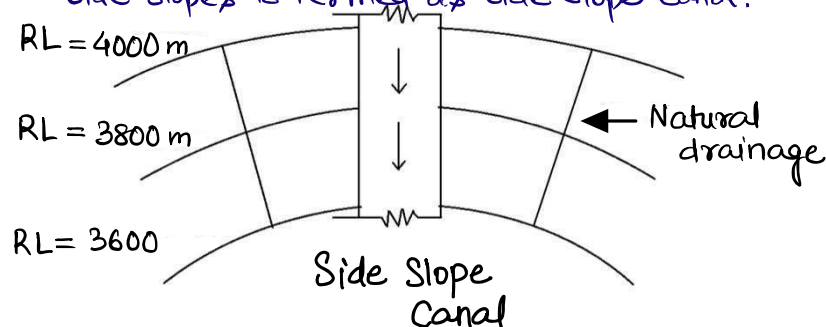
- If the Watershed is Passing through Villages or towns, the Canal may have to leave the ridge or Watershed for some distance.
- The depression in the ridge line may also required Construction of 'Viaducts' or 'Siphons' to maintain Canal FSL.

⑥ CONTOUR CANAL: The Watershed Canal cannot be provided in case of hilly areas. Since, the Conditions in hills are vastly different compared to Plains where river flows in Valleys. hence, it is not possible to align the Canal along the ridge line.

- In Such conditions, Contour Canals are provided which are aligned along the Contour except for giving the required longitudinal Slope of the Canal.
- A Contour Canal can irrigate Only One Side because area on the other Side is on Higher level.
- As the drainage flow is always at right angles to the Contour, Such Canals also meet drainage in between. hence, these Canals also required Cross-drainage work.
- Since, Slope of river is much Steeper than Canal, the Canal Surrounds more and more area between itself and the river.



⑦ SIDE-SLOPE CANAL: A Canal which is aligned at right Angle to the Contour i.e. along the Side Slopes is termed as Side Slope Canal.



- Since, natural drainage also flows at right angles to contour, it never meet this Canals, Hence, No cross-drainage Work is required in this Case.

• TERMINOLOGY

① GROSS COMMAND AREA :- It is the total Area, bounded Within the irrigation boundaries of the Projects Which can be Economically irrigated Without Considering the Limitations of the Quantity of Available Water.



- It includes both Cultivable as well as Un-Cultivable area.
ex :→ Residential Area, roads, Forest Area.

② CULTURABLE OR CULTIVABLE COMMAND AREA : It is the part of 'GCA' and includes all land of GCA on which Cultivation is Possible.

- At any given time, all the Cultivable land may not be actually irrigated Under Cultivation.
- Hence, CCA is further divided into two Categories -
 - ① Cultivable Portion of CCA.
 - ② Cultivable and not Cultivated portion of CCA In absence of any delta, CCA may be assumed to be 80% of GCA.

③ INTENSITY OF RAINFALL {SEASONAL OR ANNUAL} : The entire Cultivated Portion of Culturable Command area is not desired to be irrigated at 1 time to avoid intensive irrigation at a Particular Area.

- As it may Cause harmful effect like Water-logging, Salinity and dampness (which promote Water borne disease in a Particular area) and required high demand of Water.
- Due to this reason, Only a Small % of CCA is brought Under Irrigation.
- This % age of CCA to be Irrigated in a Season/Annual is termed of Irrigation for that Season or particular year.
Ex :→ IOI for Kharif = 30%
IOI for Rabi = 35%
Hence, In above Case Annual IOI = $30 + 35 = 65\%$.
- The annual IOI is Usually found to be in the range 40-60 %, but is desired upto 100-180% by Cultivating Larger Parts of CCA with more than One Crop in a year.

④ NET AND GROSS SOWN AREA : In Some Cases, two Crops in two Season are grown, during a Particular year on the Same area hence, Such an area will be Sown more than Once during the year.

- If this area is added to area which is Sown Only Once (net Sown area), it is termed as Gross Sown Area.

$$\text{Gross Sown Area} = \text{Net Grown Area} + \text{Area Sown more than during Same Year.}$$

⑤ NET AND GROSS IRRIGATED AREA: The Area which is Irrigated Once during a year is called 'NIA' and When to this area is added the area Irrigated more than Once Gross Irrigated Area is Obtained.

Gross Irrigated Area = Net irrigated Area + Area Irrigated More than Once.

⑥ AREA TO BE IRRIGATED: The area Proposed to be irrigated in any One Crop Season or Over an year is called Area to be irrigated.

$$AI = IOI \times CCA$$

⑦ TIME FACTOR: To check the Case of over Irrigation leading to Water logging Salinity etc, Channels are not allowed to operate on all the days during any Crop Season.

— The ratio of actual operating time of channel to the Crop Period is Called Time Factor.

⑧ CAPACITY FACTOR (C_v): The capacity factor for Canal is the ratio of the mean Supply discharge in a Canal during period to its designed full Capacity discharge.

$$C_v = \frac{\text{Mean Discharge}}{\text{Design Discharge}}$$

— Since Canal have to run almost to their full design Capacity during Kharif Season, C_v is approximately 0.9 – 0.95.

— However, Since Water requirement reduces in Rabi Season C_v reduces to about 0.6 – 0.7.

⑨ FULL SUPPLY COEFFICIENT/DUTY ON CAPACITY: It is the design duty at the head of the Canal.

— It can also be defined as no. of hectares of area irrigable per Cumec of Canal Capacity at its head.

$$\text{Full Supply Coeff/Duty On Capacity} = \frac{\text{Area to irrigated during Base Period (B)}}{\text{Design Full discharge at the head of Canal.}}$$

⑩ NOMINAL DUTY: It is the ratio of area actually irrigated by the farmer to the mean Supply discharge from the outlet of the distributary over the Crop period.

For eg.:-

Area to be irrigated = A
Crop period = 150 days
Time Factor = 0.8
Rate of Supply = x cumec

$$\text{Nominal Duty} = \frac{A}{\frac{x \times 120}{150}} = \frac{A}{0.8x} \text{ (hec/cumec)}$$



⑨ INTENSITY OF CROPPING: It is defined as ratio of Gross Cropped area to the net Cultivated area.

$$K = \frac{\text{Gross Cropped Area}}{\text{Net Cultivated Area}}$$

•• FIXING THE DESIGN CAPACITY OF IRRIGATION CANAL

- In Order to fix the design Capacity of Canal, all the likely Crops that would be Sown in an area is to be Considered.
- The Peak rate of Water requirement of all the Crops in each Season of a year is Computed.
- Peak rate Correspond to Kor Period and Kor depth.
The Canal Capacity Should be Sufficient to fulfill the max. of Peak demand of all the Crops that are required to be irrigated at any One time amongst all the Seasons.

Note:- In Order to get Precise result, the interval of analysis should be as small as Possible.

- Generally, Calculations are done for Monthly water requirement and the result is increased by 20-25 % to account for Peak demand.

Q. The GCA of a distributory is 6000 hec, 80% of which is Culturable, the Intensity of Irrigation for Rabi Season is 50% and that of Kharif Season is 25%, if the avg. duty at the head of distributory is 2000 hec/cumec for rabi and 900 hec/cumec for Kharif Season. Find out the discharge required at the head of the distributory from average demand Consideration.

Ans:-> GCA = 6000 hec.

IOI for Rabi = 50% and for Kharif = 25%.

$$[(\text{Duty})_{\text{avg}}]_{\text{Rabi}} = 2000 \text{ hec/cumec}$$

$$[(\text{Duty})_{\text{avg}}]_{\text{Kharif}} = 900 \text{ hec/cumec}$$

$$\text{CCA} = 0.8 \times \text{GCA} = 0.8 \times 6000 = 4800 \text{ hec.}$$

$$\text{Area to be irrigated in rabi Season} = 0.5 \times 4800 = 2400 \text{ hec.}$$

$$\Rightarrow \text{Full Supply Coefficient} = 0.25 \times 4800 = 1200 \text{ hec.}$$

$$\text{Water Required at head of the distributory to irrigate rabi} = \frac{2400}{2000} = 1.2 \text{ Cumec.}$$

$$\text{Kharif} = \frac{1200}{900} = 1.33 \text{ Cumec}$$

From average demand Consideration, water required at the head of distributory = 1.33 m³/sec.



Q. The Culturable Command area of a Water Course is 1200 hec, IOI of Sugarcane and Wheat Crops are 20% and 40% respectively, the duty for the Crop at the head of Water Course are 730 hec/cumec and 1800 hec/cumec respectively. Find the discharge required at the head of Water Course. Determine the design of Outlet Considering a time factor of 0.8.

Ans → Discharge requirement for Sugarcane = $\frac{1200 \times 20}{100 \times 730} = 0.328 \text{ cumec.}$

Discharge requirement for Wheat = $\frac{1200 \times 40}{1800} = 0.266 \text{ cumec.}$

Discharge required at head of Course = $0.328 + 0.266$
 $= 0.594 \text{ m}^3/\text{sec.}$



(i) Since, Sugarcane is Perennial crop so, it will require water for almost entire year.

(ii) Design discharge = $\frac{0.594}{0.8}$
 $= 0.744 \text{ cumec.}$

Note: Kor Period or Kor depth
 Sugarcane 3 weeks 17 cm
 Wheat 4 weeks 13.5 cm

$D_s = \frac{864 \cdot B}{\Delta} = \frac{864 \times 3 \times 7}{17} = 1067 \text{ hec/cumec.}$

$D_w = \frac{864 \cdot B}{\Delta} = \frac{864 \times 4 \times 7}{13.5} = 1792 \text{ hec/cumec}$

$Q_s = \frac{240}{1067} = 0.22$ and, $Q_{\text{wheat}} = \frac{480}{1792} = 0.267.$

∴ Total = $0.22 + 0.267 = 0.49 \text{ cumec.}$

Q. A main Canal, which off takes from a Storage reservoir, has to irrigate Crops in a certain Country having 3-Season in a year.

Data for the irrigated Crop is given as below -

(a) Find the discharge required at the head of the main Canal taking time factor to be 0.7.

(b) What should be the gross Storage Capacity of reservoir

Assuming conveyance loss to be 20%, reservoir losses due to Evaporation and absorption to be 10%.

Name of Crop	Crop Period (B days)	Area Irrigated in hec.	Duty at the head of Main Canal (hec/cumec)
(a) Sugarcane	280	315	630
(b) Overlapp	100	70	630
(c) Jawar (Rabi)	120	4800	1600
(d) Bajra (Monsoon)	120	5600	2800
(e) Vegetable (Hot season)	120	350	700

Ans $\rightarrow (A)_s = 315/630 = 0.5 \text{ cumec}$

$(A)_{\text{overlap}} = 70/630 = 0.11$

$(A)_T = 3$

$(A)_B = 2$

$(A)_V = 0.5$



A required for each Season -

(a) Hot Season = $0.5 + 0.11 + 0.5 = 1.11$

(b) Monsoon Season = $0.5 + 2 = 2.5$

(c) Winter Season = $0.5 + 3 = 3.5$

A required at the head of Canal = $\frac{3.5}{0.7} = 5 \text{ Cumec.}$

Note $\rightarrow$ The above Values can be increased by 20-25 % to find Peak discharge.

Part (B) $\rightarrow V_s = (0.5 \times 280 \times 86400) = 12.096 \text{ mm}^3$

$V_o = 0.95$

$V_j = 3 \times 120 \times 86400 = 31.1 \text{ mm}^3$

$V_B = 2 \times 120 \times 86400 = 20.736 \text{ mm}^3$

$V_g = 0.5 \times 120 \times 86400 = 5.180 \text{ mm}^3$

Total Volume required = 70.07 M-m^3

Gross Volume = $\frac{70.07 \times 1.05}{0.9 \times 0.9} = 90.831 \text{ mm}^3$

Q. Monthly Water Requirement Studies are given below for Various Crops that are grown in a Particular area. A Reservoir is proposed to be Constructed to Command an area equal to $1.2 \times 10^5 \text{ hec.}$

- The Various crops are Paddy, Groundnut, Maize, Cotton, Sugarcane and Chillies.
- The area Under irrigation of these Crops are going to be 20%, 8%, 5%, 10%, 10%, 13% of Command respectively.
- Determine the annual Storage requirement for the reservoir, assuming Canal losses as 25% of head discharge and reservoir, dead Storage losses are 25% of Gross Capacity.
- Also fix the Channel Capacity for the Crop Pattern.
- Assume Suitable delta which is not given.

Month	Field Irrigation Requirement (FIR) cm					
	Paddy	Ground Nut	Maize	Cotton	SugarCane	Chillies
June	19				26	
July	6				7	
August	7				7	
Sept	8				6	1.7
Oct	30		3	4	35	24
Nov	21	6	15	8	42	22
Dec		16	24	21	18	16
Jan		22	21	23	22	11
Feb		14		14	25	
March					36	
April					40	
May					50	

Ans: 1.2×10^5 hac.

Area to be irrigated for each crop,

Crop	A	B	C	D	E	F
IOI	20%	5%	5%	10%	10%	3%
AI = IOI X CCA (hec)	24000	6000	6000	12000	12000	3600

Month	Water Required in $M-m^3$					
June	45.6				31.2	
July	14.4				8.4	
August	16.8				8.4	
Sept	19.2				7.2	0.612
Oct	7.2		1.8	4.8	4.2	8.64
Nov	50.4	3.6	9	9.6	50.4	7.92
Dec		9.6	14.4	25.2	21.6	5.76
Jan		13.2	12.6	27.6	26.4	3.96
Feb		8.4		16.8	30	
March					43.2	
April					48	
May					60	
Total	218.4	34.8	37.8	84	334.82	26.9

$$\text{Total Volume Area} = 736.71 \text{ M-m}^3$$

at Outlet,

$$\text{Volume reqd at Canal head} = \frac{736.71}{0.75} = 982.3 \text{ M-m}^3$$

$$\text{Gross Storage Requirement} = \frac{982.3}{0.75} = 1309.7 \text{ M-m}^3.$$

Month	Water Required ($M-m^3$)						Total Vol. of Water reqd by all Crops → Σ of Col (2) to Col. (7)	Discharge reqd in $M-m/d$ = $\frac{\text{Col. (8)}}{\text{intervalin days.}}$
	Col. (2)	Col. (3)	Col. (4)	Col. (5)	Col. (6)	Col. (7)	Col. (8)	Col. (9)
June	45.6				31.2		76.8	2.56
July	14.4				8.4		22.8	0.735
August	16.8				8.4		25.2	0.81
Sept	19.2				7.2	0.612	27.012	0.9004
Oct	7.2		1.8	4.8	4.2	8.64	64.44	2.07
Nov	50.4	3.6	9	9.6	50.4	7.92	130.92	4.36
Dec		9.6	14.4	25.2	21.6	5.76	76.56	2.469
Jan		13.2	12.6	27.6	26.4	3.96	83.76	2.701
Feb		8.4		16.8	30		53.2	1.971
March					43.2		43.2	1.393
April					48		48	1.6
May					60		60	1.935

The Maximum Value, out of all the discharges in Col. (9) is 4.36 M-m/day.

$$= \frac{4.36 \times 10^2 \times 10^4}{24 \times 60 \times 60}$$

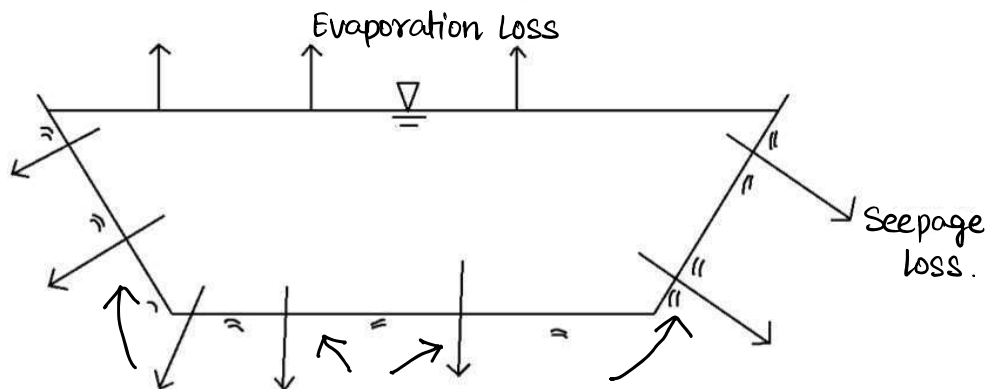
$$= 50.46 \text{ cumec.}$$



Assume that the Peak demand of the Month is 1.25 times the Average Demand.
Then, the Peak demand of the Most demanding Month = $1.25 \times 50.46 = 63.25$

Design Capacity of the outlet point = $63.25 \text{ m}^3/\text{sec}$.

- LOSSES OF WATER IN CANAL: During the flow of water from the main Canal to the outlet at the head of the Water Course, water is lost either by Evaporation from the Surface or by Seepage through Peripheries of the Channel.



- These losses may extend upto the Order of the 20-25 % of Water diverted into the main Canal.

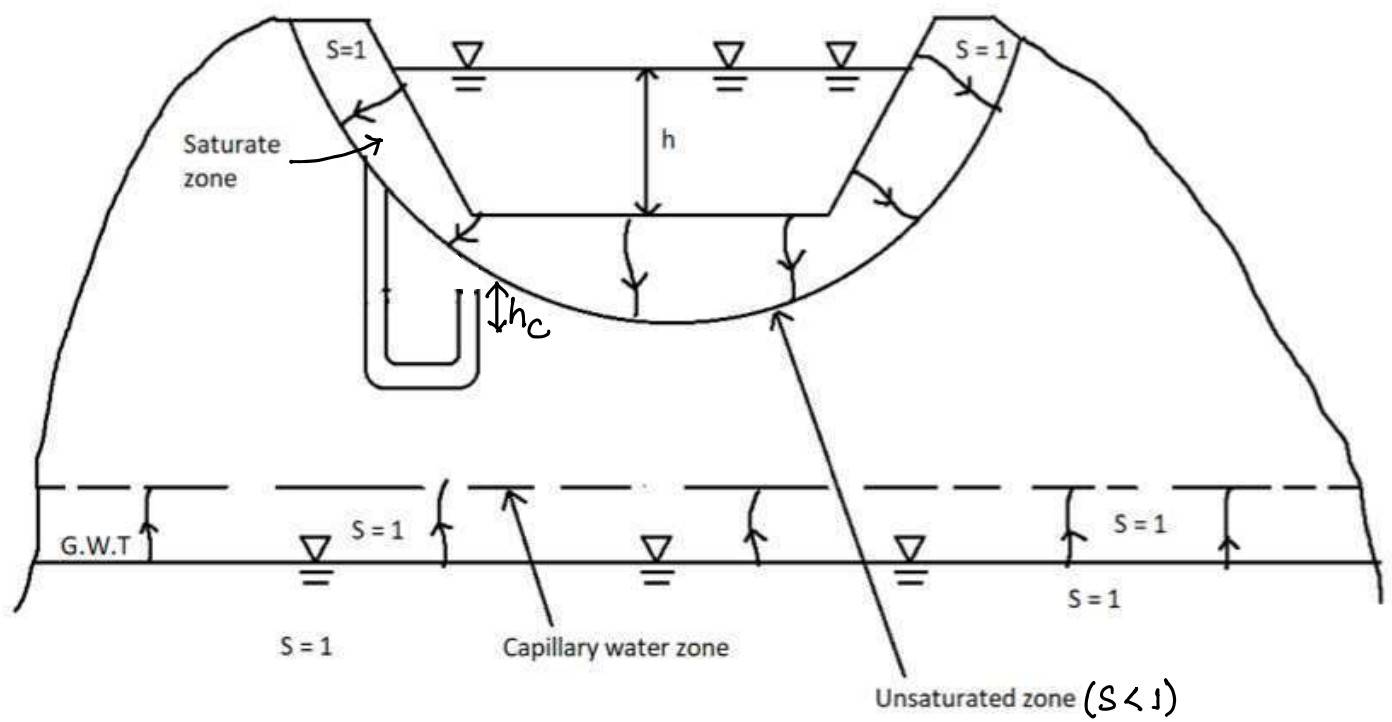
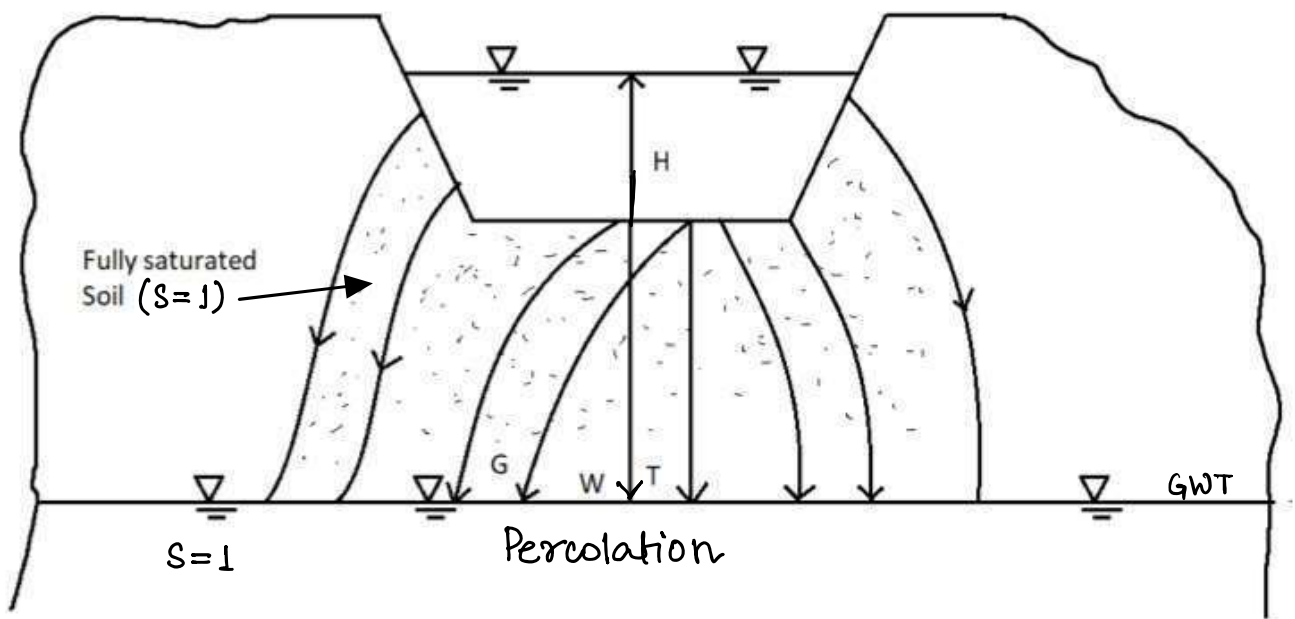
① EVAPORATION LOSSES: The Water lost by evaporation is generally Very Small as Compared to Water lost by Seepage.

- It is in Order of 2-3 % of total losses.
- It depends upon temperature, Wind Velocity and humidity at a Particular location.
- This loss Comes out to be more Corresponding to Summer.

② SEEPAGE LOSSES: These losses are further of two types :-
(i) Percolation.
(ii) Absorption.

① PERCOLATION: In Percolation, there exist a Zone of Continuous Saturation from Canal to the Water table and a direct flow is established.

- Almost all the Water lost from the Canal Joins the Ground Water table.
- The loss of Water depends upon the distance of the top Water-Surface level and level of the Water table (H).



① ABSORPTION: In Absorption, a small saturated zone exists around the canal section and is surrounded by a zone of decreasing saturation.

— Above water table there also exists a zone of capillary saturation.

— In this case, rate of flow depends upon the head of water in canal (h) and capillary head.

• FACTORS AFFECTING SEEPAGE LOSS

① TYPE OF SEEPAGE

— If percolation losses are more & if absorption losses are less.

② TYPE OF SOIL

— If soil is permeable, losses are more {as loss would be due to percolation}.
If soil is porous, losses are less {as loss would be due to absorption}.

③ CONDITION OF CANAL

— If the canal is new, it is unsilted and all the voids of soil are open. Hence, seepage losses are observed to be more than old silted canal whose voids are closed.

④ AMOUNT OF SILT CARRIED BY CANAL

— The more the silt, more the choking of the voids, lesser are the losses.

⑤ VELOCITY OF WATER IN CANAL

— More is the velocity of water, lesser is the time available for seepage. Hence, lesser are the losses.

⑥ SECTION OF CANAL

— More the wetted perimeter, more are the seepage losses.

Note: The channel losses can be computed empirically as follows:-

$$(a) \Delta Q = \frac{1}{200} (B + y)^{2/3}$$

Where,

ΔQ = Channel losses in cumecs/km length of channel.

B = Bed width of the channel in metres.

y = Depth of flow (m).

$$(b) \Delta Q = 1.9 \cdot Q^{1/6}$$

Where,

ΔQ = losses in cumecs per million sq. m of wetted perimeter.

Q = Discharge in cumec.



PREVIOUS YEAR QUESTIONS

ESE

Q. 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 that year? [2009]

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

Ans :

© 66 %.

$$\begin{aligned} \text{— In Kharif Season, Irrigated Area} &= 100 - 76 \\ &= 24 \% \end{aligned}$$

$$\begin{aligned} \text{— In Rabi Season, Irrigated Area} &= 100 - 58 \\ &= 42 \% \end{aligned}$$

$$\therefore \text{Total Intensity} = 66 \%.$$

Q. A Persian Wheel with an average discharge of 230 litre/minute irrigates 1 ha Wheat Crop in 50 hours. The average depth of Irrigation will be nearly.

- (a) 4 cm
- (b) 5 cm
- (c) 6 cm
- (d) 7 cm

[2020]

Ans :

(d) 7 cm

$$\text{— } Q \times \text{time} = \text{Area} \times \text{Average Depth of Irrigation.}$$

$$= \frac{230 \times 10^{-3} \times 50 \times 60}{10^4} \times 10^2$$

$$= 6.9 \text{ cm} \cong 7 \text{ cm.}$$

Q. The discharge Capacity Required at the Outlet to irrigate 3000 ha of Sugarcane having a kor depth of 173 mm and a Kor Period of 30 days is.

- (a) 2 m³/s
- (b) 1 m³/s
- (c) 20 m³/s
- (d) 0.20 m³/s

[2015]

Ans:

① $2 \text{ m}^3/\text{s}$

$$- Q_{\text{required}} = \frac{3000 \times 10^4 \times 173 \times 10^{-3}}{30 \times 24 \times 3600} \\ = 2 \text{ m}^3/\text{sec}.$$

GATE

Q. A field channel has Cultivable Command Area of 2000 ha. The Intensity of Irrigation for gram and wheat are 30% and 50% respectively. Gram has Kor Period of 18d, Kor depth 12 cm, While Wheat has 18 days and 15 cm. The discharge (m^3/s) required in the field channel to Supply Water to the Command area during Kor Period is _____. [2015, Shift-II]

Ans:

Gram and Wheat are Rabi Crops. Hence, both are grown in the same Period.

Discharge Required for gram,

$$Q_1 = \frac{A_1}{D_1} \\ = \frac{2000 \times 0.3}{8.64 \times \frac{18}{0.12}} = 0.463 \text{ m}^3/\text{s}$$

Discharge required for wheat,

$$Q_2 = \frac{A_2}{D_2} \\ = \frac{2000 \times 0.5}{8.64 \times \frac{18}{0.15}} = 0.964 \text{ m}^3/\text{s}.$$

$$\text{Total discharge, } Q = Q_1 + Q_2 \\ = \underline{1.427} \text{ m}^3/\text{s}.$$

Q. The Capacity of Irrigation System required to irrigate Crop X in 36 hectares is [2010]

- ① 83 l/s
- ② 67 l/s
- ③ 57 l/s
- ④ 53 l/s

Ans:

⑥ 67 l/s

— For Crop X, Peak rate of moisture use = 5 mm/day
Area = 36 hectare
= $36 \times 10^4 \text{ m}^2$

Discharge of Pump = $Q \text{ m}^3/\text{s}$.

Water Supplied by Pump in a day When Works for 10 h in a day with 75 % efficiency,
 $V = Q \times (10 \times 3600) \times 0.75 \text{ m}^3$.

This Volume Should be equal to Volume of Moisture Uses per day,
 $V = Q \times 10 \times 3600 \times 0.75$
 $= 5 \times 10^{-3} \times 36 \times 10^4$

$$\begin{aligned}\Rightarrow Q &= 0.0666 \text{ m}^3/\text{s} \\ &= 66.66 \text{ l/s} \\ &= 67 \text{ l/s}.\end{aligned}$$

Q. 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 distributory should be. [2007]

- ① 2.85 m^3/s
- ② 3.21 m^3/s
- ③ 4.63 m^3/s
- ④ 5.23 m^3/s

Ans:

③

— Area to be Irrigated, $A = \text{Culturable Command Area} \times \text{Intensity of Irrigation}$.

$$\begin{aligned}A &= 20000 \times 0.5 \\ &= 10000 \text{ ha}.\end{aligned}$$

$$\begin{aligned}\text{Kor Period, } t &= 30 \text{ days} \\ &= 30 \times 86400 \text{ sec}.\end{aligned}$$

$$\begin{aligned}\text{Kor Water depth } \Rightarrow d &= 120 \text{ mm} \\ &= 0.12 \text{ m}.\end{aligned}$$

$$\begin{aligned}\text{Water Supplied by distributory in Kor Period} &= \text{Water required by Crop} \\ Q \times t &= A \times d \\ Q \times 30 \times 86400 &= (10000 \times 10^4) \times 0.12 \\ Q &= 4.63 \text{ m}^3/\text{s}.\end{aligned}$$

SSC - JE

Q. Intensity of Irrigation _____.

[2016]

(a) Is the Percentage of Culturable commanded area Proposed to be irrigated Annually.

(b) Is always more than 100%.

(c) Is the Percentage that Could be Ideally Irrigated.

(d) All the Options are Correct.

Ans:

(a)

Q. Which of the following is not a Cause of Water logging?

(a) Excess tapping of the ground water.

(b) Frequent Irrigation

(c) High water table.

(d) Seepage from Unlined Canal.

[2017]

Ans:

(a) Excess tapping of the ground water is not a Cause of Water logging as it Results in the fall of water table.

DESIGN OF CANALS

- The Area bounded by Indo-Gangetic Plain Used to be in form of depression with Water flowing over it.
- With the Passage of time, it was filled with loosely Carried Silt Particles thereby forming Alluvial Soil.
- Almost all the rivers in northern India flows through such Soil hence, carries certain amount of Sediments. Since, artificial Canals have to Carry Water from these rivers they have the tendency of getting Silted and Scoured.
- Hence, design of such Canal is to be done which neither silts nor scours these type of Canals/Channels are known as Stable / Regime Channel.

• **REGIME CHANNEL**: A Channel is said to be in a State of Regime, if the flow is such that Silting and Scouring does not take place. {takes place upto Minimal or manageable extent}

- Such a State cannot be attained Practically in natural Channels, but can be attained in artificial Channel if designed Properly.
- The Principle of designing such a channel is that whatever silt has entered the Channel at its head is kept in Suspension Only so that it does not settle and the Velocity of Water should be such that it does not Carryout the Scouring of the Channel. (Bed and Slope).
- Designing of the Channel can be done by any of the following methods -

① **KENNEDY THEORY**: On the Basis of Observation, he concluded that the Silt Supporting Power in a Channel Cross-Section mainly depends upon generation of the eddies over the bed of the Channel, rising to the Surface.

- These eddies are generated due to friction of the flowing water with horizontal. {The distribution of which is non Uniform across the section}.
- The Vertical Component of these eddies try to keep the Sediment in Suspension, while the weight of Sediment tries to bring it down.
- So, if the Velocity is Sufficient to generate the eddies, so as to keep these Solids in Suspension Silting will be avoided.
- The Velocity at which this State occurs is termed as "Critical Velocity".
- According to Kennedy, this Velocity is given by -

$$V_0 = C_1 \cdot y^{C_2}$$

Where,

C_1 & C_2 are constants depends on type of Silt.
 $C_1 = 0.55$, $C_2 = 0.65$
 y = depth of flow.



Note: Since the above expression is valid Only for Upper Bari Doab Region, in Order to make this theory applicable to other region, also Kennedy introduces a Constant termed as "Critical Velocity Ratio" {C.V.R} Which depends upon Silt grade, and type of Silt.

Now, Critical Velocity is given by

$$V_0 = 0.55.m.y^{0.64}$$

for Coarse Sand, $m = 1$ to 1.2

for fine Sand, $m = 1$ to 1.7

TYPE OF SILT	<u>m</u>
Silt of 9ndus	0.7
Light Sandy Silt	1.0
Coarse Sandy Silt	1.1
Sandy loamy Silt	1.2
Hard Soil	1.3

..DESIGN STEPS IN KENNEDY THEORY

Step 1 - From the given value of discharge, by assuming a suitable trial depth, compute the Critical Velocity of flow.

$$V_0 = m.C_1.y^{C_2}$$

Guidelines for assuming trial depth, $y(m)$.

S.no.	Discharge, Q (m^3/s)	Trial depth, y (m)
1	0 - 20	1
2	20 - 40	2
3	40 - 80	2.5
4	80 - 100	3
5	> 100	3.5

Step 02 - Calculate Cross-Section area of the Canal -

$$A = \frac{Q}{V_0}$$

Step 03 - By Assuming a trapezoidal Section and Side Slopes of 0.5H : 1V, Calculate the dimensions of Canal.

Step 04 - Calculate Hydraulic radius

$$R = \frac{A}{P}$$



Step 05 - Calculate Actual Velocity (mean) of flow by Using Chezy's equation-

$$V = C\sqrt{RS}$$

$$C = \frac{\frac{1}{N} + \left[23 + \frac{0.0015S}{S}\right]}{1 + \left[23 + \frac{0.0015S}{S}\right] \frac{N}{\sqrt{R}}}$$

Where,

C - Chezy's Constant given by Kutter's equation

N - Manning's Constant

S - Canal bed Slope

R - Hydraulic Radius.

Step 06 - If the above Velocity is same as Critical Velocity then the design is Correct or else the above Steps are repeated by assuming other trial depth.

if Bed Slope is not given, then it can be assumed between $\frac{1}{3500}$ to $\frac{1}{5000}$.

if Roughness Constant is not given it can be assumed as follows -

Types of Surface	Manning's Constant or Kutter's Rugosity Coefficient
(i) Poorly Maintained Earthen Channel	0.025 - 0.030
(ii) Good Earthen Channel	0.022 - 0.025
(iii) Concrete Lined Channel	0.015 - 0.018

Q. Design an Irrigation Channel to carry $50 \text{ m}^3/\text{sec}$ of discharge. The Channel is to be laid at a Slope of 1:4000. The Critical Velocity ratio for the Soil is 1.1. Use Kutter's rugosity Coefficient as 0.023.

Ans: → Step 1

Assume $y = 2.5$

$$V_0 = m \cdot C_1 \cdot y^{C_2} = 1.1 \times 0.55 \times (2.5)^{0.64}$$

$$V_0 = 1.0875 \text{ m}^3/\text{sec}.$$

Step 2

$$A = \frac{Q}{V_0} = \frac{50}{1.0875} = 45.97 \text{ m}^2.$$

Step 3

$$A = y(Ny + B)$$

$$45.97 = 2.5 \times (0.5 \times 2.5 + B)$$

$$B = 17.140 \text{ m}$$

Step 4

$$P = B + 2\sqrt{y^2 + (Ny)^2}$$

$$= B + 2y\sqrt{1 + N^2}$$

$$P = 17.14 + 2 \times 2.5 \sqrt{1 + (0.5)^2}$$



$$P = 22.73 \text{ m}$$

$$R = \frac{A}{P} = \frac{45.97}{22.73} = 2.022 \text{ m}$$

Step 5

$$V = C \sqrt{RS}$$

$$V = \left[\frac{\frac{1}{n} + \left(23 + \frac{0.00155}{1/4000} \right)}{1 + \left(23 + \frac{0.00155}{1/4000} \right) \frac{n}{\sqrt{R}}} \right] \sqrt{RS}$$

$$V = \left[\frac{\frac{1}{0.023} + \left(23 + \frac{0.00155}{1/4000} \right)}{1 + \left(23 + \frac{0.00155}{1/4000} \right) \frac{1/0.023}{\sqrt{2.023}}} \right] \sqrt{(2.023) \times \frac{1}{4000}}$$

$$V = 1.1 \text{ m/sec}$$

Since, $V (1.1 \text{ m/sec}) > V_0 (1.087 \text{ m/sec})$
So, increase the depth.

Assume $y = 2.6 \text{ m}$

Step 1

$$\begin{aligned} V_0 &= m C_1 y^{C_2} \\ &= 1.1 \times 0.55 \times (2.6)^{0.64} \\ &= 1.115 \text{ m/sec} \end{aligned}$$

Step 2

$$A = \frac{50}{1.115} = 44.843 \text{ m}^2$$

Step 3

$$\begin{aligned} A &= (B + Ny) \cdot y \\ 44.843 &= (B + 0.5 \times 2.6) 2.6 \\ 44.843 &= (B + 1.35) 2.6 \end{aligned}$$

$$B = 15.89 \text{ m.}$$

Step 4

$$\begin{aligned} P &= B + 2\sqrt{y^2 + (Ny)^2} \\ &= B + 2y\sqrt{1 + N^2} \end{aligned}$$

$$P = 15.89 + 2 \times 2.6 \sqrt{1 + (0.5)^2}$$

$$P = 21.70 \text{ m}$$

$$\begin{aligned} R &= \frac{A}{P} = \frac{44.843}{21.70} \\ &= 2.066 \text{ m.} \end{aligned}$$



Step 5

$$V = C\sqrt{RS}$$

$$V = \left[\frac{1/n + \left(23 + \frac{0.00155}{S}\right)}{1 + \left(23 + \frac{0.00155}{S}\right) \frac{n}{\sqrt{R}}} \right] \sqrt{RS}$$

$$V = 1.1 \text{ m/s } \{ \text{Same as the depth } 2.5 \text{ m} \}.$$

$$A = 44.83 \text{ m}^2, R = 2.06 \text{ m}, B = 16 \text{ m}, m = 0.5, y = 2.6, S = \frac{1}{4000}$$

Q. Design an Irrigation Channel to carry 40 cumec of discharge, with B/D {Base width/depth} ratio as 2.5. The Critical Velocity ratio is 1.0. Use Kennedy theory and assume suitable value of Kutter's rugosity Coefficient.

Ans: →

Step 1

$$C.V.R = 1.0$$

$$V_0 = m \cdot 0.55 \cdot (y)^{0.64} \\ = 1 \times 0.55 \times (\Delta)^{0.64} \quad \{ y = \Delta \}$$

$$V_0 = 0.55 \cdot \Delta^{0.64}$$

Step 2

$$A = \frac{Q}{V_0} \\ = \frac{40}{0.55 \times (\Delta)^{0.64}} \quad \text{--- (i)}$$

Step 3

Assume Section to be Trapezoidal with Side Slope of 0.5H:1V

$$A = (B + my)y = (B + 0.5\Delta) \cdot \Delta \quad \text{--- (ii)}$$

$$B = 2.5\Delta$$

$$A = (2.5\Delta + 0.5\Delta) \cdot \Delta$$

$$A = 3\Delta^2 \quad \text{--- (iii)}$$

From equation (i) we get,

$$\frac{40}{0.55 \times (\Delta)^{0.64}} = 3\Delta^2$$

$$\Delta = 3.34 \text{ m}$$

$$B = 8.35 \text{ m}$$

$$V_0 = 1.19 \text{ m/sec.}$$

Step 4

$$P = B + 2y\sqrt{1+m^2} \\ = 8.35 + 2 \times 3.34 \cdot \sqrt{1+m^2} \\ P = 15.84 \text{ m}$$

$$R = \frac{A}{P} = \frac{3\Delta^2}{P}$$

$$= \frac{3 \times (3.34)^2}{15.8} = 2.12 \text{ m.}$$



Step 5

$$V = C\sqrt{RS}$$

Assume $N = 0.023$.

$$C = \left[\frac{1/0.023 + \left\{ 23 + \frac{0.00155}{S} \right\}}{\left\{ 1 + 23 + \frac{0.00155}{S} \right\} \frac{0.023}{\sqrt{2.12}}} \right]$$

for Stable Channel, $V = V_0$

$$1.19 = \frac{1/0.023 + \left\{ 23 + \frac{0.00155}{S} \right\}}{\left\{ 1 + 23 + \frac{0.00155}{S} \right\} \times \frac{0.023}{\sqrt{2.12}}}$$

$$S = \frac{1}{3700}$$

$$y = 3.34 \text{ m, } B = 8.35 \text{ m, } A = 35.46 \text{ m}^2, P = 15.84 \text{ m, } R = 2.12 \text{ m and } S = \frac{1}{3700}.$$

Since, there are lot of Calculations involved in Kennedy's theory. Hence, a graphical Solutions was proposed by "GARRET".

DRAWBACKS

- ① Kennedy Observed that Eddy Currents are developed Only from the Canal bed, Whereas in actual, they are also developed from Sides also.
- ② He did not proposed any Mathematical equation to Calculate Canal Bed Slope.
- ③ He did not proposed any Mathematical equation to Calculate Mean Velocity of flow, Instead used Chezy's equation. Therefore, all the limitations of Chezy's Theory are also included in this theory.
- ④ He did not Proposed any relation to Calculate C.V.R and randomly related it with Silt grade (type and Size of Silt).

② LACEY THEORY (IS 1772): It was Stated by Kennedy that a Channel is said to be in State of 'Regime' if there is neither Silting nor Scouring in the Channel.

- Opposite to this Lacey proposed that even a channel showing no Silting and no Scouring may actually not be in regime. hence, the difference between following regime Conditions -

- (a) True Regime
- (b) Initial Regime
- (c) Final Regime.



① TRUE REGIME: A channel is said to be in regime, if there is neither silting nor scouring for this condition to be satisfied, the silt entering into the channel must remain in suspension and carried through the channel.

- This can be achieved if following conditions are satisfied -

- ① Flow is Uniform.
- ② Discharge is constant.
- ③ Silt amount / Silt charge constant.
- ④ Type and size is same (silt grade).
- ⑤ Channel is flowing through a material which can be scoured as easily as it can be silted, { it is termed as incoherent alluvial }.
- ⑥ If the channel material is of same grade as it is transported through it.

- Hence, a designed channel shall be in true regime if all the above conditions are fulfilled which is practically not possible.

② INITIAL REGIME: → When only the bed slope of a channel varies due to silting and its cross-section or perimeter remains unaffected even though the channel exhibits no silting, no scouring properties, then the channel is termed as INITIAL REGIME.

- This regime condition is observed as the sides of the channel are restrained against silting or scouring due to the presence of clayey soil or due to presence of plants/grass.



- Their slopes & velocity are higher and cross-section narrower than what would have been if the sides were not rigid.

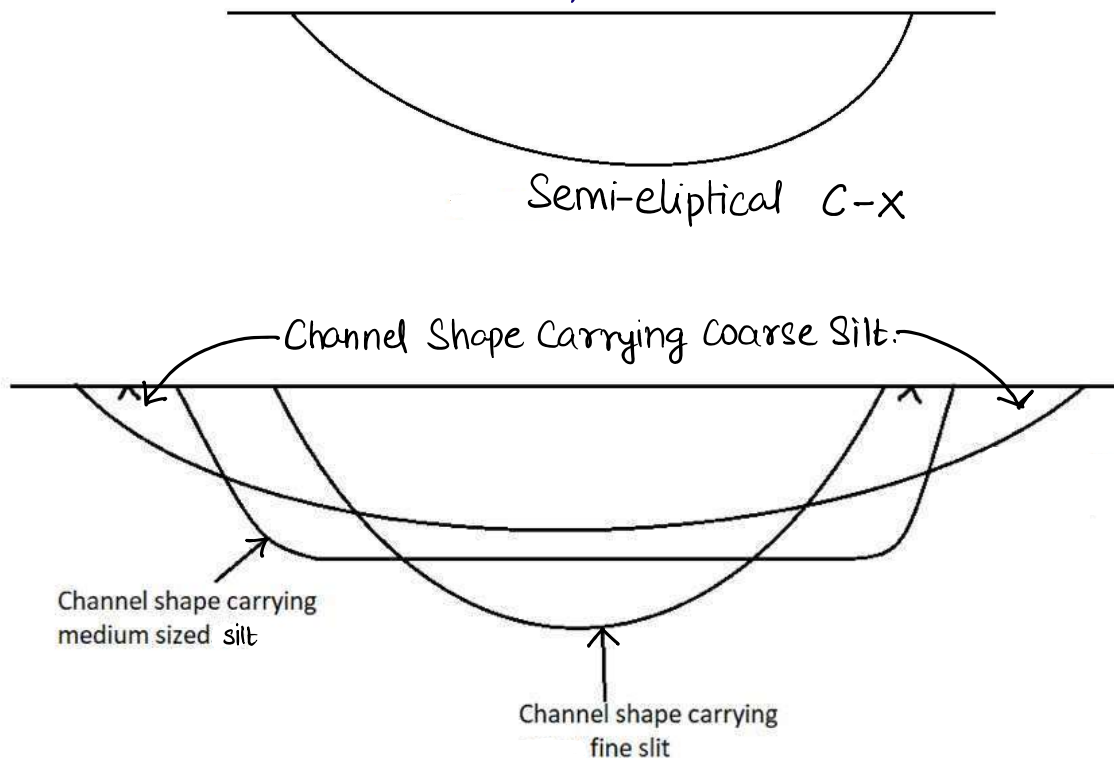
- Lacey theory is not applicable to these channels as they are not the channel in alluvial.

③ FINAL REGIME: If there is no resistance from the sides and all the variables such as perimeter, depth, slope are equally free to vary and finally get adjusted according to discharge and silt grade such that no silting and no scouring takes place, the channel is said as FINAL REGIME.

- Regime theory is applicable to this condition.

- Such a channel assumes semi-elliptical cross-section instead of trapezoidal section assumed by Kennedy.

- The finer the silt, the more nearly the section attains semi-circular geometry.



Note: Lacey theory also differs from Kennedy's theory on following points -

- ① Kennedy Considers the development of eddies supporting the silt in suspension only on the channel bed but in actual eddies developed on side slopes also serves the same purpose.
- ② Hence in Kennedy theory analysis was based on depth of flow whereas Lacey theory based on hydraulic mean depth of flow.
- ③ Lacey also considered the size of particles forming the channel in its designing in terms of factors termed as silt factor which is more appropriate than C.V.R. Used by Kennedy.

• LACEY REGIME THEORY - Lacey Proposed following Relationship between Parameters for designing of Canals.

$$\textcircled{a} V = \sqrt{\frac{2}{5}} \cdot \sqrt{FR} \text{ ————— } \textcircled{I}$$

$$\textcircled{b} AF^2 = 140 \cdot V^5 \text{ ————— } \textcircled{II}$$

Velocity of flow as per Lacey was also reported as -

$$V = 10.8 \cdot R^{2/3} \cdot S^{1/3} \text{ ————— } \textcircled{III}$$

- These 3 basic Conditions are further used to find the relationship between other Parameters -

① P-A Relationship

$$V = \sqrt{\frac{2}{5}} \cdot \sqrt{FR} \Rightarrow V^4 = \frac{4}{25} \cdot F^2 R^2 \text{ ————— } \textcircled{1}$$

$$\Rightarrow AF^2 = 140 \cdot V^5 \text{ ————— } \textcircled{II}$$

From ① & ②,



$$\Rightarrow V^4 = \frac{4}{25} \cdot \frac{140 \cdot V^5}{A} \cdot R^2$$

$$= 22.4 \cdot \frac{V^5 \cdot R^2}{A}$$

$$\Rightarrow V^4 = \frac{4}{25} \cdot \left\{ \frac{140 \cdot V^5}{A} \right\} \cdot \left(\frac{A}{P} \right)^2$$

$$\Rightarrow 1 = 22.4 \cdot \frac{VA}{P^2} \Rightarrow P^2 = 22.4 \cdot A$$

$$\boxed{P = 4.75\sqrt{A}} \quad **$$

② V-Q-F Relationship

$$AF^2 = 140 \cdot V^5$$

$$\Rightarrow AVF^2 = 140 \cdot V^5 \cdot V$$

$$\Rightarrow V^6 = \frac{AF^2}{140}$$

$$\Rightarrow V = \left[\frac{A \cdot F^2}{140} \right]^{1/6}$$

③ S-F-Q Relationship

$$V = 10.8 \cdot R^{2/3} \cdot S^{1/3} \quad \text{--- ①}$$

And,

$$V = \sqrt{\frac{2}{5}} \cdot \sqrt{FR} \quad \text{--- ②}$$

Cubing On both sides in equation ① we get,

$$V^3 = 1260 \cdot R^2 \cdot S$$

$$S = \frac{V^3}{R^2} \cdot \frac{1}{1260}$$

$$S = \left(\frac{V^2}{R} \right)^{3/2} \cdot \frac{1}{1260 \cdot R^{1/2}}$$

$$= \left(\frac{2}{5} \cdot F \right)^{3/2} \cdot \frac{1}{1260 \cdot R^{1/2}} \quad \left\{ \text{From equation ②} \right\}$$

$$S = \left(\frac{2}{5} F \right)^{3/2} \cdot \frac{1}{1260} \left(\frac{4.75\sqrt{A}}{A} \right)^{1/2}$$

$$V = \left(\frac{A \cdot F^2}{140} \right)^{1/6} \Rightarrow \frac{Q}{A} = \left(\frac{A \cdot F^2}{140} \right)^{1/6} \Rightarrow A = Q \left(\frac{140}{Q \cdot F^2} \right)^{1/6}$$

$$S = \left(\frac{2}{5} \cdot F \right)^{3/2} \cdot \frac{1}{1260} \left[\frac{4.75\sqrt{A}}{A \left(\frac{140}{Q \cdot F^2} \right)^{1/6}} \right]^{1/2}$$



$$S = \frac{F^{5/3}}{Q^{1/6} \cdot 3340} **$$

Note: Here, Silt factor can be computed as
 $F = 1.76\sqrt{d_{50}}$

Where,

d_{50} = Size below which 50% of Particles are finer. (mm).

•• DESIGN STEPS

① Calculate the Velocity of flow.

$$V = \left[\frac{Q \cdot F^2}{140} \right]^{1/6}$$



② Work out hydraulic Mean depth of flow.

$$R = \frac{5}{2} \left[\frac{V^2}{F} \right]$$

③ Compute the area of Channel

$$A = \frac{Q}{V}$$

④ Compute Wetted Perimeter.

$$P = 4.75\sqrt{Q}$$

⑤ From Area and Perimeter, dimensions of the Channel can be Computed.

⑥ Compute the Slope of the Channel.

$$S = \frac{F^{5/3}}{3340 \cdot Q^{1/6}}$$

Q. Design a Regime Channel for a discharge of 50 Cumec and $F=1.1$. Use Lacey's theory.

Ans :->

$$V = \left[\frac{50 \times 1.1^2}{140} \right]^{1/6} = 0.869 \text{ m/sec}$$

$$R = \frac{5}{2} \left(\frac{0.869^2}{1.1} \right) = 1.718 \text{ m}$$

$$A = \frac{Q}{V}$$

$$= \frac{50}{0.869} = 57.53 \text{ m}^2$$

$$P = 4.75\sqrt{50} \\ = 33.58 \text{ m}$$

$$S = \frac{1.1^{5/3}}{3340 \cdot 50^{1/6}} = \frac{1.1^{5/3}}{3340 \cdot (50)^{1/6}} \\ = \frac{1}{5469.15}$$

Assume trapezoidal Channel of 0.5H:1V.



$$P = B + 2y\sqrt{1+m^2}$$

$$\Rightarrow 33.58 = B + 2y\sqrt{1+0.5^2}$$

$$\Rightarrow 33.58 = B + 2.23y$$

$$\Rightarrow A = (B + my)y = (B + 0.5y)y$$

$$\Rightarrow 57.53 = By + 0.5y^2$$

$$\Rightarrow 57.53 = (33.58 - 2.23y)y + 0.5y^2$$

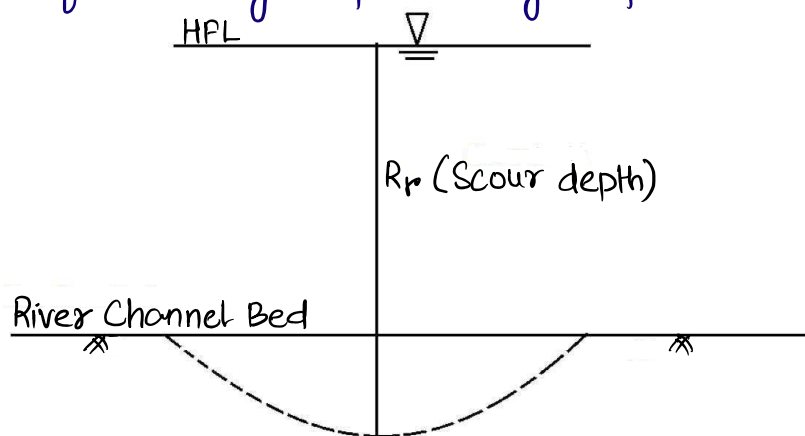
$$= 0.5y^2 + 33.58y - 2.23y^2$$

$$\Rightarrow 1.73y^2 - 33.58y + 57.53 = 0$$

$$y = 1.9 \text{ m and } B = 29.25 \text{ m.}$$

Note — Lacey also proposed the Relationship to find the Scour depth.

— Scour depth is the maximum depth in the river or Channel bed upto which the Soil will be eroded or Scoured from the highest point or highest flood level of the River.



$$V^2 = \frac{2}{5} FR$$

$$\Rightarrow R = \frac{V^2 \cdot 5}{F} \quad \text{--- (I)}$$

$$V^2 = \left[\frac{Q \cdot F^2}{140} \right]^{2/6}$$

$$= \left[\frac{Q \cdot F^2}{140} \right]^{1/3} \quad \text{--- (II)}$$

$$R = \frac{5}{2} \left(\frac{Q \cdot F^2}{140} \right)^{1/3} \cdot \frac{1}{F}$$

$$R = 0.481 \left(\frac{Q}{F} \right)^{1/3}$$

The above Relationship Can also be Written in terms of discharge per Unit Width.

$$Q = AV$$

$$\Rightarrow \frac{Q}{B} = \frac{A}{B} \times V$$

$$= R \times V$$



For wide channel, $\frac{A}{B} = R$

$$Q = \left\{ \frac{5}{2} \times \frac{V^2}{F} \right\} \times V = \frac{5}{2} \times \frac{V^3}{F}$$

$$Q = \frac{5}{2} \left(\frac{Q \cdot F^2}{140} \right)^{3/6} \cdot \frac{1}{F} = 0.211 \times Q^{1/2}$$

$$Q = 22.46 \cdot q^2 \quad **$$

$$\Rightarrow R_r = 0.481 \left(\frac{22.46 \cdot q^2}{F} \right)^{1/3}$$

$$R_r = \frac{1.357 \cdot q^{2/3}}{F^{1/3}} \quad **$$

• LINING OF CANAL : $\rightarrow$ By Lining the Canal, signifies that the Earthen Surface of the Channel is Lined with a Stable lining Surface Such as Asphalt, bricks, Concrete tile etc.

— Depending Upon the type of Lining adopted, the Seepage losses can be reduced upto 2-3% of their Original Values.

— Apart from Reduction in Seepage losses it also offers Other advantages as follows —

① SEEPAGE CONTROL : $\rightarrow$ Depending upon the type of lining Used, a particular extent of Seepage Control is achieved.

Type of Lining	Initial Rate of Seepage (Cumec/Million sq. m of Area)	Final Rate of Seepage in Cumec/Million sq. m of Area.
(a) Unlined channel	7.5	3.5
(b) Cement Mortar	0.17	0.009
(c) Cement Mortar with Brick blast.	0.13	0.007
(d) Cement lime Concrete	0.4	0.13

② PREVENTION OF WATER LOGGING : Uncontrolled Seepage often Raises the Water table level Surrounding the fields upto or near to the ground level, thereby Causing Water-logging, lining of Canal Prevents Seepage.

③ INCREASE IN DISCHARGE CAPACITY : The Capacity of Canal depends on roughness of the Channel.

$$(Q \propto \frac{1}{N})$$



- By Lining the Canal, roughness is reduced that in turn increases the discharge carrying capacity of the Canal.

Type of Canal	N
(a) Cast in situ Concrete trowel Finish	0.015 - 0.018
(b) Cement Plastered Masonary	0.012 - 0.015
(c) P.C.C Slabs or tiles	0.018 - 0.020
(d) Brick Lining	0.018 - 0.020
(e) Split Boulder Lining	0.020 - 0.025
(f) Round Boulder Lining	0.025 - 0.035

④ INCREASE IN COMMAND AREA : A lined Canal can be design to Carry same discharge at reduced area, thereby it increases Cultural Command Area.

⑤ ELIMINATION OF FLOOD : An Unlined Canal founded on Weaker Foundation have always the tendency to fail & Cause the flooding of nearby fields.

- Lining of these Canals, avoids this.

⑥ MAINTENANCE COST IS REDUCED : Lining of Canal Reduced the Maintenance Cost incurred for-

- Periodic removal of Silt deposited on the bed and Sides of Canal.
- Minor Repairs (Cracks & Cuts).
- Removal of Weeds and Water Plants.

•• FINANCIAL JUSTIFICATION OF LINING

- Mathematically, Expenditure On lining is Justified if the Resultant Annual benefits Exceeds Annual Cost.

- i.e. Benefit Cost ratio is more than '1'.

- Financial Justification of lining Can be Computed as follows -

① ANNUAL BENEFIT

- Irrigation Water is Sold to the Cultivator that generates Revenue.

- Let Rate of Irrigation Water be R rupees per Cumec.

- If x Cumec of Water is Saved by lining then, additional revenue generated Would be 'x R₁' rupees.

- Let R_2 rupees be the maintenance cost of the Canal (Unlined) On lining cost (Maintenance) is saved by $y\%$.

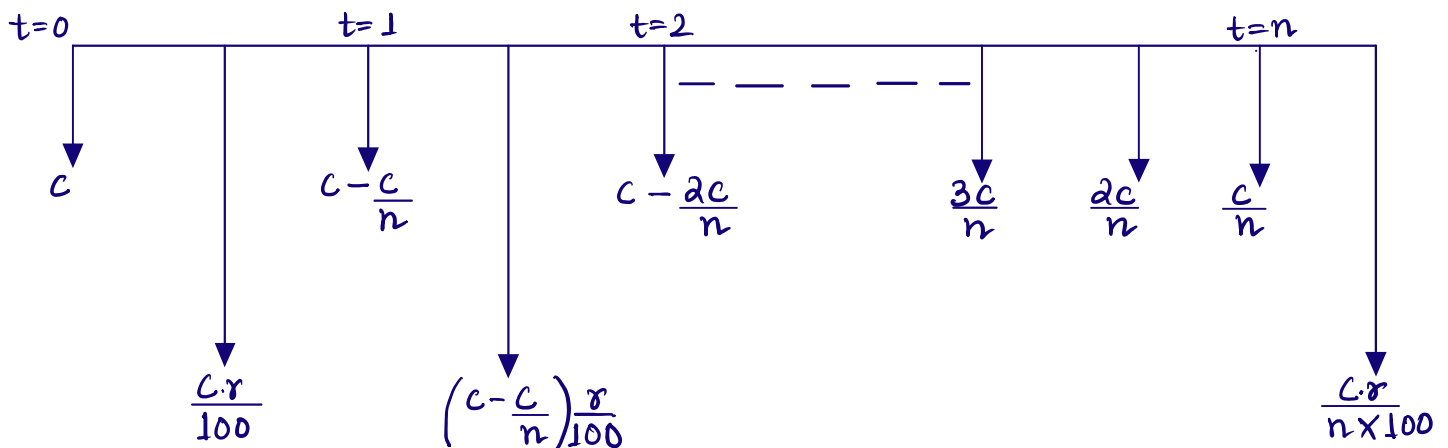


$$\text{Total Annual Benefit} = xR_1 + y.R_2$$

② ANNUAL COST

- If the Capital expenditure On lining be ' c ' rupees and lining has life of ' n ' years, then the annual depreciation cost would be $\frac{c}{n}$ rupee/year.

- If $r\%$ be the rate of Simple Interest then the interest lost on an average per year would.



$$\begin{aligned} \text{Average Annual Interest Cost} &= \frac{\frac{c \cdot r}{100} + \frac{c \cdot r}{n \times 100}}{2} \\ &= \frac{c \cdot r}{100} \left(1 + \frac{1}{n} \right) \frac{1}{2} \end{aligned}$$

$$\text{Since } n \text{ is comparatively large} \Rightarrow 1 + \frac{1}{n} = 1$$

$$\text{Hence, Avg. Annual Interest Cost} = \frac{c \cdot r}{2 \times 100}$$

$$\text{Total Annual Cost} = \frac{c}{n} + \frac{c \cdot r}{2 \times 100}$$

$$\text{Benefit Cost Ratio} = \frac{xR_1 + yR_2}{\frac{c}{n} + \frac{c \cdot r}{2 \times 100}}$$

Note: There are certain benefits of lining of Canal which can not be represented in terms of Monetary Values i.e. reduced, risk of breaching, reduced break out of water borne diseases etc.

4. An Unlined Canal giving a Seepage loss of 3.3 Cumec per Million sq. metres of Wetted Area is proposed to be lined with 10 cm thick Cement Concrete lining, which costs Rs 180 per 10 sq. m. Given the following data, Workout the economics of lining and benefit Cost Ratio.

Annual Revenue per Cumec of Water from all Crops = Rs. 3.5 lakhs.

Discharge in the Channel = 83.5 Cumec.

Area of the Channel = 40.8 sq. m.

Wetted Perimeter of the Channel = 18.8 m

Wetted Perimeter of the Lining = 18.5 m

Annual Maintenance Cost of Unlined Channel per 10 m² = Rs. 1

Assume 40% of Maintenance is Saved by Lining

Rate of Interest = 7%

Assume Life of lining = 40 years.

Ans : → Let Channel length be 1 Km.

$$\begin{aligned}\text{Wetted Perimeter Surface} &= 18.8 \times 10^3 \\ &= 18,800 \text{ sq. m}\end{aligned}$$

(i) Annual Benefits

(a) Seepage

$$\begin{aligned}\text{Seepage loss in Unlined Canal} &= \frac{3.3}{10^6} \times 18,800 \\ &= 0.062 \text{ Cumec/Km}\end{aligned}$$

$$\begin{aligned}\text{Seepage loss in Lined Canal} &= \frac{3}{100} \times 0.062 \\ &= 1.86 \times 10^{-3} \text{ Cumec/Km}\end{aligned}$$

$$\begin{aligned}\text{Net Saving of Irrigation Water} &= 0.062 - 1.86 \times 10^{-3} \\ &= 0.0601 \text{ Cumec/Km.}\end{aligned}$$

$$\begin{aligned}\text{Annual Revenue Saved per Km of Channel} &= 0.0601 \times 3.5 \times 10^5 \\ &= 21035 \text{ Rupee/Km.}\end{aligned}$$

(b) Saving in Maintenance

$$\begin{aligned}\text{Maintenance Cost of Unlined Channel} &= \frac{1}{10} \times 18800 \\ &= 1880 \text{ Rs/Km.}\end{aligned}$$

$$\begin{aligned}\text{Saving achieved by Lining} &= 0.4 \times 1880 \\ &= 752 \text{ Rs/Km.}\end{aligned}$$

$$\begin{aligned}\text{Total Annual Benefit} &= 21035 + 752 \\ &= 21787 \text{ Rs/Km.}\end{aligned}$$

(ii) Annual Costs -

$$\begin{aligned}\text{Area of Lining per Km of channel} &= 18.5 \times 10^3 \\ &= 18,500 \text{ m}^2.\end{aligned}$$



$$\begin{aligned}\text{Cost of Lining} &= \frac{180}{10} \times 18,500 \\ &= 3,33,000 \text{ Rs/Km.}\end{aligned}$$

$$\begin{aligned}\text{Depreciation Cost} &= \frac{3,33,000}{40} \\ &= \text{Rs } 8325\end{aligned}$$

$$\begin{aligned}\text{Annual Interest Cost} &= \frac{C \cdot r}{2 \times 100} = \frac{3,33,000 \times 7}{2 \times 100} \\ &= 11,655 \text{ Rs/Km.}\end{aligned}$$

$$\begin{aligned}\text{Total Annual Cost} &= 8325 + 11,655 \\ &= 19,980 \text{ Rs/Km.}\end{aligned}$$

$$\text{Benefit Cost ratio} = \frac{21035}{19980} = 1.052 > 1$$

Hence, Lining is Justified.

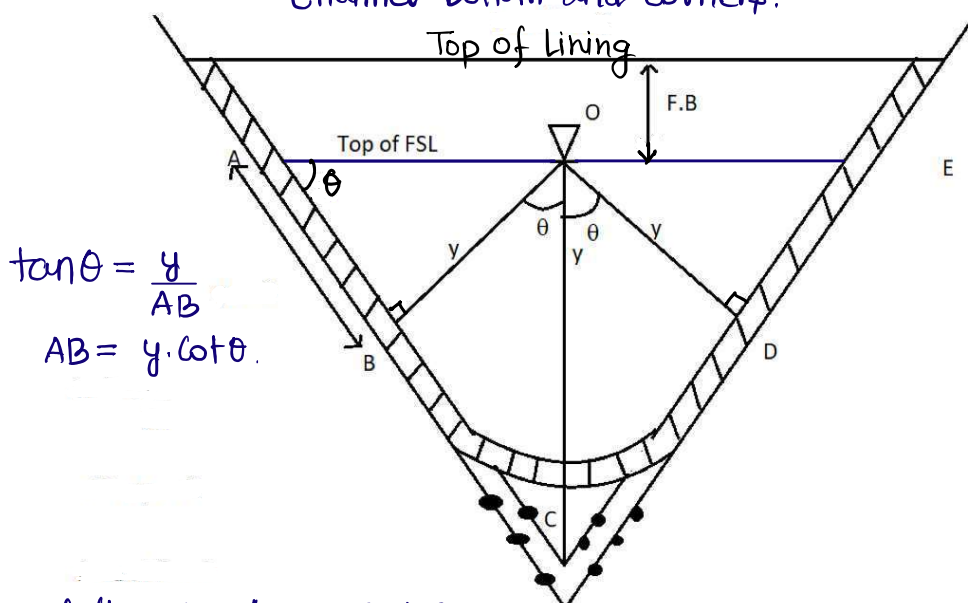
•• DESIGN OF LINED CANAL (IS 10430)

- Irrigation Canals should be aligned and laid out so that velocity of flow is uniform under all its conditions so that the water reaches the irrigated area at an elevation sufficient to insure even distribution of water.

- Generally, two type of Channel Sections are adopted

- ① Triangular { for low Q }
- ② Trapezoidal { for High Q }

① TRIANGULAR CHANNEL: In Order to increase the discharge carrying capacity (Q.dR), Hydraulic Mean depth of flow is increased by reducing the Perimeter for given Area, that is achieved by rounding off the Channel bottom and corners.



$$\begin{aligned}\Rightarrow \text{Area of flow } A &= \text{Area } ABCDEO \\ &= \text{Area } AOB + \text{Area } BCDO + \text{Area } ODE\end{aligned}$$



$$= \frac{1}{2} \cdot y \cot \theta \cdot y + \frac{\pi y^2 (2\theta)}{2\pi} + \frac{1}{2} y \cdot \cot \theta \cdot y$$

$$= y^2 \cot \theta + y^2 \theta$$

$$A = y^2 (\theta + \cot \theta)$$



⇒ Perimeter of flow P = length of ABCDE
 = length AB + length BCD + length DE.

$$P = y \cdot \cot \theta + \frac{2\pi \cdot y (2\theta)}{2\pi} + y \cdot \cot \theta$$

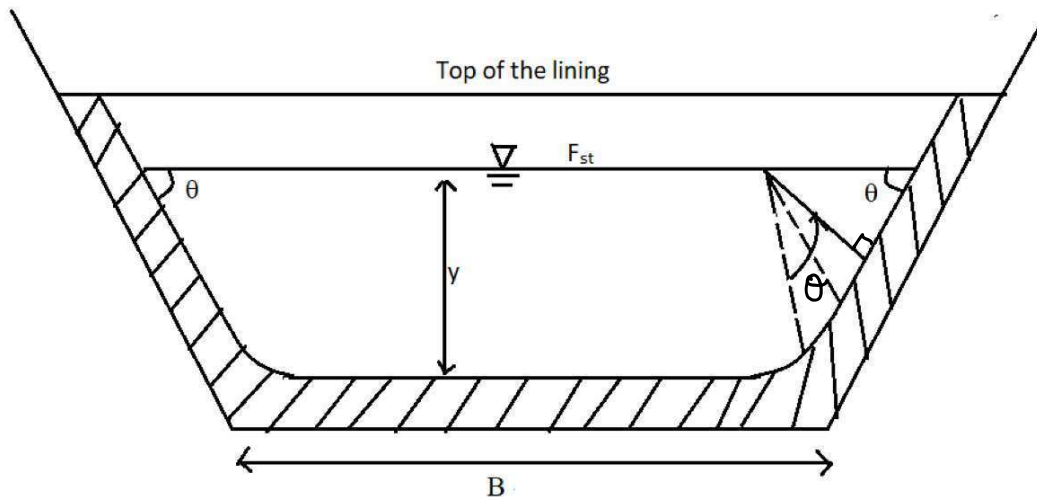
$$= 2y \cot \theta + 2y \theta$$

$$P = 2y \cdot (\cot \theta + \theta)$$

⇒ Hydraulic mean depth of flow R =

$$R = \frac{A}{P} = \frac{y}{2} \quad **$$

② TRAPEZOIDAL CHANNEL :



$$\Rightarrow A = B \cdot y + 2y^2 \frac{\theta}{2} + 2y^2 \cot \theta \cdot \frac{1}{2}$$

$$= y \left(B + 2y \cdot \frac{\theta}{2} + \frac{2y \cdot \cot \theta}{2} \right)$$

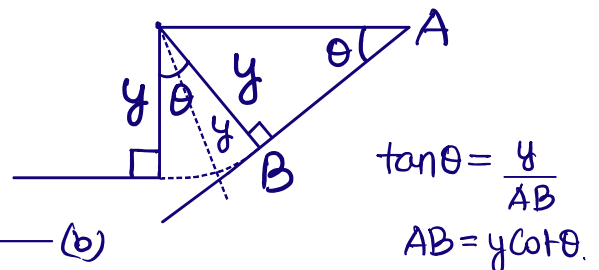
$$A = y(B + y \cot \theta + y \theta) \quad \text{--- (a)}$$

$$\Rightarrow \text{Perimeter, } P = 2y \cdot \cot \theta + 2 \left(\frac{2\pi y \cdot \theta}{2\pi} \right) + B$$

$$P = 2y \cdot \cot \theta + 2y \theta + B \quad \text{--- (b)}$$

$$\Rightarrow R = \frac{A}{P}$$

$$R = \frac{y [B + y \cot \theta + y \theta]}{[B + 2y \cdot \theta + 2y \cot \theta]} \quad **$$



In Order to limit the Scouring of the Material Used for Lining max. velocity is limited in channel as per the type of Material and type of lining.

Type of Lining	Permissible Velocity (m/sec)
Cement Concrete Lining	2 - 2.5
Burnt Clay Brick/Tile Lining	1.8
Boulder Lining	1.5

- In Order to prevent the Overflow from the channels during excess flow, additional Space is being Provided (F. B) at the top { above FSL } which depends upon the discharge to be carried by the Channel.

Type of Canal	Discharge in Cumec	Free Board (m)
① Main Canal and Branch	$Q > 10$	0.75
② Branch Canal and distributaries.	(i) $5 < Q < 10$	0.60
	(ii) $1 < Q < 5$	0.50
③ Minors	$Q < 1$	0.30
④ Water Courses	$Q < 0.06$	0.1 - 0.15

Q. Design a lined Channel to Carry a discharge of 15 Cumec. The available and accepted Slope is 1:9000. Consider Side Slope to be $1\frac{1}{4}H : 1V$ and $n = 0.015$.

Ans: →

Provide triangular Channel to Carry discharge = 15 Cumec.

$$\tan \theta = \frac{1}{\frac{5}{4}} = \frac{4}{5}$$

$$\theta = 38.66^\circ$$

$$= 0.675 \text{ radians}$$

$$\cot \theta = \frac{5}{4} = 1.25$$

$$A = y^2(\theta + \cot \theta)$$

$$= y^2(0.675 + 1.25)$$

$$= 1.925 y^2$$

$$P = 2y(\theta + \cot \theta)$$

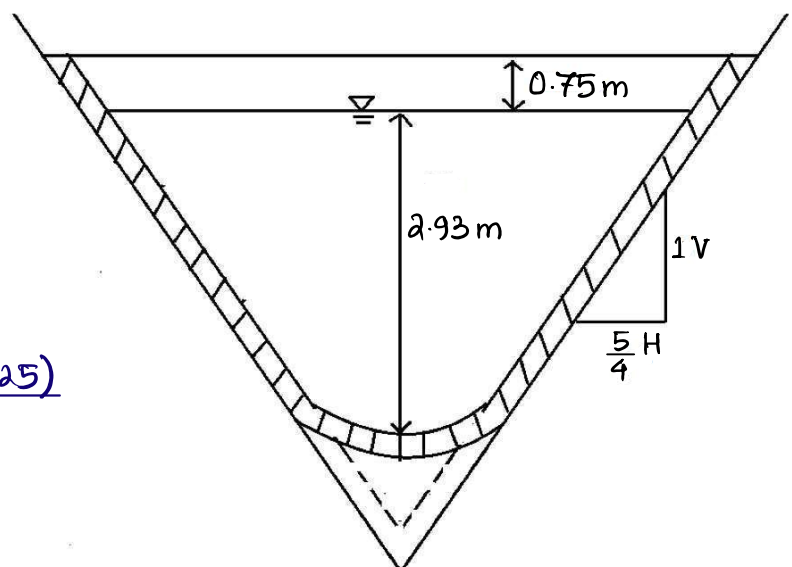
$$= 2y(0.675 + 1.25)$$

$$P = 3.85y$$

$$R = \frac{A}{P} = \frac{y^2(0.675 + 1.25)}{3.85y}$$

$$= \frac{y(1.925)}{3.85}$$

$$= 0.5y$$



$$\begin{aligned}
 Q &= AV \\
 &= A \cdot \frac{1}{N} \cdot R^{2/3} \cdot S^{1/2} \\
 &= \frac{S^{1/2} \cdot A^{5/3}}{N \cdot P^{2/3}}
 \end{aligned}$$

$$\Rightarrow Q = 15 = \left(\frac{1}{9000} \right)^{1/2} \cdot \frac{1}{(0.015)} \cdot \frac{(1.925y^2)^{5/3}}{(3.85y)^{2/3}}$$

$$y = 2.93 \text{ m.}$$

Provide F.B. of 0.75 m, Side Slope $\frac{5}{4} \text{ H} : 1 \text{ V}$

$$\begin{aligned}
 \text{Total depth of Channel} &= 2.93 + 0.75 \\
 &= 3.7 \text{ m}
 \end{aligned}$$

Q. Design a Concrete Lined Channel to Carry a discharge of 350 Cumec at a Slope of $\frac{1}{5000}$. The Side Slope of Channel may be taken as $\frac{3}{2} \text{ H} : 1 \text{ V}$, $n = 0.015$. Assume

Limiting Velocity in the Channel as 2m/sec.

Ans: $\rightarrow$

Provide Trapezoidal Channel to Carry discharge = 350 Cumec.

$$\tan \theta = \frac{1}{\frac{3}{2}} = \frac{2}{3}$$

$$\Rightarrow \theta = 0.588 \text{ radian.}$$

$$\cot \theta = 1.50$$

$$\begin{aligned}
 A &= y[B + y \cot \theta + y \theta] \\
 A &= y[B + y \times 1.5 + y \times 0.588]
 \end{aligned}$$

$$A = By + y^2 \cdot 0.588 + 1.5 \cdot y^2 \quad \text{--- (I)}$$

$$\begin{aligned}
 P &= 2y \cot \theta + 2y \theta + B \\
 &= 2y \cdot 1.5 + 2y \times 0.588 + B \\
 &= B + 1.176y + 3y
 \end{aligned}$$

$$P = B + 4.176y \quad \text{--- (II)}$$

$$V = \frac{1}{N} \cdot R^{2/3} \cdot S^{1/2}$$

$$\Rightarrow 2 = \frac{1}{0.015} \left[\frac{By + 2.05By^2}{B + 4.176y} \right] \left[\frac{1}{5000} \right]^{1/2} \quad \text{--- (A)}$$

$$\begin{aligned}
 Q &= AV \\
 \Rightarrow A &= \frac{Q}{V} = \frac{350}{2} = 175 \text{ m}^2
 \end{aligned}$$

$$\Rightarrow 175 = By + 2.058y^2 \quad \text{--- (B)}$$



$$\Rightarrow d = \frac{1}{0.015} \left[\frac{175}{B + 4.176y} \right] \left[\frac{1}{5000} \right]^{1/2}$$

$$\Rightarrow B + 4.176y = 164.99$$

$$\Rightarrow B = 164.99 - 4.176y$$

$$\Rightarrow 175 = y(164.99 - 4.176y) + 2.058y^2$$

$$\Rightarrow 175 = 164.99y - 4.176y^2 + 2.058y^2$$

$$\Rightarrow 2.118y^2 - 164.99y + 175 = 0$$

$$\Rightarrow y^2 - 77.89y + 82.625 = 0$$

$$\Rightarrow y = 1.075 \text{ m.}$$

Provide F.B = 0.75 m.

$$B = 1.075 + 0.75$$

$$= 1.825 \text{ m.}$$

•• TYPES OF LINING

— Various types of Canal lining which are commonly used are as follows —

(a) Hard Surface Lining.

- Cement Concrete Lining
- Shotcrete
- Tile/Cement Concrete tile
- Asphaltic Lining
- Boulder Lining

(b) Earth type Lining

- Compacted Earth Lining.
- Soil Cement.

① CEMENT CONCRETE LINING : It offers longer life, less permeable.

- More resistant to erosion
- It permits fast construction
- Maintenance cost is less, high initial cost.
- It has high tendency of development of crack.
- Skilled supervision required.

② SHOTCRETE

- It is suitable to be used for smaller jobs.
- It is also useful for repair work.
- It is very helpful in case, canal has irregular cross-section.



③ CEMENT CONCRETE TILE/BRICK

- It does not require any skilled supervision.
- No Expansion Joints are required.
- Even if Section is not of regular geometry it can be used.
- Large no. of labour is required.



④ ASPHALTIC LINING

- It is highly flexible.
- It does not decrease Roughness Coefficient.
- It also permits growth of weeds.

⑤ BOULDER LINING

- It is also termed as dry Stone lining/Stone Pitching.
- The biggest advantage of such lining is that it is pervious lining, allowing free flow of water.
- It is generally provided to impart stability to the channel.

⑥ COMPACTED EARTH LINING

- When cost is limited, locally available soil compacted to wet of OMC can also be used as lining.

⑦ SOIL CEMENT LINING

- When more strength and less permeability is required, cement slurry mixed with soil can also be used as lining material.

•• PROPERTIES REQUIRED IN LINING

- ① Impermeability
- ② Hydraulic Efficient
- ③ Economical
- ④ Resistant to Erosion
- ⑤ Structural Ability
- ⑥ Durability
- ⑦ Reparability.

•• SEDIMENT TRANSPORT MECHANISM

- When water flows in channel it carries out scouring of the surface due to which these particles are swept downstream by the moving water. It is termed as sediment transportation.
- The sediment in a canal is a burden to be carried by the flowing water and is termed as sediment load.

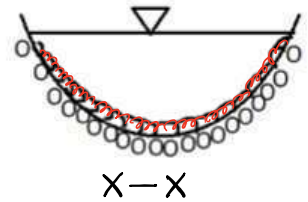
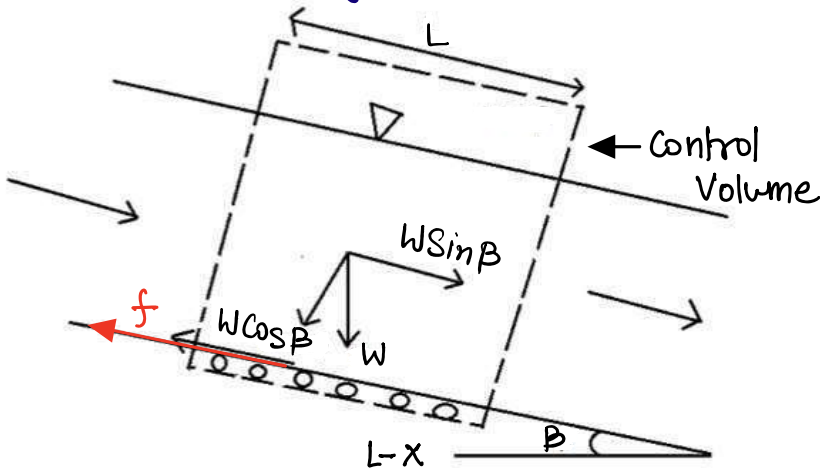
—The Sediment may move in water either as bed load or as suspended load.

• Assumptions in Mechanism of Sediment Transport as per 'Shield's theory'.

① Soil is incoherent { i.e. Cohesive force between the particles is zero, $c=0$ }.

② Sediment load is Uniform.

③ The Basic Mechanism of Sediment transport is the Weight of Water, exerting a Shear force or drag force or tractive force on the medium Particle.



⇒ Shear force Acting On Soil Particle (F) = $W \cdot \sin \beta$
 Where β is very small,
 $\beta \approx \sin \beta \approx \tan \beta$

Frictional Force, $F = W \cdot \tan \beta$
 $\tan \beta = \text{Slope of Channel bed} = S$

$$F = W \cdot S$$

$$= \gamma_w \cdot V_w \cdot S$$

Shear force $F = (A \cdot L) \cdot \gamma_w \cdot S$ { $\because \text{Volume} = \text{Area} \times \text{Length}$ }
 $V = A \times L$

$$\text{Shear Stress} = \frac{F}{\text{Wetted Area}}$$

$$\tau = \frac{A \cdot L \cdot \gamma_w \cdot S}{P \cdot L}$$

$$\tau = \gamma_w \cdot \left(\frac{A}{P} \right) \cdot S$$

$$\boxed{\tau = \gamma_w \cdot R \cdot S}^* \longrightarrow \text{Avg. Shear Stress.}$$

Where,

$$R = \frac{A}{P} = \text{Hydraulic Mean depth}$$



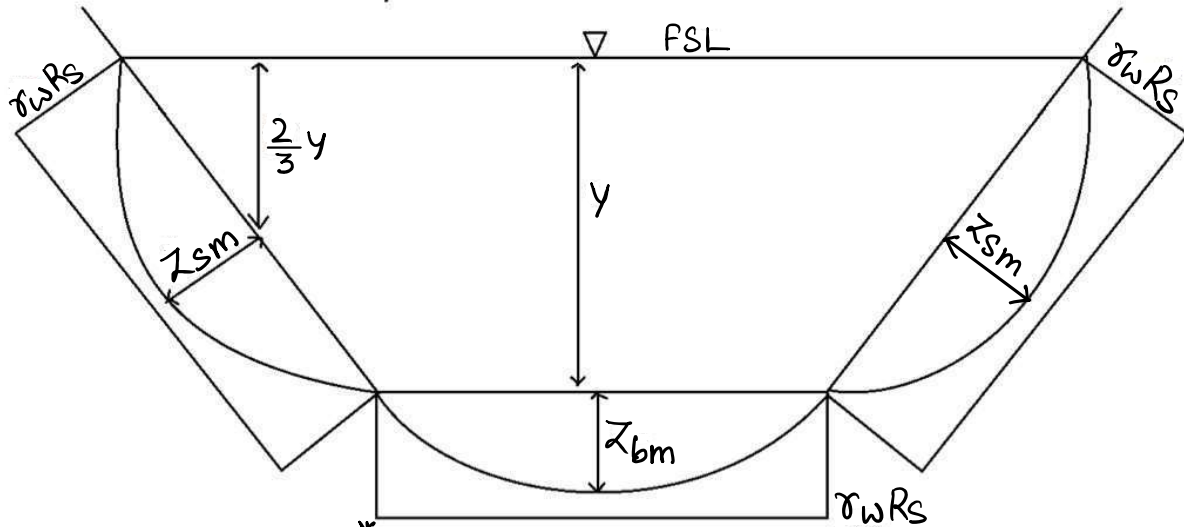
Note: ① The Above expression represents average Shear/friction/tractive Stress.

② The Maximum Stress induced by any flow occurs at the pt. of greatest depth or at the Centre of any Channel. (With horizontal bed).

$$\tau_{\max} = \gamma_w \cdot y \cdot S$$

Where,
 y = Water depth in Channel.

(iii) Actual Shear Stress distribution in the Channel is observed as follows -



$$\tau_{s\max} = 0.75 \cdot \gamma_w \cdot y_s^*$$

$$\tau_{b\max} = 0.97 \cdot \gamma_w \cdot y_s^*$$

To Prevent Scouring of Canal bed Soil Particle -
 Shear Stress acting on Canal bed Soil Particle $\leq$ Shear strength of Canal bed Soil Particle.

$$0.97 \cdot \gamma_w \cdot R \cdot S \leq 0.056 \cdot \gamma_w \cdot d \cdot (G-1)$$

$$0.97 \cdot R \cdot S \leq 0.056 \cdot d \cdot (G-1)$$

$$0.97 \cdot R \cdot S \leq 0.056 \cdot d \cdot (2.65-1)$$

$$11RS \leq d^{**}$$

To Prevent Scouring of Canal Side Slope Particle -
 Shear Stress on Canal Side Slope Particle $\leq$ Shear strength of Canal Side Slope Particle.

$$0.75 \cdot \gamma_w \cdot R \cdot S \leq \left[0.056 \cdot \gamma_w \cdot d \cdot (G-1) \right] / \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \phi}}$$

Where,
 ϕ = Angle of internal friction of Soil Particle
 θ = Side Slope.

Assume $G = 2.65$

$$0.75 \cdot R \cdot S \leq \frac{d}{11} \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \phi}}^{**}$$



Q. Water flows at a depth of 0.6m in a wide Stream having a bed slope of 1 in 2500, the Mean diameter of the Sand bed is 1mm. Determine Whether the Soil grains are Stationary or Moving and Comment as Whether the Stream bed Scouring or Non-Scouring.

Ans:→

$$\begin{aligned} \text{For No Scouring of Channel Bed Slope, } d &> 11.RS \\ \Rightarrow d &> 11 \times 0.6 \times 10^3 \times \frac{1}{2500} \\ \Rightarrow d &> 2.64 \text{ mm} \end{aligned}$$

$$\text{For Wide Channel, } R = y \Rightarrow d = 1 \text{ mm} < 2.64 \text{ mm.}$$

The Soil grains will be not Stationary and the Scouring and Sediment transport will take place.

Q. An Unlined Irrigation Canal has its bed and Side Composed of Cohesionless Material having mean diameter of 6 mm and Angle of internal friction is 40° , the Width of the Channel Can be taken as 5m and Side Slope of 1.5H:1V. Find the discharge that Can be admitted into the Canal without Sediment transport. Assume Channel Bottom Slope of $\frac{1}{5000}$ and $n = 0.025$.

Ans:→ $d = 6 \text{ mm}$, $\phi = 40^\circ$, $b = 5 \text{ m}$

$$0.75.RS \leq \frac{d}{11} \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \phi}}$$

$$0.75 \times R \times \frac{1}{5000} \leq \frac{6 \times 10^{-3}}{11} \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \phi}}$$

$$\text{For } \tan \theta = \frac{1}{1.5} = \frac{2}{3}$$

$$\theta = 33.69^\circ$$

$$\Rightarrow \sin \theta = 0.554$$

$$0.75.R \times \frac{1}{5000} \leq \frac{6 \times 10^{-3}}{11} \sqrt{1 - \frac{\sin^2(33.69)}{\sin^2(40^\circ)}}$$

$$R \leq 1.837 \text{ m.}$$

For No Sediment Transport on Channel bed,

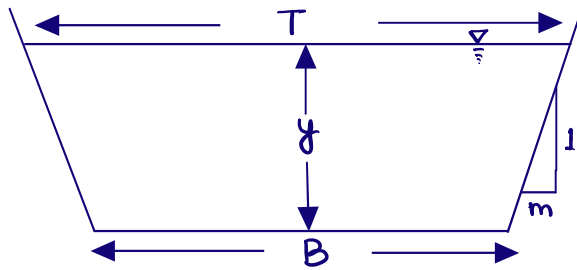
$$d \geq 11.RS$$

$$\Rightarrow R \leq \frac{d}{11.S}$$

$$\Rightarrow R \leq \frac{6 \times 5000}{11 \times 1 \times 1000} \leq 2.73 \text{ m.}$$

$$\begin{aligned} \text{So, adopt } R &= 1.837 \\ V &= \frac{1}{0.025} \times R^{2/3} \times S^{1/2} \end{aligned}$$





$$T = B + 2my$$

$$A = \frac{1}{2}(B + T)y$$

$$= (B + my)y$$

$$P = B + 2y\sqrt{1 + m^2}$$

$$\Rightarrow R = \frac{A}{P} \Rightarrow 1.837 = \frac{[5 + 1.5y]y}{5 + 2y\sqrt{1 + 1.5^2}}$$

$$\Rightarrow y = 3.07 \text{ m.}$$

$$\Rightarrow A = (5 + 1.5y)y$$

$$= (5 + 1.5 \times 3.07) \times 3.07$$

$$= 29.53 \text{ m}^2.$$

$$\Rightarrow Q = A \cdot V = A \cdot \frac{1}{n} \cdot R^{2/3} \cdot S^{1/2}$$

$$\Rightarrow Q = 29.48 \times \frac{1}{0.025} \times (1.837)^{2/3} \times \left(\frac{1}{5000}\right)^{1/2}$$

$$\Rightarrow \text{Discharge, } Q = 25.05 \text{ Cumec.}$$

Note: For Rigid Section i.e. Lined Canals and Non-Alluvial Canal, Manning's Constant Can also be Computed empirically Using 'STRICKLERS EQUATION'.

$$n = \frac{d^{1/6}}{24}$$

$d(\text{m})$: Average Size of Particle.

Q. An Irrigation Channel is to be Constructed in Coarse alluvium gravel with D_{75} Size as 5 cm. The Channel has to Carry 3 Cumec of discharge and the longitudinal Slope is 0.01. The banks of the Channel will be Protected by grass against Scouring. Find the Maximum width of the Channel.

Ans: $\Rightarrow$

$$n = \frac{\left(\frac{5}{100}\right)^{1/6}}{24} = 0.0252$$

$$Q = 3 \text{ Cumec, } S = 0.01$$



For no Scouring On Channel bed,

$$d \geq 11.8S$$

$$\Rightarrow R \leq \frac{d}{11.8}$$

$$\Rightarrow R \leq \frac{5 \times 10^{-2}}{11 \times 0.01}$$

$$\Rightarrow R \leq 0.454 \text{ m.}$$



Assume $R = y$,

$$3 = \frac{B}{0.025} \times R^{5/3} \times S^{1/2}$$

$$3 = \frac{B}{0.025} \times (0.454)^{5/3} \times (0.01)^{1/2}$$

$$B = 2.80 \text{ m.}$$

PREVIOUS YEAR QUESTIONS

ESE

Q. Which of the following are the principal factors influencing the choice of a Particular method of Lining?

1. Availability and cost of the material at the site or within easy reach.
2. Velocity of flow in the channel.
3. Cost of Maintenance.

- (a) 1 and 2 Only
- (b) 1 and 3 Only
- (c) 1, 2 and 3
- (d) 2 and 3 Only.

[2020]

Ans:

(c)

— Factors Influencing Choice of a Method of Lining:

1. Availability and cost of Material.
2. Velocity of flow.
3. Cost of Maintenance
4. Climate Condition.
5. Size of Canal
6. Position of Water table.

Q. A Channel designed by Lacey's theory has a mean Velocity of 1.1 m/s. The Silt factor is 1.1. Then hydraulic mean Radius will be

- (a) 1.13 m
- (b) 2.75 m
- (c) 3.13 m
- (d) 4.27 m.

[2016]

Ans:

(b) 2.75 m

$$\begin{aligned} \text{— } R &= \frac{5}{2} \frac{V^2}{f} \\ &= \frac{5}{2} \times \frac{1.1^2}{1.1} \\ R &= 2.5 \times 1.1 \\ &= 2.75 \text{ m.} \end{aligned}$$

Q. Consider the following Statement Regarding design of Channel by Lacey and Kennedy:

1. The theoretical Concept of Silt transportation is the same in both the theories.
2. Lacey improved upon Kennedy's formula.
3. There are no defects in either the theories of Lacey or of Kennedy.

Which of the above Statement are correct?

- (a) 1 and 2 Only
- (b) 1 and 3 Only

- © 2 and 3 Only.
 (d) 1, 2 and 3.

[2017]

Ans:

(a)

— Lacey and Kennedy both approved that, with the flowing streams silt also gets transported from one place to other place.

GATE

Q. The depth of flow in an 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.

[2009]

Ans:

(b)

— Critical Velocity Ratio,

$$m = 1.1$$

$$\text{Depth } y = 1.5 \text{ m}$$

As per Kennedy's equation,

$$\begin{aligned} V &= 0.55 \cdot m \cdot y^{0.64} \\ &= 0.55 \times 1.1 \times (1.5)^{0.64} \\ &= 0.784 \text{ m/s.} \end{aligned}$$

Q. A Stable Channel is to be designed for a discharge of $Q \text{ m}^3/\text{s}$ with silt factor f as per Lacey's method. The mean flow velocity (m/s) in the channel is obtained by

- (a) $\left[\frac{Q f^2}{140} \right]^{1/6}$
 (b) $\left[\frac{Q \cdot f^2}{140} \right]^{1/6}$
 (c) $\left[\frac{Q^2 \cdot f^2}{140} \right]^{1/6}$
 (d) $0.48 \cdot \left[\frac{Q}{f} \right]^{1/3}$

[2008]

Ans:

(a) $V = \left[\frac{Q \cdot f^2}{140} \right]^{1/6}$

Q. On Which of the Canal System, R.G. Kennedy, executive engineer in the Punjab Irrigation Department made his Observation 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.

[2005]

Ans:

- (d) Upper Bari Doab Canals.

SSC-JE

Q. Calculate the Velocity of flow through a Channel if the mean diameter of the Soil particles is 0.5 mm and discharge is 60 Cumec.

- (a) 0.75
- (b) 0.86
- (c) 0.93
- (d) 0.98

[2017]

Ans:

- (c) 0.93

— $d_{mm} = 0.5 \text{ mm}$
 $Q = 60 \text{ m}^3/\text{sec}$

From Lacey's theory,

Silt factor (f) = $1.76 \sqrt{d_{mm}}$

and,

Velocity (V) = $\left[\frac{Q \cdot f^2}{140} \right]^{1/6} \text{ m/sec.}$

$$f = 1.76 \sqrt{0.5} \\ = 1.2445$$

and $V = \left[\frac{60 \times (1.2445)^2}{140} \right]^{1/6}$
 $V = 0.934 \text{ m/sec.}$

Q. Calculate the Critical velocity (m/sec) of a Channel Using Kennedy's theory, the depth of flow is 3m.

- (a) 0.84
- (b) 1.11
- (c) 2.7
- (d) 6

[2017]

Ans:

- (b) 1.11

— Depth of flow (y) = 3m

Critical Velocity by Kennedy's theory is,

$$V_0 = 0.55 m y^{0.64}$$
$$\Rightarrow V_0 = 0.55 \times 1 \times (3)^{0.64}$$
$$\Rightarrow V_0 = 1.11 \text{ m/s.}$$

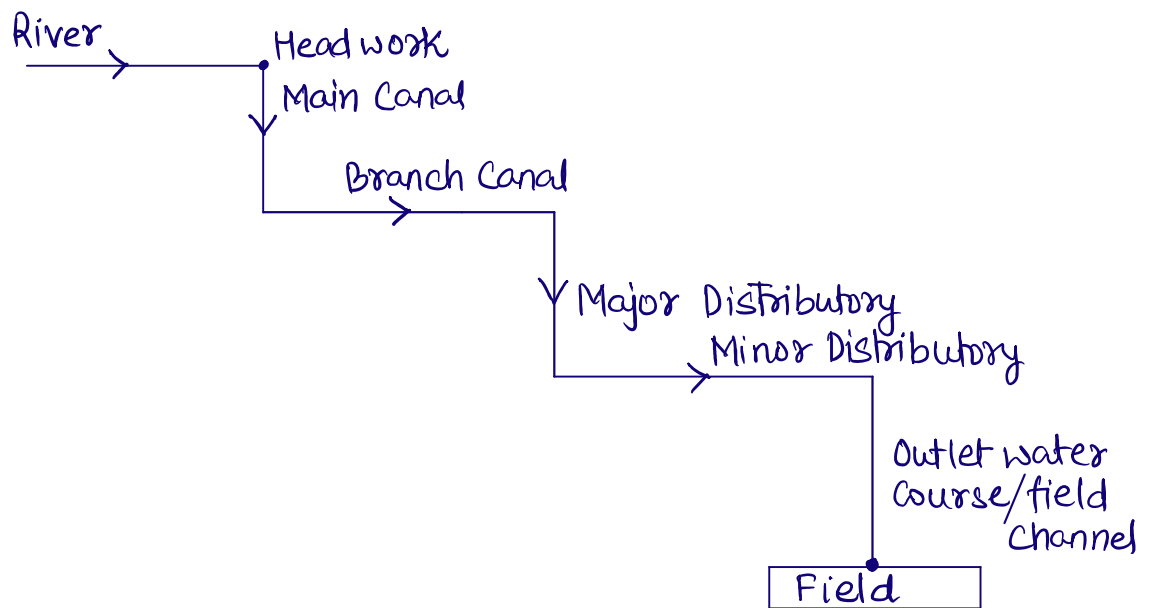
Q. Pick up the Correct Sequence of the part of a Canal System from the following -

- ① Head Work - distributory - Branch Canal - Minor.
- ② Head Work - Main Canal - Branch Canal - distributory - Minor.
- ③ Head Works - Main Canal - Branch Canal - Minor - distributory.
- ④ Head Works - Branch Canal - Main Canal distributory - Minor.

[2016]

Ans :

- ② Head Works - Main Canal - Branch Canal - Distributory - Minor.

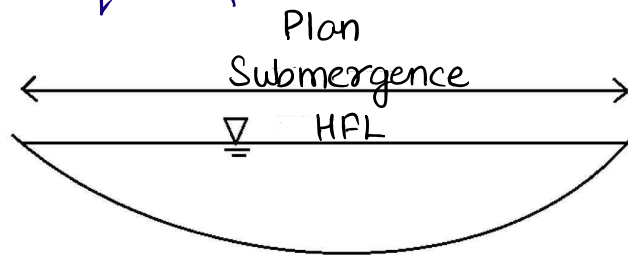
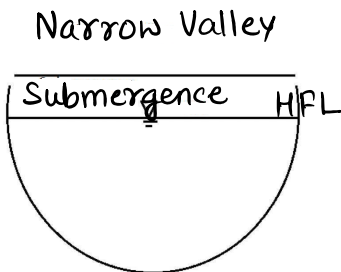


ANALYSIS OF GRAVITY DAM

- Dam is a Physical Obstruction or barrier Constructed across the Width of river, So as to Store Water in its Upstream.
- The Pool of Water is Stored On the upstream side of the dam is Called as 'RESERVOIR'.
- If the dam Resist all the External forces by its Self weight Only, is termed as Gravity dam.

•• FACTORS AFFECTING SELECTION OF DAM SITE

- ① TOPOGRAPHY OF THE REGION: Narrow Valleys having low Submergence area, Should be Preferred, in Comparison to plain Region and also due to this less Clearance will be required, hence time required for Construction would be Reduced.



- ② FOUNDATION: Foundations Should be Impermeable due to this Seepage will be less, hence Uplift will also be less.

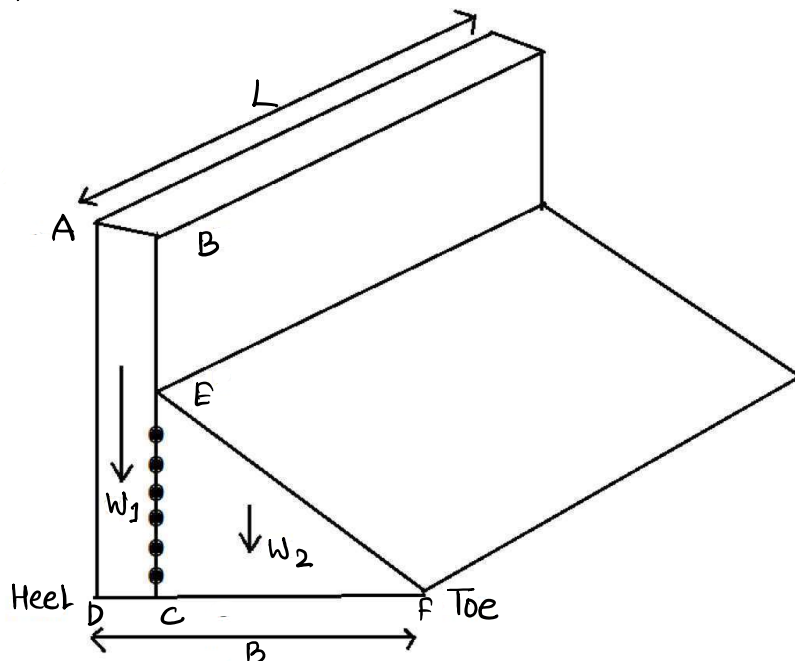
- ③ River should be Perennial in Nature.

- ④ River should be free from Seismic Activity.

- ⑤ Construction material and Labour Should be locally Available.

•• FORCES ACTING ON DAM

① SELF WEIGHT



$$W = W_1 + W_2$$

$$= \gamma_c \cdot A_1 \cdot L + \gamma_c \cdot A_2 \cdot L$$



Generally, We Assume $L = 1\text{m}$.

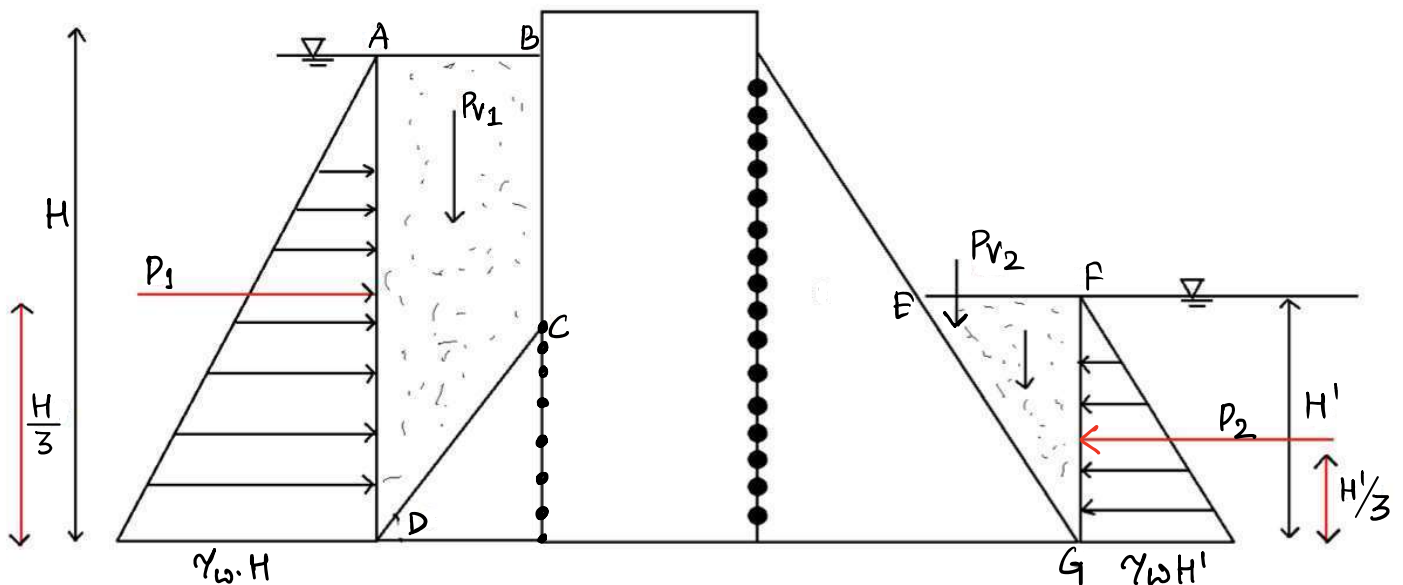
— Total weight of dam acts in vertically downward direction and Pass through its C.G.

② WATER PRESSURE FORCE

— Water Pressure is the most External force Acting On the dam.

— The horizontal Water Pressure Exerted On the Dam by the weight of the water Stored on upstream and downstream side of the Dam Can be Computed by Hydrostatic Pressure distribution.

— When the face of the dam is inclined, Vertical Component due to the Weight of Water and inclined portion is also to be Considered.



$$P_1 = \frac{1}{2} \cdot \gamma_w \cdot H^2 \quad , \quad \bar{Z}_1 = \frac{H}{3}$$

$$P_2 = \frac{1}{2} \cdot \gamma_w \cdot H'^2 \quad , \quad \bar{Z}_2 = \frac{H'}{3}$$

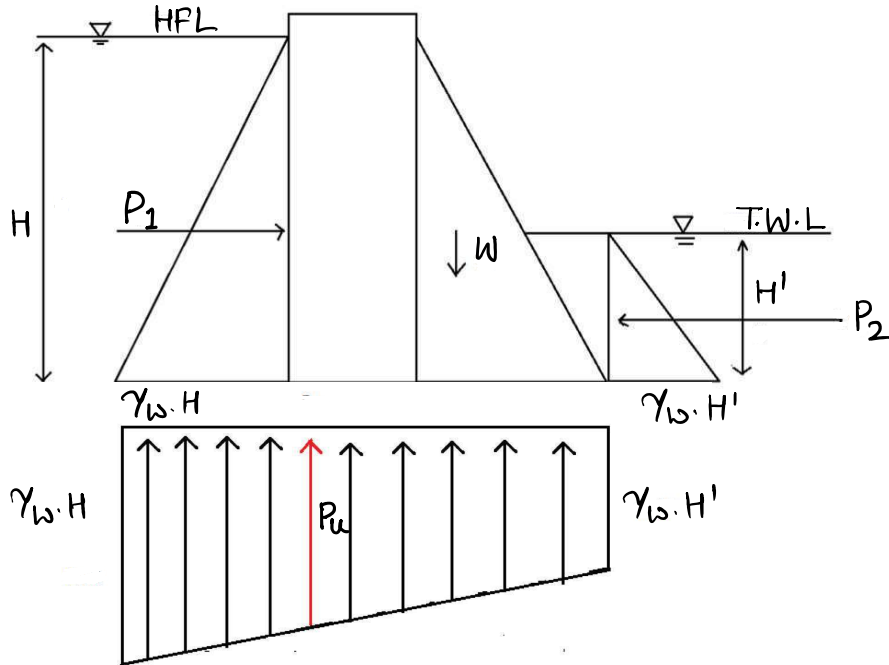
P_{V1} = Weight of Water Column ABCD.

P_{V2} = Weight of Water Column EFG.

③ UPLIFT PRESSURE FORCE

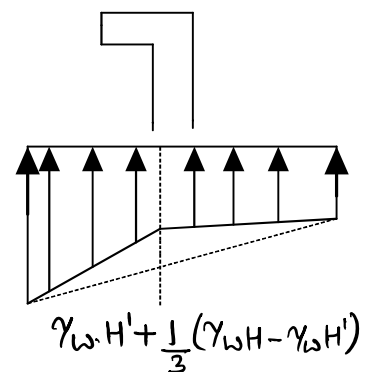
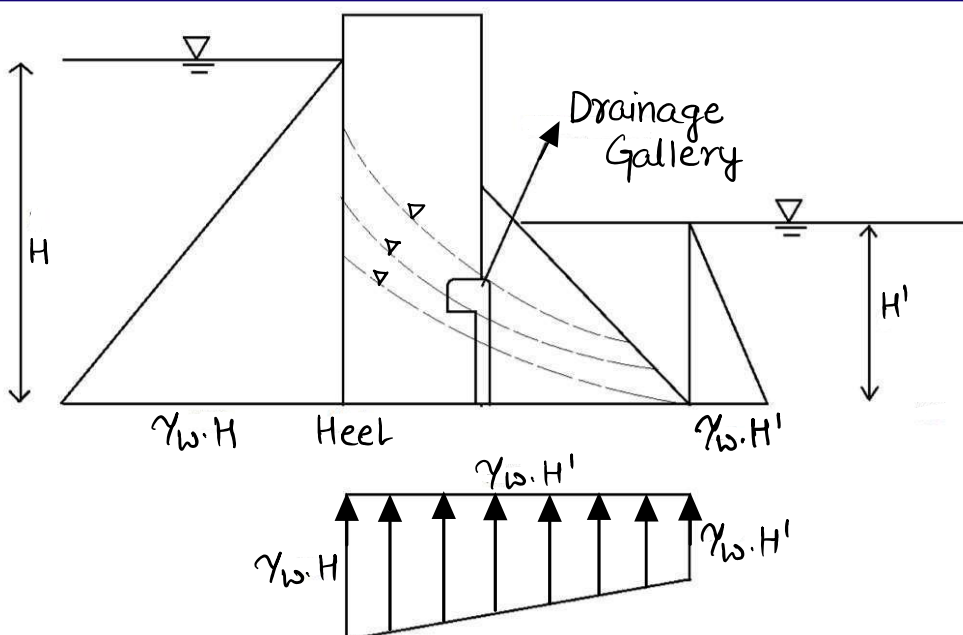
— Water Seeping through the pores, cracks and fissures of the foundation material and Water Seeping through dam body then to the bottom through joints between the body of dam and its foundation, exert the Uplift pressure On the base of the dam.

- It is the Second Major External force Acting On the dam, as it reduces the downward weight of the dam. thereby, reduces its stability.
- it is not accounted Precisely Uphill now, but for Calculation, uplift pressure intensity at heel and toe at dam should be taken equal to their Respective Hydrostatic Pressure and joined by a straight line.

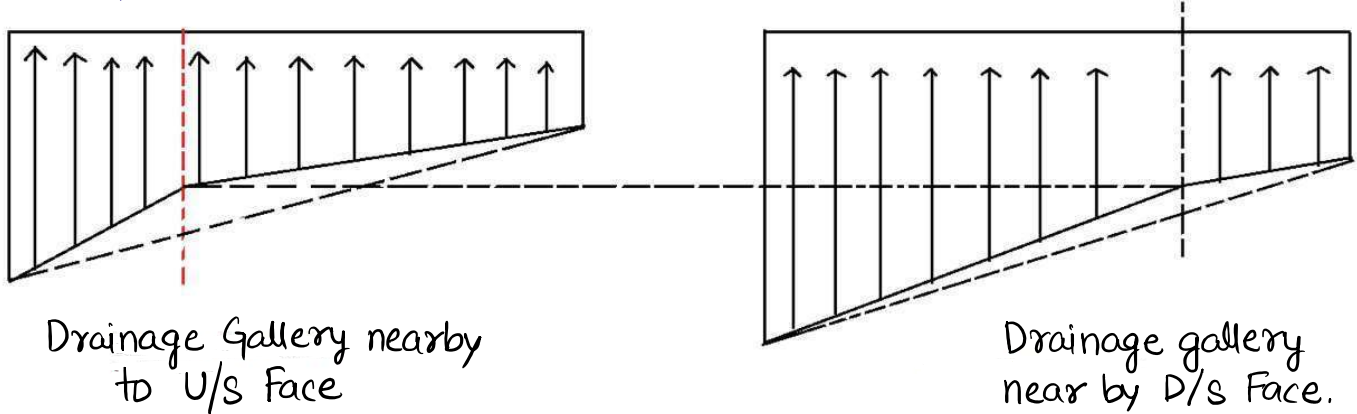


Note : $\rightarrow$ When drainage Gallery is provided to relieve the Uplift, the recommended Uplift at the face of gallery is equal to hydrostatic Pressure at toe ($\gamma_w \cdot H'$ plus $\frac{1}{3}$ rd of the difference of the hydrostatic Pressure at heel and toe of the dam $\frac{1}{3} [\gamma_w \cdot H - \gamma_w \cdot H']$)

$$\Rightarrow \text{Uplift Pressure at face of drainage gallery} = \gamma_w \cdot H' + \frac{1}{3} [\gamma_w \cdot H - \gamma_w \cdot H']^{**}$$



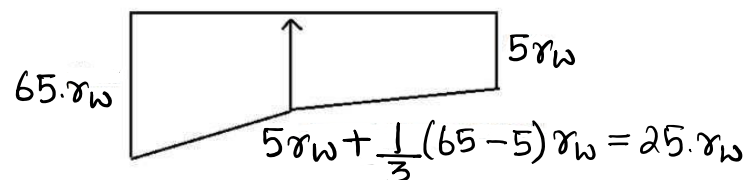
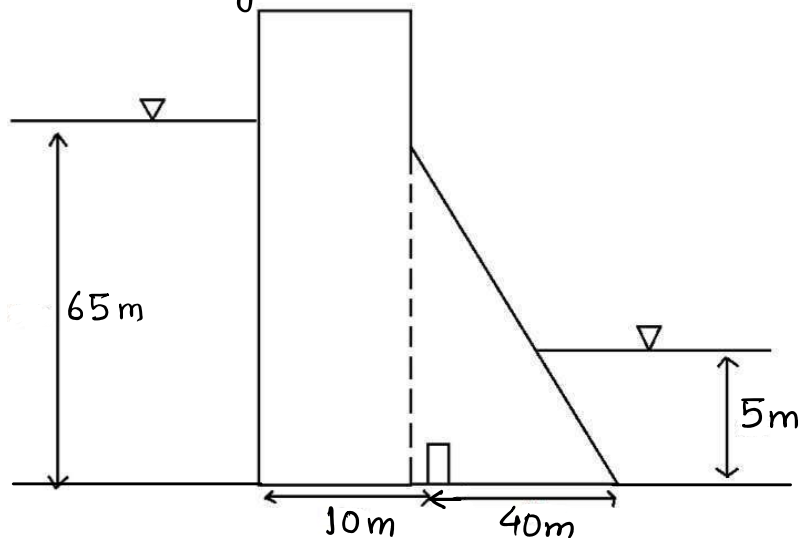
- Basic Dimension of Drainage Gallery is $1.5\text{ m} \times 1.8\text{ m}$
- The Value of Uplift Pressure at the location of drainage gallery is independent of the location of drainage gallery.
- Hence, drainage gallery should be provided as close to Upstream face as possible, so that Uplift Pressure is reduced to Minimum.
- By Providing drainage gallery Uplift Pressure reduces at all level below the upstream Water level.



- It is also considered that Uplift Pressure are not affected by Earthquake forces.

Q. Calculate Uplift Force acting On dam.

Ans:



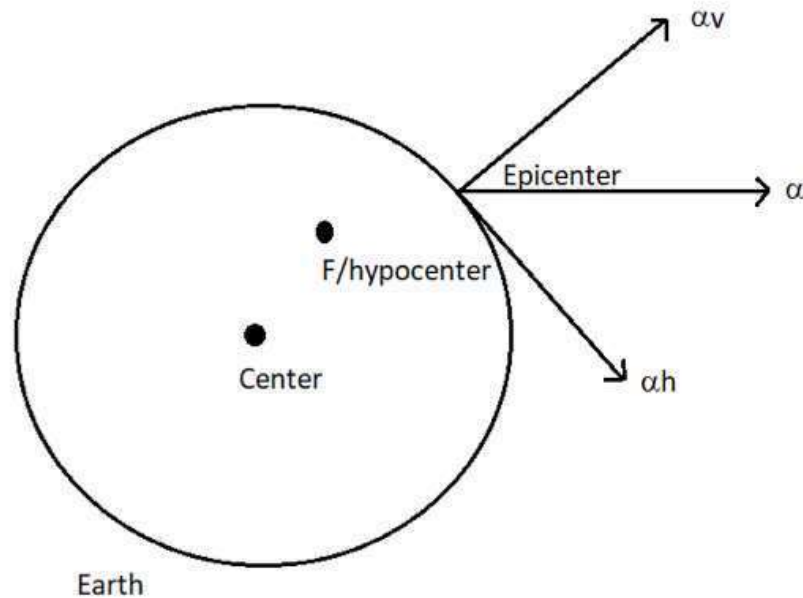
$$\begin{aligned}
 \Rightarrow \text{Uplift force} &= \frac{1}{2} [65 \gamma_w + 5 \gamma_w] \times 10 + \frac{1}{2} [25 \gamma_w + 25 \gamma_w] \times 40 \\
 &= 1050 \gamma_w \\
 &= 10500 \text{ kN/m}
 \end{aligned}$$



Note:- The Uplift Pressure Can be Controlled by Constructing the sheet pile wall Under the upstream face of the dam.

- It Can also be Control by Constructing the drainage channels between the dam and its foundation, or by grouting the foundation.

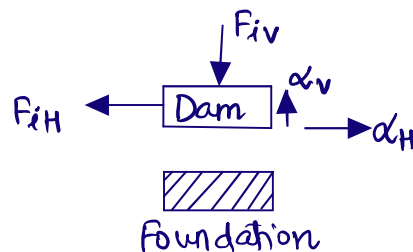
④ EARTHQUAKE FORCE



F = Focus/ Hypocenter
E = Epicenter
 $\alpha_h = K_h \cdot g$

$$\alpha_v = K_v \cdot g$$

$$\alpha_v = 0.75 \cdot \alpha_h$$



- If dams are located in a region which is Susceptible to Earthquakes, then designing is to be done to resist stresses generated by seismic forces.
- An earthquake produces wave which are capable of shaking the Earth upon which dam is resting on, it will be subjected to inertial forces.
- Hence, impact of earthquake is equivalent to imparting an acceleration to the foundation of the dam and it resist in development of inertial force in dam in opposite direction.
- Earthquake wave may move in any direction and for design it is considered in vertical and horizontal direction.
- This Acceleration is generally expressed as % of acceleration due to gravity (K).
- Generally its value lies in the range of (0.05 - 0.3).

① EFFECT OF VERTICAL ACCELERATION

— A Vertical Acceleration can act both in downward or Upward direction resulting in development of inertial force in opposite direction. Hence, total force on dam in Vertical direction would be

$$\begin{aligned} &= W \pm F_i \\ &= W \pm \frac{W}{g} \cdot K_v \cdot g \\ &= W \pm W \cdot K_v \\ &= W [1 \pm K_v] \end{aligned}$$

— From here it can be Concluded Worst or Critical Case would be When acceleration due to earthquake is in downward direction {away} from the body of dam. Thus, reducing effective weight of dam.

② EFFECT OF HORIZONTAL ACCELERATION

— Horizontal Acceleration may Cause the following two forces —

- (a) Hydrodynamic force
- (b) Horizontal Inertial force.



③ HYDRODYNAMIC FORCE

— Horizontal acceleration acting towards the reservoir causes a Momentary increase in the Water Pressure as foundation and dam accelerate towards the reservoir and water resist this Moment due to inertia.

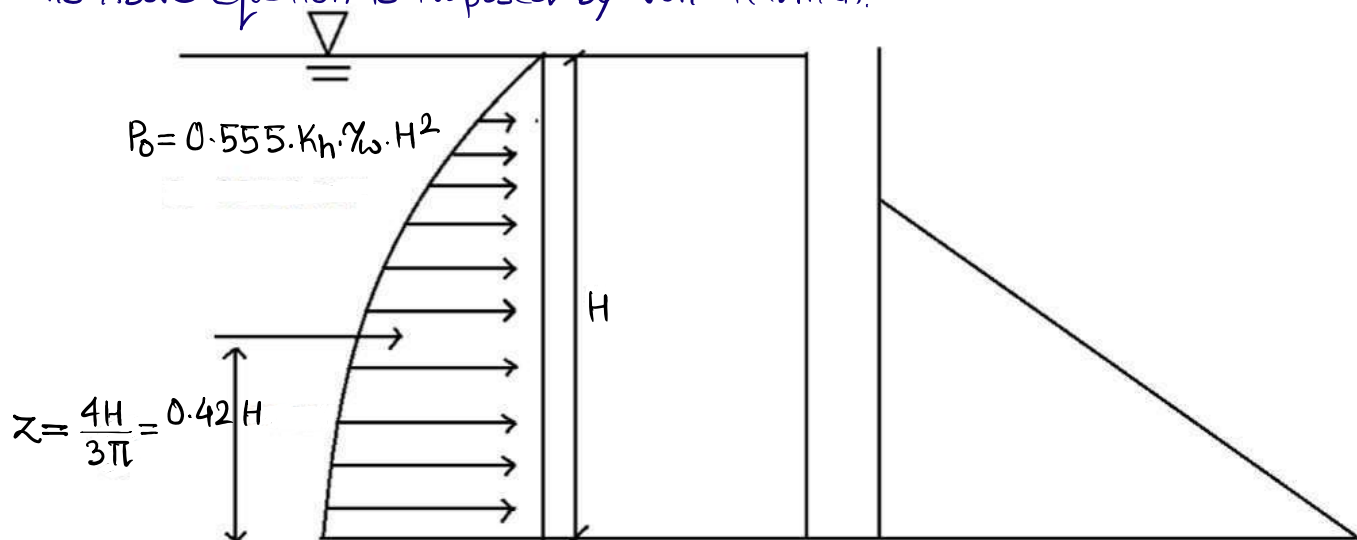
— This extra Pressure Exerted by this process is termed as Hydrodynamic Pressure.

— Magnitude of this Pressure force is given by —

$$P_0 = 0.555 \cdot K_h \cdot \gamma_w \cdot H^2$$

acting at a height of $\frac{4H}{3\pi}$ from the base of the dam.

— The Above equation is Proposed by Von-Karman.



Moment of this force, $M_e = P_e \times \frac{4H}{3\pi}$

Note:- Hydrodynamic Pressure force can also be Computed Using Zanger equation.

$$P_e = 0.726 \cdot P_e \cdot H$$

$$P_e = C_m \cdot K_h \cdot \gamma_w \cdot H$$

$$P_e = 0.726 \cdot C_m \cdot K_h \cdot \gamma_w \cdot H^2$$



Where,

C_m = Max. value of Pressure Coefficient for a given Constant Slope.

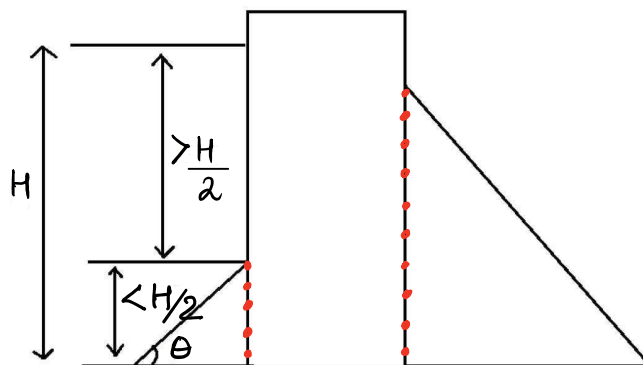
$$C_m = 0.735 \left[\frac{\theta}{90^\circ} \right]$$

θ = Angle in degree which the Upstream face of dam makes with the horizontal.

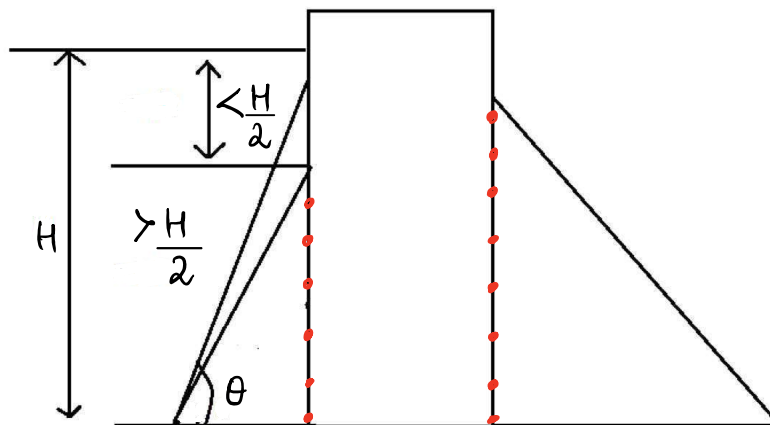
Moment of this force about the base,

$$M_e = 0.299 \times P_e \cdot H^2$$

$$= 0.299 \times \frac{P_e}{0.726} \times H = 0.41 \cdot P_e \cdot H$$



- In Such Case, Upstream face of dam is Considered Vertical, $\theta = 90^\circ$.



- In Such Case, upstream face of dam is Considered to be inclined throughout the height of the dam.

⑥ HORIZONTAL INERTIAL FORCE

- Since an earthquake may impart either upstream or downstream acceleration Critical Case is to decide as per the Water Availability in Reservoir.

① WHEN RESERVOIR IS FULL.

- The Worst Result in this would be observed when horizontal inertia force is additive to hydrostatic pressure force. Hence, acceleration due to earthquake must be towards the Upstream.

② WHEN RESERVOIR IS EMPTY

- In this case, Earthquake force imparts Unstability when it is Acting towards the Reservoir. i.e. acceleration is in DownStream Side, as it would lead to tension Near the toe.

- The Magnitude of this horizontal Inertial force is equal to
$$\begin{aligned} &= M \cdot \alpha_H \\ &= \frac{W}{g} \cdot K_H \cdot g \\ &= W \cdot K_H \end{aligned}$$

- ⑤ SILT PRESSURE: Silt from the Water gets deposited on Upstream and Downstream Side of the Dam and exerts the forces over it than can be computed as follows :-

$$P_{\text{silt}} = \frac{1}{2} \times \gamma' \times h^2 \times K_a$$

Where,

h = height of Silt.

K_a = Active Earth Pressure Coefficient.

$$= \frac{1 - \sin \phi}{1 + \sin \phi}$$

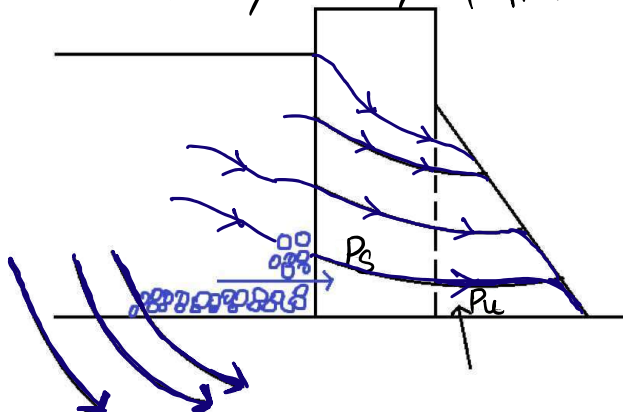
- If the upstream face of dam is inclined. Then, weight of Silt above the inclined Portion is also Considered.

Note :- (i) The Analysis of Silt may be done by Considering it as equivalent fluid Exerting a force with a Unit weight equals to 3.6 kN/m^3 in horizontal direction and a Vertical force with Unit weight of 9.2 kN/m^3 .

- (ii) In Designing of Most the gravity Dam, silt pressure is not Considered as-

(a) In the initial Stage, it is not Present.

(b) In the final Stage, When it is deposited it makes the reservoir Impermeable thereby Reduces Uplift Pressure.



⑥ WAVE PRESSURE

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

$$h_w = 0.032 \sqrt{V_F} + 0.763 - 0.271(F)^{1/4} \quad , F < 32 \text{ Km.}$$

$$h_w = 0.032 \sqrt{V_F} \quad , F > 32 \text{ Km.}$$

Where,

h_w = height of water from top of crest to the bottom trough (m).

V = Wind Velocity (km/hr)

F = Fetch i.e. Straight length of water (km).

- The maximum pressure intensity due to wave action is given by -

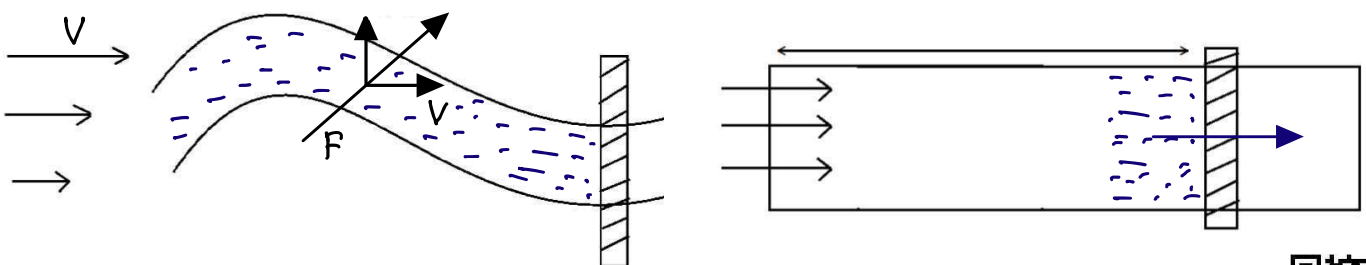
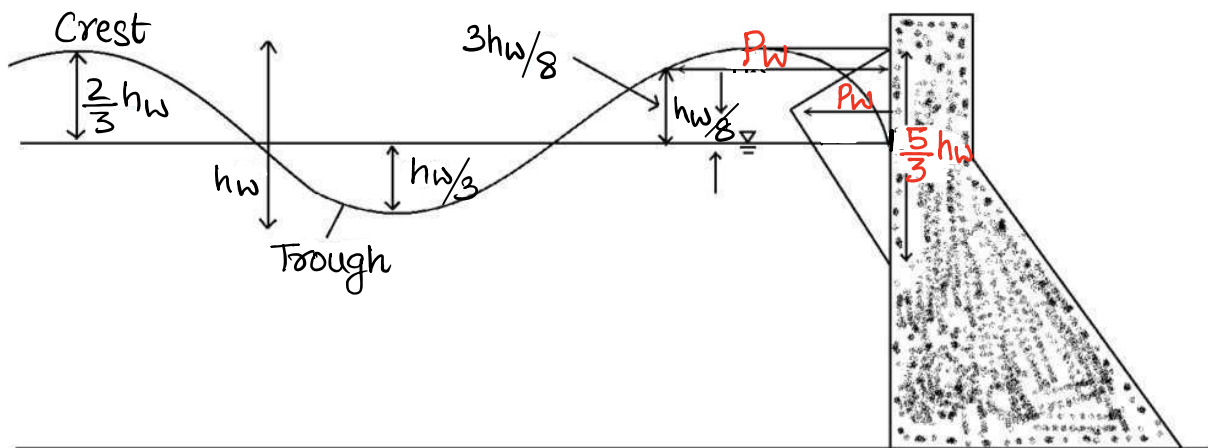
$$P_w = 2.4 \cdot \gamma_w \cdot h_w \text{ acting at } \frac{h_w}{8} \text{ from still water surface.}$$

- The pressure distribution due to wave is considered to be triangular of height $\frac{5 \cdot h_w}{3}$

- The total force due to wave action is -

$$P_w = \frac{1}{2} (2.4 \cdot \gamma_w \cdot h_w \times \frac{5}{3} \cdot h_w)$$

$$P_w = 19.62 \cdot h_w^2 \text{ KN/m, acting at a distance of } \frac{3 \cdot h_w}{8} \text{ above Reservoir Surface.}$$



(vii) ICE PRESSURE: The Ice may be formed Over the Surface of the reservoir and get Expanded, thereby exerts the Pressure Over the dam of Reservoir level. Its Magnitude Varies between 250-1500 kN/m² descending upon temperature.

•• MODES OF FAILURE OF DAM

— A Gravity Dam may Fail in the following ways —

(a) BY OVERTURNING { ROTATION } ABOUT THE TOE.

— If the Resultant of all the forces acting On a Dam at any of the Section Passes outside the toe, the dam shall rotate and Overturn about its toe.

— Such Condition doesnot arise as the dam will Fail much earlier by Compression.

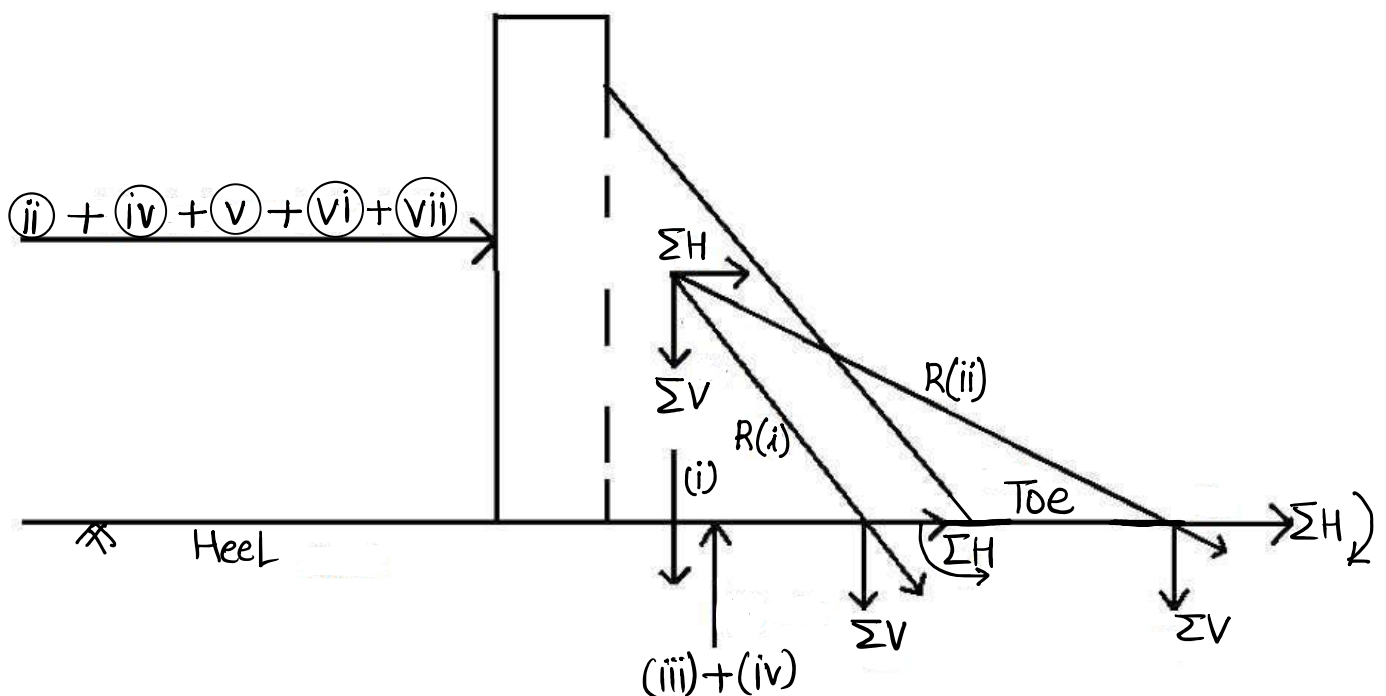
— For No Overturning, total Resisting Moment $\geq$ Overturning Moment.

To Prevent Overturn,

$$\Sigma M_R \geq \Sigma M_O$$

$$\text{FOS/OT} = \frac{\Sigma M_R}{\Sigma M_O} \geq 1$$

$$\{ \text{FOS/OT} = 1.5 \text{ to } 3 \}$$



(b) COMPRESSION FAILURE

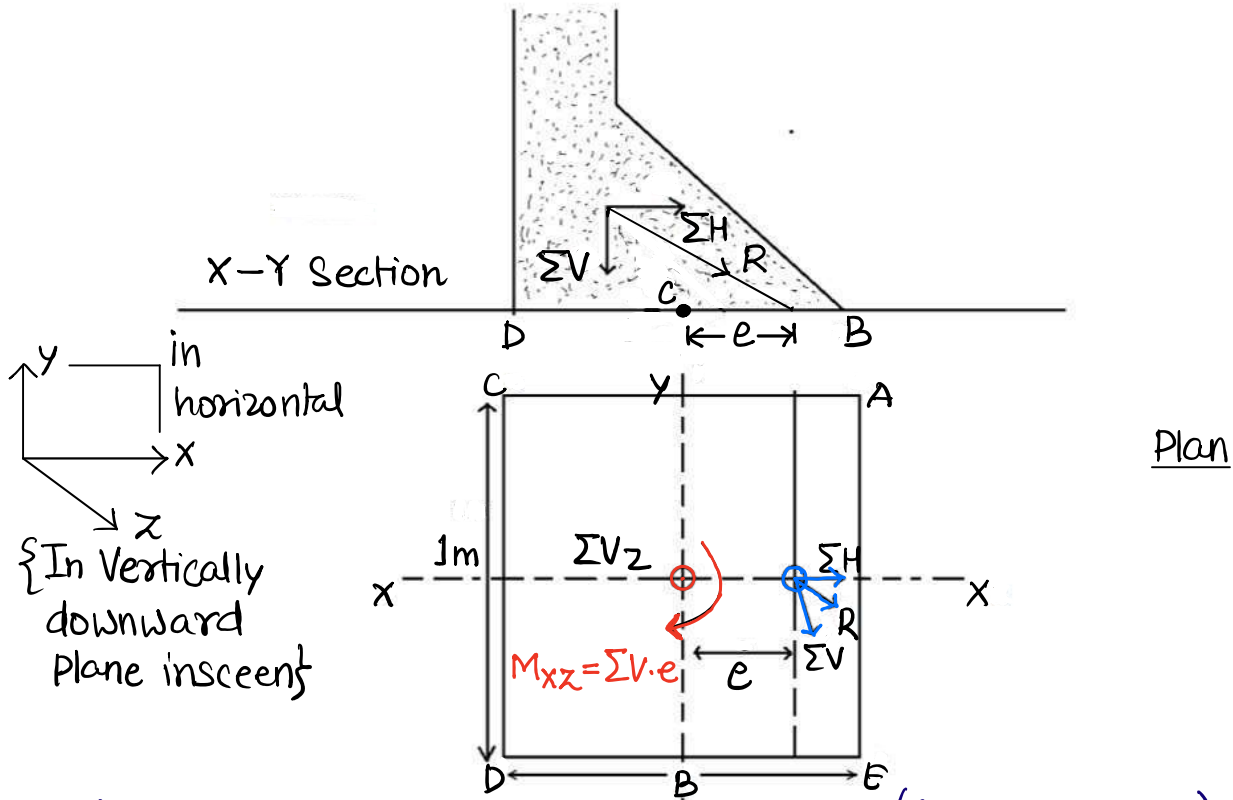
① WHEN RESERVOIR IS FULL.

— A Dam may fail by the failure of its material against Compressive Stresses Produced in Dam.

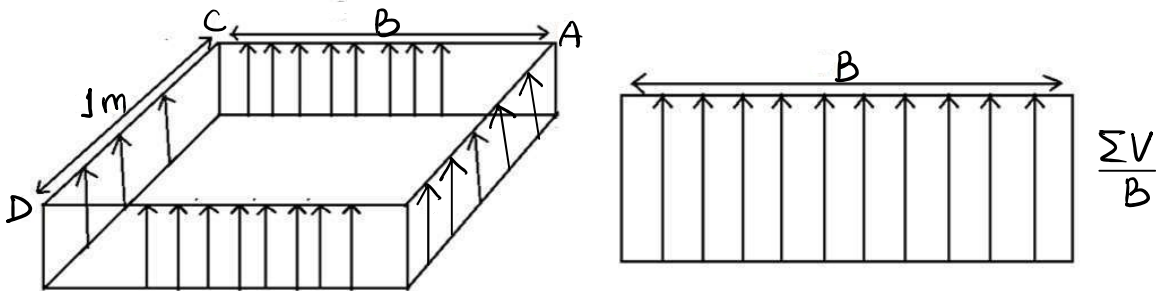
— The total Stress { Vertical } Produced would Consist of Both direct Stress and Bending Stress.

$$P = \text{Direct Stress} + \text{Bending Stress.}$$





— Axial Direct Stress = $\frac{\text{Axial force}}{\text{Cross-Section Area}} = \frac{\Sigma V}{B \times 1}$ (Compressive, +ve)



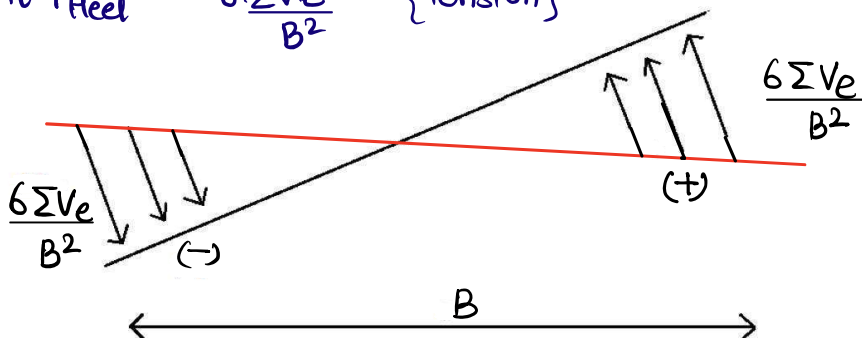
— Bending Stress (F) = $\frac{M}{I} \times y$ $\left\{ \because \frac{M}{I} = \frac{F}{y} \right\}$

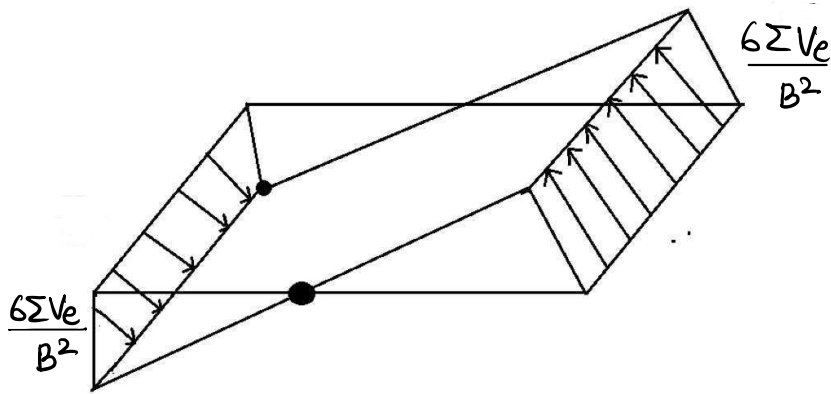
$$= \frac{\Sigma V \cdot e}{\frac{B^3}{6}}$$

$$= \pm \frac{6 \cdot \Sigma V \cdot e}{B^2}$$

At Toe = $\frac{6 \cdot \Sigma V \cdot e}{B^2}$ {Compressive}

At Heel = $-\frac{6 \cdot \Sigma V \cdot e}{B^2}$ {Tension}



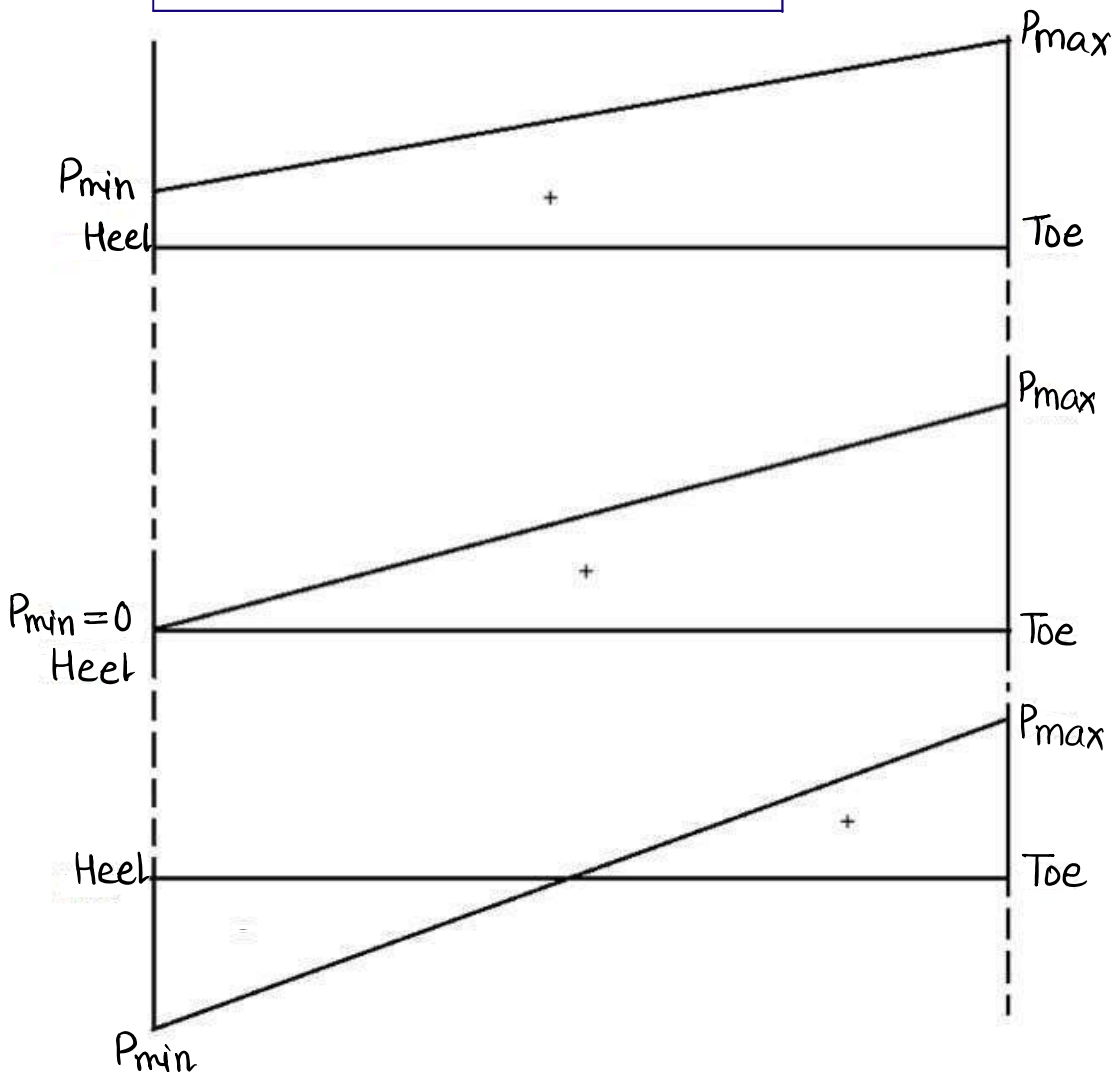


$$P = \frac{\Sigma V}{B} \pm \frac{6 \Sigma V \cdot e}{B^2}$$

$$P = \frac{\Sigma V}{B} \left[1 \pm \frac{6e}{B} \right] \quad ***$$

$$P_{\max} = \text{Toe} = \frac{\Sigma V}{B} \left[1 + \frac{6e}{B} \right] \quad **$$

$$P_{\min} = \text{Heel} = \frac{\Sigma V}{B} \left[1 - \frac{6e}{B} \right]$$

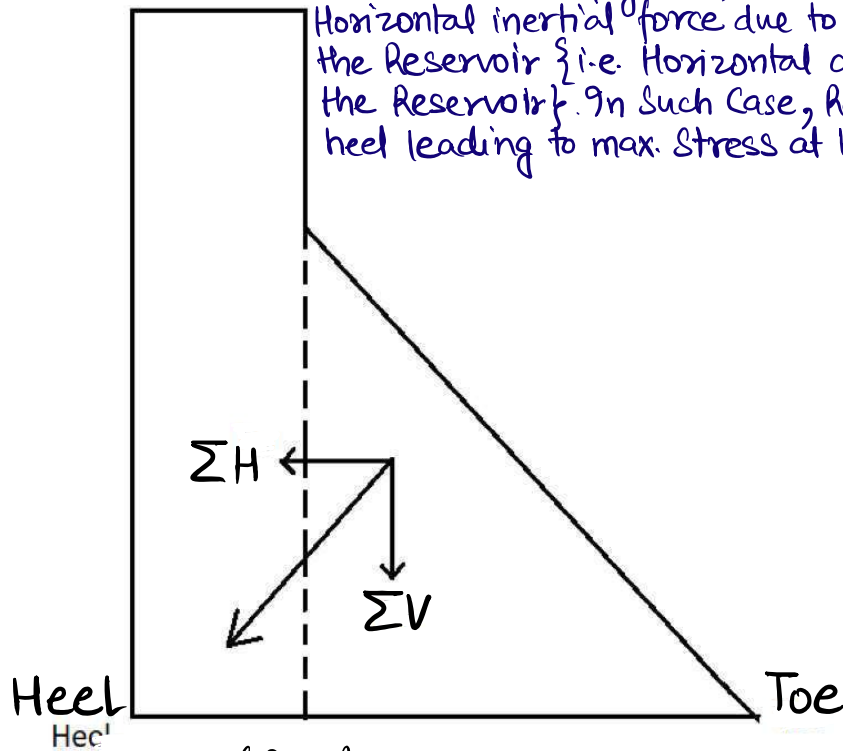


$$e < \frac{B}{6}$$

$$e = \frac{B}{6}$$

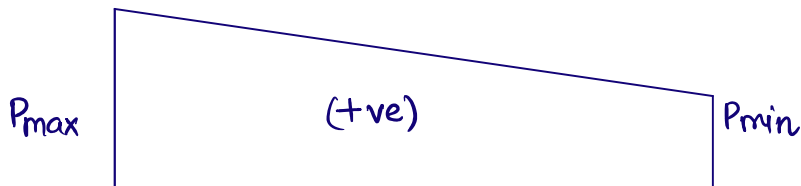
$$e > \frac{B}{6}$$

② RESERVOIR EMPTY CASE: In Reservoir Empty Case, critical Condition develops when Horizontal inertial force due to Earthquake acts towards the Reservoir {i.e. Horizontal acceleration is away from the Reservoir}. In Such Case, Resultant Passes near to the heel leading to max. Stress at heel and min. Stress at toe.



Note: → The max. Stress { P_{max} } will be Produce on the End Which is near to the Resultant.

$$P_{max} = \frac{\Sigma V}{B} \left[1 \pm \frac{6e}{B} \right]$$



— P_{min} Comes Out to be -ve, it means tension Shall be Produced at toe.

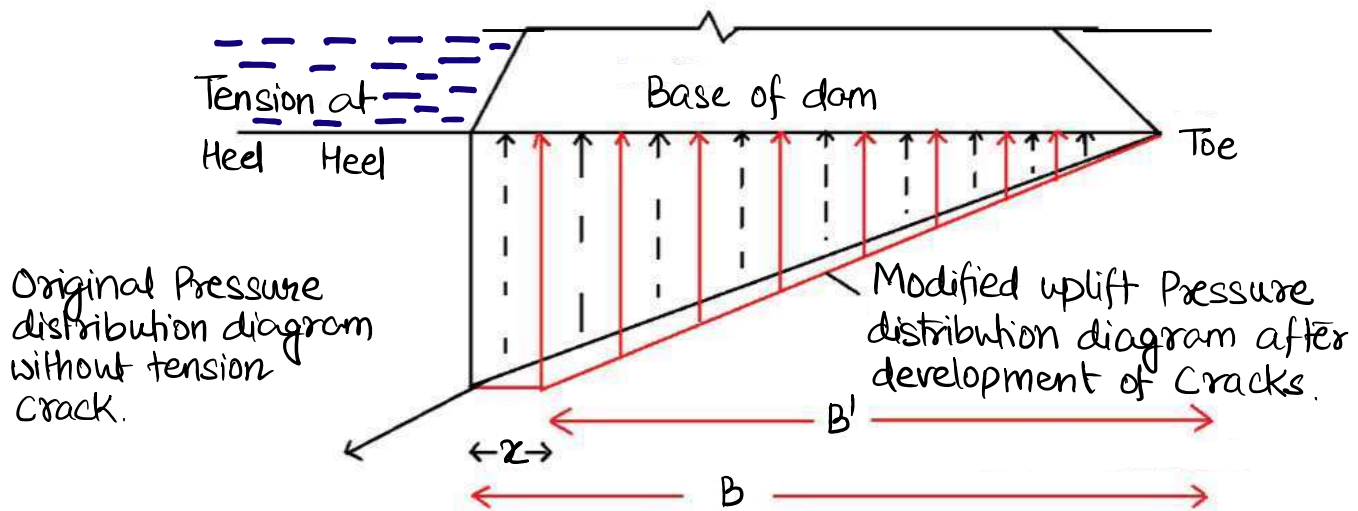
— If P_{max} Exceeds the allowable Compressive Stress {Compressive Strength} of Dam i.e. 3000 KN/m^2 for Concrete, the dam may Crush and fail by Crushing.

③ TENSION FAILURE: Gravity dams are designed in Such a way that No tension is developed anywhere in Section as it is Weak in tension.

— The maximum Permissible tensile Stress for high Concrete gravity dam is 500 KN/m^2 .

— If tensile Stresses are developed, it results in formation of tension Crack due to Which effective width available to transfer load Reduces ($B' < B$) as the area Over Which tension Cracks develops losses the Contact with the bottom foundation.

— Hence, it results in increase in P_{max} moreover the uplift Pressure also increases and modified Uplift Pressure Distribution diagram is as follows —



- Since, the uplift increases, the net effective Downward force decreases, hence the Resultant will shift More towards the toe, which further increases the Compressive Stress P_{max} and Lengthening of tension Cracks.
- This Process Continues upto the Extent the failure of the dam takes place due to Crushing When $P_{max} > f$.

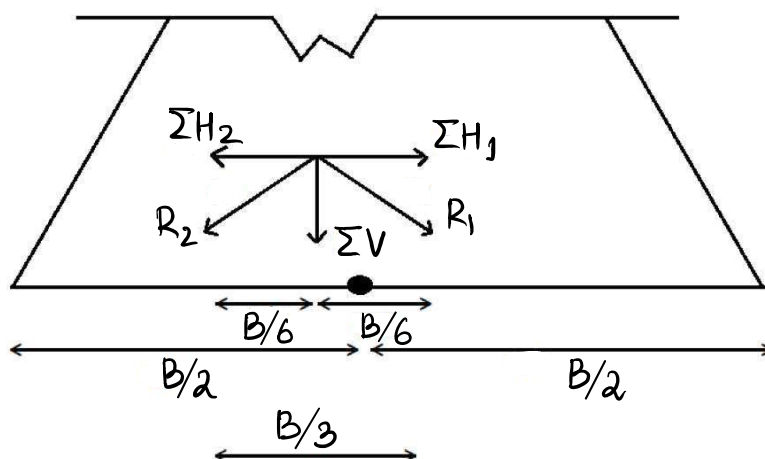
Note :→ Hence tension Crack itself does not fail the dam but it leads to the failure of Dam by Producing Excessive Compressive Stresses. Hence, in Order to Ensure no tension is developed.

$$P_{max/min} = \frac{\sum V}{B} \left[1 \pm \frac{6e}{B} \right]$$

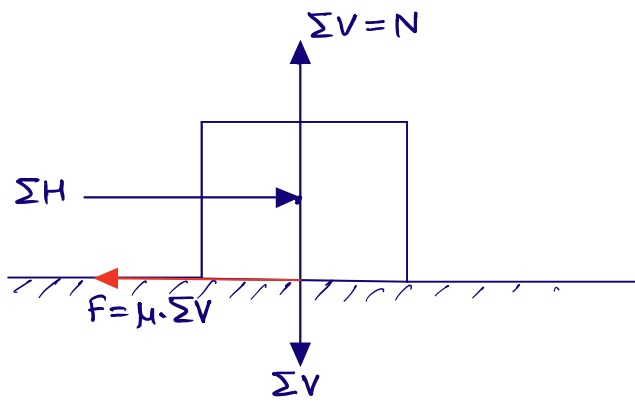
$$\frac{\sum V}{B} \left[1 - \frac{6e}{B} \right] = 0$$

$$\Rightarrow e = \frac{B}{6}$$

Hence, Maximum value of Eccentricity that Can be Permitted On either Side of Centre is equal to $\frac{B}{6}$. thereby, Resultant force must be within middle $\frac{1}{3}$ rd region of Dam.



④ SLIDING



— Sliding or Shear failure will not occur when net Horizontal force above any plane in the Dam (ΣH) does not exceeds friction Resistance (F) Developed at that level.

$$\begin{aligned}\Sigma H &< F \\ \Sigma H &< \mu \cdot N \\ \Sigma H &< \mu \cdot \Sigma V\end{aligned}$$

For Against Sliding,

$$\frac{F}{\Sigma H} = \mu \cdot \frac{\Sigma V}{\Sigma H}$$

— In low dams, Shear Strength of joints is not Considered for Safety against Sliding but in case of large dam, it must be Consider for economic Design.

$$\begin{aligned}\text{For Against Sliding} &= \frac{F + q(B \times 1)}{\Sigma H} \\ &= \frac{\mu \cdot \Sigma V + q \cdot B}{\Sigma H}\end{aligned}$$

Here,

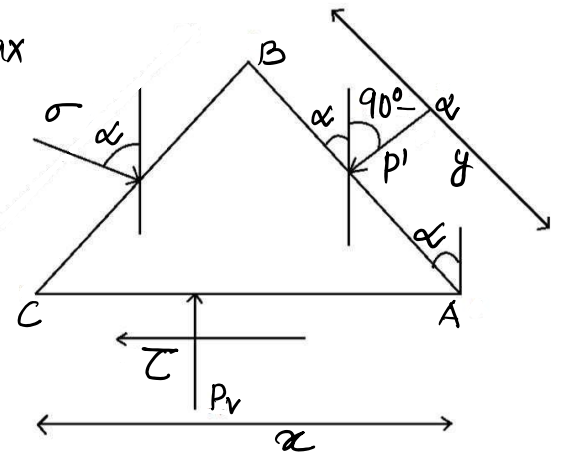
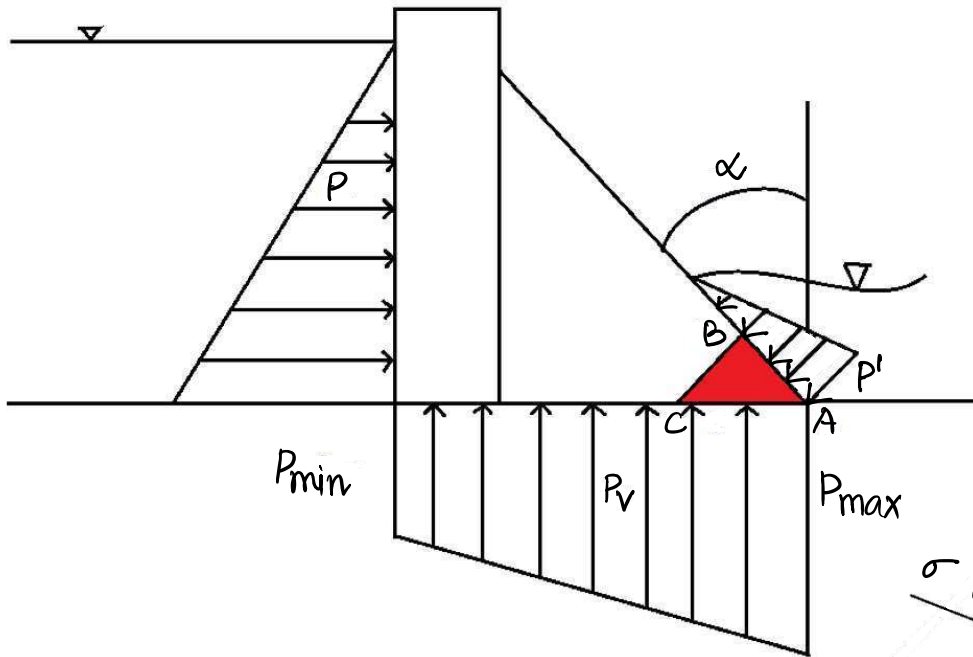
$$\begin{aligned}q &= \text{avg. Shear Strength of Joints (1400 N/mm}^2\text{)} \\ \mu &= 0.65 - 0.75\end{aligned}$$

Note — Principal and Shear Stress

- The Vertical Stress $P_{\max}/P_{\min}$ is not the max or min direct Stress produced anywhere in the dam.
- The max. Normal Stress, will be the major Principal Stress, that acts on major Principal plane.
- Hence, for no failure in Crushing/Compression $\{\sigma_1 < f\}$, major Principal Stress, Should be less than max. Permissible Compressive Strength in Concrete.

① PRINCIPAL STRESS AT TOE: Let Downstream Face of Dam be inclined at an angle ' α ' to the Vertical.





- From Condition of static equation,
 $\sum F_y = 0$ ————— ①
 $\sum F_x = 0$

From equation ①, we get
 $\sigma_2 \times 1 \times \cos \alpha + P'_y \times 1 \times \sin \alpha - P_v \times x \times 1 = 0$

$$\Rightarrow \sigma_2 \cdot \frac{\cos \alpha}{x} = P_v \cdot \frac{x}{x} - \frac{P'_y \cdot \sin \alpha}{x}$$

$$\Rightarrow \sigma \cdot \cos^2 \alpha = P_v - P' \cdot \sin^2 \alpha$$

$$\Rightarrow \sigma = \frac{P_v - P' \cdot \sin^2 \alpha}{\cos^2 \alpha}$$

$$\Rightarrow \boxed{\sigma = P_v \cdot \sec^2 \alpha - P' \cdot \tan^2 \alpha}$$

Major Principal Stress.

- For σ to be max, P' must be Zero i.e. When no tail water is Present.

$$\sigma = P_v \cdot \sec^2 \alpha$$

Since, $\sec \alpha > 1$
 $\Rightarrow \sigma > P_v$

- Hence, for no failure in Compression and crushing, Major Principal Stress (σ) must be less than Compressive Strength of Concrete.

Note : $\Rightarrow$ ① If hydrodynamic Pressure is also considered

$$\sigma_T = P_v \cdot \sec^2 \alpha - (P' - P_e) \cdot \tan^2 \alpha.$$



⑫ If Principal stress is to be considered at heel of dam, it is given by

$$\sigma_H = P_v \cdot \sec^2 \beta - (P + P_e) \cdot \tan^2 \beta$$

Where,

β = it is the angle which Upstream face makes with Vertical.

⑬ Since, Water Pressure P is always more than P' and P_v at toe is more than P_v at heel.

$$\sigma_T > \sigma_H$$

⑭ Since, Water Pressure P is always more than σ . Hence, σ is minor Principal Stress.

$$\sum F_x = 0$$

$$z \times 1 - \sigma(z \times 1) \sin \alpha + P'(y \times 1) \cos \alpha = 0$$

$$\tau = \sigma \cdot \frac{z}{2} \cdot \sin \alpha - \frac{P' y}{2} \cdot \cos \alpha$$

$$\tau = \sigma \cdot \cos \alpha \cdot \sin \alpha - P' \cdot \sin \alpha \cdot \cos \alpha$$

$$\tau = (\sigma - P') \cdot \cos \alpha \cdot \sin \alpha$$

$$\tau = (P_v \cdot \sec^2 \alpha - P' \cdot \tan^2 \alpha - P') \cdot \sin \alpha \cdot \cos \alpha$$

$$\tau = P_v \cdot \sec^2 \alpha \cdot \sin \alpha \cdot \cos \alpha - P'(\tan^2 \alpha + 1) \cdot \sin \alpha \cdot \cos \alpha$$

$$\tau = P_v \cdot \tan \alpha - P' \cdot \tan \alpha$$

$$\tau = (P_v - P') \cdot \tan \alpha \quad **$$

If there is no tail water, $z = P_v \cdot \tan \alpha$

- If Hydrodynamic Pressure is also considered,

$$\tau = [P_v - (P' - P_e)] \cdot \tan \alpha$$

$$\Rightarrow \tau_H = [P_v - (P' - P_e)] \cdot \tan \beta \quad **$$

•• DESIGN STEPS OF GRAVITY DAM

Step ① Consider Unit length of gravity dam and find the magnitude and direction of all the Vertical and Horizontal force.

Step ② Calculate lever arm at all the forces about toe point, and Restoring and Overturning Moment.

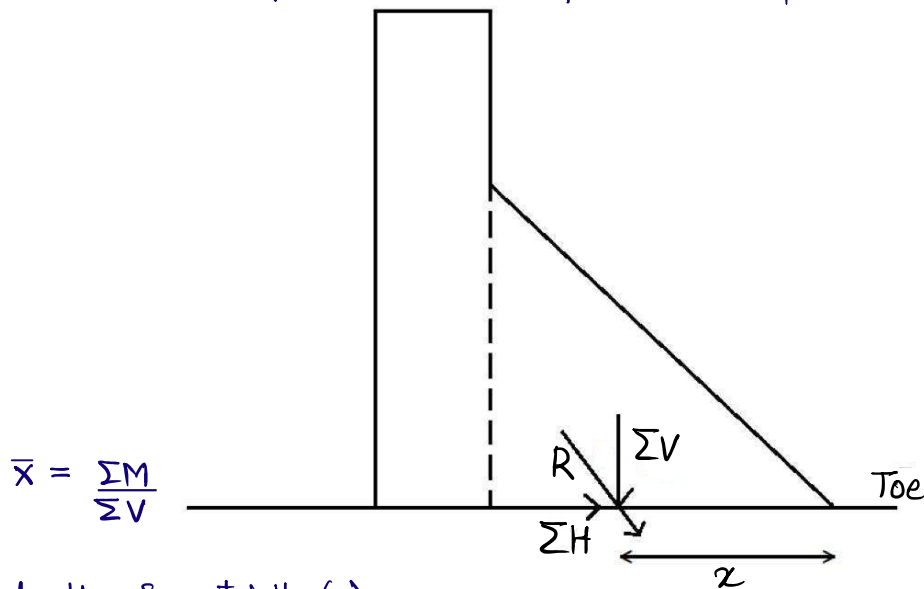
Step ③ Calculate Summation of Restoring and Summation of Overturning Moment.

Step ④ Calculate Resultant Moment

$$\sum M = \sum M_R - \sum M_o$$



Step (V) Calculate the location of Resultant force from the toe point.

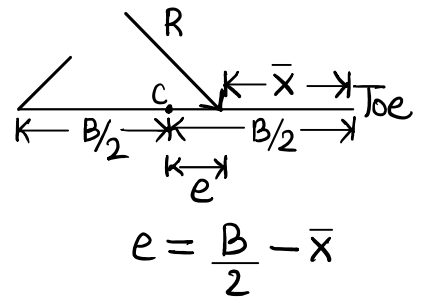


Step (VI) Calculate the eccentricity (e)

$$e = \frac{B}{2} - \bar{x}$$

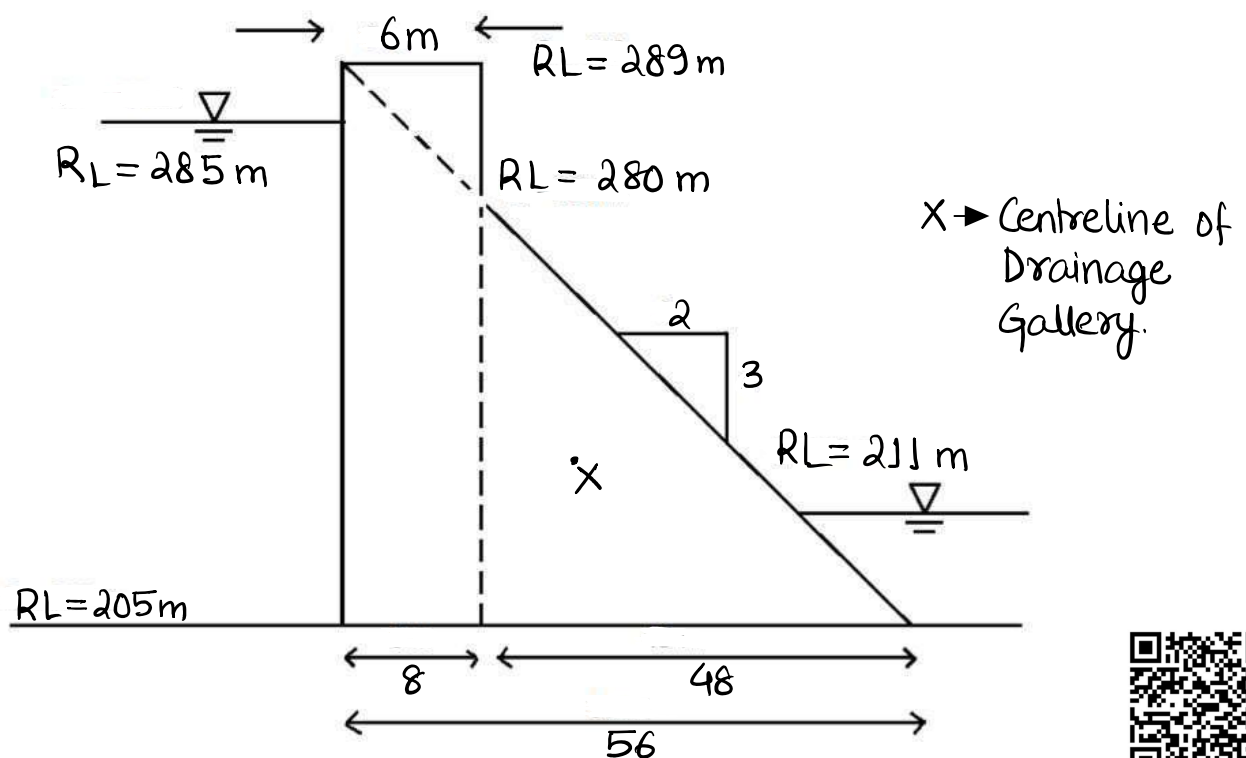
Step (VII) Calculate P_v

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



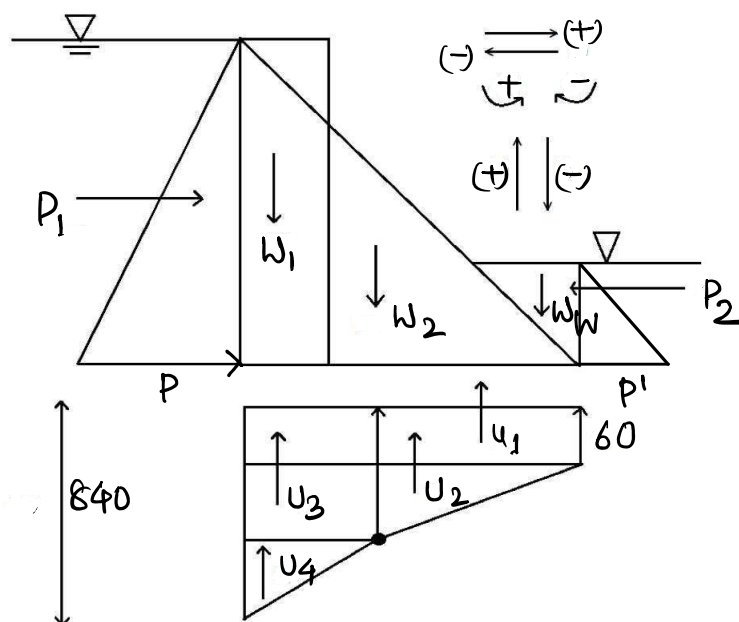
Q. Calculate the following for Gravity Dam as shown -

- ① The maximum Vertical Stress at the heel and toe of the Dam.
- ② The major Principal Stress at the toe of the Dam.
- ③ The Intensity of Shear Stress On a horizontal Plane near the toe, neglect Earthquake effects.



Ans :→





Force	Magnitude	Lever Arm	Moment About toe (kN-m)
W_1	$84 \times 6 \times 24 \times 1$ $= 12096 (+)$	53	641088 (+ve)
W_2	$\frac{1}{2} \times 50 \times 24 \times 75$ $= 45000 (+)$	33.33	1499850 (+ve)
W_3	$\frac{1}{2} \times 6 \times \frac{6}{3} \times 2 \times 1 \times 10$ $= 120 (+)$	1.33	160 (+ve)
	$\Sigma V_1 = 57216$		$\Sigma M = 2141248$
U_1	$56 \times 10 \times 6 \times 1$ $= 3360$	28	94086 (-ve)
U_2	$\frac{1}{2} \times 48 \times \left[\frac{1}{3} \times 10 \times (80-6) \right]$ $= 5920$	32	189440 (-ve)
U_3	$8 \times \frac{1}{3} \times 10 (80-6) \times 1$ $= 1973.3$	52	1026272 (-ve)
U_4	$\frac{1}{2} \times 8 \times 49.33 \times 10$ $= 1973.3$	53.33	105244.27 (-ve)
	$\Sigma V_2 = 43990$		$\Sigma M_2 = 1415042.27$

P_1	$\frac{1}{2} \times 10 \times 80 \times 80$ $= 32000$ (+ve)	26.67	853120 (-ve)
P_2	$\frac{1}{2} \times 6 \times 60$ $= 180$ (-ve)	2	360 (+ve)
	$\Sigma H = 31820$ (+ve)		$\Sigma M_3 = 853080$ (-ve)

$$\begin{aligned}\Sigma M &= \Sigma M_1 - \Sigma M_2 - \Sigma M_3 \\ &= 2141246 - 1415042.27 - 853080 \\ &= 736776.2 \text{ KN-m (+ve)}\end{aligned}$$

$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{736776.2}{43990} = 16.75 \text{ m}$$

$$\begin{aligned}e &= \frac{B}{2} - \bar{x} = \frac{56}{2} - 16.75 \\ &= 11.25 \text{ m}\end{aligned}$$

$$\begin{aligned}P_v &= \frac{\Sigma V}{B} \left[1 \pm \frac{6e}{B} \right] = \frac{43990}{56} \left[1 \pm \frac{6 \times 11.25}{56} \right] \\ &= 785.94 \left[1 \pm 1.205 \right]\end{aligned}$$

$$P_v = 1732 \text{ KN/m}^2 \text{ or } -161.3 \text{ KN/m}^2$$

$$(P_{\max})_{\text{Toe}} = 1732 \text{ KN/m}^2$$

$$(P_{\min})_{\text{Heel}} = -161.3 \text{ KN/m}^2$$

$$\textcircled{\text{II}} \quad \sigma = P_v \cdot \sec^2 \alpha - P' \cdot \tan^2 \alpha$$

$$\tan \alpha = \frac{d}{3} \text{ and } 1 + \tan^2 \alpha = \sec^2 \alpha$$

$$= 1732 \times \frac{13}{9} - 60 \times \frac{4}{9}$$

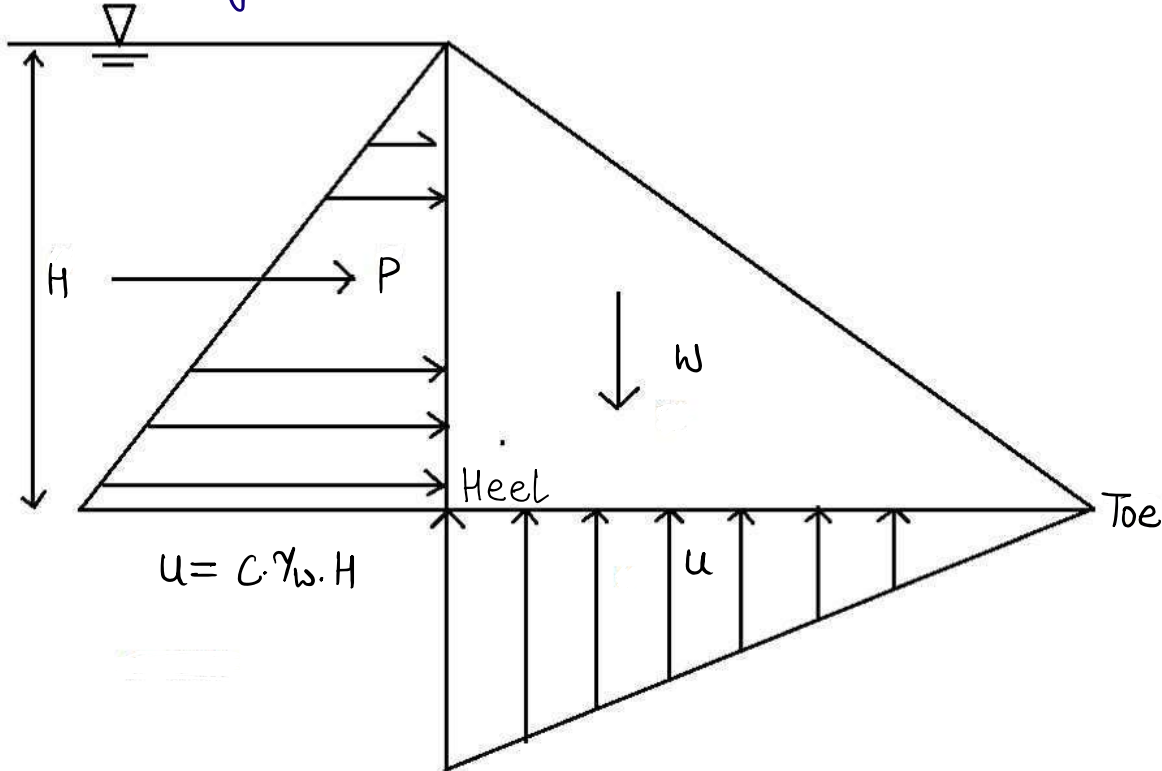
$$\sigma = 2475.11 \text{ KN-m} < 3000 \text{ KN/m}^2$$

$$\begin{aligned}\textcircled{\text{III}} \quad \tau &= (P_v - P') \cdot \tan \alpha \\ &= (1732 - 60) \times \frac{2}{3} \\ &= 1114.67 \text{ KN/m}^2\end{aligned}$$



•• ELEMENTARY PROFILE OF DAM

- The elementary Profile of the Dam Subjected to Only external water Pressure on the Upstream Side is a right Angled triangle with Zero width at the Upstream water level and Base width (B) at the Bottom.
- It is of the Shape which is similar to that of Hydrostatic Pressure Distribution Diagram.



$$P = \frac{1}{2} \cdot \gamma_w \cdot H \cdot H = \frac{1}{2} \cdot \gamma_w \cdot H^2$$

$$W = \frac{1}{2} \times B \times H \times 1 \times \gamma_c$$

$$= \frac{1}{2} \times B \times H \times \gamma_c \times \gamma_w$$

$$U = \frac{1}{2} \times B \times C \cdot \gamma_w \times H$$

Where,

C = Seepage Constant

$C = 1$, When Uplift is Considered

$C = 0$, When Uplift is not Considered.

① TO PREVENT OVERTURNING

$$\Sigma M_R \geq \Sigma M_o$$

$$W \times \frac{2B}{3} \geq \frac{P \cdot H}{3} + u \times \frac{2B}{3}$$

$$\frac{1}{2} \times B \cdot H \cdot \gamma_c \cdot \gamma_w \cdot 2B = \frac{1}{2} \cdot \gamma_w \cdot H^2 \cdot H + \frac{1}{2} \cdot B \cdot C \cdot \gamma_w \cdot H \cdot 2B$$

$$2 \cdot \gamma_c \cdot B^2 = H^2 + 2C \cdot B^2$$



$$B^2(2G_c - 2c) \geq H^2$$

$$B \geq \frac{H}{\sqrt{2(G_c - c)}}$$

For B_{min} , $c = 0$

$$B_{min} = \frac{H}{\sqrt{2 \cdot G_c \cdot c}} \quad **$$

② TO PREVENT DEVELOPMENT OF TENSION

$$e \leq \frac{B}{6}$$

$$\frac{B}{2} - \bar{x} \leq \frac{B}{6}$$

$$\frac{B}{2} - \frac{B}{6} \leq \frac{\sum M}{\sum V}$$

$$\frac{B}{3} \leq \frac{\sum M}{\sum V}$$

$$\frac{B}{3} \leq \frac{\sum M_R - \sum M_o}{\sum V}$$

$$\sum V \cdot \frac{B}{3} \leq \sum M_R - \sum M_o$$

$$(W - U) \frac{B}{3} \leq W \cdot \frac{2B}{3} - \frac{PH}{3} - U \cdot \frac{2B}{3}$$

$$(W - U) B \leq 2B(W - U) - PH$$

$$B(W - U) \geq PH$$

$$B \left(\frac{1}{2} B H G_c \gamma_w - \frac{1}{2} B c \gamma_w H \right) \geq \frac{1}{2} \gamma_w H^2 H$$

$$B(BG_c - Bc) \geq H^2$$

$$B^2(G_c - c) \geq H^2$$

$$\Rightarrow B \geq \frac{H}{\sqrt{G_c - c}} \quad \text{--- It is more Critical than Overturning.}$$

For B_{min} , $c = 0$

$$B_{min} = \frac{H}{\sqrt{G_c}}$$

Assume $G_c = 2.5$

$$(B_{min})_{OT} = 0.44 \cdot H$$

$$(B_{min})_T = 0.632 \cdot H$$

Assume $H = 100 \text{ m}$,

$$(B_{min})_{OT} = 44 \text{ m} \quad \text{and,} \quad (B_{min})_T = 63.2 \text{ m.}$$



③ TO PREVENT SLIDING FAILURE

— To Prevent Sliding Failure,
Restoring force $\geq$ Sliding force

$$\text{Friction Force} \geq \Sigma H$$

$$\mu \cdot N \geq \Sigma H$$

$$\Rightarrow \mu \cdot \Sigma V \geq \Sigma H$$

$$\mu(W - U) \geq P$$

$$H \left[\frac{1}{2} \cdot B H \times 1 \times G_c \cdot \gamma - \frac{1}{2} \cdot B \cdot c \gamma_w \cdot H \times 1 \right] \geq \frac{1}{2} \cdot \gamma_w \cdot H \times H$$

$$H B [G_c - c] \geq H$$

$$\Rightarrow \boxed{B \geq \frac{H}{\mu(G_c - c)}}$$

$$\text{For } B_{\min}, c = 0 \Rightarrow B_{\min} = \frac{H}{\mu \cdot G_c}$$

$$\text{Assume } G_c = 2.5, \mu = 0.7$$

$$B_{\min} = \frac{H}{0.7 \times 2.5} \\ = 0.571 \cdot H$$

$B_{\min}$ from tension Criteria $0.632 \cdot H$ {More Critical}.

④ TO PREVENT COMPRESSION FAILURE

— To Prevent Compression failure
 $\sigma_1 \leq F$

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

$$= \frac{W - U}{B} \left[1 \pm \frac{6 \cdot B}{B} \right]$$

$$= \frac{\frac{1}{2} \cdot B \cdot H \cdot G_c \cdot \gamma_w - \frac{1}{2} B \cdot c \gamma_w \cdot H}{B} (1 \pm 1)$$

$$= \frac{1}{2} \cdot \gamma_w \cdot H (G_c - c) (1 \pm 1)$$

$$P_{\max} = \gamma_w \cdot H \cdot (G_c - c)$$

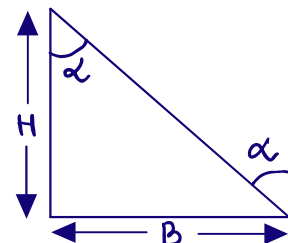
(Toe)

$$P_{\max} = 0$$

(Heel)

$$\sigma_1 = P_v \cdot \sec^2 \alpha - P' \cdot \tan^2 \alpha$$

$$P' = 0, \tan \alpha = \frac{B}{H}$$



$$\begin{aligned}\sigma_1 &= \gamma_w \cdot H (G_c - c) (1 + \tan^2 \alpha) \\ &= \gamma_w \cdot H (G_c - c) \left[1 + \left(\frac{B}{H} \right)^2 \right] \text{---(1)}\end{aligned}$$



from tension Criteria, $B = \frac{H}{\sqrt{G_c - c}}$

$$\left(\frac{B}{H} \right)^2 = \left(\frac{1}{G_c - c} \right) \text{---(11)}$$

from (1) and (11),

$$\begin{aligned}\sigma_1 &= \gamma_w \cdot H (G_c - c) \left[1 + \frac{1}{G_c - c} \right] \\ &= \gamma_w \cdot H (G_c - c) \frac{G_c - c + 1}{G_c - c}\end{aligned}$$

$$\sigma_1 = \gamma_w \cdot H [G_c - c + 1]$$

Assume $G_c = 2.5$, when $c = 1$

$$\sigma_1 = 2.5 \cdot \gamma_w \cdot H$$

$$\sigma_1 = 2.5 \cdot H$$

For No Compression failure, $\gamma_w \cdot H \cdot (G_c - c + 1) \leq F$

$$\Rightarrow \boxed{H \leq \frac{F}{\gamma_w \cdot (G_c - c + 1)}}^*$$

Note :- $\tau_{Toe} = (P_v - P'_v) \cdot \tan \alpha$
Since, $P'_v = 0$ and $\tan \alpha = \frac{B}{H}$

$$\begin{aligned}\tau_{Toe} &= P_v \cdot \frac{B}{H} \\ &= \gamma_w \cdot H \cdot (G_c - c) \times \frac{1}{\sqrt{G_c - c}}\end{aligned}$$

$$\boxed{\tau_{Toe} = \gamma_w \cdot H \cdot \sqrt{G_c - c}}^{**}$$

•• HIGH AND LOW GRAVITY DAM

— The Principal Stress at toe for elemental Profile of Dam.

$$\sigma_1 = \gamma_w \cdot H [G_c - c + 1] \text{ Depends on Height of Dam.}$$

— Hence, for no Compression failure $\sigma_1 \leq f_t$ from here the limiting height of the dam can be Computed as -

$$H \leq \frac{F}{\gamma_w (G_c - c + 1)}$$

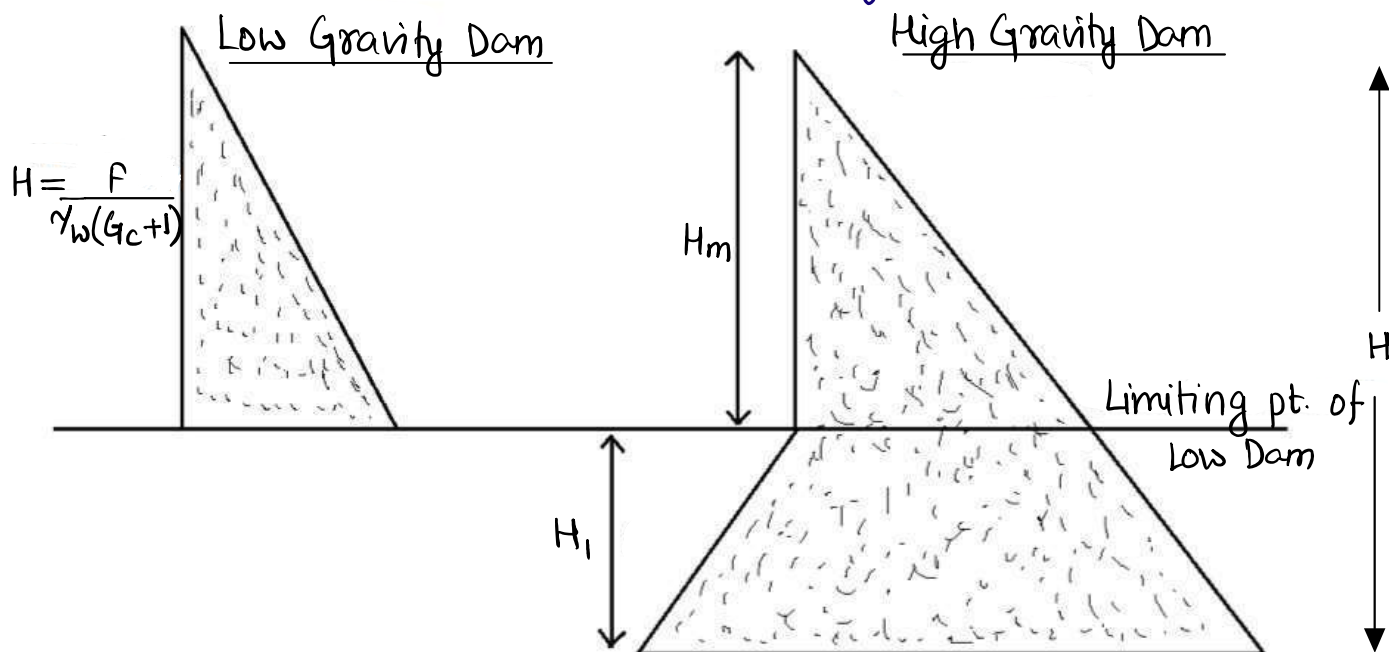
for Critical height,

$$c=0, G_c=2.5$$

$$H_{\max} = \frac{3000}{9.81(2.5-0+1)} = 87.337 \cong 90 \text{ m}$$



- Hence, if the height of the dam having elementary Profile is more than $H_{\max} \cong 90 \text{ m}$, the maximum Compressive Stress generated will exceed.



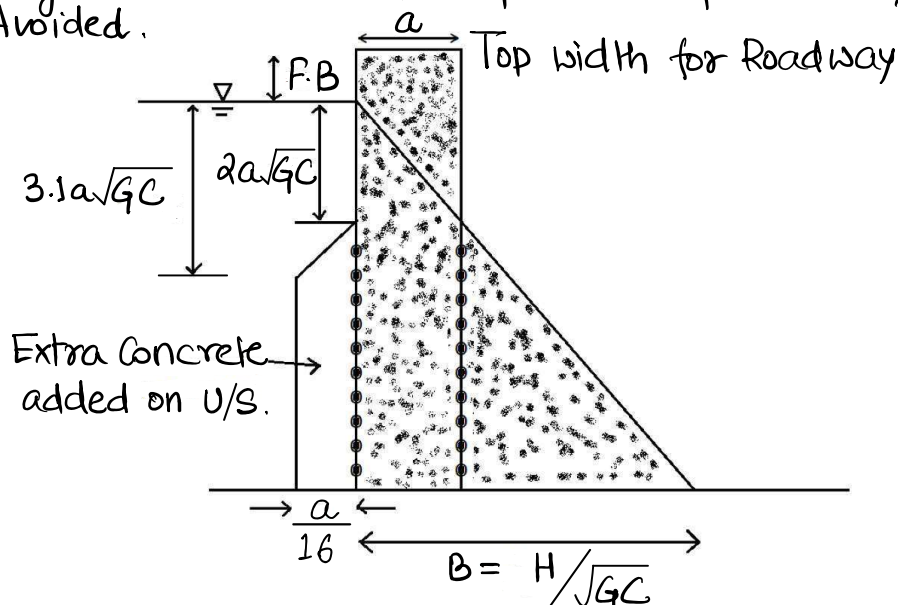
- For low Gravity Dam, $H \leq H_{\max}$
For High Gravity Dam, $H > H_{\max}$.

Note :→ The elementary Profile of dam is Only a theoretic Profile in Which Certain changes will have to be made in Order to Cater to the Practical need.

- These needs are -

(a) Providing a straight top width, for road Construction Over the top of the dam.

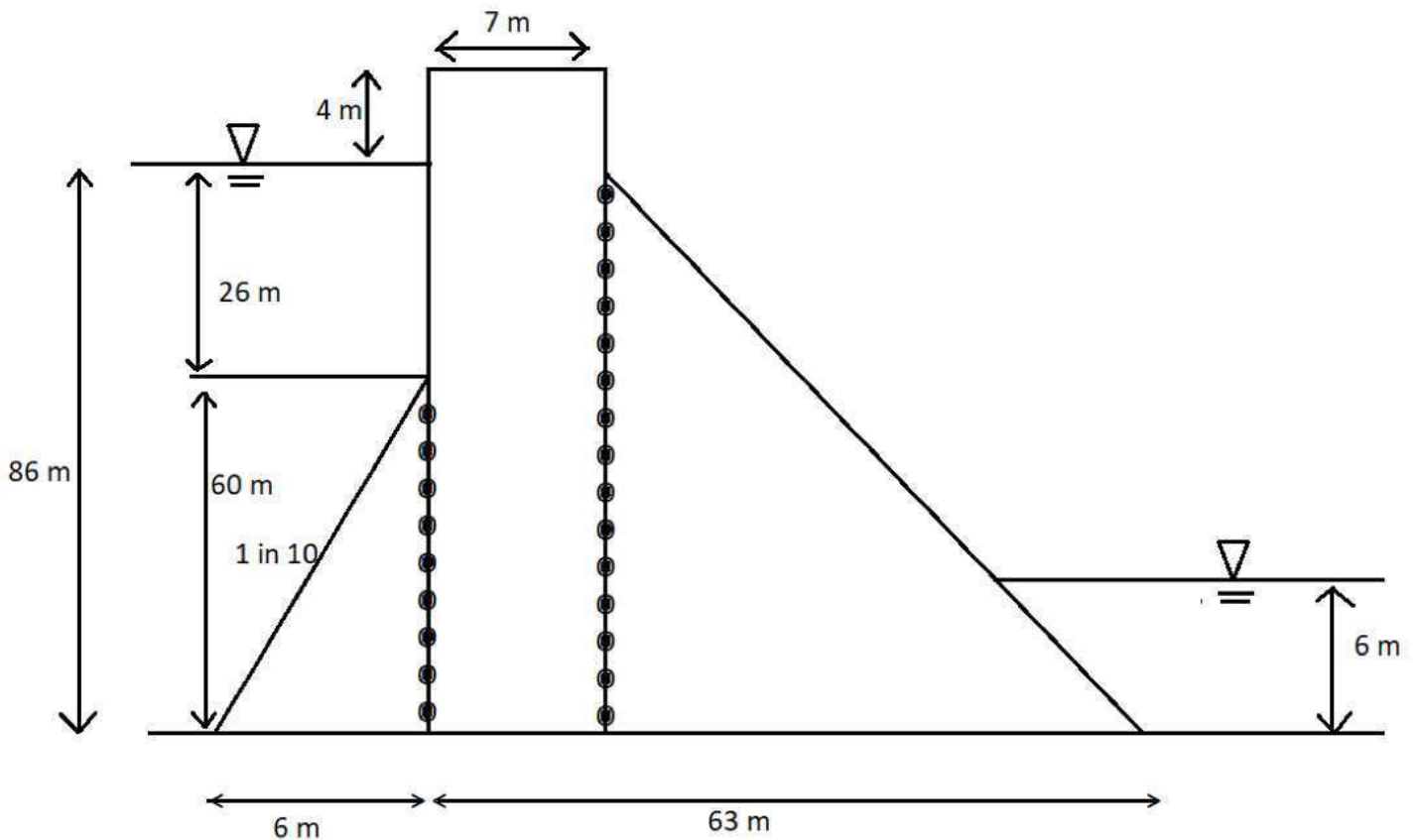
(b) For Providing Free board above the top water surface level so, that Overflow can be Avoided.



- The Addition of the two Provision, will Cause the resultant force to shift towards heel.
- The Resultant force, When the Reservoir is Empty was earlier Passing through Middle third point, this will therefore Shift more towards the heel. Thereby, would lead to development of tension at the toe.
- In Order to avoid this additional Concrete is provided On Upstream Side of Dam.

Note $\rightarrow$ The Free Board provided to prevent the Overflow from the dam is taken to be 4-5 % of dam height or $\frac{3}{2} h_w$, where h_w is the height of the Wave.

Q. Examine the stability of the given dam at the Base, The Earthquake Forces may be taken as equivalent to 0.1g for horizontal and 0.005 g for Vertical forces. The Uplift may be taken as equal to the hydrostatic pressure at the either ends and is Considered to act Over 60% of the area of the Section. A tail water depth of 6m is assumed to be Present When Reservoir is full and there is no tail water When Reservoir is empty. Assume $\gamma_c = 24 \text{ kN/m}^3$, $\gamma_w = 10 \text{ kN/m}^3$.



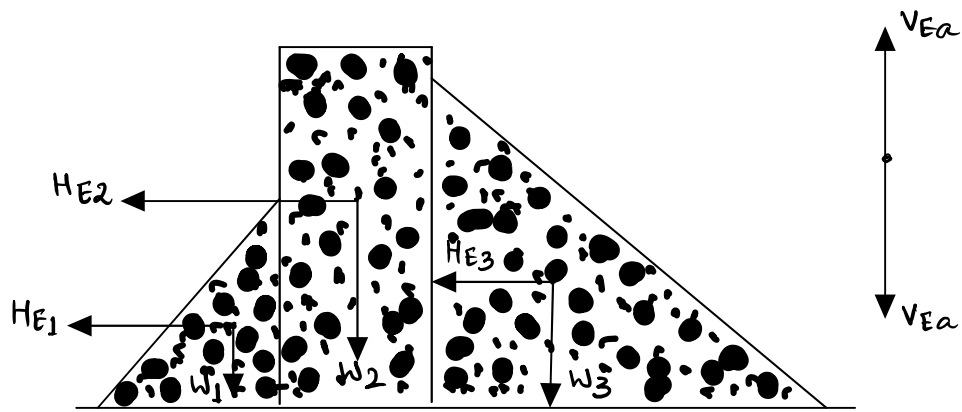
Ans $\rightarrow$ ① When Reservoir is Empty

Case A

Vertical Earthquake force acting Downward.

Case B

Vertical Earthquake force acting upward.



Force	Magnitude in kN	Lever Arm	Moment
W_1	$\frac{1}{2} \times 60 \times 6 \times 1 \times 24$ $= 4320$	65	2,80,800 (+)
W_2	$7 \times 90 \times 1 \times 24$ $= 15120$	59.5	8,99,640 (+)
W_3	$\frac{1}{2} \times 56 \times 80 \times 24$ $= 53760$ $\Sigma W_1 = 73200$	37.33	2007040 (+) $\Sigma M_1 = 3187480 (+)$
H_{E1}	$0.1 \times W_1$ $= 432$	20	8640 (+)
H_{E2}	$0.1 \times W_2$ $= 1512$	45	68040 (+)
H_{E3}	$0.1 \times W_3$ $= 5376$ $\Sigma H = 7320$	26.67	143,324 (+) $\Sigma M_2 = 220004$
V_{E1}	0.05×4320 $= 216 (+)$	63	14040 (+)
V_{E2}	0.05×15120 $= 756 (+)$	59.5	44982 (+)
V_{E3}	0.05×53760 $= 2688 (+)$ $\Sigma V_3 = 3660$	37.33	100352 (+) $\Sigma M_3 = 159374$

$$\begin{aligned}\Sigma M &= \Sigma M_1 + \Sigma M_2 + \Sigma M_3 \\ &= 3566858 \text{ KN-m}\end{aligned}$$

$$\begin{aligned}\Sigma V &= \Sigma V_1 + \Sigma V_3 \\ &= 76860 \text{ KN (+)}\end{aligned}$$

$$\begin{aligned}\bar{X} &= \frac{\Sigma M}{\Sigma V} = \frac{3566858}{76860} \\ &= 46.4 \text{ m.}\end{aligned}$$

$$\begin{aligned}e &= \frac{B}{2} - \bar{X} = \frac{69}{2} - 46.4 \\ &= -11.9 \geq \frac{B}{6} = 11.5\end{aligned}$$

$$\begin{aligned}P_V &= \frac{\Sigma V}{B} \left[1 \pm \frac{6 \cdot e}{B} \right] \\ &= \frac{76860}{69} \left[1 \pm \frac{6 \times 11.9}{6.9} \right] \\ &= 1113.91 (1 \pm 1.0347)\end{aligned}$$

$$(P_V)_{\text{Heel}} = 2266.5 \text{ KN/m}^2 < 3000 \text{ KN/m}^2$$

$$(P_V)_{\text{Toe}} = -37.87 \text{ KN/m}^2 < 500 \text{ KN/m}^2$$

$$\begin{aligned}\sigma_{\text{Toe}} &= P_V \cdot \sec^2 \alpha = P_V \cdot (1 + \tan^2 \alpha) \\ &= -37.87 \left(1 + \left(\frac{63}{90} \right)^2 \right) \\ &= -56.42 \text{ KN/m}^2 < 500 \text{ KN/m}^2\end{aligned}$$

$$\begin{aligned}\sigma_{\text{Heel}} &= P_V \cdot \sec^2 \beta = P_V \cdot (1 + \tan^2 \beta) \\ &= 2266.5 (1 + 0.1^2) \\ &= 2289.45 \text{ KN/m}^2 < 3000 \text{ KN/m}^2\end{aligned}$$

$$\begin{aligned}Z_{\text{toe}} &= P_V \cdot \tan \alpha \\ &= -37.87 \times \left(\frac{63}{90} \right) \\ &= -26.51 \text{ KN/m}^2 < 500 \text{ KN/m}^2\end{aligned}$$

$$\begin{aligned}Z_{\text{Heel}} &= P_V \cdot \tan \beta \\ &= 2266.5 (0.1) \\ &= 226.65 \text{ KN/m}^2 < 500 \text{ KN/m}^2\end{aligned}$$

- Case (B) Reservoir Empty and Vertical Earthquake force acting in the upward direction.

$$\begin{aligned}\Sigma M &= \Sigma M_1 + \Sigma M_2 + \Sigma M_3 \\ &= 3248110 \text{ KN-m.}\end{aligned}$$

$$\begin{aligned}\Sigma V &= \Sigma V_1 + \Sigma V_2 + \Sigma V_3 \\ &= 69540 \text{ KN.}\end{aligned}$$

$$\bar{x} = \frac{3248110}{69540} = 46.70 \text{ m.}$$

$$e = \frac{B}{2} - \bar{x}$$

$$= \frac{69}{2} - 46.70 = -12.2 \text{ m} > 11.5$$

$$P_v = \frac{\Sigma V}{B} \left(1 \pm \frac{6e}{B} \right) = \frac{69540}{69} (1 \pm 1.06)$$

$$(P_v)_{\text{Heel}} = 2077.87 < 3000 \text{ KN/m}^2.$$

$$(P_v)_{\text{Toe}} = -62.22 < 500 \text{ KN/m}^2.$$

$$\sigma_{\text{Toe}} = P_v \cdot \sec^2 \alpha$$

$$= -62.22 \cdot \left[1 \pm \left(\frac{63}{90} \right)^2 \right]$$

$$= -92.42 \text{ m}^2 < 500 \text{ KN/m}^2.$$

$$\sigma_{\text{Heel}} = P_v \cdot (1 + \tan^2 \beta)$$

$$= 2077.87 \times [1 + 0.1^2]$$

$$= 2098.648 < 3000 \text{ KN/m}^2.$$

$$\tau_{\text{Heel}} = 2077.87 \times \frac{63}{90} = 207.7 \text{ KN/m}^2$$

$$\tau_{\text{Toe}} = -62.22 \times 0.1$$

$$= -43.35 \text{ KN/m}^2.$$

Reservoir is Safe in empty Condition.

— Reservoir is **Full**

Case

When horizontal Acceleration due to earthquake is towards the Reservoir and Vertical acceleration is in Downward direction and uplift Pressure is Considered.

Force	Magnitude (KN)	Lever Arm	Moment
P	$\frac{1}{2} \times 86^2 \times 10$ $= 36980 (+)$	28.67	1060093.37 (+)
P'	$\frac{1}{2} \times 10 \times 6 \times 6$ $= 180$	2	360 (+)
W ₃	$\frac{1}{2} \times (88 + 26) \times 6$ $= 3360 (+)$	63.2	212352 (+)

W_{w2}	$\frac{1}{2} \times 6 \times 4.2 \times 1 \times 10$ $= 126 (+)$	1.4	176.4
U_1	$10 \times 6 \times 69 \times 0.6$ $= 2484$	34.5	85698 (+)
U_2	$\frac{10 \times (86 - 6) \times 69 \times 0.6}{2}$ $= 16560$	46	761760 (-)

$$\Rightarrow P_e = 0.702 \times 0.1 \times 10 \times 86$$

$$= 60.4 \text{ KN/m}^2 (-)$$

$$p_e = 0.726 \times 60.4 \times 86$$

$$= 3771.1344 (-)$$

$$M_e = 0.412 \cdot P_e \cdot H$$

$$= 0.412 \times 3771.13 \times 86$$

$$= 133618.89 \frac{\text{KN-m}}{\text{m}} (-)$$

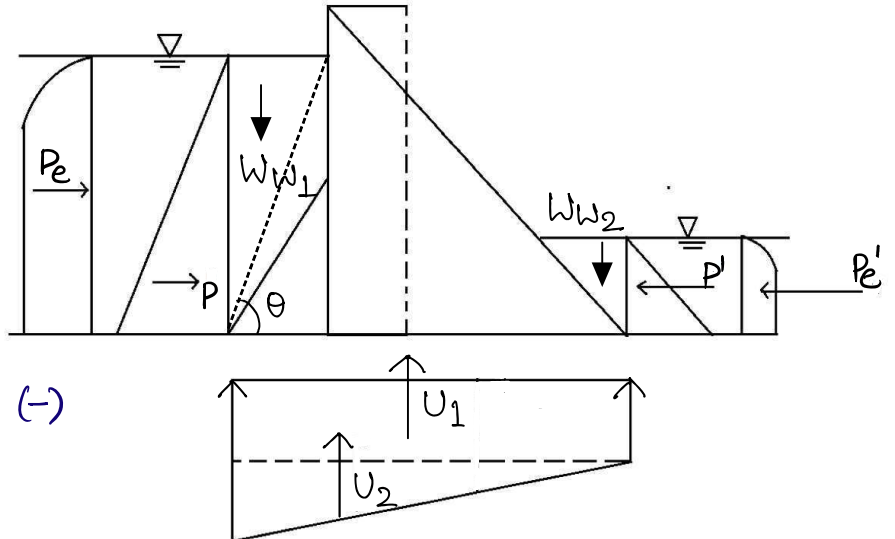
$$C_m = 0.735 \cdot \frac{\theta}{900}$$

$$\tan \theta = \frac{86}{6}$$

$$\theta = \tan^{-1}\left(\frac{86}{6}\right) = 86^\circ$$

$$C_m = 0.735 \times \frac{86}{90}$$

$$= 0.702$$



— Hydrodynamic force on the side is neglected.

$$\Sigma M = 3187480 + (-220004) + (-159374) + \Sigma M'$$

$$= 979882.21 \text{ KN-m/m } (+)$$

$$\Sigma V = 73200 - 7320 + \Sigma V'$$

$$= 50322 \text{ KN } (+)$$

$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{979882.21}{50322}$$

$$= 19.47 \text{ m}$$

$$e = \frac{B}{2} - \bar{x} = \frac{69}{2} - 19.47$$

$$= 15.02 \text{ m} > \frac{B}{6} = 11.5 \text{ m}$$

$$P_v = \frac{\sum V}{B} \left[1 \pm \frac{6 \cdot e}{B} \right] = \frac{50322}{69} \left[1 \pm \frac{6 \times 15.02}{69} \right]$$

$$(P_v)_{toe} = 1681.83 \text{ KN/m}^2 < 3000 \text{ KN/m}^2$$

$$(P_v)_{heel} = -223.23 < 500 \text{ KN/m}^2$$

$$\begin{aligned} \sigma_{toe} &= P_v \cdot \sec^2 \alpha - P' \cdot \tan^2 \alpha \\ &= 1681.33 \left\{ 1 + \left(\frac{63}{90} \right)^2 - 60 \left(\frac{63}{90} \right)^2 \right\} \\ &= 2475 \text{ KN/m}^2 < 3000 \text{ KN/m}^2 \end{aligned}$$

$$\begin{aligned} \sigma_{heel} &= P_v \cdot \sec^2 \beta - (P + P_e) \cdot \tan^2 \beta \\ &= -223.25 (1 + 0.12) - (86 \times 10 + 60.4) \times 0.12 \\ &= -234.66 \text{ KN/m}^2 \end{aligned}$$

$$\begin{aligned} \tau_T &= (P_v - P') \cdot \tan \alpha \\ &= (1681 - 60) \times \frac{63}{90} \\ &= 1134.7 < 3000 \text{ KN/m}^2 \end{aligned}$$

$$\begin{aligned} \tau_{heel} &= [P_v - (P + P_e)] \cdot \tan \beta \\ &= [-233.23 - (86 \times 10 + 60.4)] \times 0.1 \\ &= -114.363 \text{ KN/m}^2 < 500 \text{ KN/m}^2 \end{aligned}$$

FOS Against Overturning,

$$\begin{aligned} FOS &= \frac{\sum M(+)}{\sum M(-)} = \frac{3400368.4}{2420485.3} \\ &= 1.4 < 1.5 \end{aligned}$$

FOS Against Sliding,

Assume $\mu = 0.7$

$$\begin{aligned} \Rightarrow \frac{\mu \cdot \sum V}{\sum H} &= \frac{0.7 \times 50322}{4759.113} \\ &= 0.73 < 1 \end{aligned}$$

FOS for skin friction,

$$\begin{aligned} SFF &= \frac{\mu \cdot \sum V + q \times B \times 1}{\sum H} \\ &= \frac{0.7 \times 50322 \times 1 + 400 \times 69 \times 1}{47891.113} \\ SFF &= 2.75 \end{aligned}$$

PREVIOUS YEAR QUESTIONS

ESE

Q. Which of the following are the Causes of Failure of weirs ?

1. Rupture of floor due to Uplift.
2. Rupture of floor due to Suction Caused by Standing Wave.
3. Scour On the Upstream and downstream of the weir.

- (a) 1 and 2 Only.
(b) 1 and 3 Only.
(c) 1, 2 and 3.
(d) 2 and 3 Only.

[2020]

Ans :

(c) 1, 2 and 3.

— A Weir may fail due to following Reasons —

- (i) Piping
- (ii) Rupture of floor due to Uplift.
- (iii) Rupture of floor due to Suction Caused by Standing Wave.
- (iv) Scour at the U/S and D/S Side of the Weir floor.
- (v) Erosion of Apron Protection.

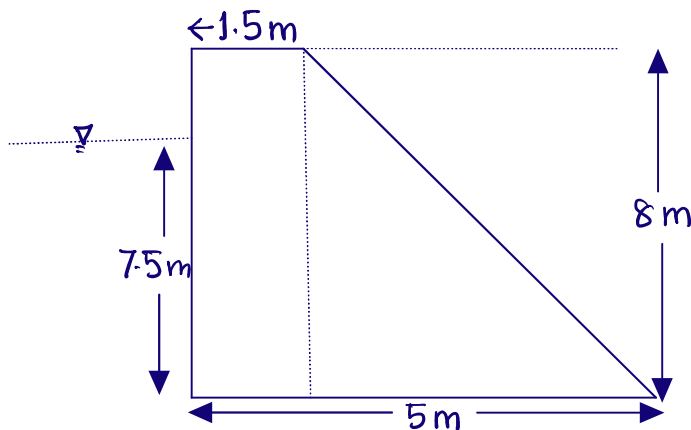
Q. A Masonry Dam 8 m high, 1.5 m wide at the top and 5 m wide at the Base retains water to a depth of 7.5 m, the water face of the dam being vertical. If the weight of water is 9.81 kN/m^3 , weight of Masonry is 22 kN/m^3 , the maximum intensity of Stress developed at the base will be nearly.

- (a) 196 kN/m^2 .
(b) 182 kN/m^2 .
(c) 160 kN/m^2 .
(d) 148 kN/m^2 .

[2019]

Ans :

(b) 182 kN/m^2 .



$$\text{Weight of Rectangular Portion} = 1.5 \times 8 \times 22$$

$$= 264 \text{ KN.}$$

$$\text{Weight of Triangular Portion} = \frac{1}{2} \times (5 - 1.5) \times 8 \times 22$$

$$= 308 \text{ KN.}$$

$$\text{Overturning moment about toe} = \frac{1}{2} \times 9.81 \times 7.5^2 \times \frac{7.5}{3}$$

$$= 689.765 \text{ KN-m.}$$

$$\text{Resisting moment about toe} = 264 \times \left\{ \frac{5 - 1.5}{2} \right\} + 308 \times 2 \times \left(\frac{5 - 1.5}{3} \right)$$

$$= 1840.67 \text{ KN-m.}$$

$$\text{Eccentricity about Centroid} = \frac{5}{2} - \bar{x}$$

$$\bar{x} = \frac{(M_R - M_o)}{\text{Vertical load}}$$

$$= \frac{1840.67 - 689.765}{(308 + 264)}$$

$$= 2.01 \text{ m}$$

$$e = \frac{5}{2} - 2.01 = 0.49 \text{ m.}$$

$$\text{Stress at toe} = \frac{W}{b} \left\{ 1 + \frac{6e}{b} \right\}$$

$$= \frac{572}{5} \left\{ 1 + \frac{6 \times 0.49}{5} \right\}$$

$$= 181.80 \text{ KN/m}^2 = \underline{182} \text{ KN/m}^2$$

Q. The Volume of Water below the minimum pool level in a reservoir is known as,

- (a) Useful Storage
- (b) Surcharge Storage
- (c) Dead Storage
- (d) Bank Storage.

Ans :

(c) Dead Storage.

GATE

Q. 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

$$\textcircled{a} \frac{b}{H} = \frac{1}{\sqrt{G - K}}$$

(b) $\frac{b}{H} = \sqrt{G-K}$
 (c) $\frac{b}{H} = \frac{1}{G-K}$
 (d) $\frac{b}{H} = \frac{1}{K\sqrt{G-K}}$

[2008]

Ans:

(a) $\frac{b}{H} = \frac{1}{\sqrt{G-K}}$

Q. Which one of the following equations represent the downstream Profile of Ogee Spillway with Vertical Upstream face? (x, y) are the Coordinates of the point on the downstream 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) $\frac{y}{H_d} = -0.5 \left[\frac{x}{H_d} \right]^{1.85}$
 (c) $\frac{y}{H_d} = -2.0 \left[\frac{x}{H_d} \right]^{1.85}$
 (d) $\frac{y}{H_d} = -2.0 \left[\frac{x}{H_d} \right]^{1/1.85}$

[2005]

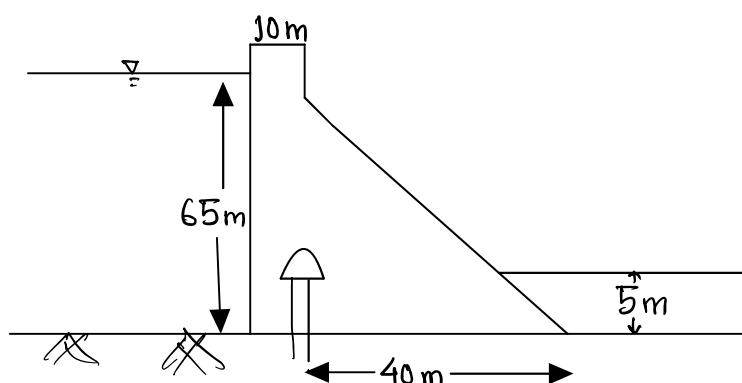
Ans:

(a) For a Spillway having Vertical Upstream face the downstream crest is given by the equation,

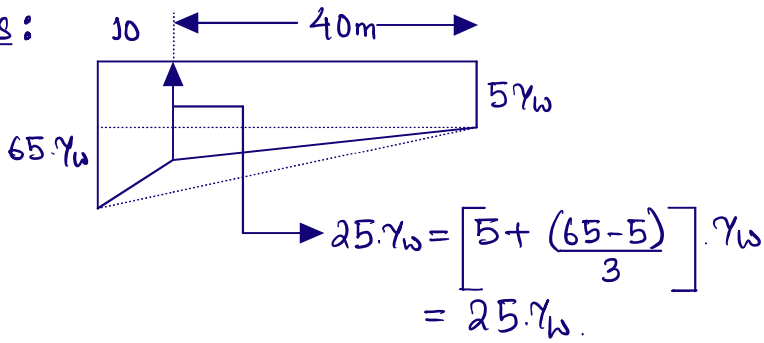
$$\begin{aligned}
 x^{1.85} &= -2 \cdot H_d^{0.85} \cdot y \\
 \Rightarrow x^{1.85} &= -2 \cdot \frac{H_d^{0.85}}{H_d} \cdot y \\
 \Rightarrow \left[\frac{y}{H_d} \right] &= -0.5 \left[\frac{x}{H_d} \right]^{1.85}
 \end{aligned}$$

Q. A Concrete gravity dam Section is shown in the figure. Assuming Unit Weight of Water as 10 kN/m^3 and Unit Weight of Concrete as 24 kN/m^3 the Uplift force per Unit length of the dam (expressed in kN/m) at PA is _____.

[2016 - Shift-1]



Ans:



$$\Rightarrow F = \frac{1}{2} \times 10(65+25) \times 10 + \frac{1}{2} (25+5) \times 40 \times 10$$

$$= 4500 + 6000$$

$$F = \underline{10,500 \text{ KN/m}}$$

SSC-JE

A. The maximum height of a Masonry dam of a triangular Section Whose base width is B and Specific gravity is S.

- (a) $B\sqrt{S}$
- (b) BS
- (c) $\sqrt{BS}$
- (d) $S\sqrt{B}$

[2017-Morning]

Ans:

(a) $B\sqrt{S}$

— The maximum height of a Masonry dam of a triangular Section is

$$H = B\sqrt{S-C}$$

$$H = B\sqrt{S}$$

{When No Uplift is Considered i.e. $C=0$ }

A. A Retaining wall of trapezoidal section having base width 'b' retains earth at its back. For no tension to be developed at base, the resultant force will intersect the base from centre line of the base, within a distance of

- (a) $b/3$
- (b) $b/4$
- (c) $b/5$
- (d) $b/6$

[2013]

Ans:

(d) $b/6$

$$\text{Since, } P_{\max/\min} = \frac{\Sigma V}{B} \left\{ 1 \pm \frac{6 \cdot e}{B} \right\}$$

$$P_{\min} = \frac{\Sigma V}{B} \left[1 - \frac{6 \cdot e}{B} \right]$$

$$\begin{aligned} \because P_{\min} = 0, \text{ then } \frac{\Sigma V}{B} \left[1 - \frac{6e}{B} \right] &= 0 \\ 1 - \frac{6e}{B} &= 0 \\ e &= \frac{B}{6} \end{aligned}$$

THEORY OF SEEPAGE

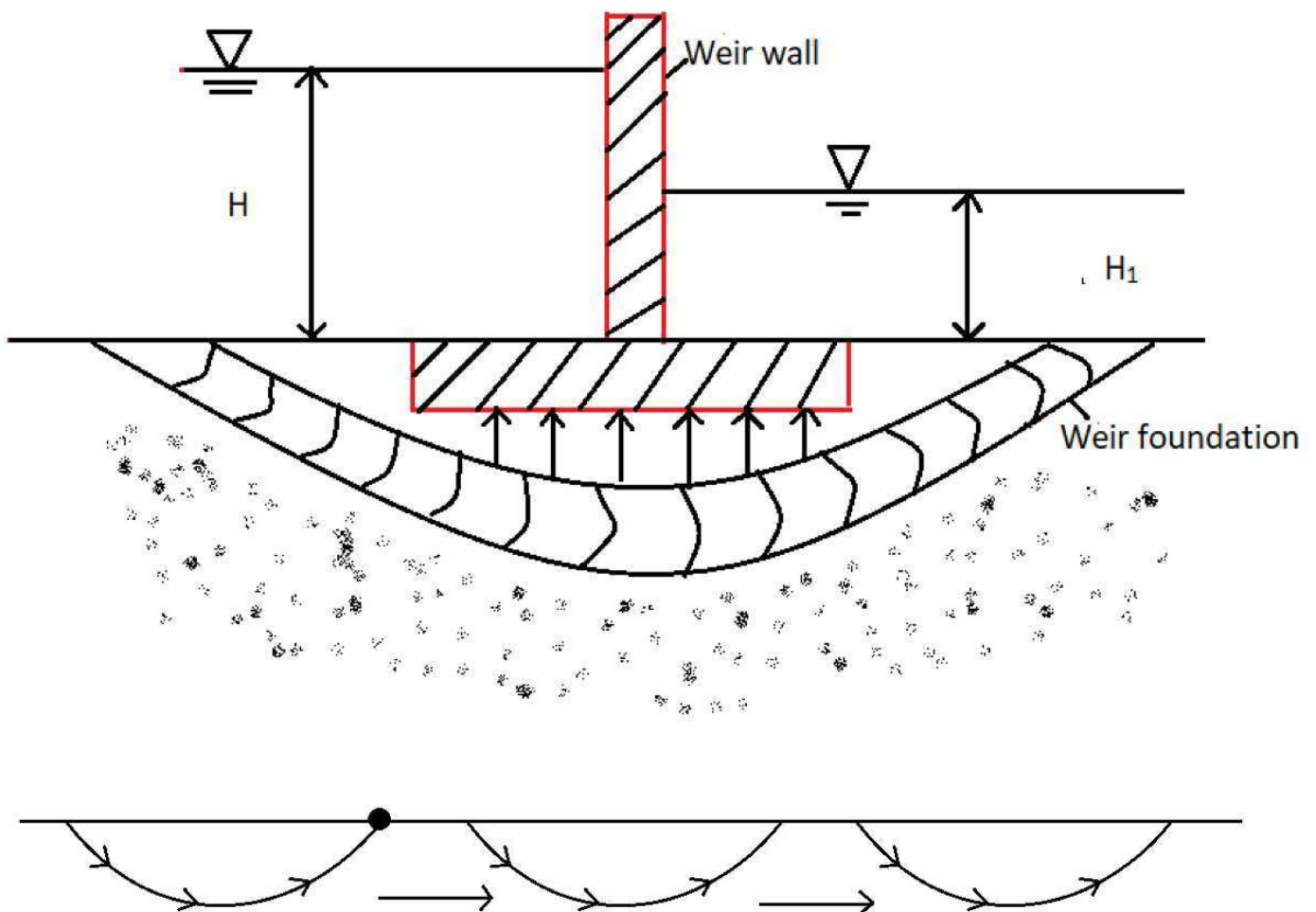
- Hydraulic Structures Such as Dam, weirs, barrage, head Regulators, Cross Drainage Works, may either be constructed on a Impervious Rock foundation or on Pervious foundation.
- Most of these Structure in India are founded On Pervious alluvial Soil foundation, hence Seepage of Water is observed beneath these foundation resulting in their failure by following modes.

- (i) Failure due to piping
- (ii) Failure due to Seepage.



① FAILURE DUE TO PIPING { HYDRULIC HEAD }

- The Percolating water travels from upstream to Downstream of these Hydraulic Structures, it has Sufficient head available with it due to which it exerts the Seepage Pressure On the medium Particles. This Pressure may Cause Moment of Medium Particles along with the flowing water at Downstream.
- This Process of erosion extend backward and Results in formation of empty space below the foundation due to which Hydraulic Structure Subsides.
- This type of failure is termed as Piping failure or Undermining.



- It Can be avoided by following method -

① By Increasing the length of flow by Provision of Sheet pile wall at Downstream Side.

② By Increasing the length of the foundation due to which head loss will be more.

② UPLIFT PRESSURE FAILURE {PRESSURE HEAD}

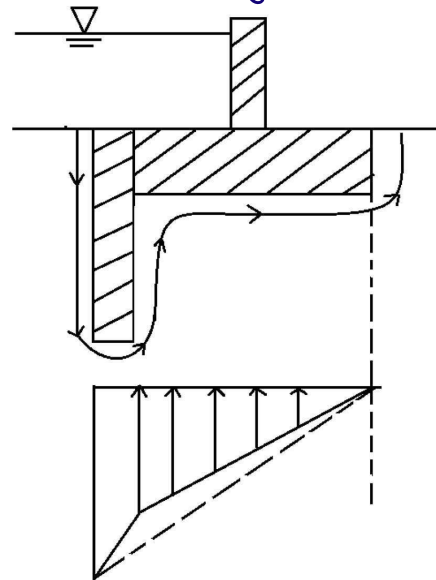
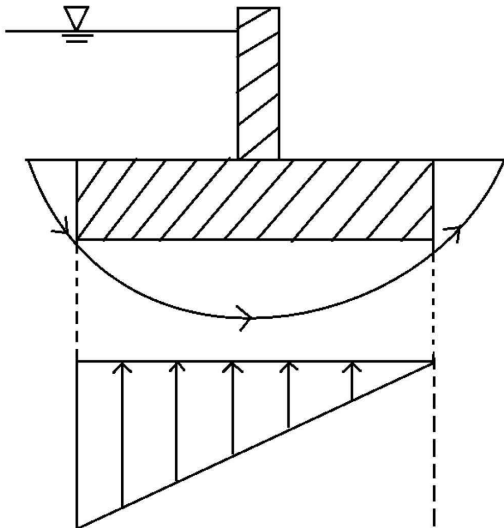
- The Percolating Water below the Impervious floor Exerts an upward Pressure on the foundation, which is termed as Uplift Pressure.

- If this Pressure is not Counter-balance by the weight of the foundation, then foundation may fall due to Rupture.

- In Order to avoid this -

① A Sheet pile at Upstream end can be provided, that helps in causing low head at all the Downstream points.

② By Increasing the thickness of foundation, due to which weight of foundation increases.



•• THEORIES OF SEEPAGE

① BLIGH'S CREEP THEORY OF SEEPAGE

Assumptions

① The Percolating Water Follows the outline of the base of foundation of the Hydraulic Structure i.e. water Creeps along the bottom Contour of the Structure. The length of Path thus travelled by the water is termed as Creep length.

② The loss of head is Proportional to length of the Creep.

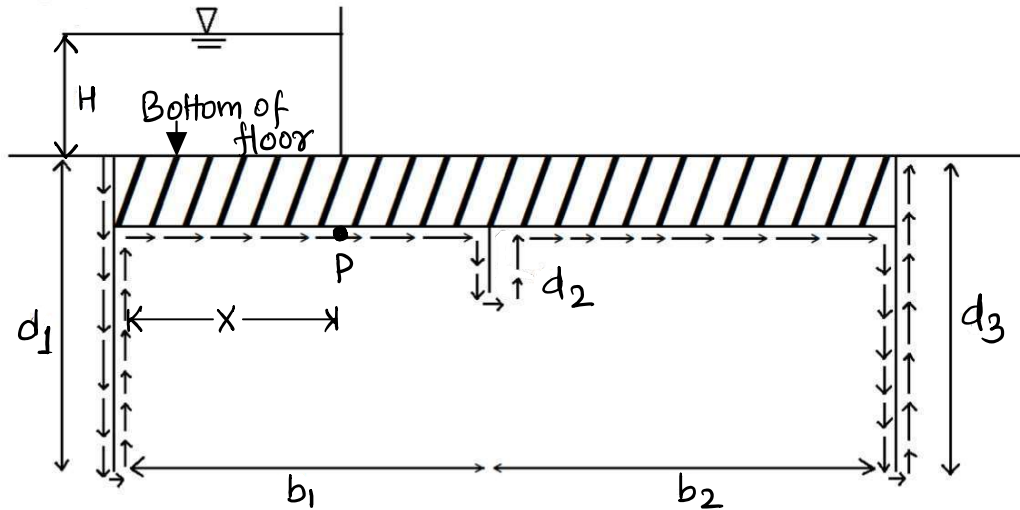
$$H_L \propto L$$

Where,

H_L = Head Under Which flow takes place.

③ There is no distinction between Horizontal and Vertical Creep.





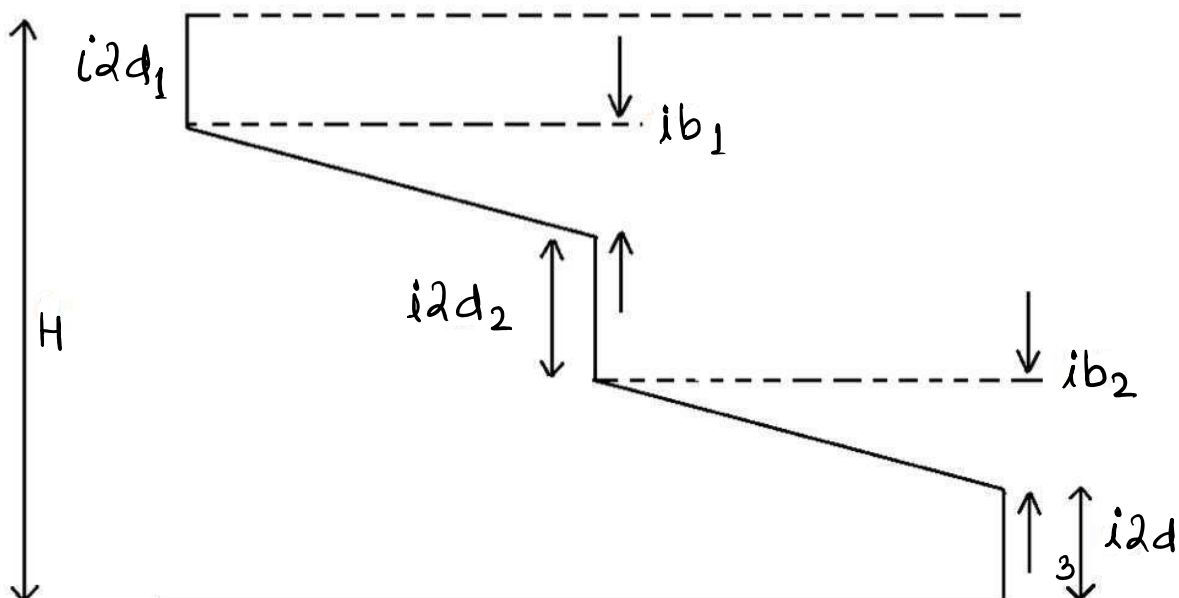
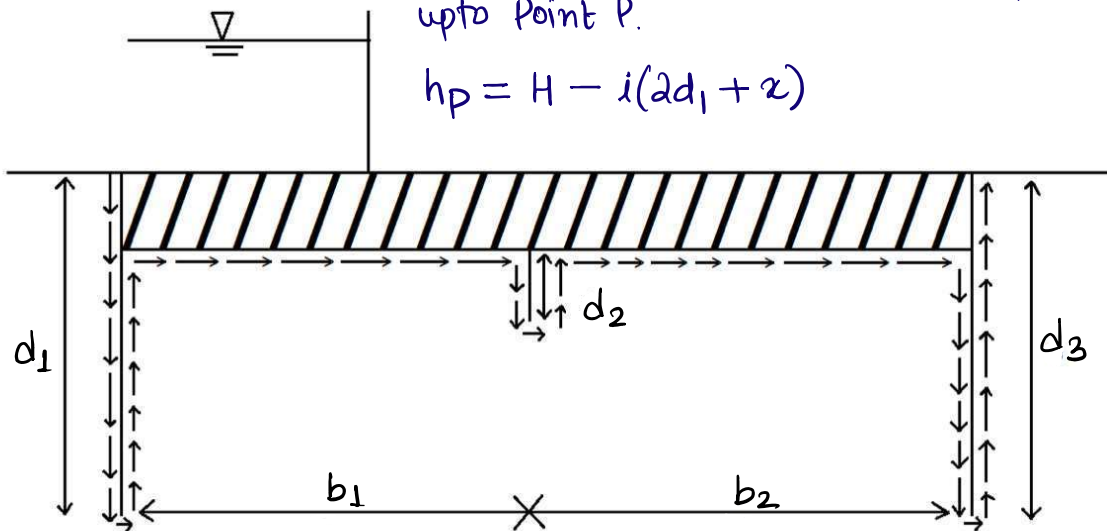
$$\text{Length of Creep (L)} = 2d_1 + b_1 + 2d_2 + b_2 + 2d_3$$

$$\text{Hydraulic Gradient} = \frac{\text{Hydraulic Head loss}}{\text{length of Creep}} = \frac{H}{L}$$

$$H \propto L$$

Head Available at point P = Total head Under Which flow takes place — Head Loss upto Point P.

$$h_p = H - i(2d_1 + x)$$



— As Per Bligh

⑥ TO PREVENT THE PIPING

Average Hydraulic gradient $\leq \frac{1}{C}$

$$\boxed{\frac{H}{L} \leq \frac{1}{C}} \quad ***$$

Where,

$L \rightarrow$ Creep length

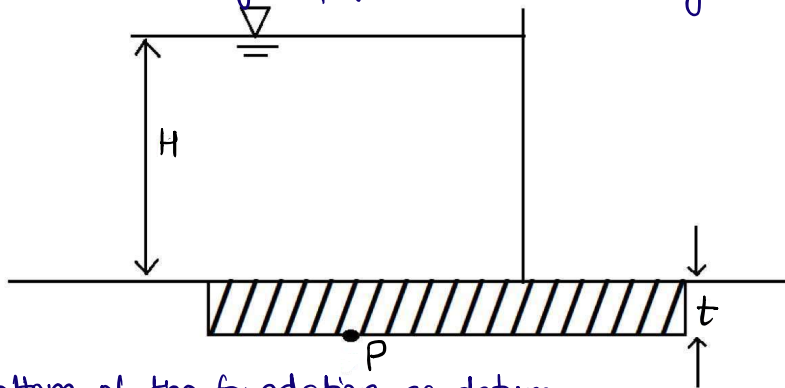
$H \rightarrow$ Total Seepage head

$C \rightarrow$ Bligh's Creep Coefficient } It depends on type of soil.

Type of Soil	C	$i = \frac{H}{L}$
① Fine Sand	15	$< \frac{1}{15}$
② Coarse Sand	12	$< \frac{1}{12}$
③ Sand Mixed with Gravel and Boulder.	5-9	$< \frac{1}{5} - \frac{1}{9}$
④ light Sand and Mud.	8	$< \frac{1}{8}$

⑥ TO PREVENT THE UPLIFT PRESSURE FAILURE

— To Prevent this failure weight of foundation must be greater than Uplift Pressure.



— Taking the bottom of the foundation as datum.

Pressure head at point, P = $h + t$

Uplift Pressure = $(h + t) \cdot \gamma_w$

Uplift Force = $(h + t) \cdot \gamma_w \cdot A$

Now, for no Uplift failure,

Weight of Foundation $\geq$ Uplift force

$$\gamma_c \cdot V_c \geq (h + t) \cdot \gamma_w \cdot A$$

$$\cancel{\gamma_c \cdot \gamma_w \cdot A} \geq (h + t) \cdot \cancel{\gamma_w \cdot A}$$



$$t(G_c - 1) \geq h$$

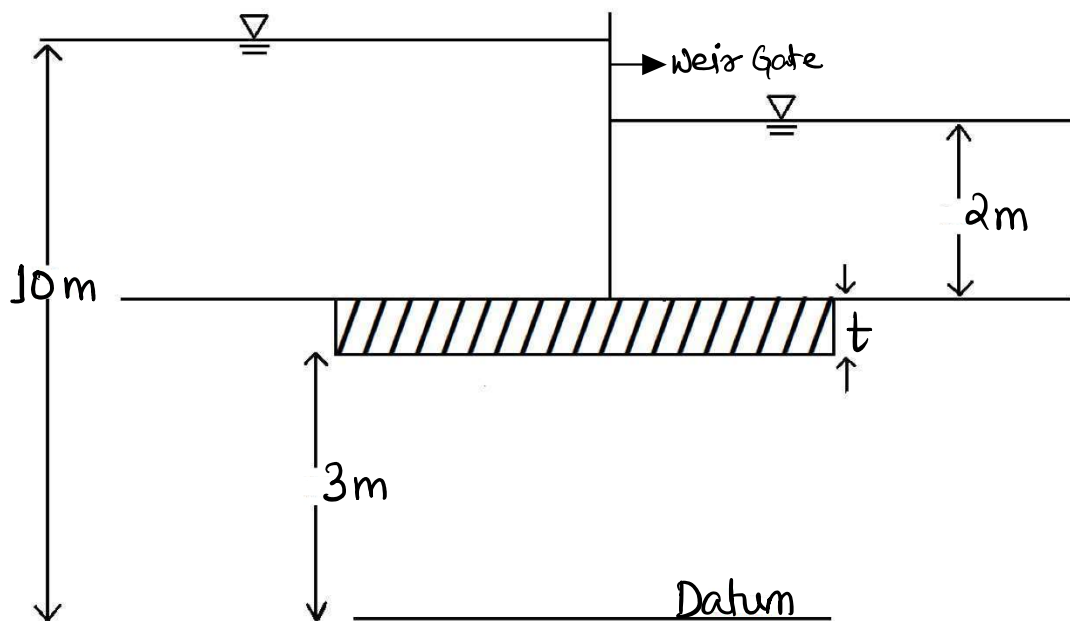
$$t \geq \frac{h}{(G_c - 1)}$$

Here, FOS of $\frac{4}{3}$ is also provide against Uplift Pressure failure.

$$F = \frac{4}{3}$$

$$t_{\text{design}} = \frac{4}{3} \frac{h}{(G_c - 1)}$$

Q. Find thickness of foundation.



$$\begin{aligned} \text{Ans :- } t &= \frac{4}{3} \frac{h}{(G_c - 1)} \\ &= \frac{4}{3} \frac{h}{(2.4 - 1)} \end{aligned}$$

$$h = (10 - (3 + t + 2)) = 5 - t$$

$$\begin{aligned} \text{for No Uplift failure} &\Rightarrow t = \frac{h}{(G_c - 1)} \\ t(2.4 - 1) &= 5 - t \end{aligned}$$

$$t = 2.083 \text{ m.}$$

Q. Calculate the Seepage head at point a and B, for following case.



Ans $\Rightarrow H = 5 \text{ m}$
 $L = 2 \times 10 + 20 + 16$
 $= 56 \text{ m}$

$$i = \frac{H}{L} = \frac{5}{56}$$

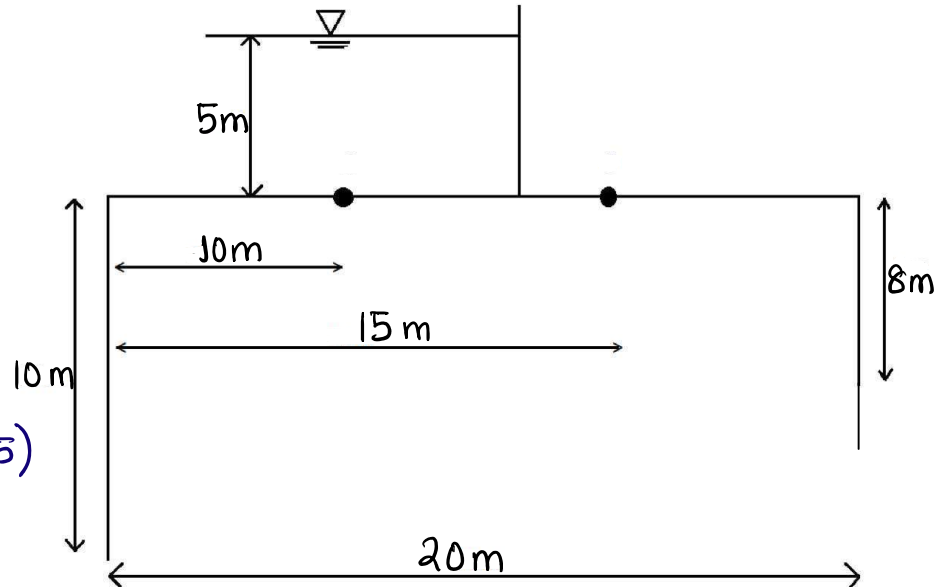
$$h_A = 5 - \frac{5}{56} \times 30$$

$$= 2.32 \text{ m}$$

$$h_B = H - i \cdot L_B$$

$$= 5 - \frac{5}{56} (2 \times 10 + 15)$$

$$= 1.875 \text{ m.}$$



— LIMITATION OF BLIGH'S THEORY

- ① Bligh's Observed no difference between horizontal and Vertical length of Creep.
- ② Bligh Observed no difference between Effectiveness of outer and inner face of Sheet Pile and Short and Long intermediate Sheet pile Wall.
- ③ Bligh did not indicate any Significance of exit gradient i.e. hydraulic gradient at the exit of the floor.
- ④ According to Bligh, the loss of head is proportional to the Creep length but is not the Case in actual.
- ⑤ Bligh did not indicate the Significance of Providing a Cut off {Sheet pile Wall} at Downstream Side of the hydraulic Structure.

② LANES WEIGHTED CREEP THEORY

— Lane Observed that Vertical Creep length is more effective in Causing head loss in Comparison to the horizontal length of the Creep.

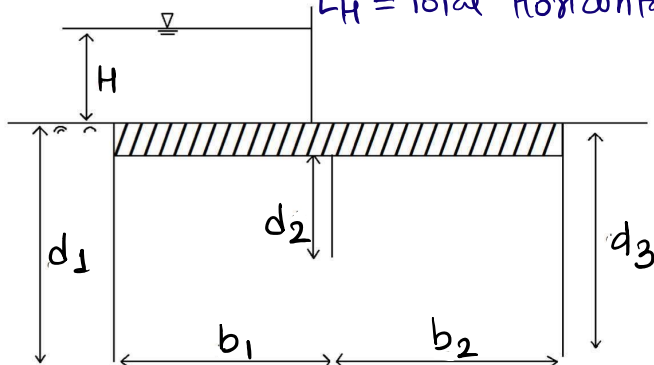
— Hence, he Proposed a Weightage factor of One to the Vertical Creep and $\frac{1}{3}$ to Horizontal Creep.

$$L_c = 1 \times L_v + \frac{1}{3} \times L_h$$

Where,

L_v = total Vertical length of the Creep.

L_h = Total Horizontal length of the Creep.



$$L_e = 2\{d_1 + d_2 + d_3\} + \frac{\{b_1 + b_2\}}{3}$$

$$L_{ep} = 2d_1 + \frac{1}{3} \cdot x$$

$$i = \frac{H}{L}$$

$$h_p(\text{head Available}) = (H - i \cdot L_{ep})$$

— All the limitations of Bligh's Creep theory are also Applicable to Lanes theory except 1st,

— As per Lane theory,

① TO PREVENT PIPING FAILURE

$$\text{Avg. } i \leq \frac{1}{C_w}$$

$$\frac{H}{L_e} \leq \frac{1}{C_w}$$

Where,

C_w = Lanes Creep Coefficient.

② TO PREVENT UPLIFT FAILURE

$$t \geq \frac{h}{(G-1)}$$

③ KHOSLA'S THEORY

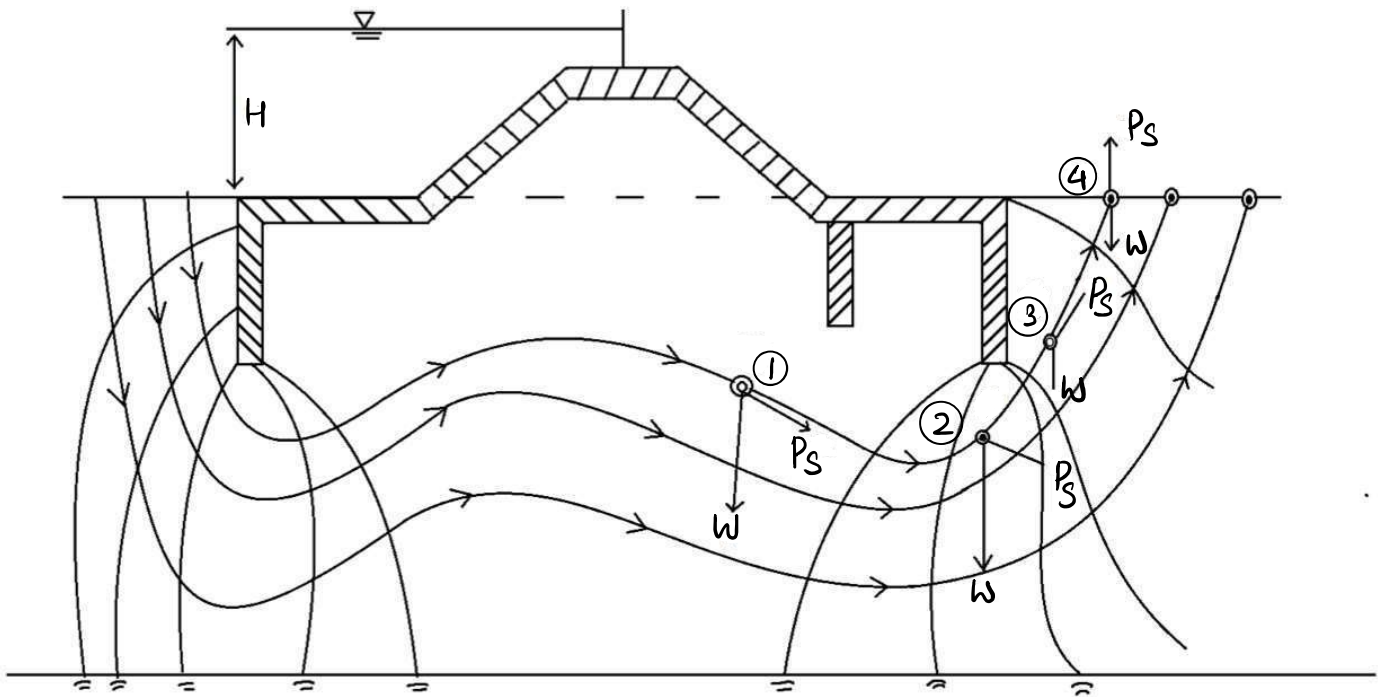
— Assumptions

- ① The Outer Face of end Sheet pile is Much more effective than the inner face of the of Horizontal length of floor.
- ② The intermediate Sheet piles is Smaller in Size (height) than the outer ones, is found to be ineffective in causing head loss.
- ③ Piping of the floor is started from the tail end, i.e. When exit gradient becomes More than i_c < Critical Hydraulic gradient.

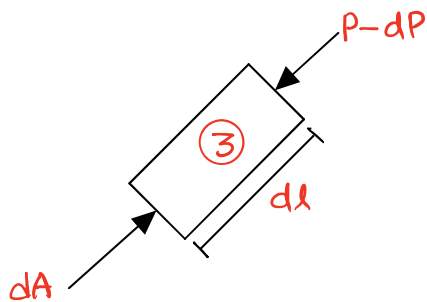
$$\boxed{i_{\text{exit}} \geq i_c} \quad \text{Piping failure}$$

- ④ It is absolutely Necessary to have a deep Sheet pile at the Downstream end of the floor in order to prevent Piping failure.
- ⑤ Seeping Water does not Creep along the bottom Contour of the floor, but instead follows the flow path / Stream line or Flow line.





— ANALYSIS OF KHOSLA THEORY



— Seepage Pressure On Soil Element (3)

$$P - (P - dP) = dP$$

$$dP = dh \cdot \gamma_w \text{ ————— (I)}$$

Weight of Soil elements (3) = $\gamma' \cdot \text{Volume}$.

$$\begin{aligned} \text{Weight of Soil elements (3)} &= \gamma' \times \frac{\text{Volume}}{\text{Area}} \\ &= \gamma' \times dl \text{ ————— (II)} \end{aligned}$$

for No Piping, weight $\geq P_s$ { Seepage Pressure }

$$\begin{aligned} \gamma' \times dl &\geq dh \cdot \gamma_w \\ \gamma' &\geq \frac{dh}{dl} \times \gamma_w \end{aligned}$$

$$\frac{(G-1) \cdot \gamma_w}{1+e} \geq i \times \gamma_w$$



$$\Rightarrow i \leq (G-1)(1-n)$$

for Critical Condition, $i = i_c$
 $i_c = (G-1)(1-n)$

for fine Sand, Coarse, Silt
 $G = 2.65$, $e = 0.65$

$$i_c = \frac{2.65 - 1}{1 + 0.65}$$

$$= 1$$

for No Piping, $i \leq 1$

for Against Piping $FOS = \frac{i_c}{i}$

— FOS is generally Kept in Range of 4-5.

$$\text{Hence, } i_{\text{exit}} = \frac{1}{4} - \frac{1}{5}$$

— FOS to be Provided depends on type of Soil as follows —

Type of Soil	FOS	Safe Exit gradient
Shingle	4-5	$\frac{1}{4} - \frac{1}{5}$
Coarse Sand	5-6	$\frac{1}{5} - \frac{1}{6}$
Fine Sand	6-7	$\frac{1}{6} - \frac{1}{7}$

— Exit Gradient as Per Khosla's theory,

$$L_{\text{exit}} = \frac{H}{d} \times \frac{1}{\pi \sqrt{\lambda}}$$

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

$$\alpha = \frac{b}{d}$$

Where,

H = Total Head Under which flow takes place.

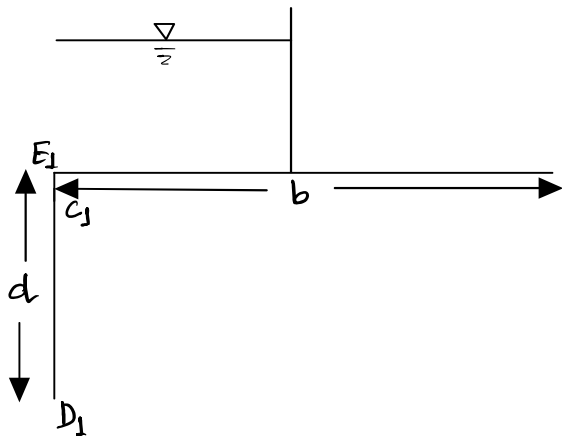
b = total horizontal length of the floor.

d = depth of Downstream Sheet pile.



•• Khosla's method of Independent Variable For Determination of Exit Gradient And Pressure.

- In Order to find how seepage takes place below the hydraulic structure it is Necessary to plot the flow net which is the gradient solution of Laplace equation.
- flow net along with other methods to solve Laplace equation are comparatively difficult. Hence, Khosla introduces a Accurate and Quick approach termed as method of Independent Variable.
- In this Method a complex Profile of hydraulic Structure is broken into no. of Simple Profile, each of which can be Solved mathematically and solution of which is given by Khosla in the form of Curve also.
- The Simple Profile which are most Useful are
 - ① The Straight horizontal floor of negligible thickness with sheet pile line on Upstream and downstream end or both.
 - ② A Straight horizontal floor depressed below the bed but without any Vertical Cutoff.
 - ③ A Straight horizontal floor of negligible thickness with sheet pile of some intermediate point.

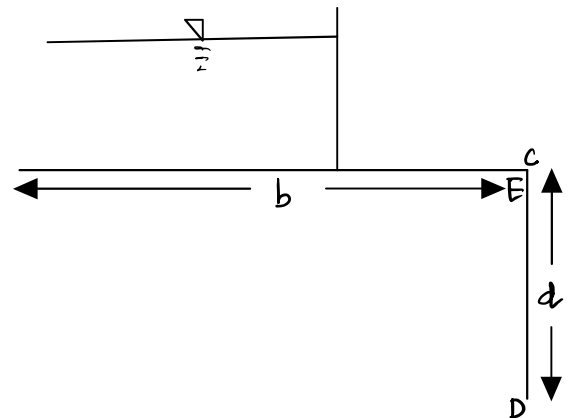


$$\phi_{C1} + \phi_E = 100$$

$$\phi_{D1} + \phi_D = 100$$

Here,

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



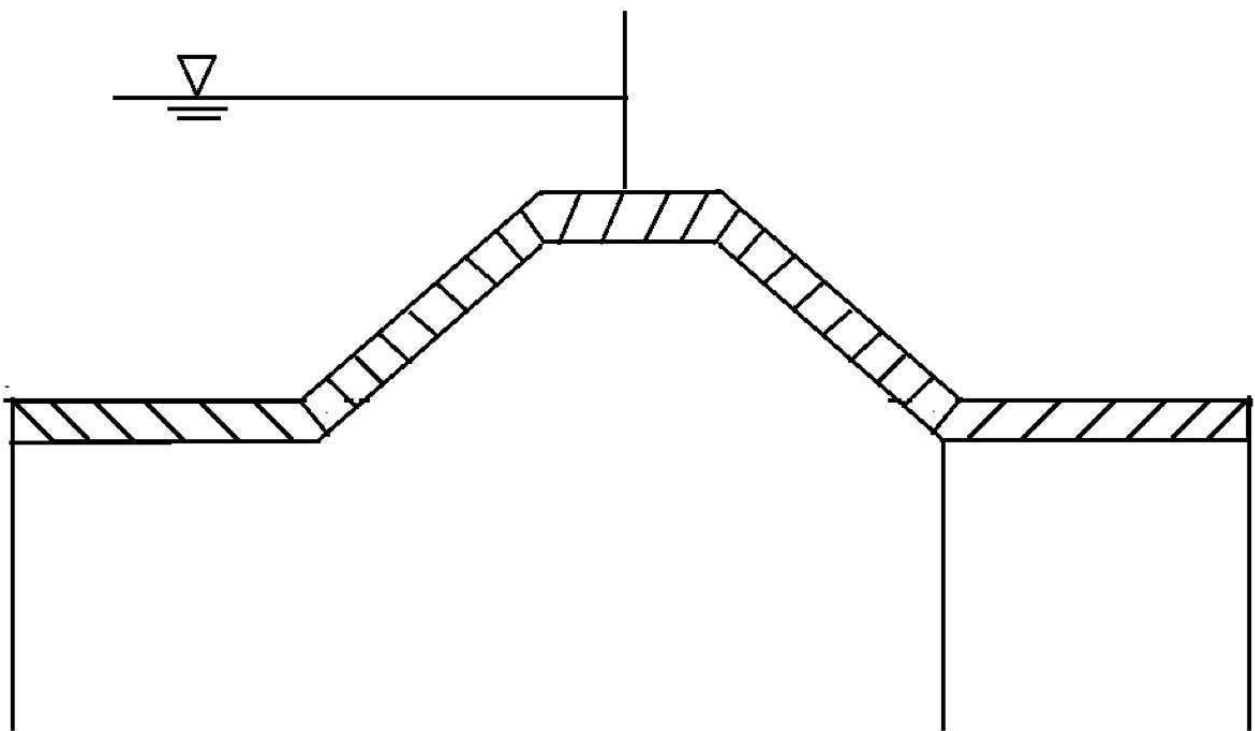
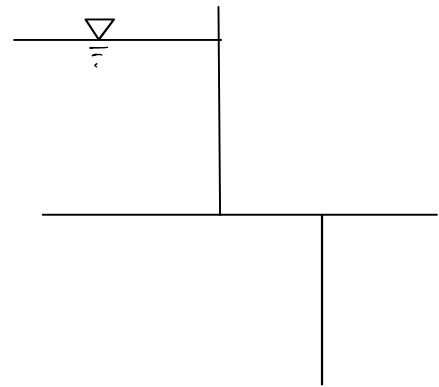
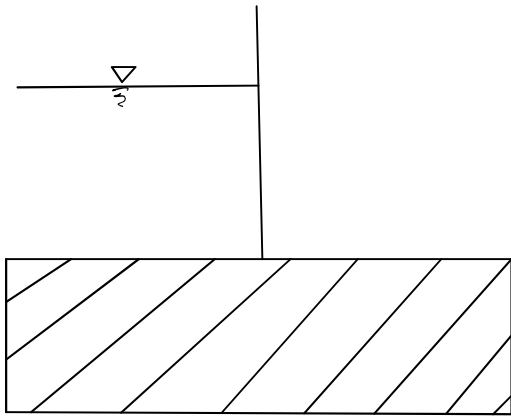
$$\phi_E = \frac{1}{\pi} \cos^{-1} \left\{ \frac{\lambda - 2}{\lambda} \right\}$$

$$\phi_D = \frac{1}{\pi} \cos^{-1} \left\{ \frac{\lambda - 1}{\lambda} \right\}$$

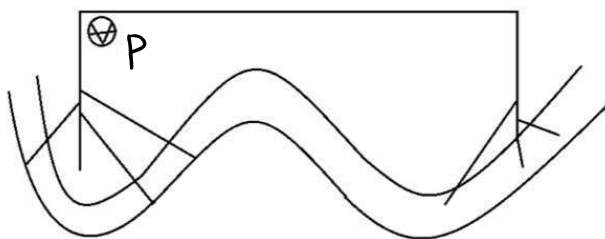
Here,

$$\lambda = \frac{b}{d} \quad \{ \text{for Respective Cases} \}$$

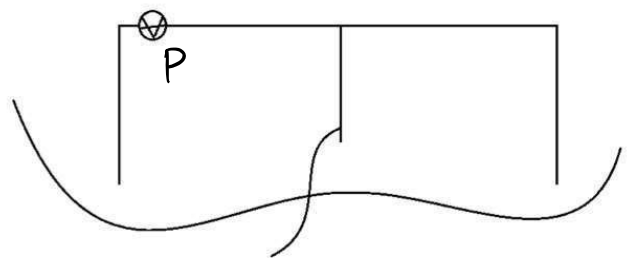




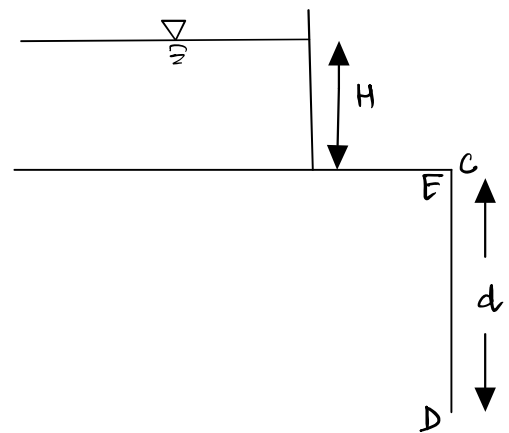
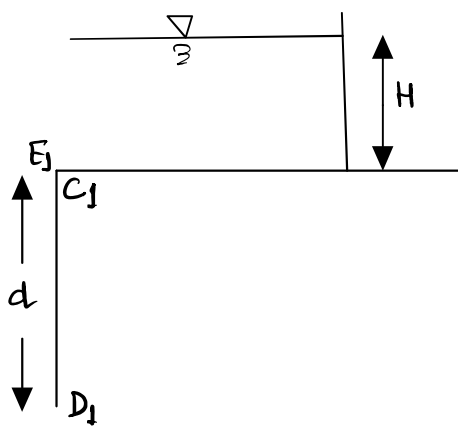
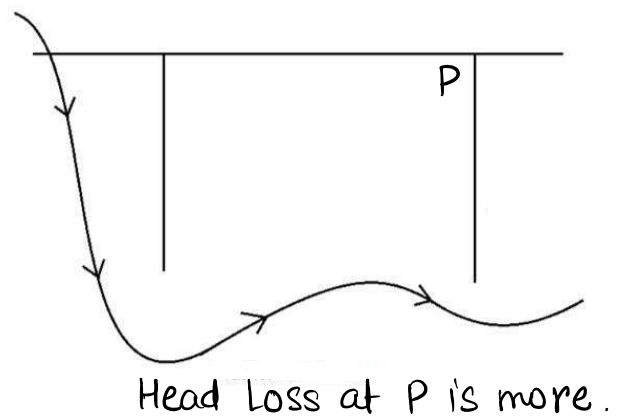
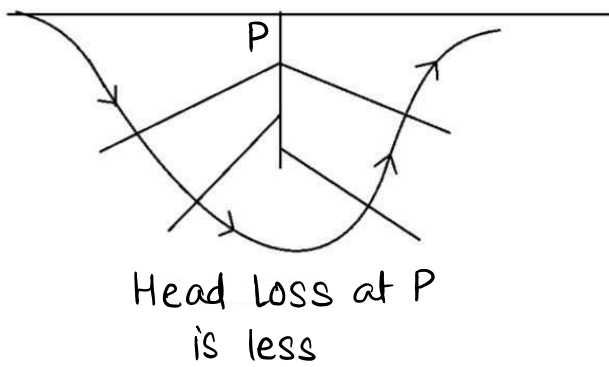
①



Head loss more



Head Loss $2 < 1$
more head available at P.



$$h_{D1} = h_{E1} - h_{L_{E1D1}}$$

$$= H - i \cdot d \quad \text{--- ①}$$

$$h_c = h_D - h_{L_{pc}}$$

$$0 = h_D - i \cdot d \quad \text{--- ②}$$

from ① & ②, $h_{D1} = H - h_D$

$$\boxed{h_{D1} + h_D = H} = 100\% \cdot H$$

$$= \phi H$$



- After Breaking the Complex hydraulic Structure Profile into no. of Simple Profile Residual Potential at Key points of Simple profile is Computed directly with the help of the Standard result given by Khosla.
- These Residual Potential at Key points is then further Corrected for the following -

① CORRECTION FOR THE MUTUAL INTERFERENCE OF PILES

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

Where,

C = Correction to be applied as % of head due to interference of Pile.

D = Depth of the Pile, the influence of which has to be determine on the Neighbouring Pile of depth d .

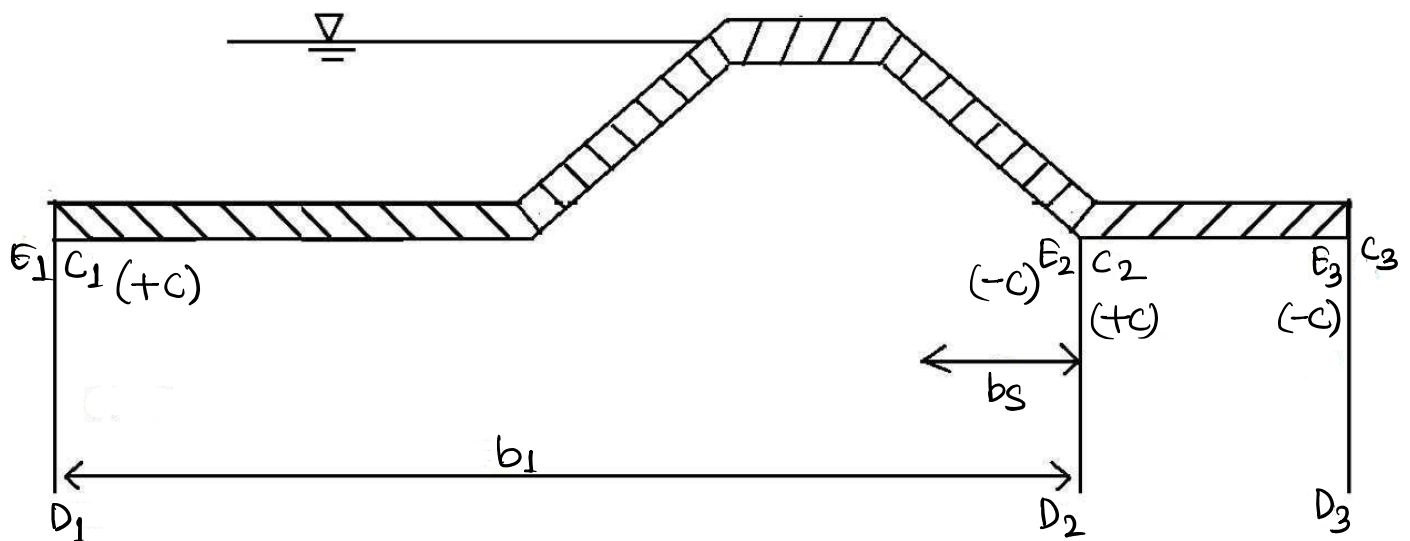
- D is to be Measured below the level of which interference is desired.

b' = The distance between the two Piles.

b = Total floor length

d = The depth of pile On which the effect is to be Considered.

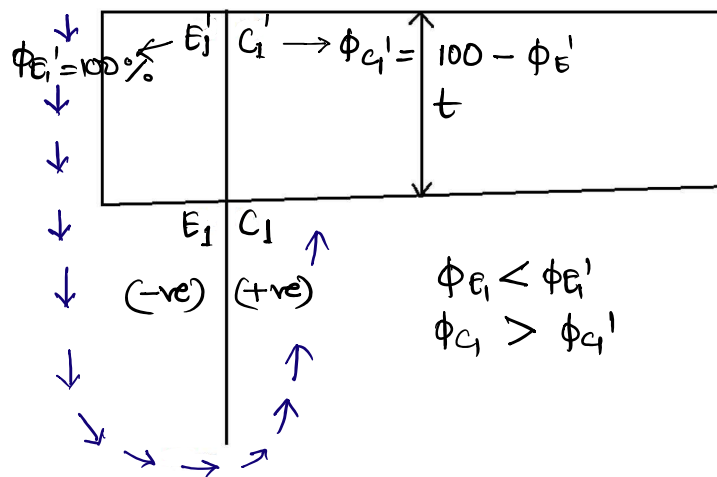
- This Correction is +ve for the Points in the rear or back water and -ve for the points forward in the direction of flow.



Note :- The Correction due to interference does not apply for intermediate Pile, if this Pile is equal or smaller than the Outer pile and is at a distance less than twice the length of the outer pile.

② CORRECTION FOR THE THICKNESS OF FLOOR

- In Standard Profile, the floor is assumed to have negligible thickness. hence, the % Pressure given by Khosla's theory is for the top level of the floor.
- But in Actual Junction Points E and C are at the Bottom of floor.
- Hence, the Pressure at actual points are calculated by Assuming Straight line Pressure Variation.
- Since, the Corrected Pressure at E_1 Should be less than the Calculated Pressure the point ' E_1 ' Shall be -ve.
- Similarly, the Pressure Calculated at C_1 is less than Corrected Pressure at C_1 . hence, the Correction to be applied at C_1 is +ve.



③ CORRECTION FOR THE SLOPE OF FLOOR

- A Correction is applied for a Sloping floor and is taken +ve for down and -ve for the up slope, following the direction of flow.
- Values of Correction of Standard Slope are as follows -

Slope {H:V}	Correction Factor
1 : 1	11.2
2 : 1	6.5
3 : 1	4.5



Note: The Correction factored Mentioned above is to be multiplied by the horizontal length of slope and divided by the distance between two Pile line between which the Sloping floor is located.

Q. An Impervious floor of a Weir on Permeable Soil is 16 m long and has Sheet piles On Both Ends. The Upstream pile is 4 m deep and downstream is 5 m deep. The Weir Creates a net head of 2.5 m neglecting the thickness of the floor. Calculate the Pressure at the Junction of inner faces of the Pile with weir floor Using Khosla's theory.

Ans:

For Upstream Pile,

$$\phi_{C_1} = 100 - \phi_E$$

$$\phi_E = \frac{1}{\pi} \times \cos^{-1} \left\{ \frac{\lambda - 2}{\lambda} \right\}$$

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

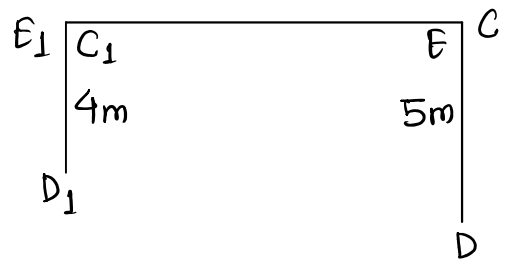
$$\alpha = \frac{b}{d} = \frac{16}{4} = 4$$

$$\lambda = \frac{1 + \sqrt{1 + 4^2}}{2}$$

$$\lambda = 2.56$$

$$\phi_E = \frac{1}{\pi} \times \cos^{-1} \left\{ \frac{2.56 - 2}{2.56} \right\} = 24.6$$

$$\phi_{C_1} = 100 - 24.6 = 75.43$$



Correction due to Downstream pile,

$$C = 19 \times \sqrt{\frac{D}{b}} \left[\frac{d+D}{b} \right]$$

$$\Rightarrow C = 19 \times \sqrt{\frac{5}{6}} \left[\frac{4+5}{16} \right]$$

$$= 6 \%$$

$$\phi_{C_1} = 75.45 + 6 = 81.38 \%$$

$$h = \frac{2.5 \times 81.38}{100} = 2.02 \text{ m}$$



For Downstream pipe,

$$\phi_E = \frac{1}{\pi} \cos^{-1} \left\{ \frac{\lambda - 2}{\lambda} \right\}$$

$$\alpha = \frac{18}{5} \\ = 3.2$$

$$\lambda = \frac{1 + \sqrt{1 + 3.2^2}}{2} \\ = 2.17$$

Correction due to Upstream Pile,

$$c = -19 \times \sqrt{\frac{4}{16}} \left[\frac{5+4}{16} \right] \\ = -5.34 \%$$

$$\phi_E = 27.1 - 5.34 \\ = 21.75 \%$$

$$h = \frac{21.75}{100} \times 2.5 \\ = 0.554 \text{ m.}$$

Q. The Concrete floor of a head regulator is level with the Channel bed and is 13 m long the floor is provided with cutoff wall at its Upstream and Downstream ends. The depth of Upstream wall is 1.5 m and that of Downstream wall is 2 m. Use Khosla theory to determine the thickness of the floor at its mid-length and also its Junction with the upstream and Downstream Cutoff Walls, The floor thickness may not be less than 30 cm anywhere. The Upstream FSL is 1.5 m above the floor level. Also Check Safety against Piping failure, if Permissible Exit Gradient is 0.18.

Ans:

① For Upstream Cutoff Wall

$$\phi_{c1} = 100 - \phi_E$$

$$\phi_E = \frac{1}{\pi} \cos^{-1} \left[\frac{\lambda - 2}{\lambda} \right]$$

$$\alpha = \frac{b}{d} = \frac{13}{1.5} \\ = 8.67$$

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} \\ = \frac{1 + \sqrt{1 + 8.67^2}}{2}$$



$$\lambda = 4.86$$

$$\Rightarrow \phi_E = \frac{1}{\pi} \times \cos^{-1} \left[\frac{4.86 - 2}{4.86} \right]$$

$$= 17.16 \%$$

$$\Rightarrow \phi_{C_1} = 100 - 17.16$$

$$= 82.84 \%$$

Correction for Downstream Pipe,

$$C = 19 \times \sqrt{\frac{D}{b_1}} \left[\frac{d+D}{b} \right]$$

$$= 19 \times \sqrt{\frac{12}{13}} \left[\frac{1.5+2}{13} \right]$$

$$= 2$$

$$\phi_{C_1} = 82.84 + 2 \%$$

$$= 84.84 \%$$

$$h = 84.84 \% H$$

$$= \frac{84.84}{100} \times 1.5$$

$$h = 1.27 \text{ m}$$

$$t = \frac{h}{(G-1)}$$

$$= \frac{1.27}{2.4-1}$$

$$t = 0.9 \text{ m} > 30 \text{ cm}$$

② Downstream pile,

$$\phi_E = \frac{1}{\pi} \times \cos^{-1} \left[\frac{\lambda - 2}{\lambda} \right]$$

$$\alpha = \frac{13}{2} = 6.5$$

$$\lambda = \frac{1 + \sqrt{1 + 6.5^2}}{2}$$

$$\lambda = 3.78$$



$$\begin{aligned}
 \phi_E &= \frac{1}{\pi} \times \cos^{-1} \left[\frac{\lambda - a}{\lambda} \right] \\
 &= \frac{1}{\pi} \times \cos^{-1} \left[\frac{3.78 - 2}{3.78} \right] \\
 &= 19.66 \%
 \end{aligned}$$

Correction due to Upstream pile,

$$\begin{aligned}
 c &= -19 \sqrt{\frac{D}{b'}} \left\{ \frac{d+D}{b} \right\} \\
 &= -19 \sqrt{\frac{1.5}{13}} \times \left[\frac{2+1.5}{13} \right] \\
 &= -1.73
 \end{aligned}$$

$$\begin{aligned}
 \phi_E &= 19.66 - 1.73 \\
 &= 17.92 \%
 \end{aligned}$$

$$\begin{aligned}
 h &= 17.92 \% H \\
 &= \frac{17.92}{100} \times 1.5
 \end{aligned}$$

$$h = 0.27 \text{ m.}$$

$$t = \frac{0.27}{2.4 - 1}$$

$$= 19.2 \text{ cm} < 30 \text{ cm}$$

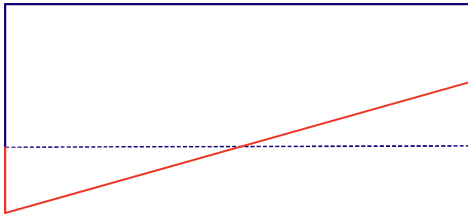
$$h_{\text{mid}} = \frac{h_u + h_D}{2}$$

$$t_{\text{mid length}} = \frac{t_u + t_D}{2}$$

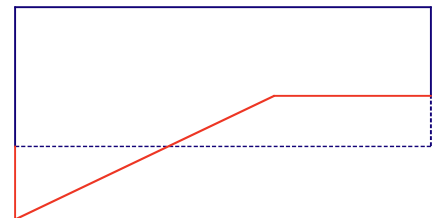
$$= \frac{90 + 19.2}{2}$$

$$= 54.6 \text{ cm} > 30 \text{ cm.}$$





Calculated



Actual

$$\textcircled{\text{iii}} \quad i_E = \frac{H}{d} \times \frac{1}{\pi \times \sqrt{\lambda}}$$

$$\alpha = \frac{b}{d} = \frac{13}{2} = 6.5$$

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

$$= \frac{1 + \sqrt{1 + 6.5^2}}{2}$$

$$= 3.78$$

$$i_E = \frac{1.5}{2} \times \frac{1}{3.14 \times \sqrt{3.78}}$$

$$= 0.122 < 0.18$$

The floor is Safe Against piping.



PREVIOUS YEAR QUESTIONS

ESE

Q. Which One of the following is not the major factor influencing Seepage from a Canal.

- (a) Characteristics of the Soil traversed by the Canal System.
- (b) Area Wetted by the Canal.
- (c) Location of the Canal.
- (d) Frequencies of Canal Usage.

[2020]

Ans:

© Location of the Canal.

Q. Khosla's formulae for assessing pressure distribution Under Weir floors are based on

- (a) Potential flow in Permeable layers just beneath the floor.
- (b) Boundary layer flow with the pressure drop longitudinally.
- (c) Conformal transformation of potential flow into the w plane.
- (d) Simplification of 3-D flow.

[2018]

Ans:

© Khosla's formula for assessing pressure distribution Under Weir floors are based on Conformal transformation of Potential flow into w-plane.

Q. A 20 m long horizontal concrete floor Under a barrage On a Permeable foundation retains a 5 m head of Water and has a 5 m deep downstream end pile. The exit gradient is

- (a) 1 in 4
- (b) 1 in 5
- (c) 1 in 6
- (d) 1 in 8

[2016]

Ans:

© 1 in 5

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

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

$$\alpha = \frac{b}{d} = \frac{20}{5} = 4$$

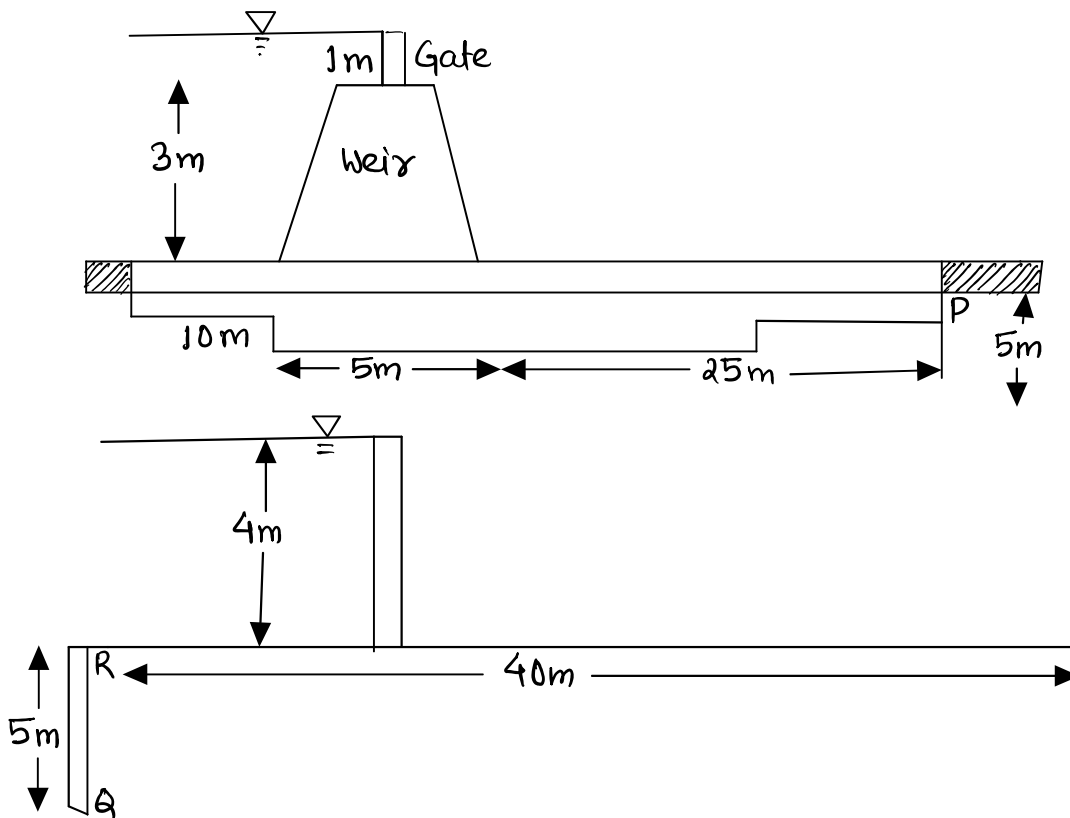
$$\lambda = \frac{1 + \sqrt{1 + 4^2}}{2} = 2.56$$

$$G_E = \frac{5}{5} \times \frac{1}{3.14 \times \sqrt{2.56}}$$

$$G_F = \frac{1}{5}$$

GATE

Q. Profile of a weir on permeable foundation is shown in figure I and an elementary profile 'Upstream pile Only Case' according to Khosla's theory is shown in fig. II. The Uplift Pressure heads at key points A, R and S are 3.14 m, 2.75 m and 0 m respectively (refer fig. II).



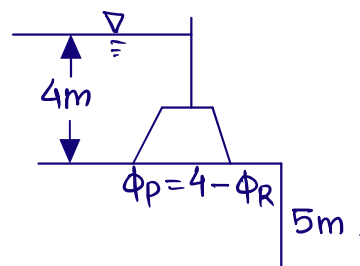
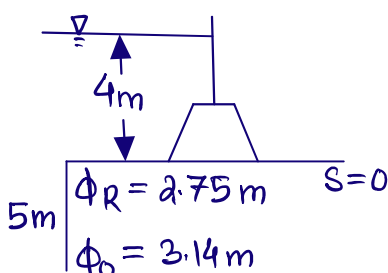
What is the Uplift Pressure head at point P downstream of the weir (Junction of floor and pile as shown in the figure I) ?

- (a) 2.75 m
- (b) 1.25 m
- (c) 0.8 m
- (d) Data Not Sufficient

[2016, Shift -II]

Ans :

(b) 1.25 m

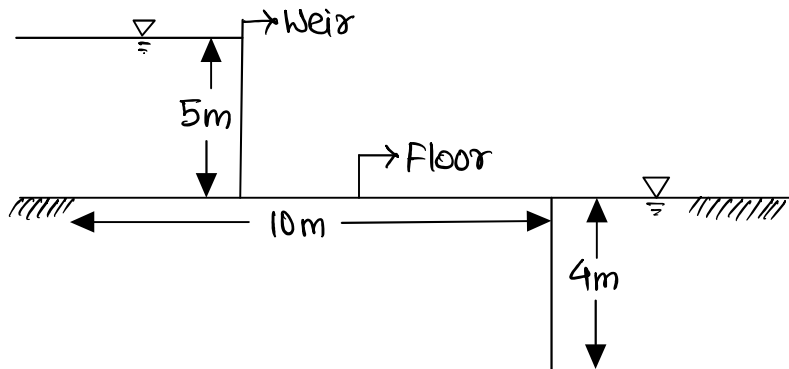


As the pile on D/s is of same length of U/s hence,

$$\phi_p = 4 - \phi_R$$

$$\Rightarrow \phi_p = 4 - 2.75 \\ = \underline{1.25 \text{ m}}$$

Q. A Weir on a permeable foundation with downstream 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

[2008]

Ans:

(c) 1 in 3.4

— As per Khosla's Method,

$$\text{Exit gradient, } G_E = \frac{H}{d} \cdot \frac{1}{\pi \sqrt{\lambda}}$$

$$\lambda = \frac{1}{2} \left[1 + \sqrt{1 + \alpha^2} \right]$$

$$\alpha = \frac{b}{d}$$

$$\alpha = \frac{10}{4} = 2.5$$

$$\lambda = \frac{1}{2} \left[1 + \sqrt{1 + 2.5^2} \right] = 1.846$$

$$G_E = \frac{H}{d} \times \frac{1}{\pi \sqrt{\lambda}} \\ = \frac{5}{4} \times \frac{1}{\pi \sqrt{1.846}} = \frac{1}{3.41}$$

Q. Uplift Pressure at Point E and D (Fig. A) of a straight horizontal floor of negligible thickness with a sheet pile at downstream end are 28% and 20% respectively. If the sheet pile is at upstream end of the floor (Fig. B) the uplift pressures at points D₁ and C₁ are

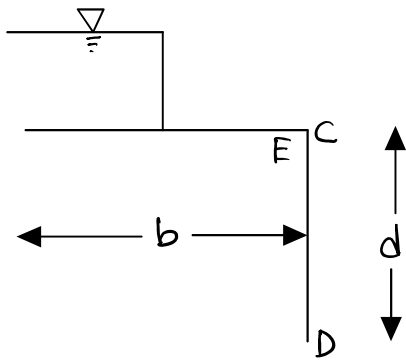


Fig. A

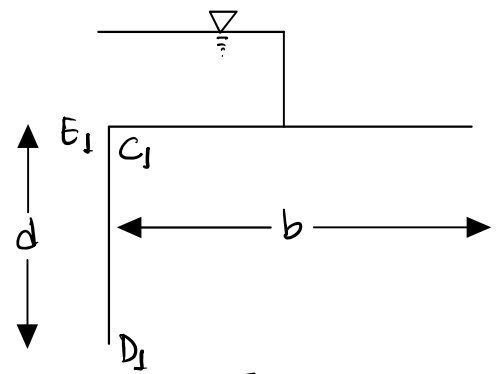


Fig. B

- Ⓐ 68 % and 60 % respectively.
- Ⓑ 80 % and 72 % respectively.
- Ⓒ 88 % and 70 % respectively.
- Ⓓ 100 % and Zero respectively.

[2005]

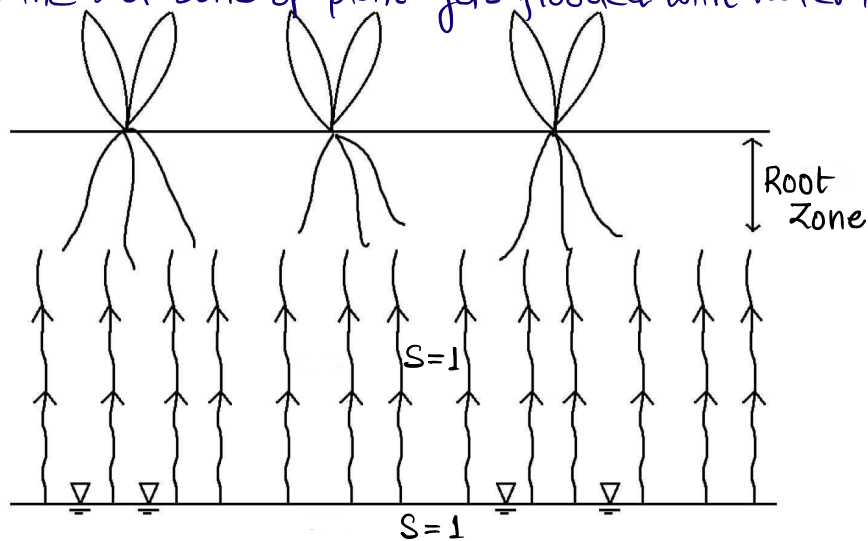
Ans: Ⓑ 80 % and 72 % respectively.
 — Uplift Pressure When Sheet pile is at Upstream
 $= 100 - \text{Uplift Pressure When Sheet pile is at D/S.}$

$$\begin{aligned} \text{Uplift Pressure at } D_1 &= 100 - \text{Uplift Pressure at D} \\ &= 100 \% - 20 \% \\ &= 80 \% \end{aligned}$$

$$\begin{aligned} \text{Uplift pressure at } C_1 &= 100 - \text{Uplift pressure at E} \\ &= 100 \% - 28 \% \\ &= 72 \% \end{aligned}$$

RECLAINATION OF WATER LOGGED AND SALINE SOIL

- An Agricultural land is said to be Waterlogged when its productivity gets effect by high water table.
- The Productivity of this land gets effected when the root zone of Plant gets Effects When the root zone of plant gets flooded with water in following ways.



① Plants requires nutrients like N, P, S in Specific form, to be Consumed by them $[NO_3^-, PO_4^{3-}, SO_4^{2-}]$ Which are formed during Oxidation reaction by aerobic Microorganisms.

- If the root Zone of the plant is flooded by water, voids in it get Chocked, thereby inhibiting entry of air (O_2) into it hence avoiding the formation of Usuable form of Nutrients thus, Effecting the growth of plant.



- ② Certain Waterlogging plants, the grass, weed grow in Waterlogged area thus effects the growth of the Crop.
- ③ The Normal Cultivation Operation like filling, ploughing Cannot be done in Water logged Area.
- ④ Water logging also leads to **SALINITY**

- If Plant root happens to come in Capillary Zone/fringe it absorbs the Water continuously for its growth.

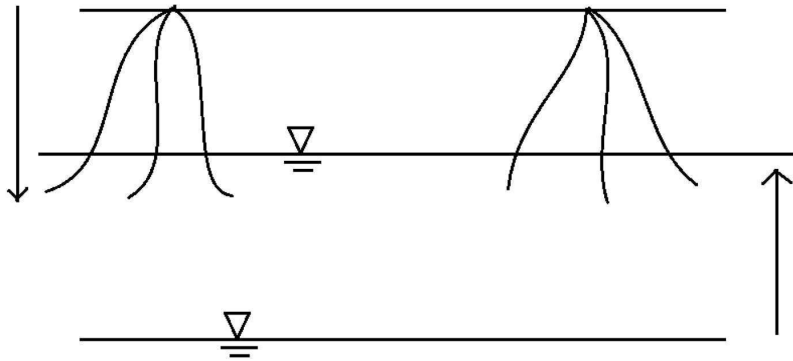


— Along with this water, salts also rises upto root zone, concentration of which increases over the period of time, which effects the osmotic activity of plants, thereby reduces plant yield. { This type of soil is termed as Saline Soil }.

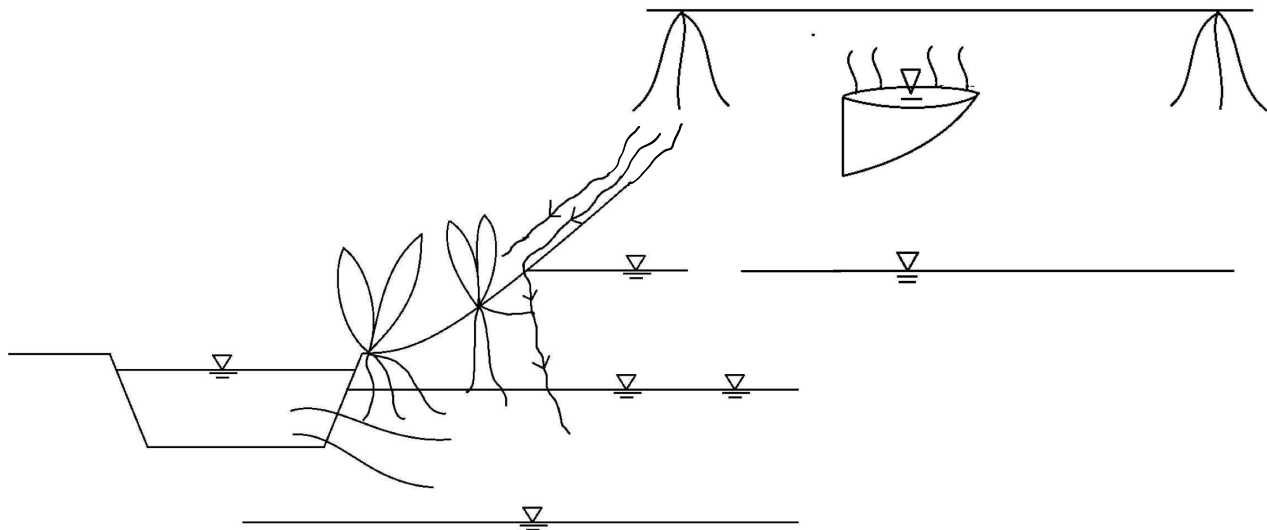
• • CAUSES OF WATERLOGGING

① Over And Intensive Irrigation

② Impervious Obstruction.



③ Seepage of Water from adjoining high land and through Canal.



④ Inadequet Surface drainage.

⑤ Excessive Rainfall

⑥ Topography { Plains }

• • PREVENTION/CONTROL OF WATER LOGGING

① Lining of Canals and Water Course.

② Adopting Extensive irrigation.

③ By Providing Efficient drainage System { Surface and Subsurface drains }.



iv) By Adopting Crop Rotation.

v) By Adopting Highly Efficient Irrigation Technique.

•• RECLAMATION OF SALINE AND ALKALINE SOIL

- It is the process by which an Unculturable Soil is made fit for Cultivation.
- Saline and water logged land gives very less Crop Yielding and are Unfit for Cultivation.
- A Water logged area Over the Period of time becomes Saline.
- Hence to reclaim Saline Soil firstly Water logging is to be Avoided by Method Which is discuss early.
- Several type of Salts are Present in Soil Some of them are beneficial for Plants While most them are injurious for growth of Plant.

Ex : $\rightarrow \text{Na}_2\text{CO}_3, \text{Na}_2\text{SO}_4, \text{NaCl}$ {also termed as alkali Soil}

$\text{Na}_2\text{CO}_3 > \text{Na}_2\text{SO}_4 > \text{NaCl}$ {Order of their harmful effect}.

Note : Na_2CO_3 is also termed as Black alkali as it dissolve some decomposed Organic Constituents of Soils in it.

- These Salts rises upto root Zone and Over the Surface of field and form a layer of thickness 5-7.5 m, by process termed as Efflorescence.
- Reclamation of Salts affected Soil is termed as '**LEACHING**'.
- In this process the land is flooded with adequate depth of Water, due to which alkali Salts present in Soil get dissolved in it and joins the Water table or drainage System.
- This process is repeated till the Salts in top layer of land reduces upto an extent Some Salt resistant Crop Can be grown like fodder, bajara etc.
- Over the period of time {two Seasons} the land becomes fit for Cultivation of all types of Crop, it is termed as Reclaimed land.
- When Sodium Carbonate (Na_2CO_3) is Present in Saline Soil, Gypsum is generally added to Soil before leaching and thoroughly mixed with Water.



- In Order to maintain equilibrium, Salinity of a given Soil and to avoid any further increases in its Salinity, the amount of Water additionally Required above Consumptive Use is termed as Leaching Requirement.

$$L.R = \frac{\text{Depth of Water drained out Per Unit area}}{\text{Depth of Irrigation water Applied Per Unit Area}}$$

$$L.R = \frac{D_d}{D_i}$$

$$D_i = C_u + D_d$$

$$D_d = D_i - C_u$$

$$L.R = \frac{D_i - C_u}{D_i} \quad **$$

$L.R$ can also be expressed in terms of Concentration of Salts in Irrigation Water (C_i) and drained out Water (C_d).

$$D \propto \frac{1}{C}$$

$$L.R = \frac{D_d}{D_i}$$

$$= \frac{C_i}{C_d}$$

$$L.R = \frac{EC_i}{EC_d}$$

Q. Estimate the leaching Requirement When 'EC' Value of a Saturated Extract of Soil is $10 \mu\text{mho/cm}$ at 25% reduction in the Yield of Crop. The EC of Irrigation water $1.2 \mu\text{mho/cm}$. What will be the requirement depth of Water to be Applied if the C_v of Crop is 80 mm, EC Value of Leaching water is assumed to be two times of that of Saturated Soil.

Ans :-

$$LR = \frac{D_i - D_u}{D_i}$$

$$= \frac{EC_i}{EC_d}$$

$$= \frac{EC_i}{2 \times EC_s}$$



$$= \frac{1.2}{2 \times 10} \times 100$$

$$= 6\%$$

$$LR = \frac{D_d}{D_i}$$

$$= \frac{D_i - C_u}{D_i} \times 100$$

$$\Rightarrow 0.06 = \frac{D_i - 80}{D_i}$$

$$\Rightarrow D_i = 85.1 \text{ mm.}$$

•• In Order to avoid Water logging Efficient Drainage System is to be Provided

— This System is of two types :-

- (i) Surface Drainage
- (ii) Sub-Surface Drainage.

(i) SURFACE DRAINAGE : It is the removal of Excess rain water falling On the field or Excess irrigation water applied to the field by the Use of Open ditches or Drains.

— Surface drains are of two types —

- (a) Shallow Surface
- (b) Deep-Surface Drains {French Drains}

— Shallow Surface drains Collect/Utilize the Water from Surface Whereas Deep Surface drains Utilizes Sub-Surface water from tile drain.

(ii) SUB-SURFACE DRAINAGE : These are the drains Required for Soil with Poor internal drainage and a high water table level.

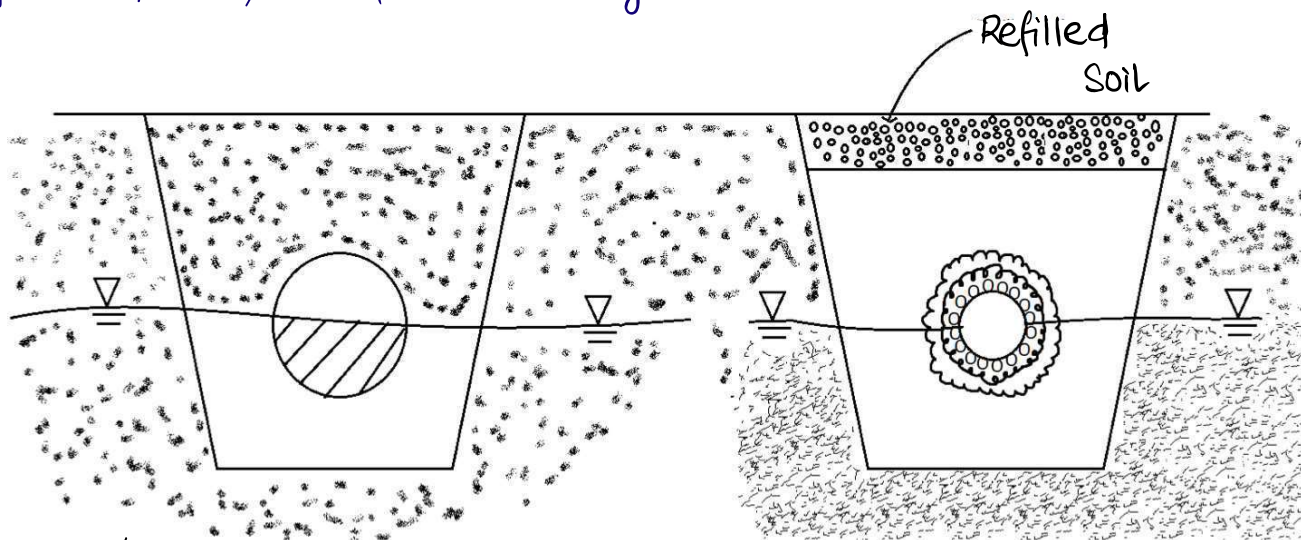
— Sub-Surface/Tile drains Offers Several advantage Over Surface drains.

- (a) Increases Air Circulation
- (b) Reduces Soil Erosion
- (c) It Reduces interference with Cultivation Operation.
- (d) It Permits deeper root Zone hence gives higher Yield.



•• DESIGN ANALYSIS OF TILE DRAINS

- Tile drains are Usually Made up of Porous Earthenware and are Circular in Section.
- The diameter of these drains may Vary from 10 - 30 cm (minimum diameter is in range of 10 - 15 cm)
- These are laid on a Certain longitudinal Slope Varying 0.05 - 3% .
- The Closed drains are generally Spaced at Such a distance as to be Capable to lower the water table level Sufficiently below the root Zone of the plants.
- For Most of the plant the top point of the water table must be atleast 1-1.5m below the ground level. {However it Varies from plant to plant and is up to 0.7 - 2.5 m}.
- The tile drains may be placed at about 0.3 m below the desired height level.
- As far as Possible these drains Should not be placed below less Pervious Soil as in that Case it may remains dry, even in the Case soil above water logged.
- In Such Case, if drains are to be Provided, They must be Surrounded by graded filters / Envelope filter having minimum thickness of 7.5 cm.

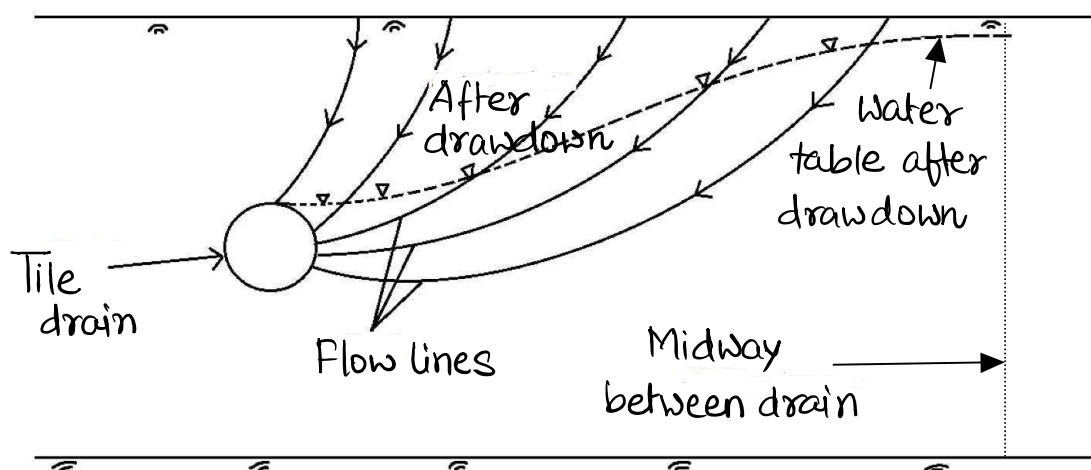
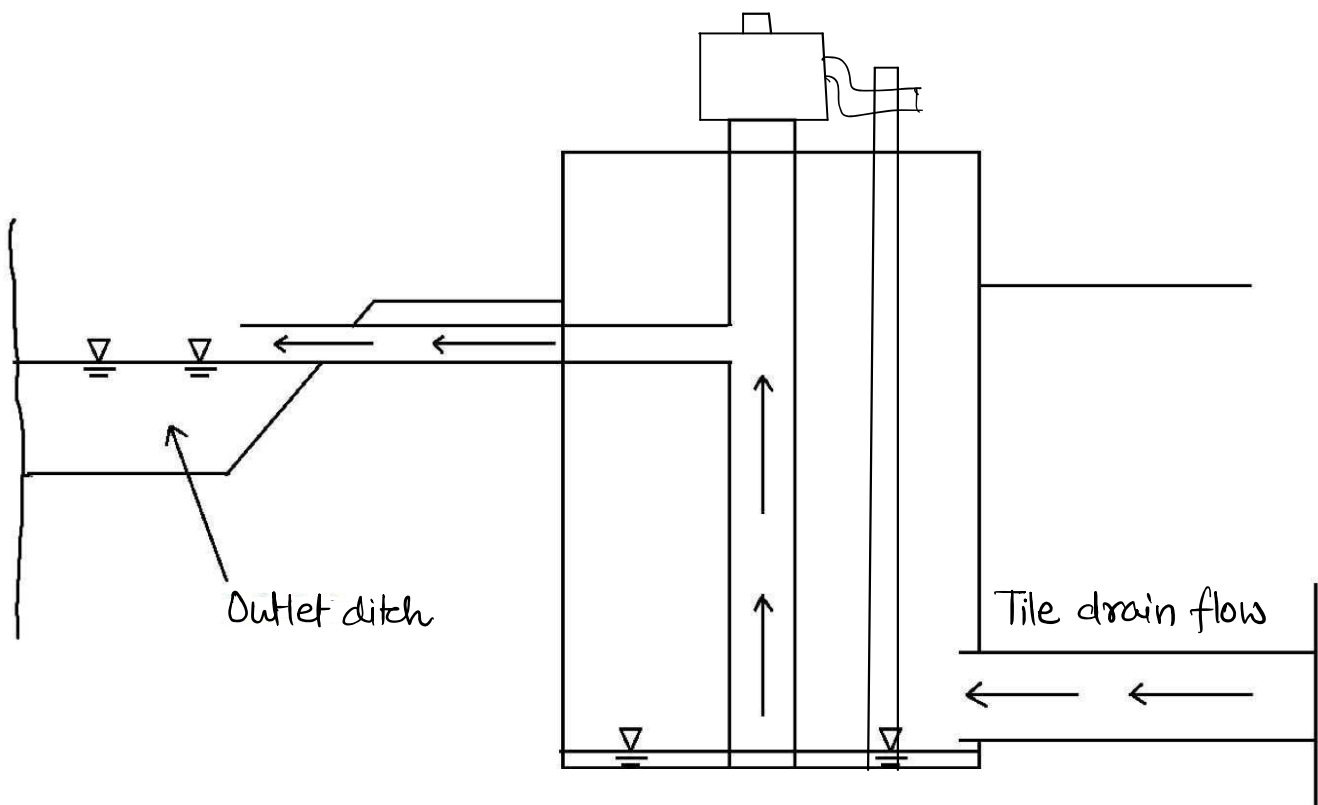
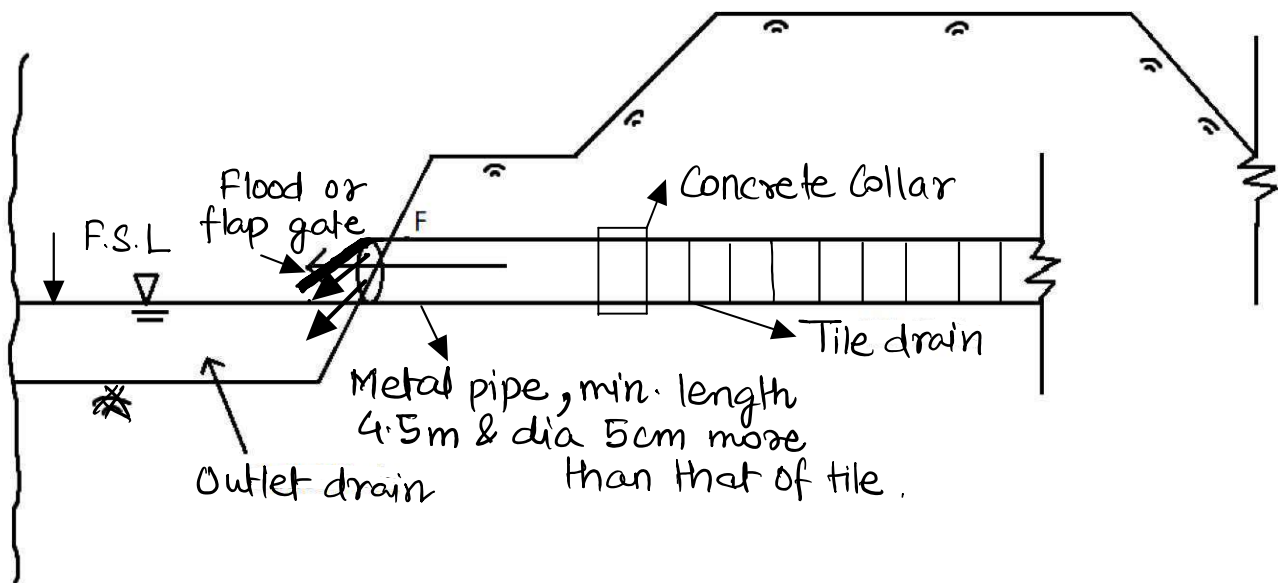


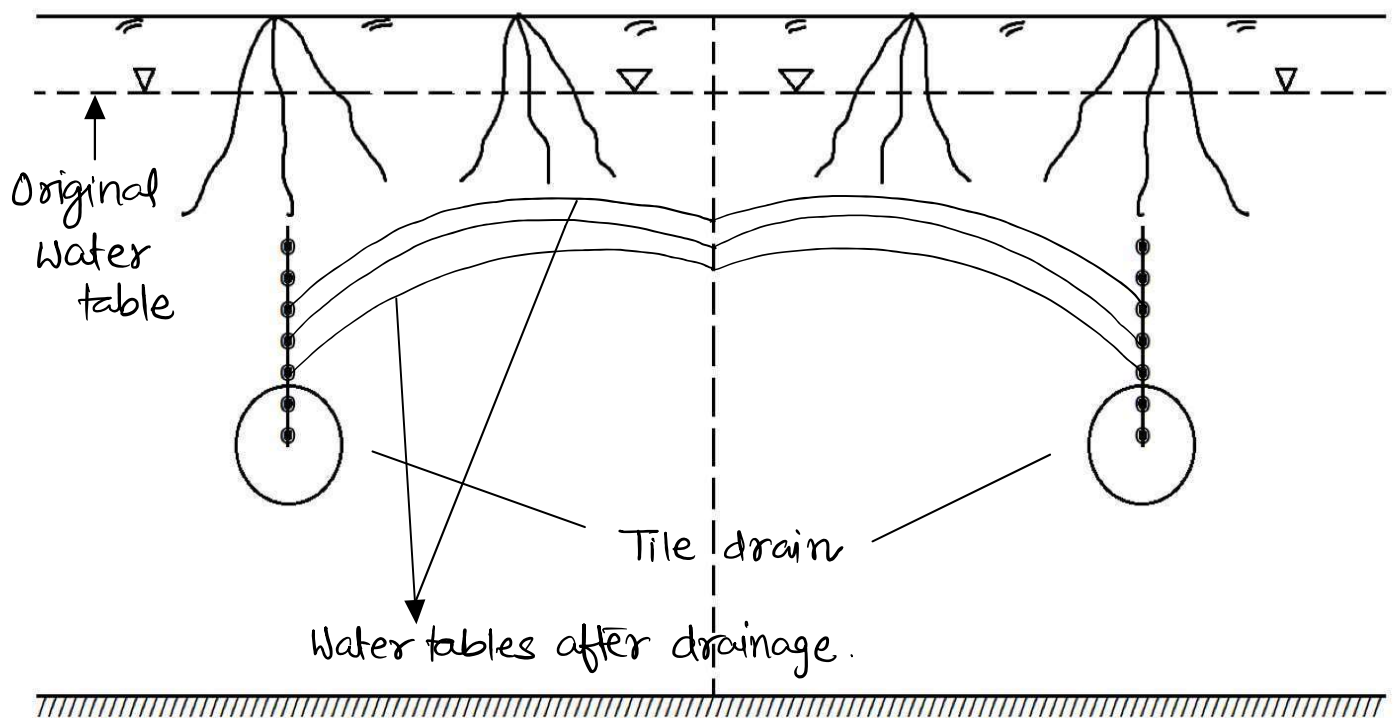
C/s of the tile drain in Pervious Soil without any filter.

Graded filter X- Section of tile drain in less Pervious Soil with Graded filter.

- The Water drained by tile drain is discharge into Some bigger drains called deep Surface drains.
- The Water from the tile drains may be discharged into an outlet drain also by gravity or by Pumping.

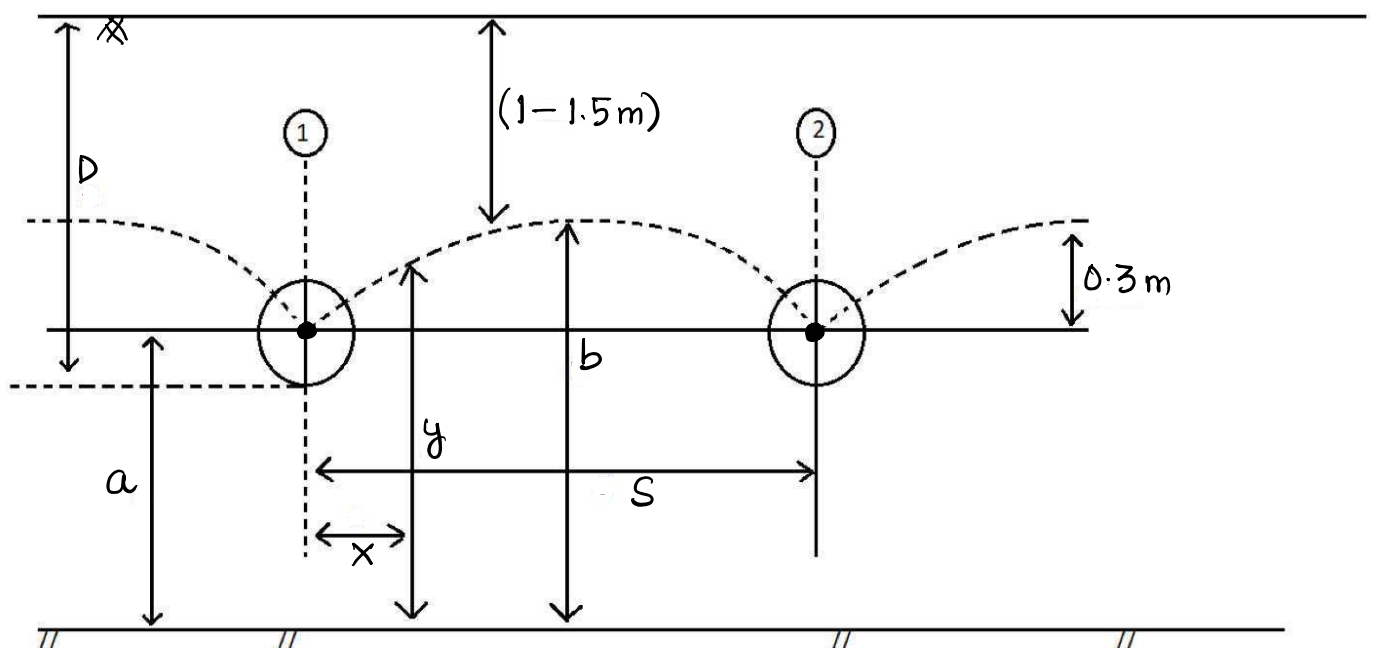






- Let 'S' be the Spacing between the drains and 'a' be the depth of Impervious Stratum from the Centre of drains.
- The maximum height of the drained water table above the impervious layer be 'b'.
- let the length of tile drain be 1m.

$$\begin{aligned} \text{Discharge, } q_x &= a_x \cdot v_x \\ q_x &= (y \times 1) \times k \cdot i \\ &= (y \times 1) \times k \cdot \frac{dy}{ds} \end{aligned}$$



$$q_x = K \cdot \frac{dy}{dx} \cdot y \quad \text{————— ①}$$



$$x = \frac{s}{2} \quad , \quad q_x = 0$$

$$x = 0 \quad , \quad q_x = \frac{q}{2}$$

Assuming q is Inversly Proportional to the distance from the Centre of the drain.

$$q_x = \frac{q}{2} - \frac{q}{2} \cdot \frac{x}{(s/2)}$$

$$q_x = \frac{q}{2} \left[1 - \frac{2x}{s} \right]$$

$$q_x = \frac{q}{2s} [s - 2x] \quad \text{————— ②}$$

From equation ① and ② we get,

$$K \cdot \frac{dy}{dx} \cdot y = \frac{q}{2s} [s - 2x]$$

$$\int_{y=a}^{y=b} y \cdot dy = \frac{q}{2s} \int_{x=0}^{x=s/2} [s - 2x] dx$$

$$K \cdot \left[\frac{y^2}{2} \right]_a^b = \frac{q}{2s} \left[s \cdot x - \frac{2x^2}{2} \right]_0^{s/2}$$

$$\frac{K}{2} [b^2 - a^2] = \frac{q}{2s} \left[\frac{s^2}{2} - \frac{2s^2}{4} \right]$$

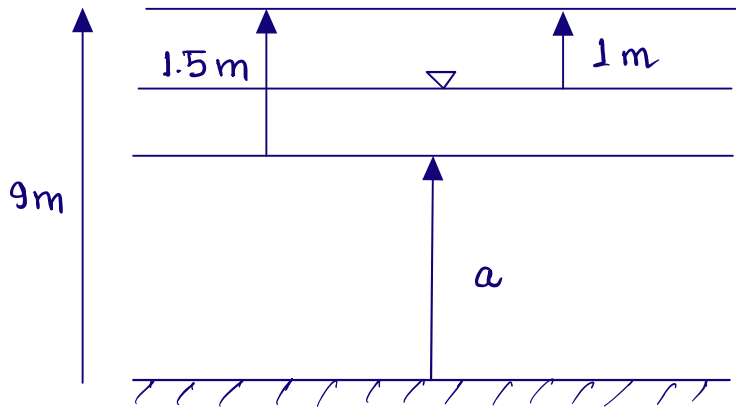
$$\frac{K}{2} [b^2 - a^2] = \frac{q}{4s} [s^2]$$

$$s = \frac{4K(b^2 - a^2)}{q} \quad **$$

Note : The Value of total discharge to be Carried by the tile drain is taken equal to 1 % of average Annual Rainfall to be drained in 24 hrs and this is termed as Drainage Coefficient.

Q. In a tile Drainage System, the drains are laid with their centres 1.5 m below the Ground level. The impervious layer is 9 m below ground level and the Annual Average rainfall in area is 80 cm if 1% of the annual Rainfall is to be drained in 24 hours to keep the highest Position of the water table to 1 m below the ground level. Determine the Spacing of the drain pipes. Use $K = 0.01 \text{ cm/sec}$.

Ans:



Here,

$$S = \frac{4K}{q}(b^2 - a^2)$$

$$b = 9 - 1 = 8 \text{ m}$$

$$a = 9 - 1.5 \\ = 7.5 \text{ m}$$

$$q = 1\% \text{ of } (80 \text{ cm} \cdot A) / 24 \text{ hr.}$$

$$= \frac{1}{100} \times 80 \times 10^{-2} \times S \times 1 \times \frac{1}{24 \times 60 \times 60}$$

$$q = (9.25 \times 10^{-8}) S \text{ (m}^3/\text{m/sec)}$$

$$S = \frac{4 \times 0.01 \times 10^{-2} (8^2 - 7.5^2)}{(9.25 \times 10^{-8}) S}$$

$$S = 183 \text{ m}$$

Q. Determine the Size of tile drain at the outlet of a 6 hectare drainage System, if the drainage Coefficient is 1 cm and the tile grade is 0.3%, $N = 0.011$.

Ans: $DC = 1 \text{ cm}$

$$q = 1 \times 10^{-2} \times 6 \times 10^4 \times \frac{1}{86400} \\ = 6.944 \times 10^{-3} \text{ m}^3/\text{sec.}$$



$$q = av = \frac{\pi}{4} \times D^2 \times \frac{1}{N} \left(\frac{D}{4} \right)^{2/3} \times S^{1/2}$$

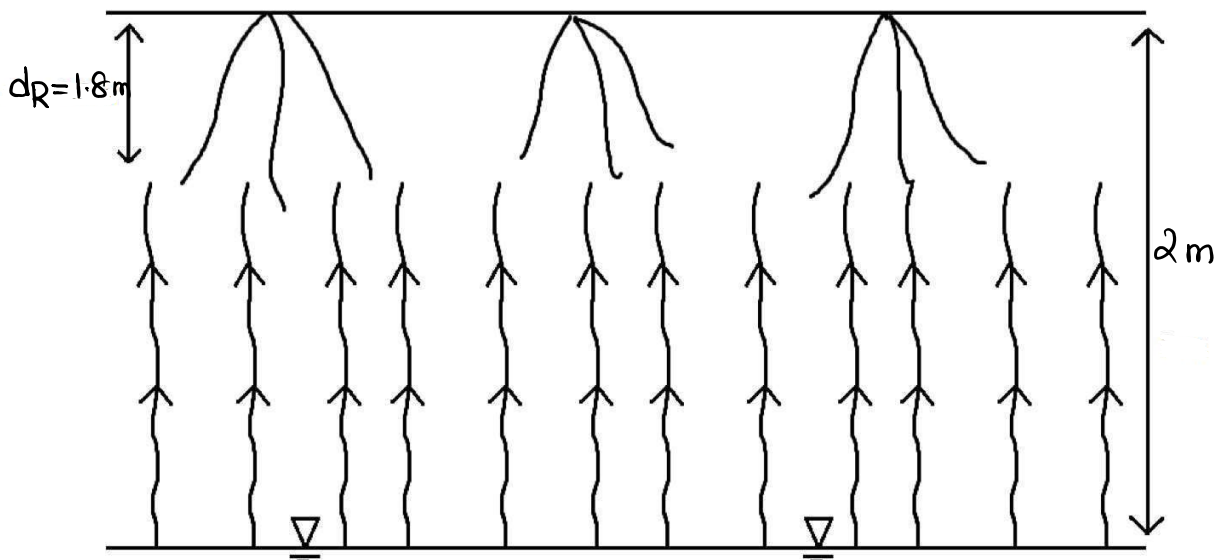
$$\Rightarrow 6.944 \times 10^{-3} = \frac{\pi D^2}{4} \times \frac{1}{0.011} \left(\frac{D}{4} \right)^{2/3} \left[\frac{0.3}{100} \right]^{1/2}$$

$$D = 13.15 \text{ cm}$$

Q. Sugarcane {root Zone depth 1.8 m} is grown in a Particular area, When the ground Water table 2m below the ground, if the Size of Soil pores is 0.08 mm in diameter and Surface Tension is 0.064 N/m, is the field Water logged.

— If So, determine the Vertical of Closed drains below ground, Spaced at 15 m. DC = 0.116 Cumec/Km², K = 10⁻⁶ m/sec and the impervious Stratum is 7m below the ground.

Ans:

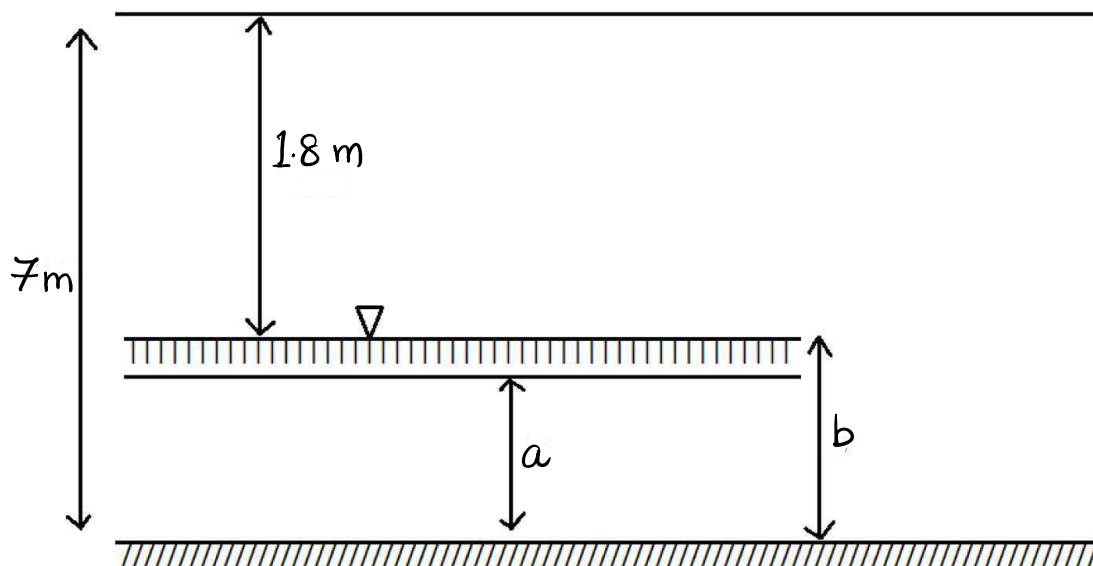


$$\begin{aligned} h_{cmax} &= \frac{4T}{\gamma_w \cdot d} \\ &= \frac{4 \times 0.054}{9810 \times 0.08 \times 10^{-3}} \\ &= 0.275 \text{ m.} \end{aligned}$$

Effective water table = 2 - 0.275
= 1.725 < 1.8 m below the Ground level hence field is Water logged.

$$b = 7 - 1.8$$





$$q = 0.116 \text{ cumec/Km}^2$$

$$q = 0.116 \times (15 \times 1) \times (10^{-3})^2$$

$$q = 1.74 \times 10^{-6} \text{ m}^3/\text{m}/\text{sec.}$$

$$15 = \frac{4 \times 10^{-6}}{1.74 \times 10^{-6}} \times \{5.2^2 - a^2\}$$

$$a = 4.53 \text{ m or } 7 - 4.53 \\ = 2.47 \text{ m from Ground.}$$



PREVIOUS YEAR QUESTIONS

ESE

- Q. Which One of the following is not the main Cause for Soil Salinity and Sodicity?
- (a) Irrigation mismanagement.
 - (b) Poor land levelling
 - (c) Use of heavy Machinery, resulting in no Soil Compaction.
 - (d) Leaching without adequate drainage.
- [2020]

Ans:

- (c) Use of heavy Machinery, resulting in no Soil Compaction.
— Main Cause of Soil Salinity and Sodility:
1. Irrigation Mismanagement.
 2. Leaching Without adequate Drainage.
 3. Poor land levelling.
 4. Deforestation.
 5. Accumulation of air borne or water borne Salts in Soil,

- Q. Consider the following data for a drain ;

$$L = 50 \text{ m}$$

$$a = 10 \text{ m}$$

$$b = 10.3 \text{ m and}$$

$$K = 1 \times 10^{-5} \text{ m/s.}$$

If the drains carry 1% of average annual rainfall in 24 hours, the average annual rainfall for which this system has been designed will be

- (a) 78 cm
- (b) 84 cm
- (c) 90 cm
- (d) 96 cm.

[2019]

Ans:

(b) 84 cm

$$— S = \frac{4K}{q} (b^2 - a^2)$$

$$\Rightarrow 50 = \frac{4 \times 10^{-5} \times (10.3^2 - 10^2)}{q}$$

$$\Rightarrow q = 4.872 \times 10^{-6} \text{ m}^3/\text{s.}$$

$$\Rightarrow q = \frac{1}{100} \times \alpha \times (S \times 1)$$
$$\frac{4.872 \times 10^{-6}}{8.64 \times 10^4}$$

$$\Rightarrow \alpha = \frac{0.841 \text{ m}}{\cong 84 \text{ cm.}}$$

- Q. Consider the following Statement regarding Water logging:

1. Water logging is the rise of groundwater table leading to possible increase in salinity resulting in a reduction in the yield of crops.

2. Water logging Cannot be eliminated in certain areas but Can be Controlled Only if the quality of Water percolating into that Soil is Checked and Reduced.

Which of the above Statement is/are Correct?

- (a) 1 Only.
- (b) 2 Only.
- (c) Both 1 and 2.
- (d) Neither 1 nor 2.

[2018]

Ans:

- (c) Both 1 and 2.

— Water logging Can be Controlled by Provision of efficient drainage to drain away the storm water and excess irrigation water by Use of Sub-Surface drainage, water logging Can be Controlled by Checking and removing Percolating water.

SSC-JE

- Q. Which One is the best method of reclamation of the alkaline Soil?

- (a) Adding of Gypsum to Soil Only.
- (b) Adding of Gypsum to Soil and Leaching.
- (c) Leaching Only.
- (d) Providing good drainage system.

[2017-Morning]

Ans:

- (b) Adding of Gypsum to Soil and Leaching

— Leaching — In this process, the land is flooded with adequate depth of water. The alkali salts present in the soil, gets dissolved in this water, which percolates down to join the water table or drained away by surface and subsurface drains. This process is known as Leaching.

— Reclamation — Land Reclamation is the process by which an uncultivable land is made fit for cultivation.

- Q. Which One is the best method of the reclamation of the acidic Soil?

- (a) Leaching
- (b) Provision of Good drainage.
- (c) Use Gypsum as a Soil amendment.
- (d) Use limestone as a Soil amendment

[2018-Morning]

Ans:

- (d) Use limestone as a Soil Amendment.

— The best method of the reclamation of the acidic soil is use limestone as a Soil amendment.

— Different lining material to reclamation of acid soil :

• Oxides — CaO

- Hydroxides — Ca(OH)_2
- Carbonate — CaCO_3
- Silicates of Calcium — CaSiO_3 .

Q. Which One of the following is not a Cause of Water Logging?

(a) Excess tapping of the ground water.

(b) Excess Rainfall.

(c) Frequent Irrigation.

(d) High Water table.

[2017-Evening]

Ans:

(a) Excess tapping of the Ground water.

RIVER, THEIR BEHAVIOUR, CONTROL AND TRAINING

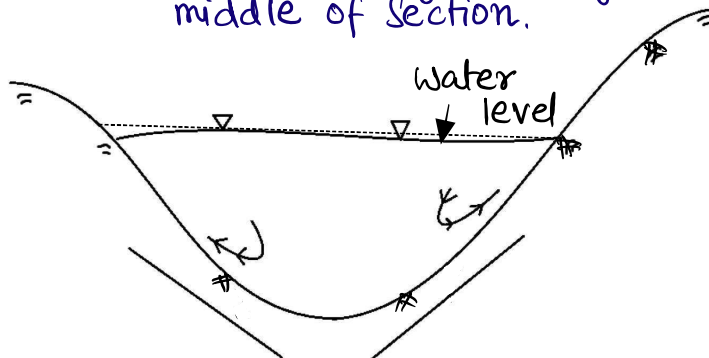
— The Chief factor which is responsible for the behaviour of river is 'SILT' which flows along the river water.

— The Silt in river water causes —

- (i) Increase of flood level.
- (ii) Silting of Reservoir.
- (iii) Silting of Irrigation Channels.
- (iv) Meandering of River.
- (v) Splitting up of River into Number of Channel.

— The Behaviour of river at Critical Points along its flow is as following

① STRAIGHT REACHES : In Straight reach of a River, Cross-Section is in Shape of trough with high Velocity of flow in the middle of Section.



— Since, Velocity of flow is more, depth of flow is comparatively less.

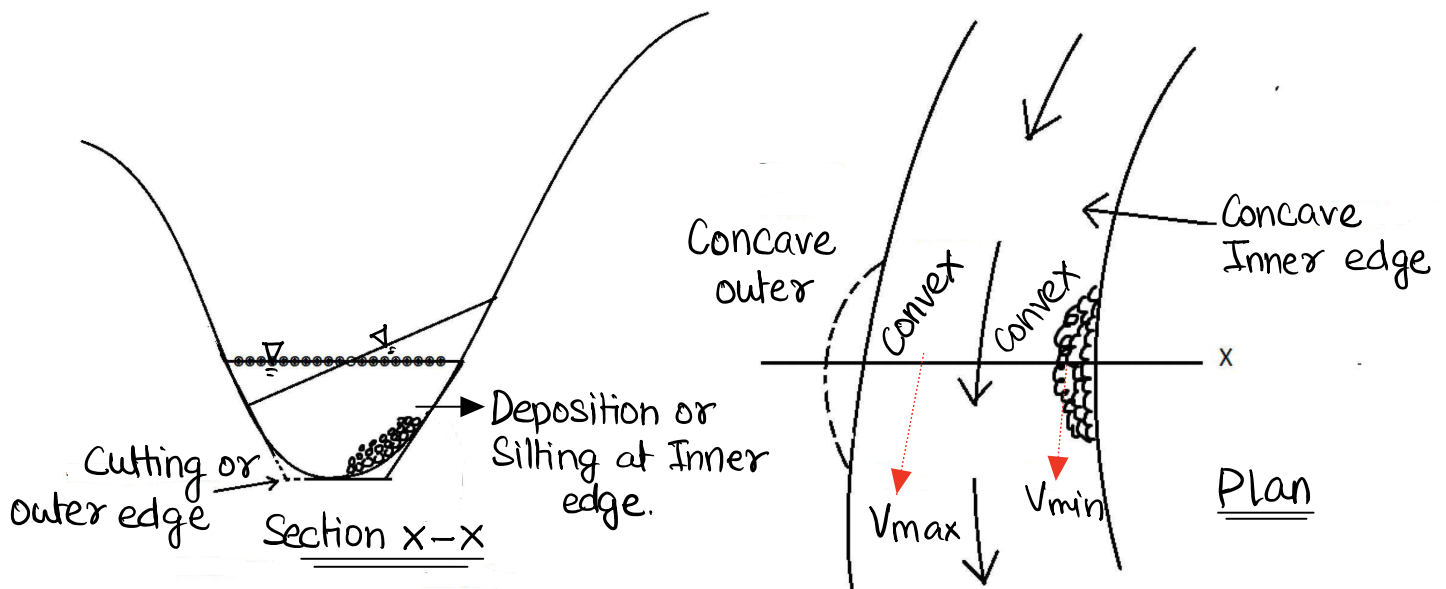
— Due to this existence of transverse gradient from Edges towards Centre, transverse Secondary flow get developed.

② BENDS : Every River in Alluvial Soil tends to develop the Bend which are characterised by Scouring on the Concave side and Silting on Convex Side.

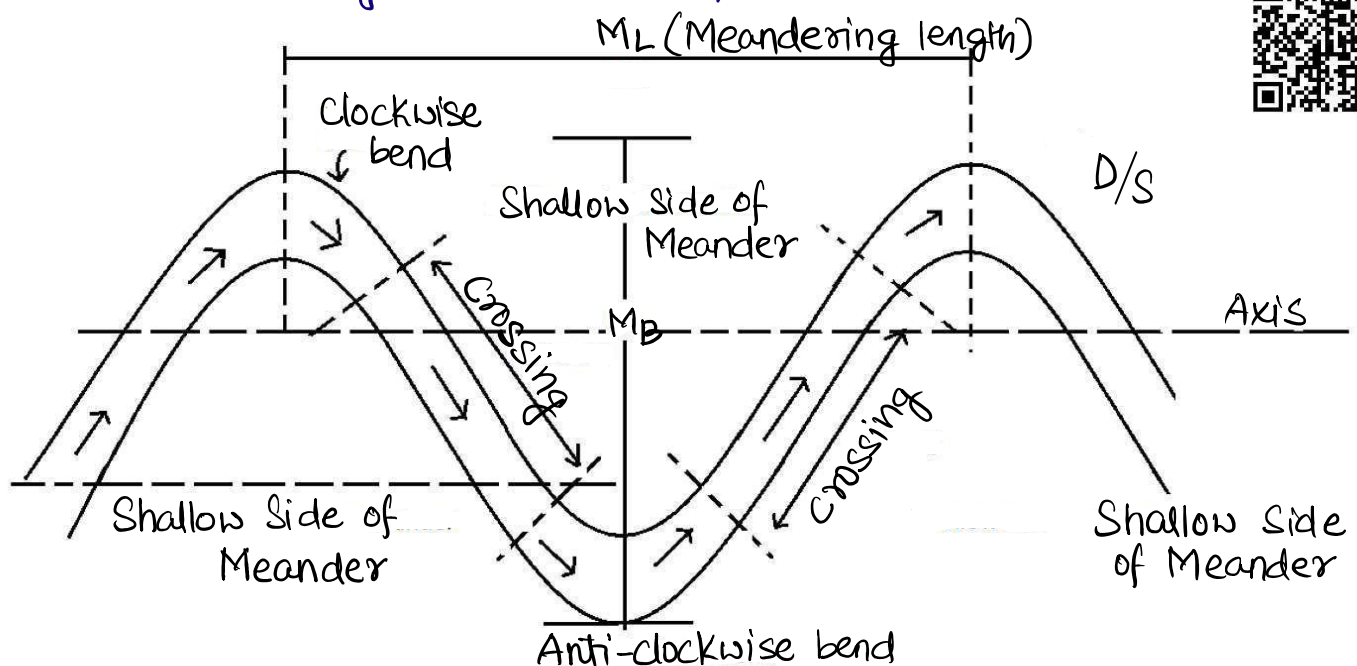
— The Silting and Scouring in the bend may Continue due to the action of Centrifugal force.

— This Silting & Scouring of Opposite Sides of river are independent of each other.

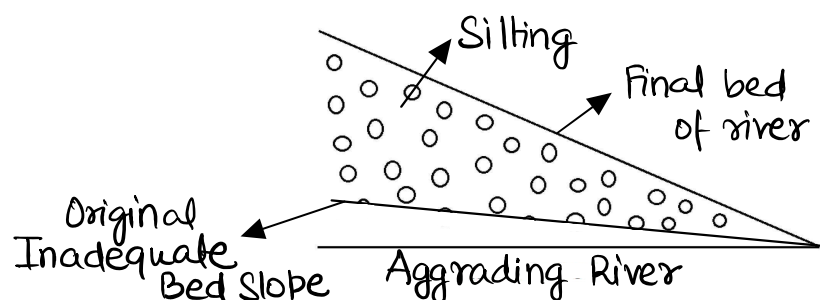
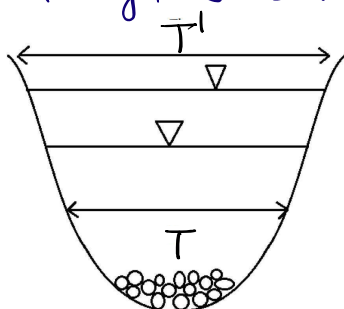


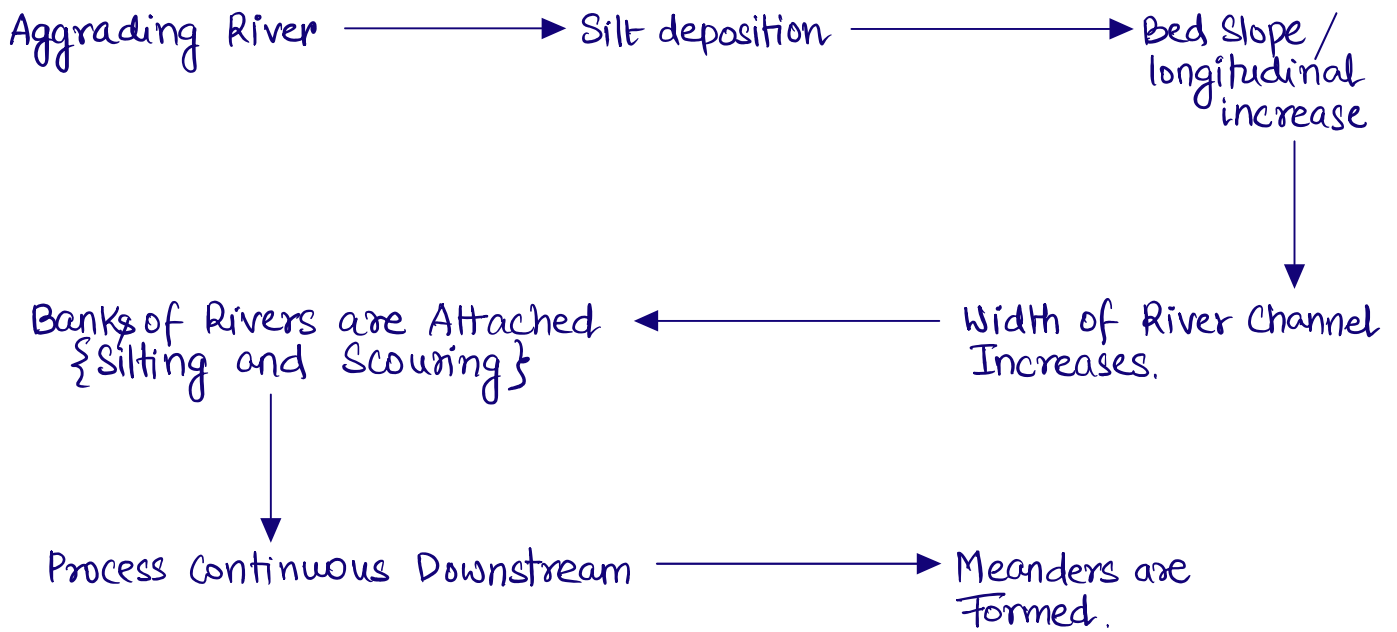


- MEANDERS: When Once a river deviates from its axial path and curvature is developed, the Process moves Downstream by building up of Silting Islands on the Convex Side which Results in further Shifting of Outer banks by Erosion.



- Formation of Successive bends of Reverse order is called Meander {S Curve}
- The River in which Meanders are Observed to be formed is termed as 'MEANDERING RIVER'.
- Meandering is Caused by the following Mechanism -





— The 4 Variables which governs the Meandering Process are as follows —

① Discharge

② Bed Slope

③ Bed Material and Side Material.

④ Silt Grade and Silt Charge.

— MEANDER LENGTH: It is the axial length of One Meander.

— MEANDER WIDTH/BELT:— It is the distance between the outer edges of Clockwise and anti-clockwise loop.

$$\text{Meander Ratio} = \frac{\text{Meander Width MB}}{\text{ML}}$$

— TORTUOSITY: It is the Ratio of length along the channel to the direct axial length.

— CROSSING/CROSSOVER:— The short straight reaches of river connecting two consecutive Clockwise and anticlockwise loop is called Crossing.

— CUTOFF:— In an Excessively Meandering river a Particular Bend may Sometime be abandoned by formation of straight and a short Channel is termed as Cutoff.



•• CONTROL AND TRAINING OF RIVER

—River training includes all works which are constructed on a river, so as to guide and confine the flow of river channel and to control and regulate river configuration so as to safely dispose of sediment load.

—River training is classified in following three forms :-

① High Water Training / Training for Discharge.

② Low Water Training / Training for Depth.

③ Mean Water Training / Training for Sediments.

① HIGH WATER TRAINING : High Water Training is undertaken with the primary purpose of flood control. It therefore aims on providing sufficient river cross-section for safe passage of flood, which is achieved by providing embankment on either side of river.

② LOW WATER TRAINING : Low water training is undertaken with purpose of providing sufficient depth for navigation during low water period, which is achieved by reducing width of river.

③ MEAN WATER TRAINING : Mean water training is undertaken to effectively dispose suspended load and bed load which is achieved by constructing groyne/spur.

—The following are generally adopted method of training of river —

① Marginal Embankment / Levees.

② Guide Banks.

③ Groyne/Spur

④ Artificial Cutoff

⑤ Pitching of Bank/launching Apron.

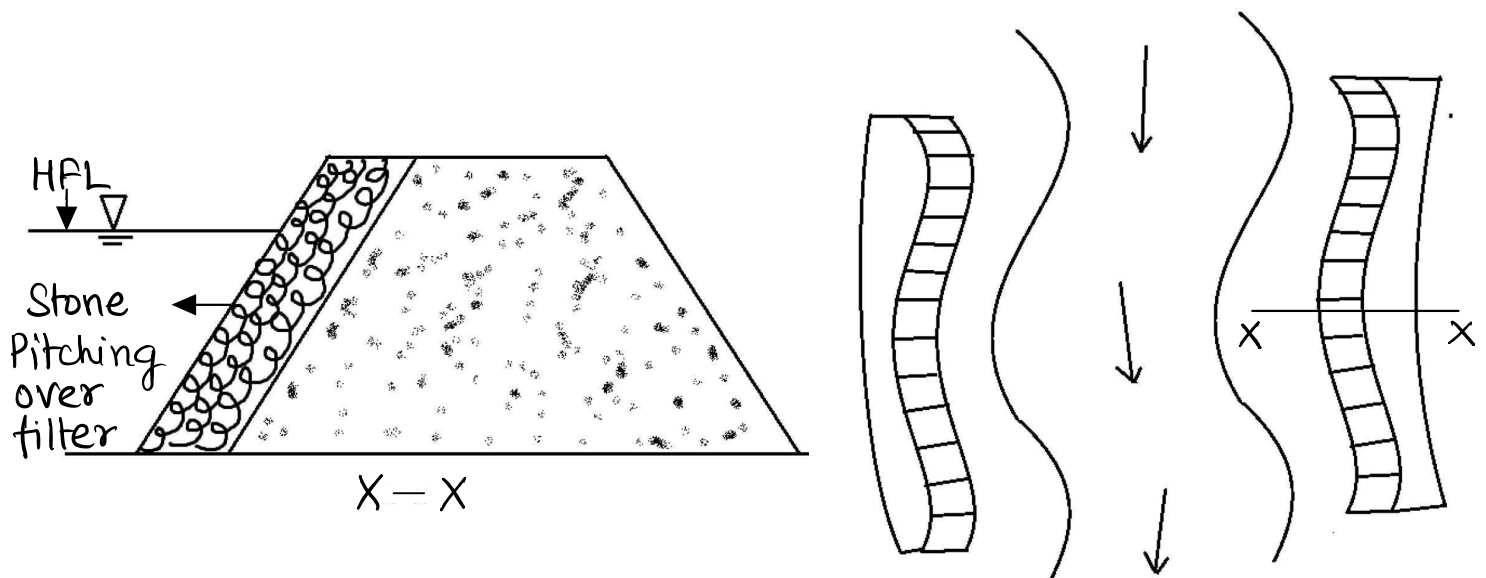
⑥ Pitched Islands

⑦ Sills, Bandalling etc.



① MARGINAL EMBANKMENT/LEVEES

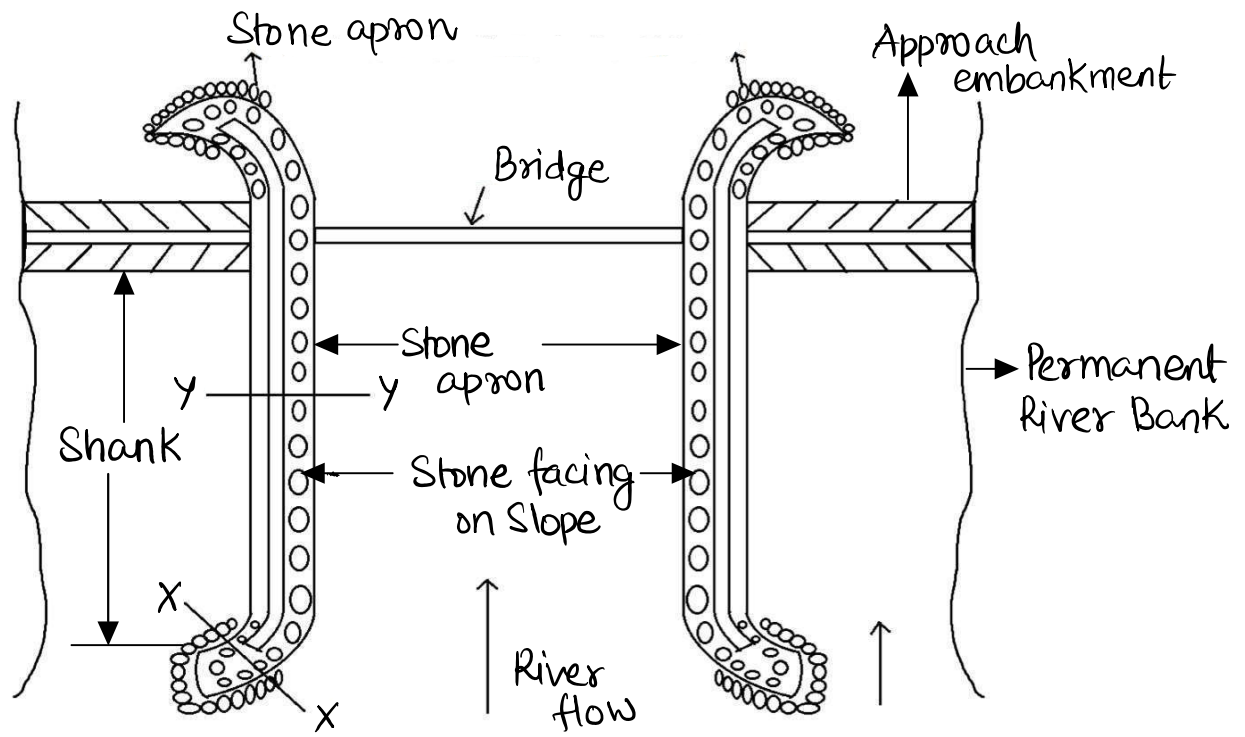
- These are generally Earthen Embankment, running Parallel to the River at Some Suitable distance from it.
- They may be Constructed On both Sides or on Only One Side of river for Some Suitable length When the River Enter or Passing Through towns, Villages, cities or any place of Importance.
- These Embankment wall Retain the flood water and thus preventing it from Spreading into the town.
- The Alignment of these levees follow the Normal meandering Pattern of the River.
- In Order to avoid the Erosion of these Embankment/Levees, Stone Pitching is also done on other Side.



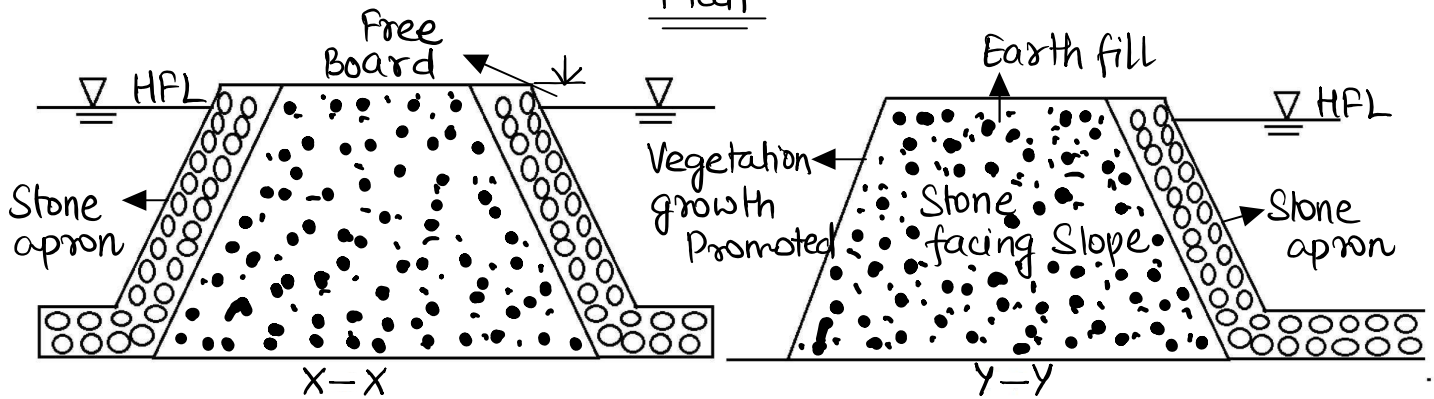
② GUIDE BANK

- If an Engineering Structure Such as Weir or Barrage or Bridge etc is Constructed across a river, the river width is reduced and drained in Such a way as to ensure not Only Safe disposal of flood water takes place but also ensures Permanent width of the waterway for the waterflow.
- If it's not ensured it would be Uneconomical to Span the entire width of the river or its Khadir Bank {Max Stretch across the width, the River may Meander}.
- These banks are generally Provided in Pairs, Symmetrical in Plane.

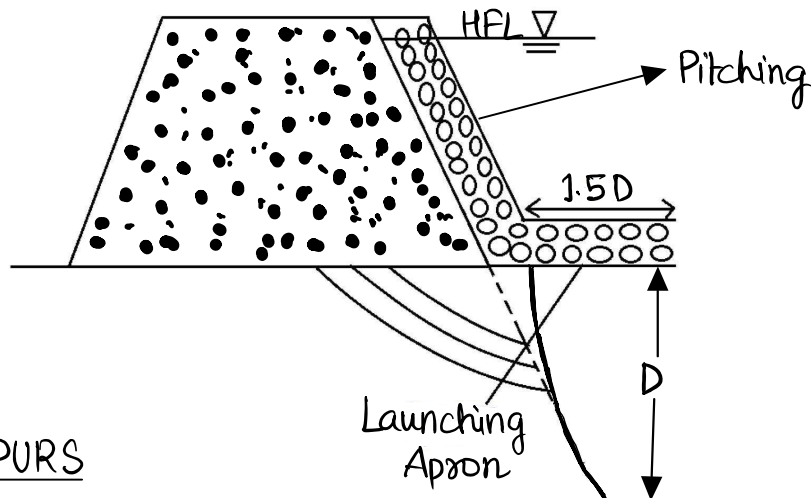




Plan



Note : Whenever a sloping face is protected by stone pitching against Scouring, the Pitching is extended beyond the bed in the form of Packed Stone which dump into the Scoured Portion and it termed as Launching Apron.



iii) GYRONES/SPURS

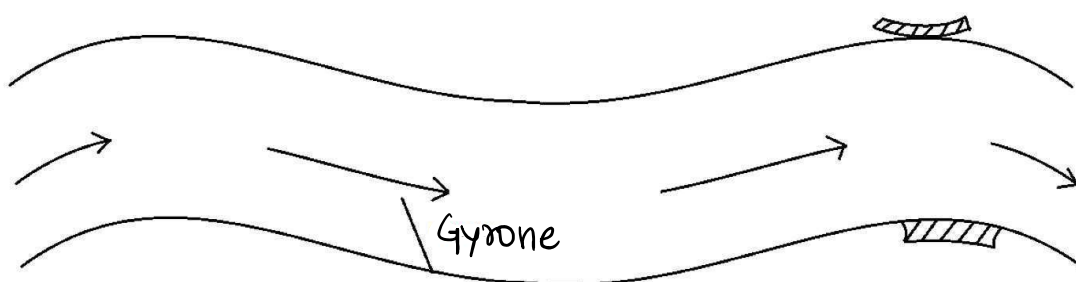
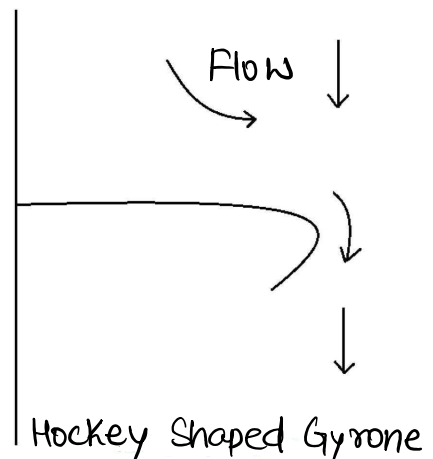
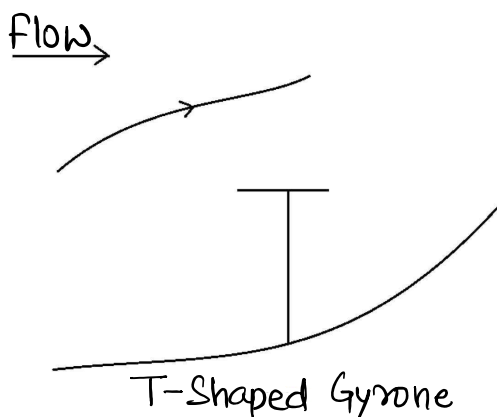
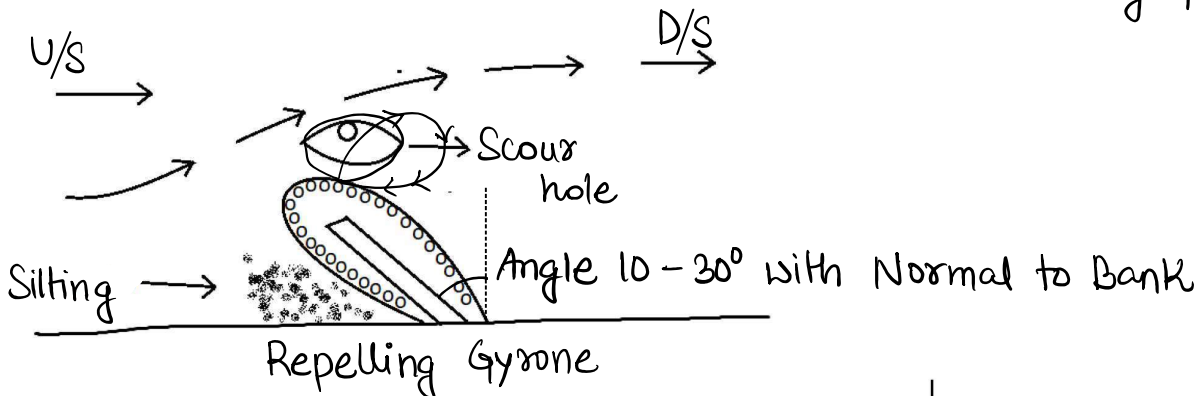
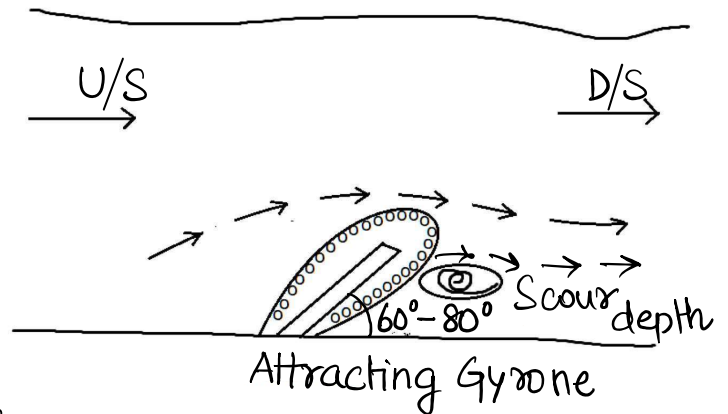
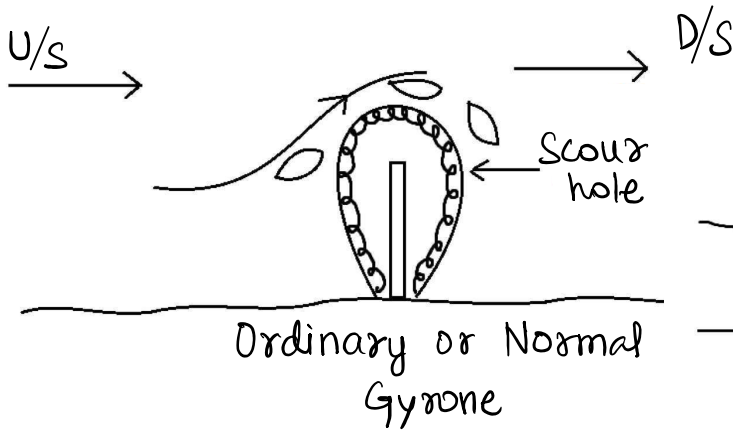
- These are the embankment type structure constructed along the transverse direction to the River flow extending from the Bank into the River.
- They protect the bank of the River from Erosion and Silting by deflecting

the current away from the Bank.

— The Gyrones are of following types —

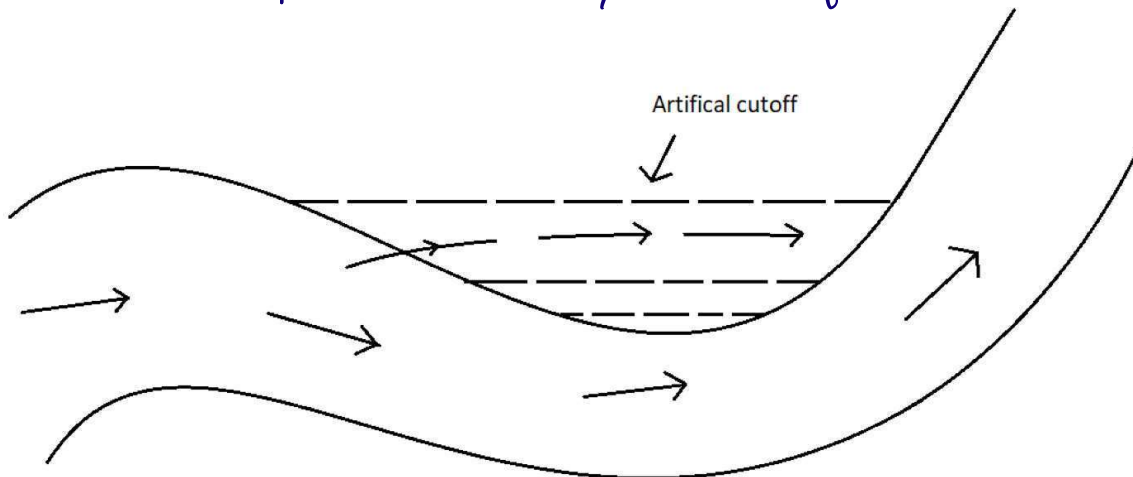


Left Bank



(iv) ARTIFICIAL CUT-OFF

- When a meander goes on increasing, it may endanger some valuable land or structure, then the river course may be straightened by inducing an artificial cut-off.
- For inducing an artificial cut-off, only a pilot channel is required to be excavated as rest of work is done by river itself.

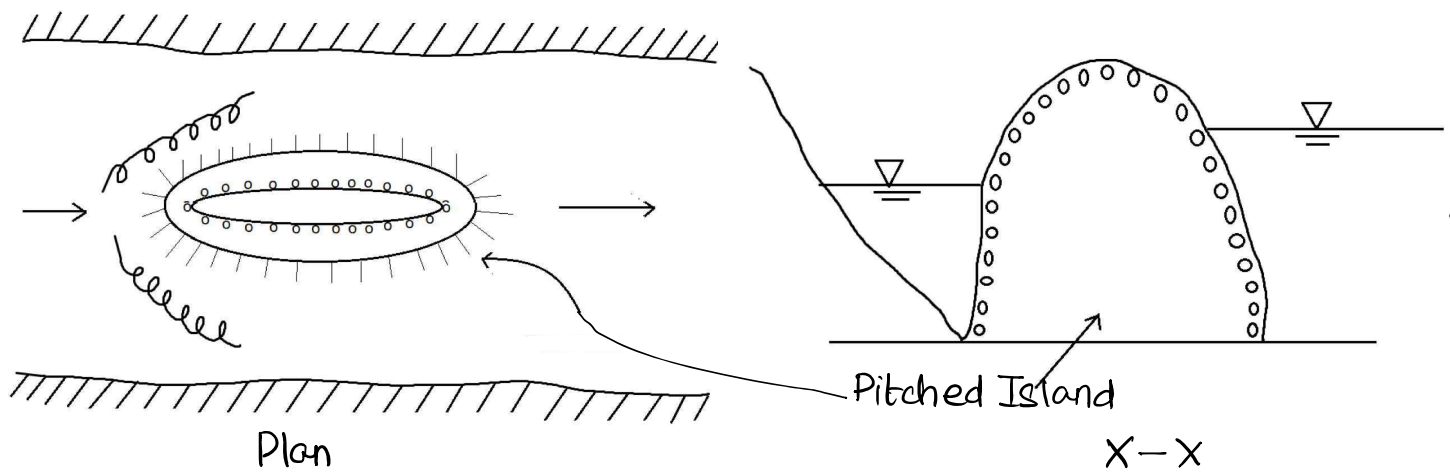


(v) PITCHING OF BANK / PROVISION OF LAUNCHING APRON

- Pitching of Bank is part of indirect river training work.
- In this bank of river are directly protected by stone pitching or by concrete blocks or by brick lining or by growing vegetable cover etc.

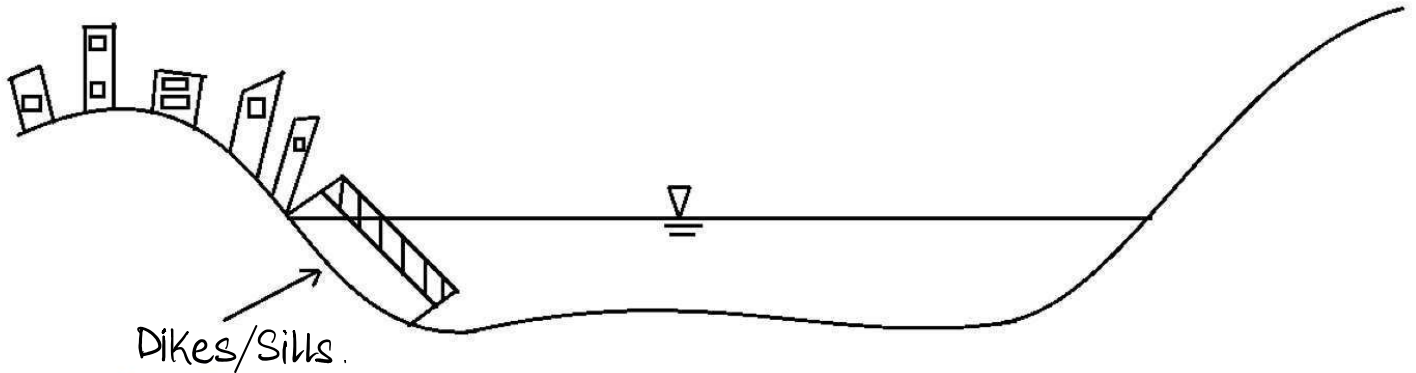
(vi) PITCHED ISLANDS

- It is an artificial island, constructed over the river bed and is protected by stone pitching on all sides.
- Because of the turbulence generated by the island in its vicinity, the river channel around the island gets deepened and water is attracted towards it, thereby, avoiding the erosion of the banks of river.



(vii) DIKES/SILLS AND BANDALLING

- Sometimes, a river may create deep cut or channel in the vicinity of certain structure and are required to be corrected.
- In such situation, sills are placed across the scoured portion of the bed with their top level at or slightly below the designed bed level.



PREVIOUS YEAR QUESTIONS

ESE

Q. Which of the following are the Objectives for River training ?

1. High flood discharge may pass safely and quickly through the reach.
2. To make the river course stable and reduce bank erosion to minimum.
3. To check flow through Canal.
4. To provide a sufficient draft for navigation as well as good course for it.

- (a) 1, 2 and 3 Only
- (b) 1, 3 and 4 Only
- (c) 1, 2 and 4 Only.
- (d) 2, 3 and 4 Only.

[2020]

Ans:

(c) 1, 2 and 4 Only.

— Objectives of River training are

1. High flood discharge may pass safely and quickly through the reach.
2. Sediment load including bed and suspended load may be transported efficiently.
3. To make the river course stable and reduce bank erosion to minimum.
4. To provide a sufficient draft for navigation as well as good course for it.

Q. Tortuosity of a Meandering river is the ratio of

- (a) Meander length to width of the Meander.
- (b) Meander length to width of the river.
- (c) Curved length along the river to the direct axial length of the river.
- (d) Direct axial length of the river to the curved length along the river.

[2017]

Ans:

(c) Curved length along the river to the direct axial length of the river.

Q. 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 the above Statements is/are correct ?

- (a) 1 and 2 Only.
- (b) 1, 2 and 3
- (c) 2 and 3 Only.
- (d) 2 Only.

[2010]

Ans:

⑥ 1, 2 and 3

— Water training Works:

High Water training — Flood Controls.

Low water training — For depth & Navigational Requirement.

Medium water training — Sediment & disposal.

SSC-JE

Q. Among the following river training works, which are aligned either parallel or at an angle to the banks, identify the one having a difference in alignment when compared to others.

① Guide Banks.

② Levees

③ Groynes

④ Flood walls.

[2020]

Ans:

③ Groynes.

Q. A River training work is generally required when the river is:

① Aggrading type

② Meandering type

③ Degradation type

④ Both ① and ③.

[2013]

Ans:

② Meandering type.

— The chief characteristics of these river reaches is the Zig Zag fashion in which they flow.

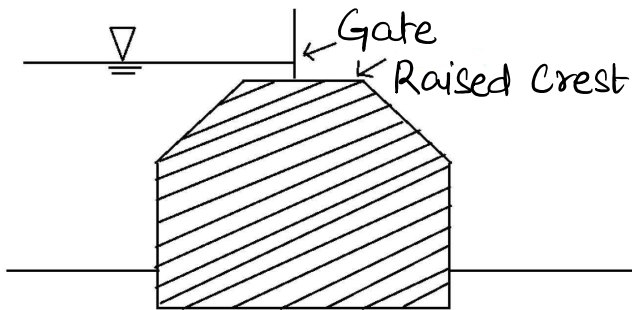
— This zig-zag fashion is called Meandering.

DIVERSION HEADWORK

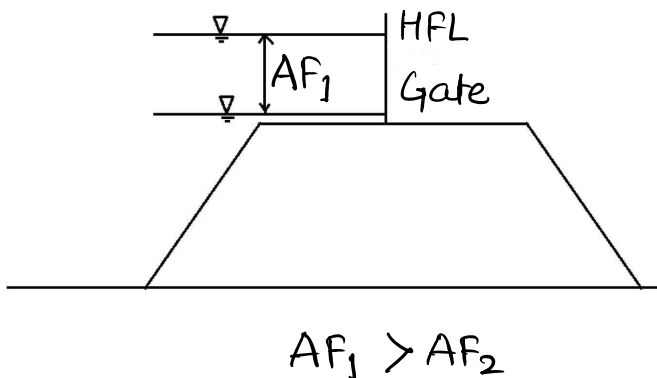
- The Work Which are Constructed at the head of Canal in order to divert the River water towards the Canal so as to ensure regulated Silt free Continuous Supply of Water, with a Certain minimum head into the Canal are termed as Diversion Headwork.
- For example : Weirs
Barrage

WEIR

- Major Ponding is Achieved from Raised Crest.

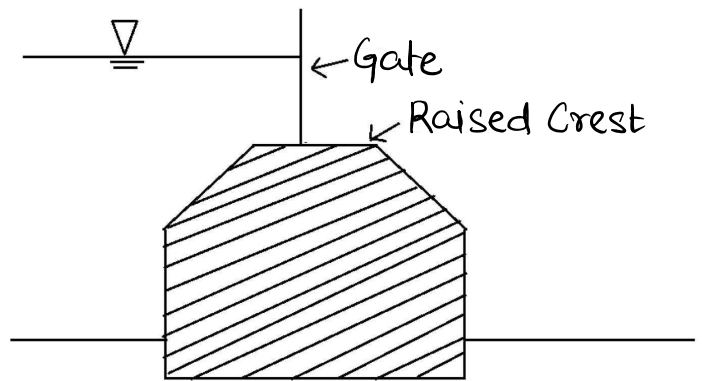


- Weir Cannot be Used as means of transportation.
- Cheap but lack Control.
- Afflux Cost during flood is Very High.

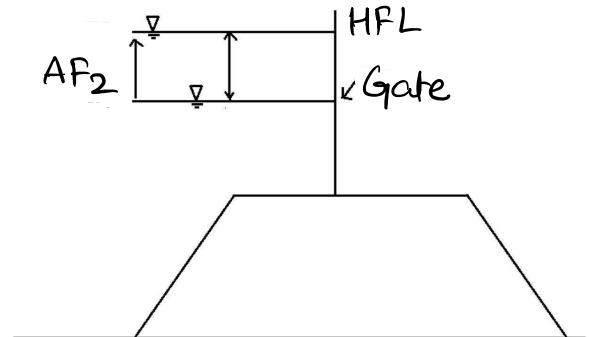


BARRAGE

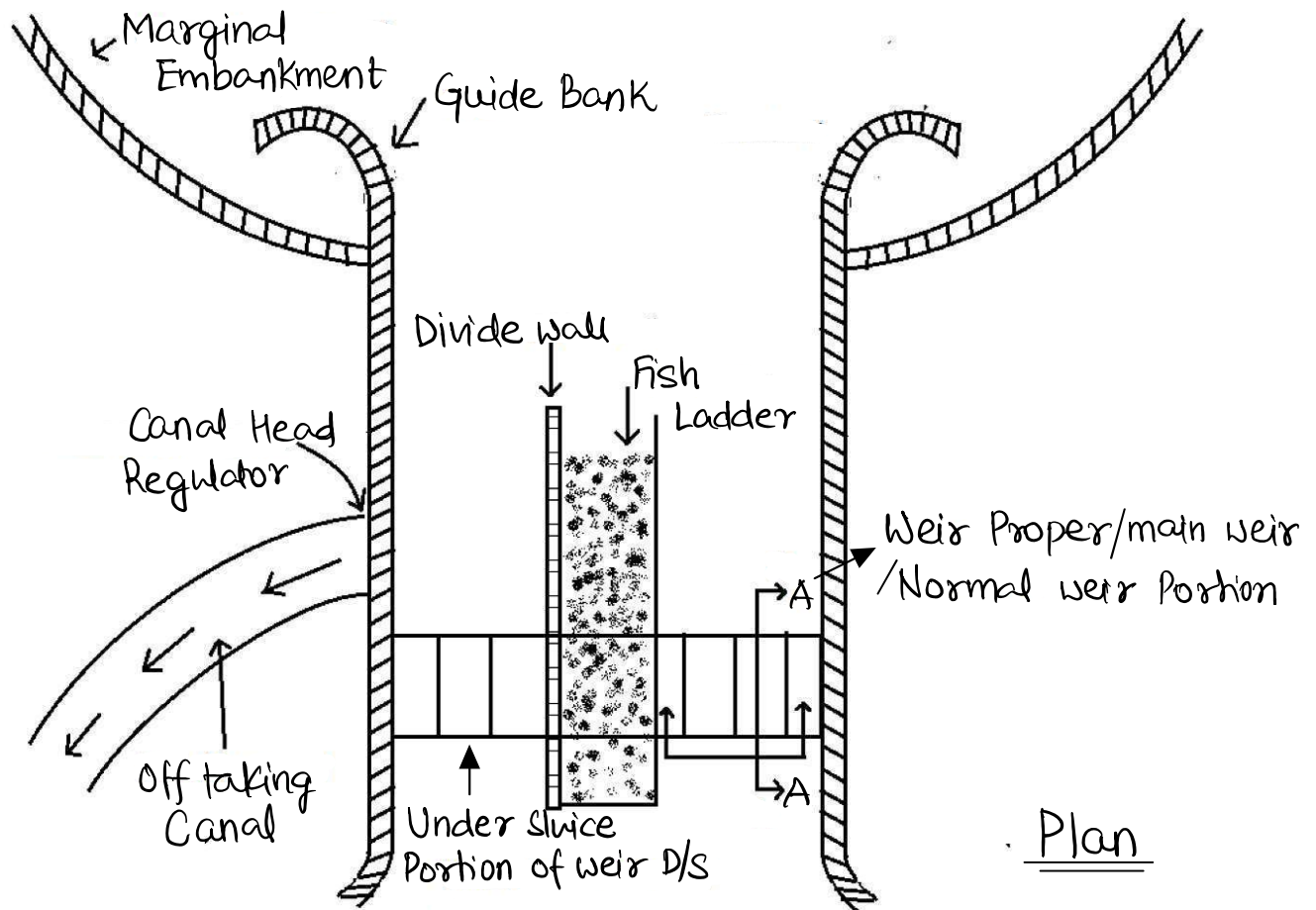
- Major Ponding is Achieved from Gate/ Shutter.



- Barrage Can be Used as Means of Transportation.
- Costly but effective Control is Provided.
- Afflux Cost during flood is Very less.



•• LAYOUT OF A DIVERSION HEADWORK AND ITS COMPONENTS



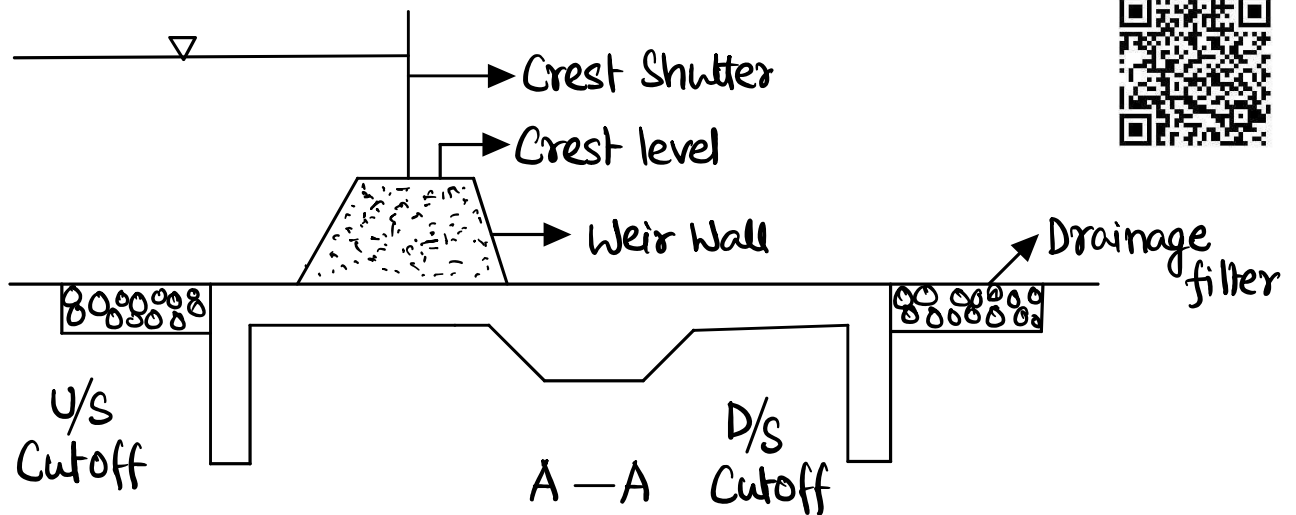
— A diversion head work generally consists of following components —

- (a) Weir Proper
- (b) Under Sluice
- (c) Divide Wall
- (d) River training Works {Guide Banks, Marginal Embankment}.
- (e) Fish Ladder
- (f) Canal head Regulator.
- (g) Silt Regulator.

(a) WEIR PROPER

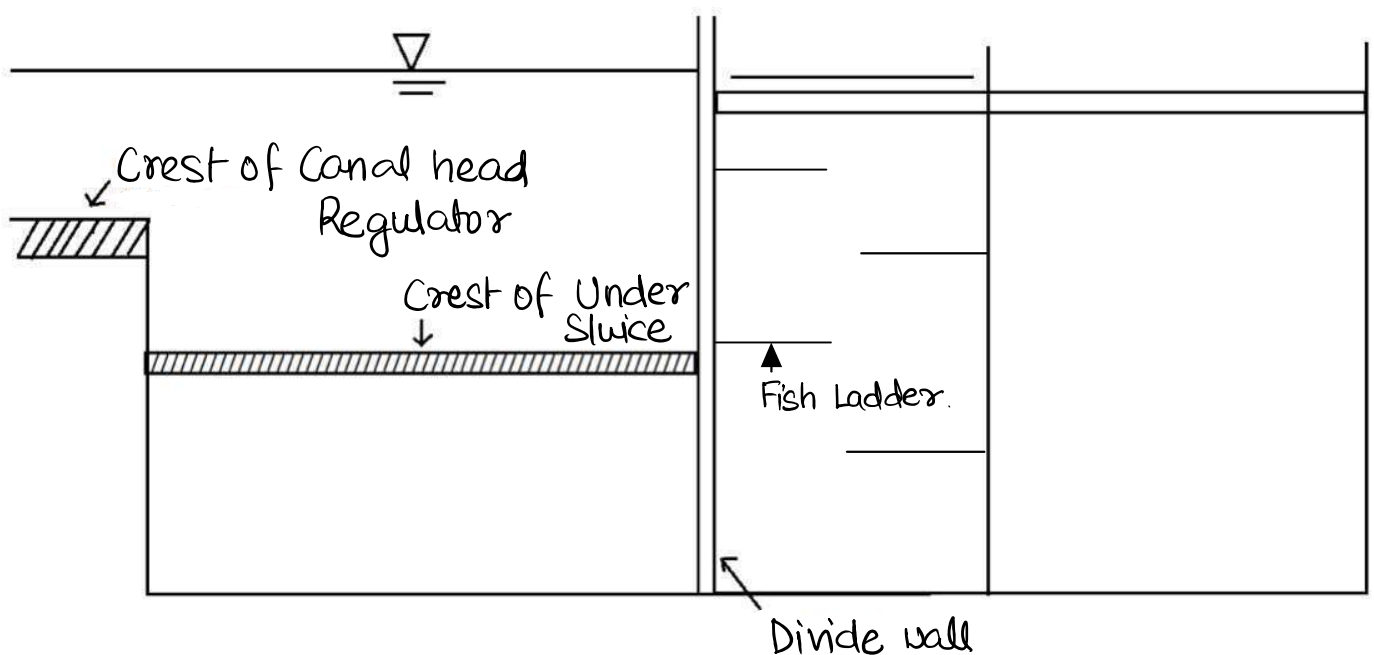
— It is a raised permanent (Pucca) structure with or without shutters/gate laid across the river width to divert the water into the canal.





⑥ UNDER SLUICE/ SCOURING SLUICE

- A comparatively less turbulent of Water is Created near the Canal head Regulator by Constructing Under Sluice Portion of the Weir.
- A divide wall separates the main Weir Portion from the Under Sluice.
- The Crest of the Under Sluice Portion of the Weir is Kept at a lower than the Crest of the Normal portion of Weir.
- Normally, the Crest level of Under Sluice is Kept equal to the deepest bed level of River during Non-Monsoon Season, Whereas Crest level of Weir is Kept higher by About 1-1.5.
- The Crest level of head Regulator is also Kept higher than the Crest level of Under Sluice, So that Only Silt free Water is Admitted into the Canal.



- Divide Wall is Provided from the point Upstream of Canal head Regulator.

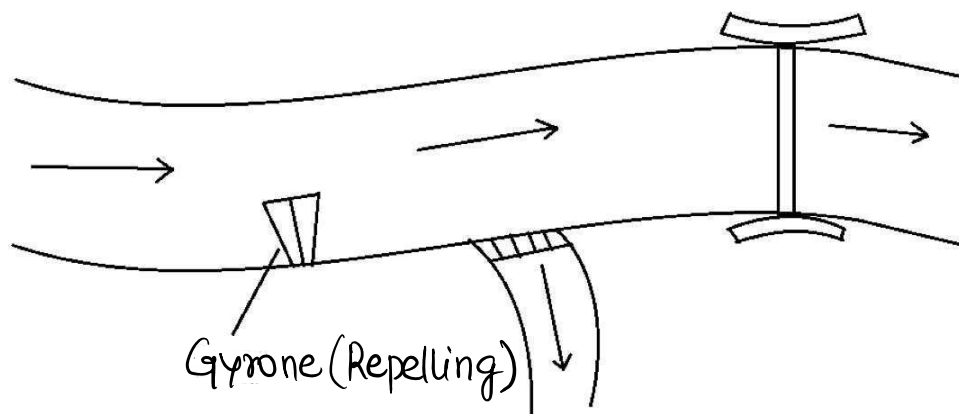
© DIVIDE WALL

- It is a Masonary or Concrete Wall Constructed at Right Angle to the axis a Weir and Seperate Weir Proper from Under Sluice.
- It Extends On the Upstream Side beyond the beginning of Canal head Regulator on the Downstream Side it extends beyond the End of the loose Protection of Under Sluice,
- The Main function of the divide Wall :
 - ① It Seperate Under-Sluice from Weir proper.
 - ② It helps in Providing Comparatively less turbulent Pocket nearby to the Canal.
 - ③ It Keeps the Cross-current away from the weir.



④ RIVER TRAINING WORK

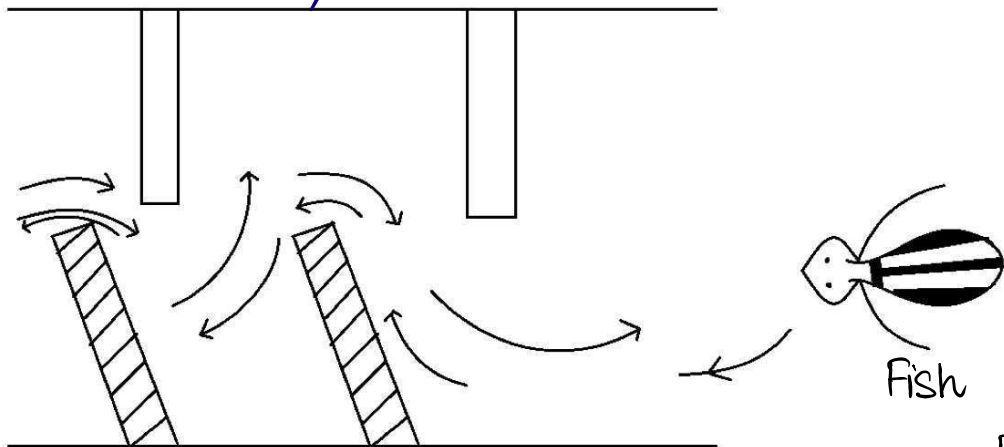
- Guide Bank, Marginal Embankment, Gyrones.



© FISH LADDER

- Generally, Rivers are inhabited by Several type of migration Fishes termed ANADROMOUS, Which moves from One part of river to another part of River, according to Season.
- When these fishes moves from Downstream to Upstream Side they have to Resist water Pressure/Current, due to Which they dies Over the period of time.
- It is Observed Most of fishes Can Resist velocity of 3-3.5 m/sec at head difference of 5-6 meter Upstream.
- In Order to Enable Movement of fish a Structure is made termed as Fish Ladder which dissipates the Energy of Water by Provision of baffle walls in it

- These ladders are suitable for weir of small height as in case of large weirs { Height 100 m } it becomes uneconomical, in that case it is Transported Mechanically.



Plan

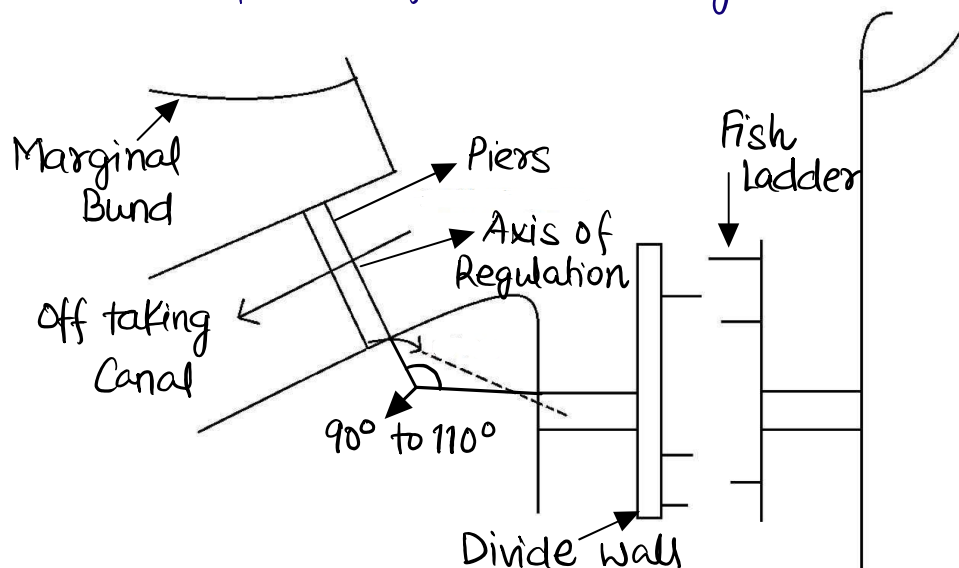
④ CANAL HEAD REGULATOR/HEAD SLUICES

- A Canal head Regulator is provided at head of off-taking Canal and Serves the following Purpose –

- ① It Regulates the Supply of Water Entering the Canal.
- ② It Controls the Entry of Silt into the Canal.
- ③ It prevents the river flood from Entering the Canal.



- The Regulator is generally aligned at right Angle to the Weir but Slight Angles can also be provided for Smooth Entry.



⑧ SILT CONTROL DEVICE

- The Entry of Silt into a Canal which take off from head work, can be Reduced by Constructing Silt Control Work, as follows –

① Silt Excluders : These are the Work Which are Constructed on the head of the river, upstream of head Regulator.

— The Clear Water Enters the head regulator and Silted water Enters to the Silt Excluder.

— In this type of Work Silt is, therefore removed from water before it enters into Canal.

② Silt Ejectors : These are device Which extract Silt from the Canal Water after Silted Water has travelled a Certain distance in to the off - take Canal.

— These Works are therefore Constructed On the head of Canal, a little downstream of the head Regulator.

•• CANAL REGULATION WORKS

— The Water Which Enters into the Main Canal from the River has to be Divided into different Branches and distributaries, in accordance with demand of Water.

— This Process of distribution is termed 'REGULATION' and Work Which are Constructed in order to Control and regulate discharge, depth of Velocity in Canal are known as 'CANAL REGULATION WORKS'.

— The Important type of Canal Regulation Work

① Canal Fall

② Canal Header

③ Canal Escape And cross Regulations

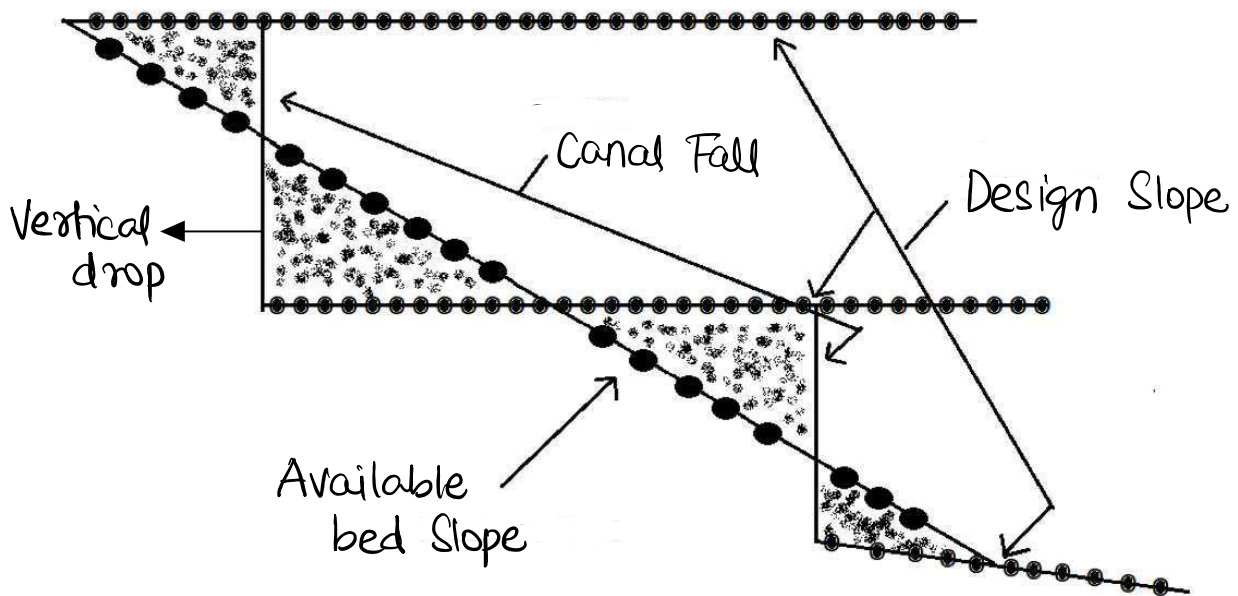
④ Metering Flumes

⑤ Canal Outlets and Modules.

① CANAL FALL

— Whenever the available Natural ground Slope is Steeper than the design bed Slope of the Channel, the difference is adjusted by Constructing Vertical 'CANAL FALLS' in the Canal bed at Suitable interval. Which are Pucca Structure.





— Location of the falls depends upon the topography and is selected such that Cost of Excavation and Cost of filling is minimum by balancing the Earthwork in Excavation and filling.

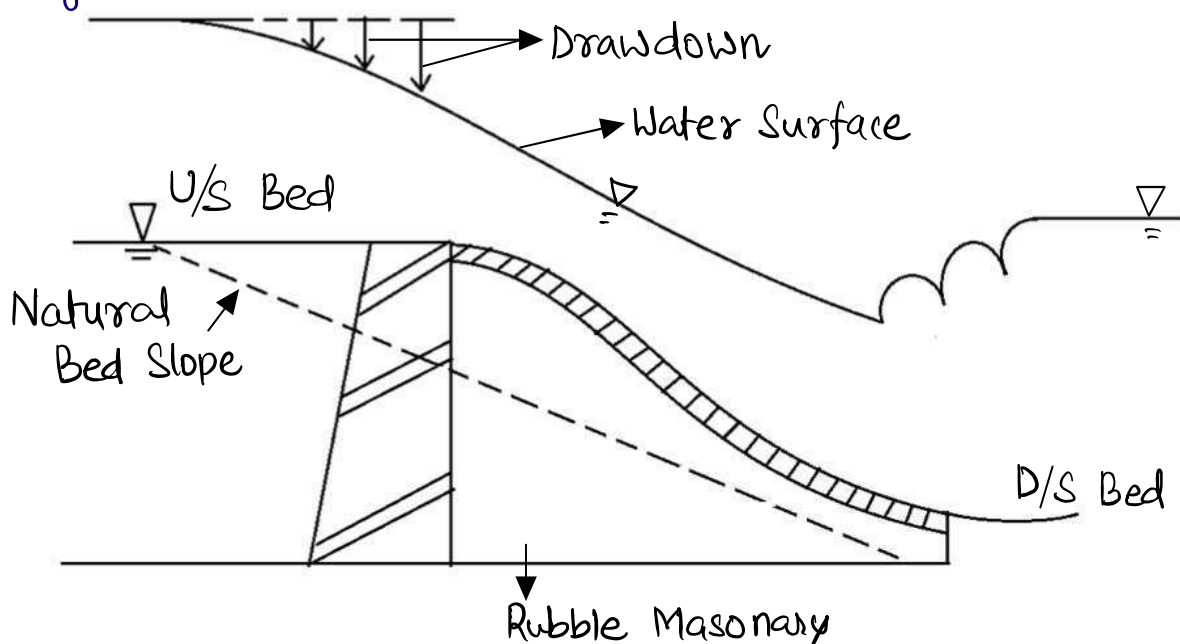
— Various types of Canal Falls are as follows —



① OGEE FALL : In this the water is gradually laid down by providing Convex and Concave Curve.

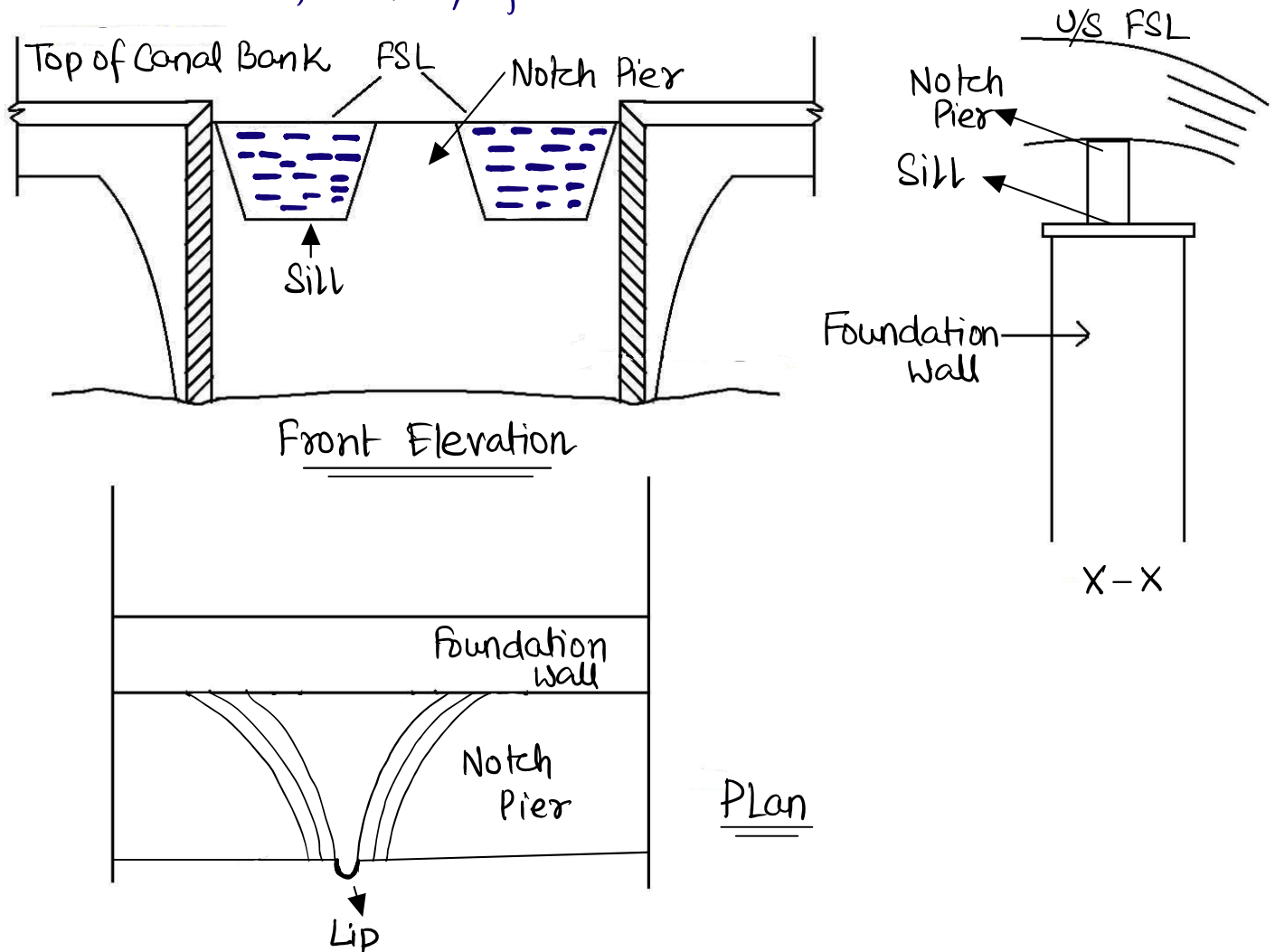
— The Performance of this was found to have following defects —

- ① There are heavy draw-down on Upstream Side, due to which low depth and higher Velocities were Observed Causing Erosion.
- ② Due to Smooth transition, the Kinetic Energy of flow was not dissipated, Causing erosion in Downstream Side.



⑥ TRAPEZOIDAL NOTCH FALL : It consist of No. of trapezoidal Notches Constructed in a high Crested wall across the channel with Smooth Entrance on a flat Circular lip projecting Downstream from each notch to Spread out the Falling Jet.

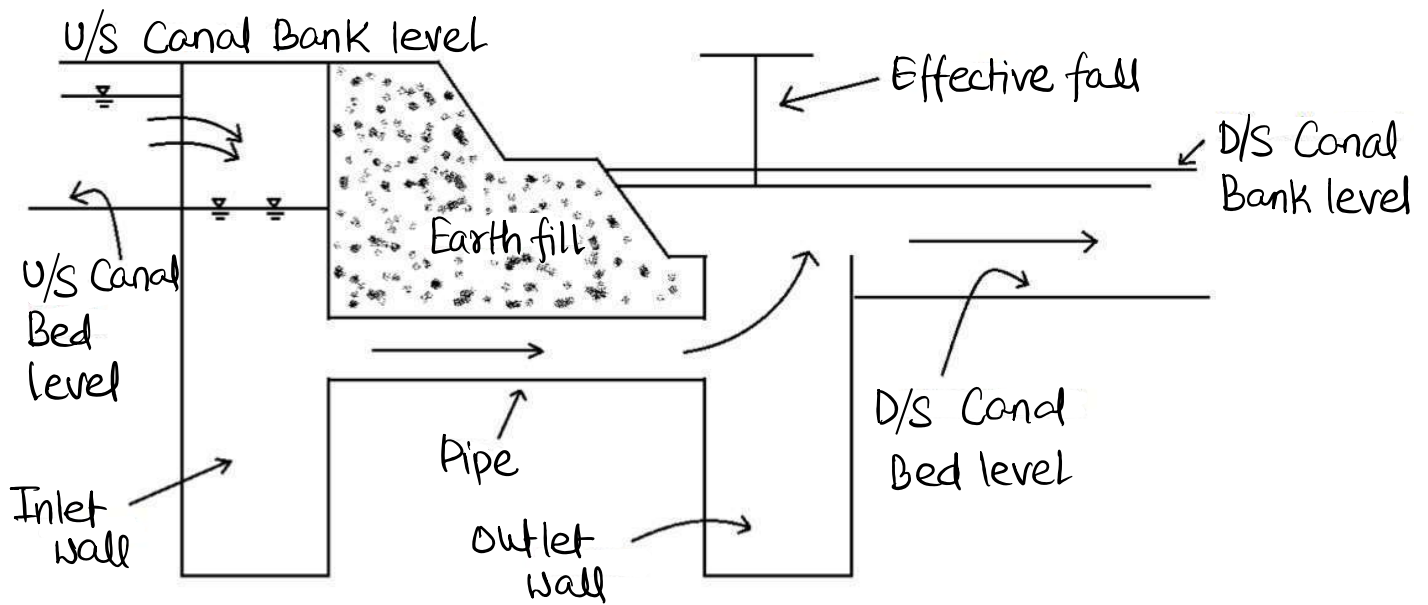
— These Jets Could be design to maintain Normal Water depth in the Upstream Channel hence, Tendency of erosion is reduced in this Case.



⑦ WELL TYPE FALL / CYLINDER TYPE / SYPHON WELL DROP

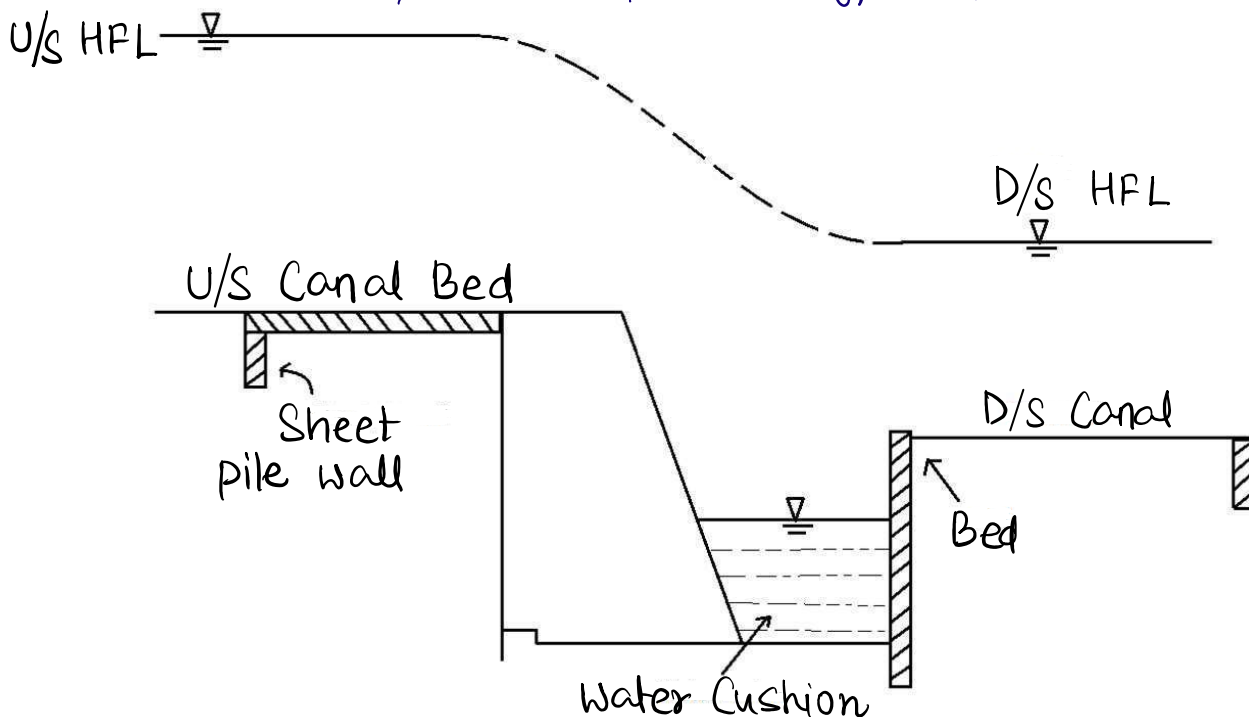
- This type of fall consists of inlet well with a pipe at its bottom, Carrying Water from inlet well to a Downstream Outlet well.
- The Downstream well is necessary in the Canal of fall greater than 1.8 m and discharge greater than $0.29 \text{ m}^3/\text{sec}$.
- This type of fall are very Useful for affecting large drop for Small discharge.
- They are Commonly Use as tail escape or Where high levelled Smaller drains are available.





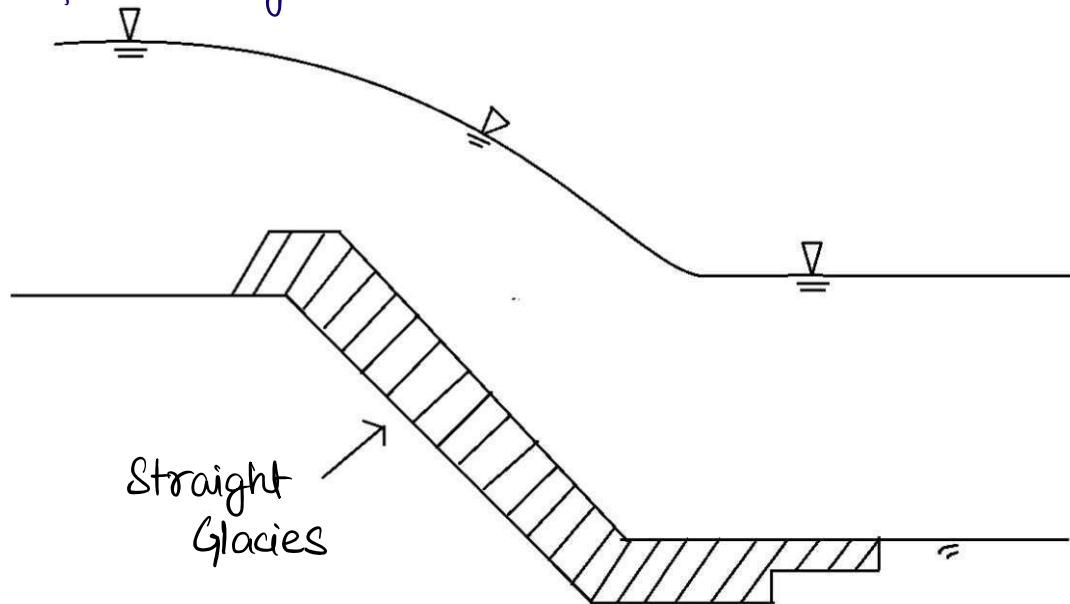
④ SIMPLE VERTICAL DROP TYPE / SARDA TYPE FALL

- A Raised Crest fall with a Vertical impact is Provided in this Case.
- It is Economical and Simple in design and Construction.
- In this type of high Crested fall, the nappe Fall into water Cushion below it.
- There is no clear hydraulic Jump and energy dissipation in this Case.

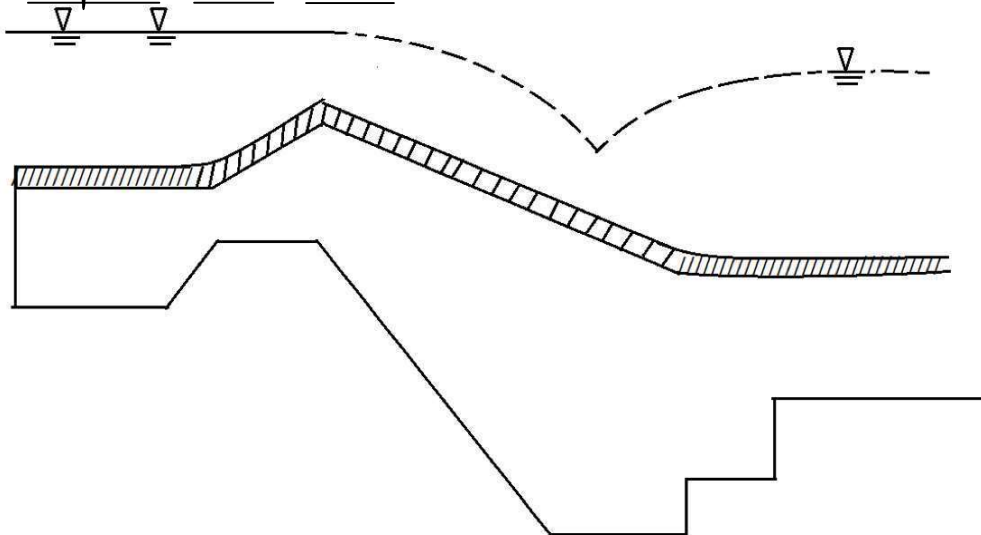


③ STRAIGHT GLACIS FALL

- In this Fall, Straight Glacis (2:1) is Provided after a Raised Crest.
- The Hydraulic Jump is made to Occur on the glacis, causing Sufficient Energy dissipation.
- This type of fall gives very good Performance however, they may be flumed for Reducing Cost.



④ MONTAGUE TYPE FALL

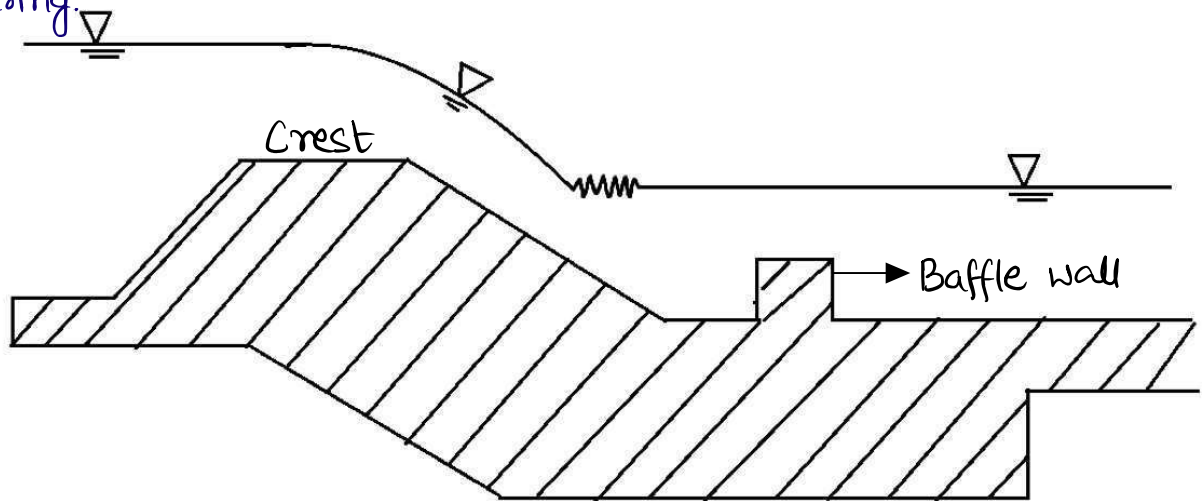


- The Energy dissipation on a Straight Glacis remain incomplete due to Vertical Component of Velocity being Unaffected Hence, to increase the dissipation Straight Glacis is reflected by Parabolic Glacis Community known as 'MONTAGUE PROFILE'.



⑨ ENGLISH FALL/BAFFLE FALL

—A Straight Glacis type fall When added with a baffle Platform and a baffle wall, it increases the head loss thereby Reducing the tendency of Scouring.



Note :

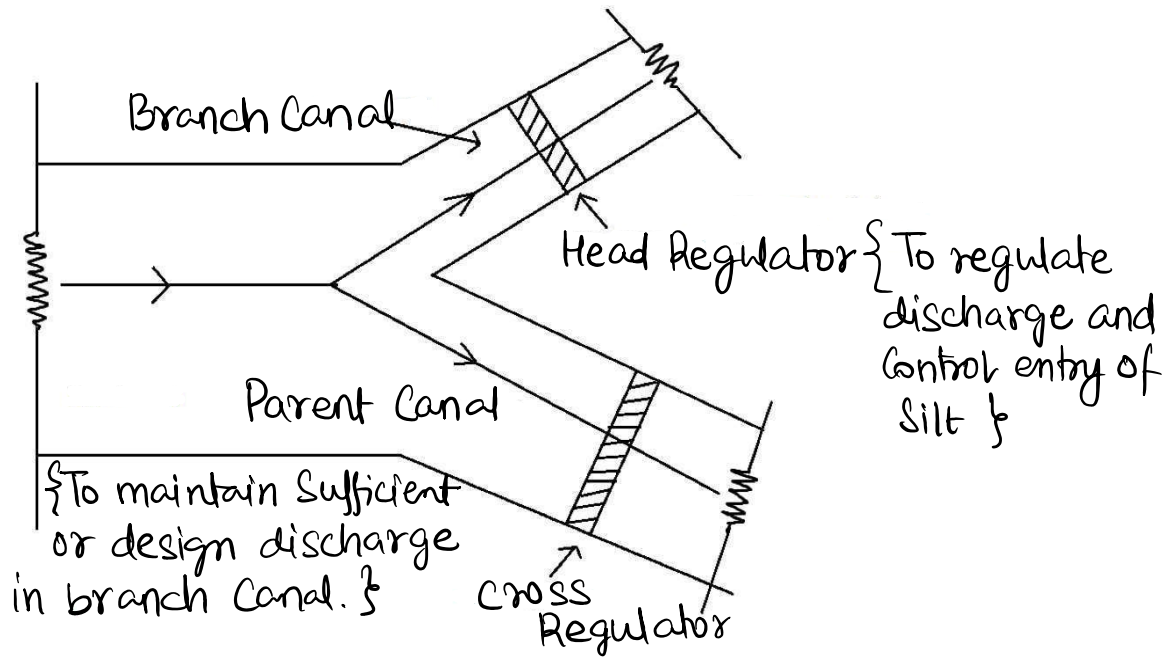
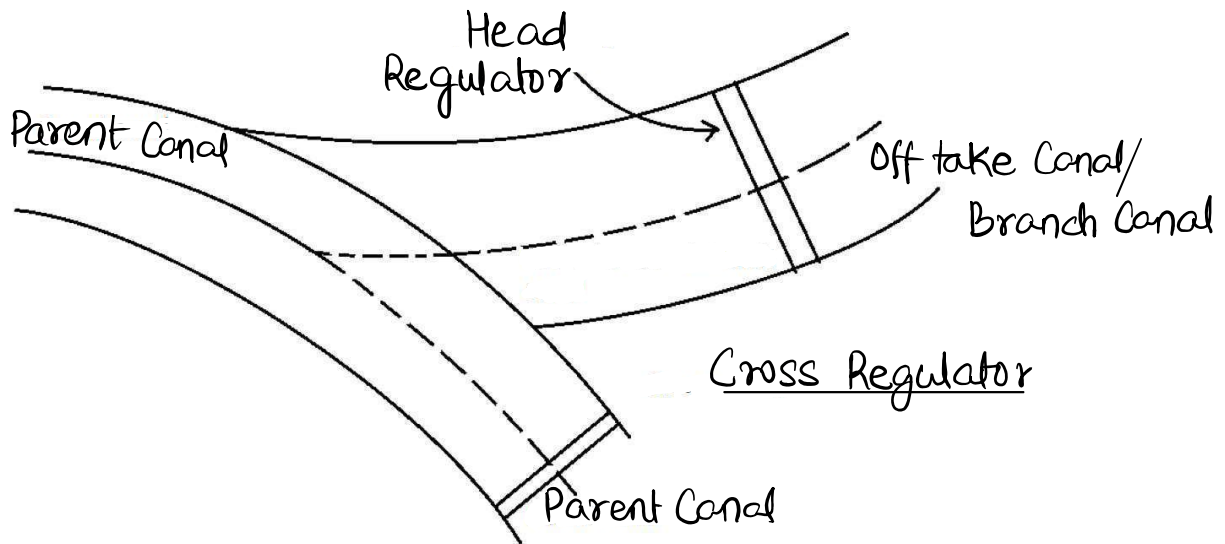
- ① If Canal falls Can be Used to Measure the discharge of the Canal, it is termed as Meter Fall and if it is not Used for Measurement of discharge it is termed as Non-Meter Fall.
- ② (i) Vertical drop Falls are Suitable for discharge upto $15 \text{ m}^3/\text{sec}$ and drop of 1.5 m .
(ii) Straight Glacis type of fall is Used for high Energy loss of discharge upto $60 \text{ m}^3/\text{sec}$.
(iii) Baffle Wall may be Used for all discharges and drop of 1.5 mtr .
(iv) Well type fall are Suitable for low discharge with high drops.

⑩ CANAL HEAD AND CROSS REGULATOR

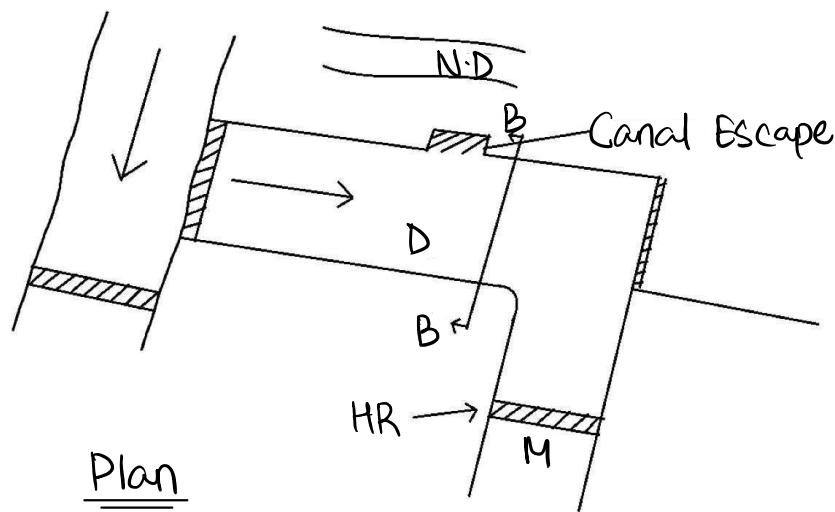
—When a branch Canal takes off from the main Canal {Parent Channel}, the off-take Channel must be Carefully designed and aligned in Such a way it makes Zero-angle with the Parent Channel and then Separates out in Transition.

—The transition will have to be properly designed So as to avoid Silting and as an alternative both the Channels Should make an angle with the parent Channel.





iii) CANAL ESCAPE



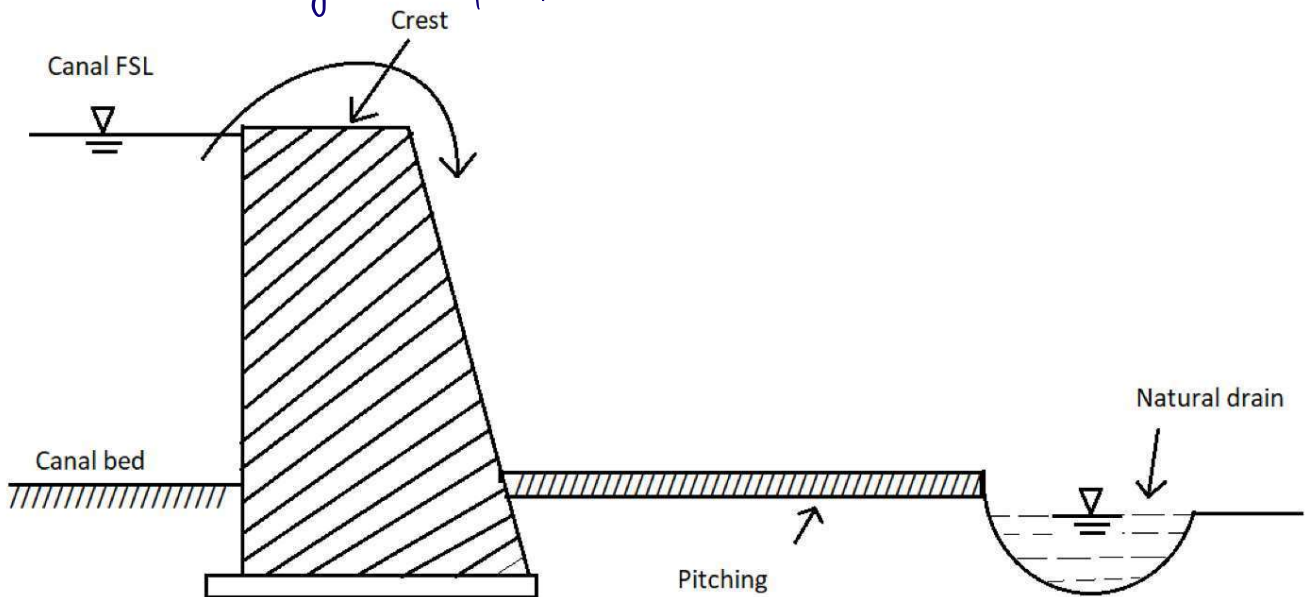
- It is a side channel constructed to remove surplus water from an irrigation channel { Main, branch, distributory } into a Natural Drain.
- The water in irrigation channel may become surplus due to,



- ① Some mistake or difficulty in regulation at the head.
- ② Due to Excessive Rainfall.
- ③ Sudden Closure of Outlet by the Cultivator.
- ④ Due to Sudden Breach at Downstream Side.

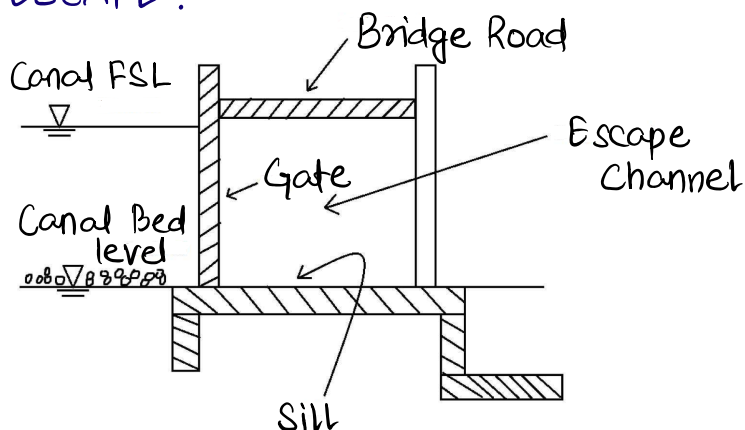
— Canal Escape are of following type —

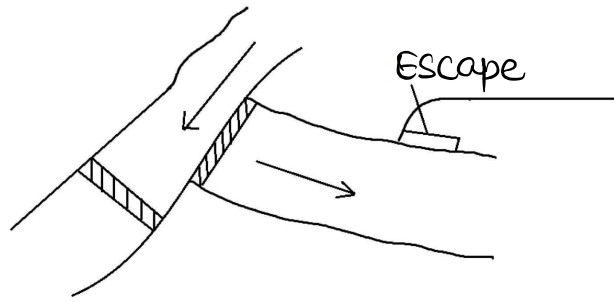
① WEIR TYPE: In this type, Crest of the weir wall is kept at RL equal to FSL of the Canal, hence When water level Rises above FSL it get Escaped.



② REGULATOR TYPE OR SLUICE TYPE CANAL

- In this type the Sill of the Escape is kept at Canal Bed level and the flow is controlled by gates.
- This type of Escape are Preferred as they give better and Can be Used for Completely Emptying the Canal.
- If this Escape is designed for Scouring of Excess bed Silt it is termed as 'SCOURING ESCAPE'.



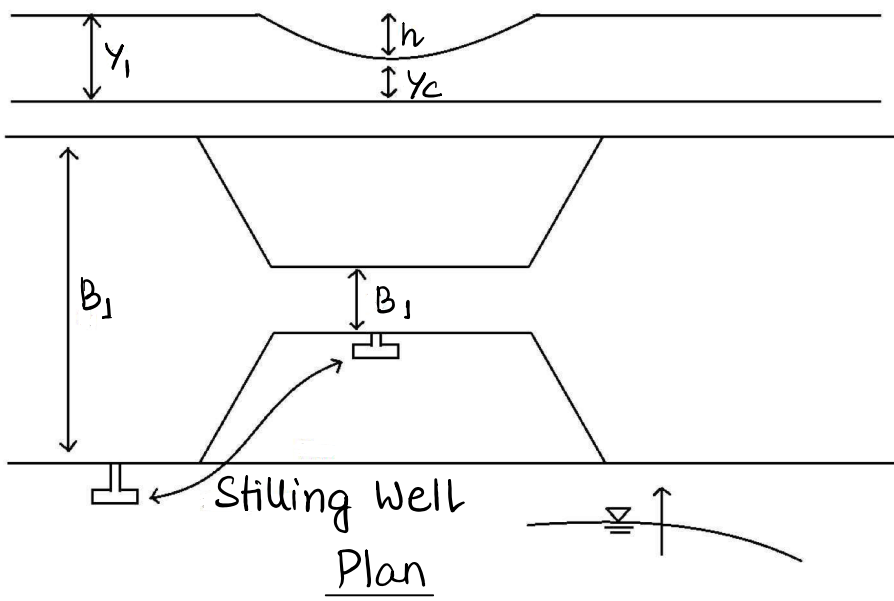


④ METERING FLUMES

- It is a Structure Constructed in Canal for Measuring its discharge accurately.
- It is an artificially flumed Section of the Channel Which Can be Utilized for Calculation of discharge.
- These are of following types –

① Venturi-flume/Non-Modular/Drawnd flume.

② Standing wave flume/Modular/free flow flume



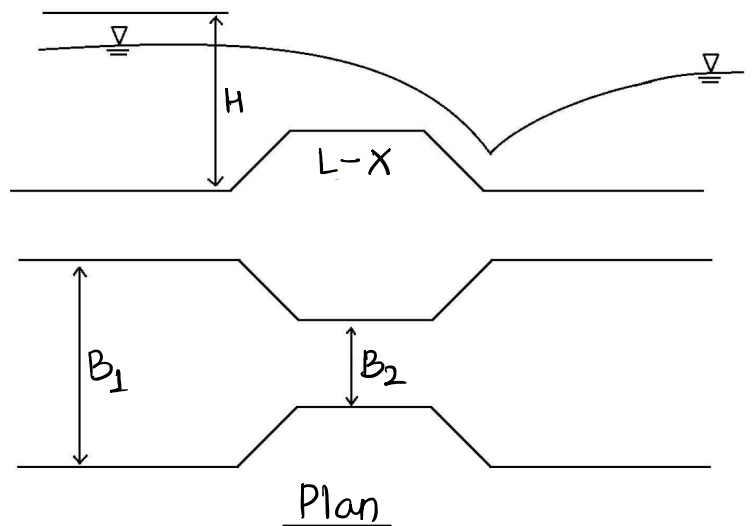
① Venturi-flume

$$Q = \frac{C_d \cdot a_1 \cdot a_2 \cdot \sqrt{2gh}}{\sqrt{a_1^2 - a_2^2}}$$



② Standing Wave flume

$$Q = 1.7 \cdot C_d \cdot B_2 \cdot H^{3/2}$$



⑤ CANAL OUTLET OR MODULES

- A Canal Outlet or a Module is a Small Structure built at the head of the Water Course so as to connect it with a minor or a distributory Channel.
- The Channel and maintenance of the Entire Network of Canals Upto the Module is Under State government and beyond it, is Under Cultivator itself.
- A Good Module Must Posses following property—
 - ① It Should draw its fair Share of Silt.
 - ② It Should be Such So as to avoid interference by Cultivator.
 - ③ It Should be Cheap.
 - ④ It Should Work efficiently with low head also.
 - ⑤ It Should be Simple and easy to Maintain.
 - ⑥ It Should fit well to the decided Principle of Water Distribution.



— Modules are of following types ÷

① NON-MODULAR OUTLET

- These are those through which discharge depends upon the difference of head between the distributory and Water Course.
- The discharge through such a module, therefore varies widely, with either the change in the water level of distributory or that of the water Course.
- Example — Open Sluice, Drowned Pipe Outlet.

② SEMI MODULE/FLEXIBLE MODULE

- These are those through which discharge is independent of water level of the water Course but depends Only upon water level of Distributory as long as minimum working head is Available.
- Example : Pipe Outlet, Venturi flume, Open flume, orifice Semi-module.

③ RIGID MODULE OR MODULAR OUTLET

- These are those discharge through which is constant and fixed within limit irrespective of fluctuations of the water level on either side.

- Example : Gibbs Module

- The behaviour and functioning of Module Can be Judged by the following Parameter :

① FLEXIBILITY : It is defined as ratio of rate of Change of Discharge of the Outlet to the rate of Change of discharge of the distributary channel.

$$F = \frac{dq/q}{dQ/Q}$$

Where,

q = discharge passing through the Outlet.
 Q = discharge passing through Distributary Channel.

$$Q = K \cdot y^n$$

$$q = C \cdot H^m$$

y = depth of water in distributary.

H = Is the head acting on the Outlet.

$$dq = C_m \cdot H^{m-1} \cdot dH$$

$$dQ = K \cdot n \cdot y^{n-1} \cdot dy$$

$$\begin{aligned} \frac{dq}{q} &= \frac{C_m \cdot H^{m-1} \cdot dH}{C H^m} \\ &= \frac{m}{H} \cdot dH \end{aligned}$$

$$\begin{aligned} \frac{dQ}{Q} &= \frac{K n \cdot y^{n-1} \cdot dy}{K \cdot y^n} \\ &= \frac{n}{y} \cdot dy \end{aligned}$$

$$F = \frac{dq/q}{dQ/Q} = \frac{\frac{m}{H} \cdot dH}{\frac{n}{y} \cdot dy}$$



$$= \frac{m}{n} \cdot \frac{y}{H} \cdot \frac{dH}{dy}$$

Here $dy = dH$,

$$F = \frac{m}{n} \cdot \frac{y}{H} \quad **$$

② PROPORTIONALITY

— The Outlet is Proportional when the rate of Change of Outlet discharge equals the rate of Change of Channel discharge i.e. The Outlet is proportional when its flexibility equal unity.

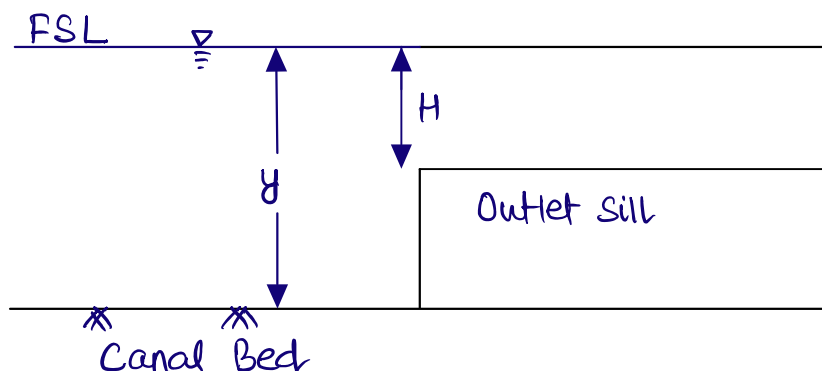
$$F = \frac{m}{n} \times \frac{y}{H} = 1$$

$$\frac{H}{y} = \frac{m}{n} = \frac{\text{Outlet Index}}{\text{Channel Index.}}$$

③ SETTING

— The Ratio of $\frac{H}{y}$ is termed as Setting of the outlet.

— It is the ratio of depth of Sill level of the outlet below the full-Supply level of the distributor to the full Supply depth of distributory.



For Proportional Outlet,
 $F=1$

$$S = \frac{H}{y} = \frac{m}{n} = \frac{\text{Outlet Index}}{\text{Channel Index}}$$



For Example, For Wide Trapezoidal channel, $n = \frac{5}{3}$

$$Q = A \cdot V$$

$$= A \cdot \frac{1}{N} \left\{ \frac{A}{P} \right\}^{2/3} \cdot s^{1/2}$$

$$= \frac{C \cdot A^{5/3}}{P^{2/3}}$$

$$Q = \frac{C [(b + my)y]^{5/3}}{[b + 2y \sqrt{1 + m^2}]^{2/3}}$$

$$B \gg y \Rightarrow b + my \simeq b$$

$$\Rightarrow b + 2y \sqrt{1 + m^2} \simeq b$$

$$Q = C_1 \cdot y^{5/3} \Rightarrow n = \frac{5}{3}$$

For Orifice type outlet,

$$Q = C_2 \cdot \sqrt{H}$$

$$\Rightarrow m = \frac{1}{2}$$

$$\Rightarrow \frac{H}{y} = \frac{1 \times 3}{2 \times 5} = 0.3$$

$$\Rightarrow H = 0.3y$$

Note :

① Hypo-proportional Outlet : An Outlet is termed to be Hypo-proportional if its flexibility is greater than Unity.

$$F > 1 \Rightarrow \frac{m}{n} \cdot \frac{y}{H} > 1$$

$$\Rightarrow \frac{H}{y} < \frac{m}{n}$$

$$\Rightarrow \text{Setting} < \frac{m}{n}$$



Hence, the Outlet is hypo-proportional if its Setting is less than $\frac{m}{n}$, thereby it is set higher.

② Sub-Proportional Outlet: It is the Outlet where flexibility is less than 1

$$F < 1 \Rightarrow \frac{m}{n} \times \frac{y}{H} < 1$$

$$\Rightarrow \frac{H}{y} > \frac{m}{n}$$

Hence, a Sub-proportional Outlet is Set-lower.

④ SENSITIVITY

— It is defined as the ratio of rate of change of discharge through the Outlet to the rate of change of water level of the distributory.

$$S = \frac{dq/q}{dy/y}$$

Hence for modular and rigid Outlet Sensitivity is Zero as it discharge does not depend on level of water in distributory {and also on level of water in water course}.

$$F = \frac{dq/q}{dQ/Q}$$

$$\Rightarrow Q = C \cdot y^n$$

$$\frac{dQ}{Q} = \frac{n}{y} \cdot dy$$

$$\Rightarrow F = \frac{dq/q}{\frac{n}{y} \cdot dy} = \frac{1}{n} \times \frac{dq/q}{dy/y}$$

$$\Rightarrow F = \frac{1}{n} \times S$$



Note :-

- (i) Minimum Modular Head : The Minimum difference between Upstream and Downstream water-level which is Required to be maintained So as to enable the Module to pass the design discharge is Called Min modular head.
- (ii) Efficiency of an Outlet : It may be defined as the ratio of head Recovered to the head Put in.
- (iii) Drowning Ratio : It is ratio of depth of Water level Over Crest on the Downstream and Upstream of the Module.

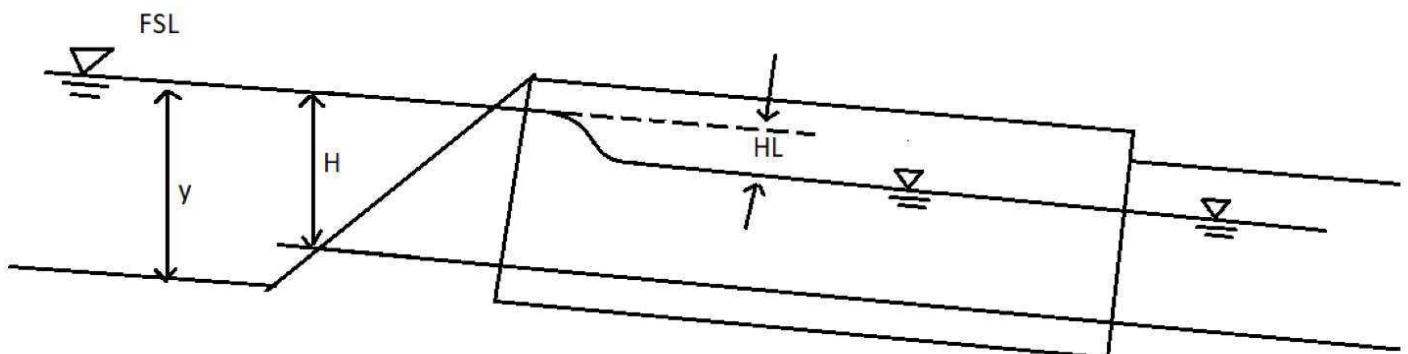
•• TYPES OF OUTLETS

(i) NON MODULAR OUTLET



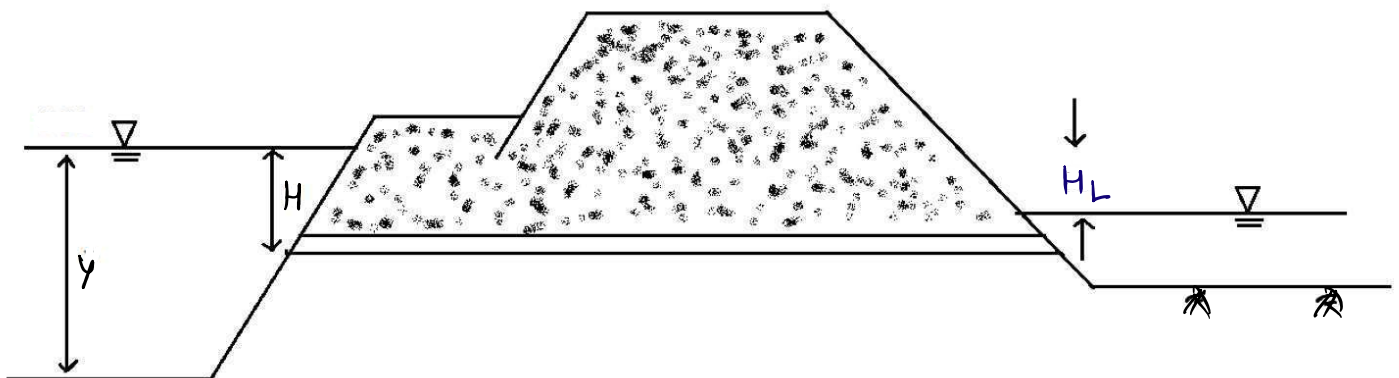
- (a) Open Sluice
- (b) Submerged Pipe.

@ OPEN SLUICE



$$q \propto H, H_L$$

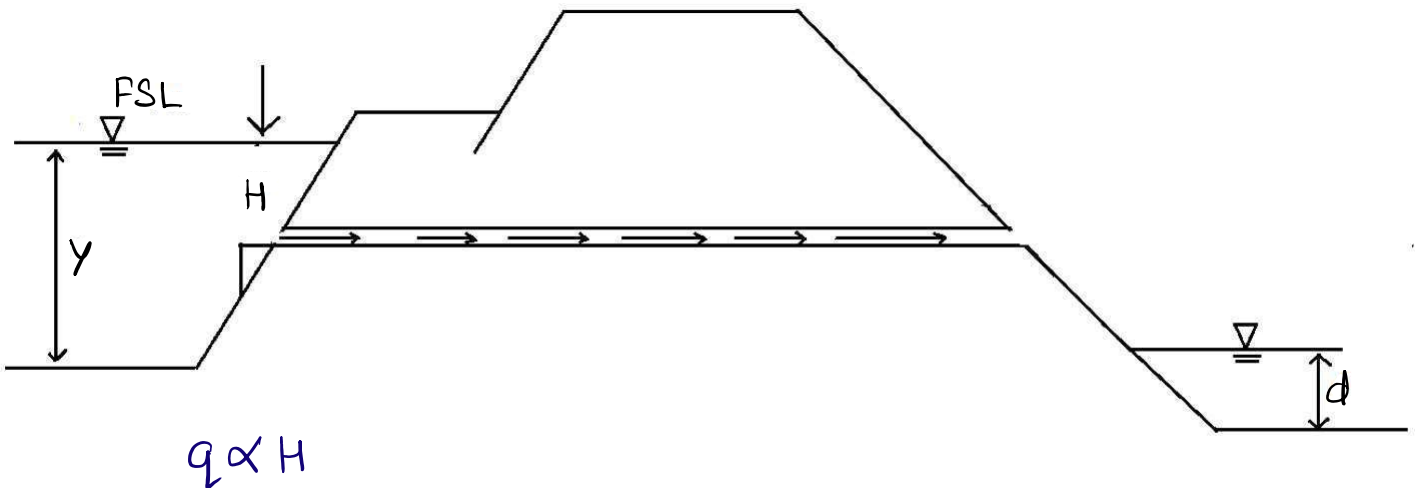
(b) SUBMERGED PIPE OUTLET



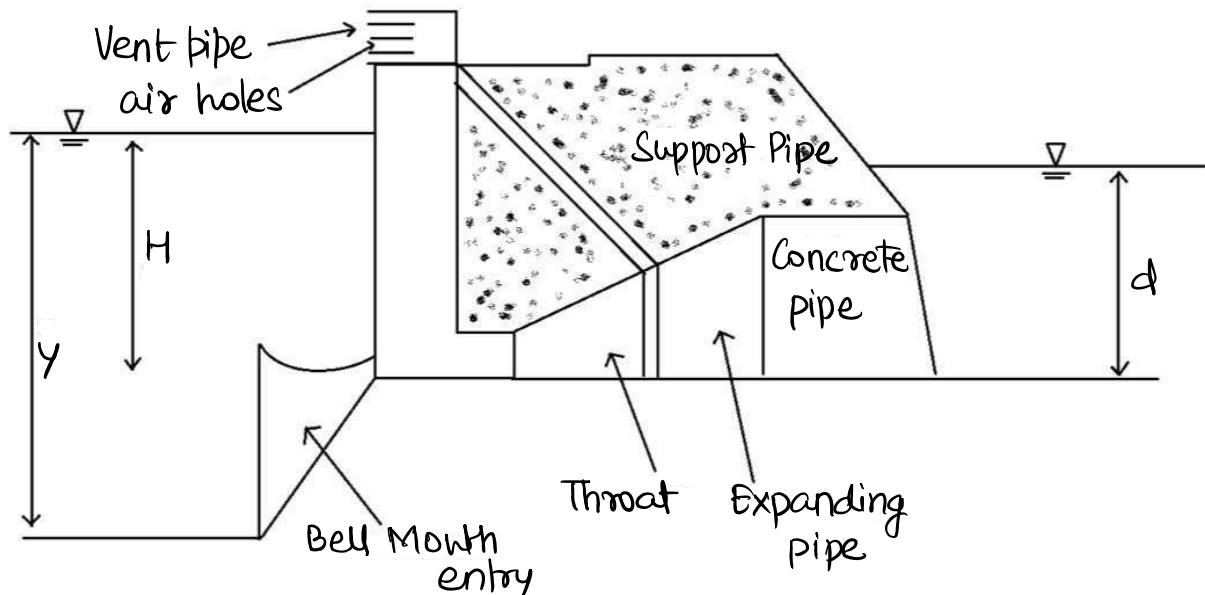
$$q \propto H_L$$

② TYPES OF SEMIMODULE/FLEXIBLE OUTLET

① FREE PIPE OUTLET



② VENTURI FLUME/KENNEDY'S GAUGE



③ OPEN FLUME OUTLET

④ ADJUSTABLE ORIFICE SEMIMODULE

③ TYPES OF RIGID MODULES

① GIBB MODULE

② KHANNAS RIGID MODULE

③ FOOTE MODULE



PREVIOUS YEAR QUESTIONS

ESE

Q. Consider the following Statement with respect to weir Under discussion :

1. Its design Corresponds to Soft Sandy foundation.
2. The difference in weir Crest and downstream river bed may not exceed 3m.
3. When water Passes Over it, the longitudinal location of the formation of Hydraulic Jump is Variable.

The Weir is of the type.

- (a) Vertical drop weir.
- (b) Masonry or Concrete Sloping weir.
- (c) Dry Stone Slope Weir.
- (d) Parabolic weir.

[2019]

Ans :

- (b) Masonry or Concrete Sloping weir.

— Weirs of this type are of recent Origin. They are Suitable for Soft Sandy foundations and are generally Used Where the difference in weir Crest and downstream river bed is limited to 3m, When water Passes Over Such a weir, hydraulic Jump is formed on the Sloping glacis.

Q. A Barrage on a major river in the Gangetic plains has been designed for a flood discharge 7000 m³/s. It has been provided with a waterway of 360 m length. The looseness factor of this Barrage is

- (a) 1.7
- (b) 1.1
- (c) 0.7
- (d) 0.90

[2016]

Ans :

- (d) 0.90

— Looseness factor = $\frac{\text{Actual Width}}{\text{Regime width}}$

Actual Width = 360 m (Given)

According to Lacey's theory,

$$\text{Regime width} \Rightarrow W = 4.75\sqrt{Q}$$

$$W = 4.75 \times \sqrt{7000}$$
$$W = 397.41$$

$$\text{Looseness factor} = \frac{360}{397.41} = 0.90.$$

Q. Flexibility of an outlet may be defined as the ratio of the rate of change of :

- (a) Outlet discharge to the rate of Change of water level of the Parent Channel.
- (b) Outlet discharge to the rate of Change of the discharge of the Parent Channel.

- © Parent Channel discharge to the rate of Change of the outlet discharge.
④ Parent Channel Water level to the rate of Change of the Outlet discharge.

[2011]

Ans :

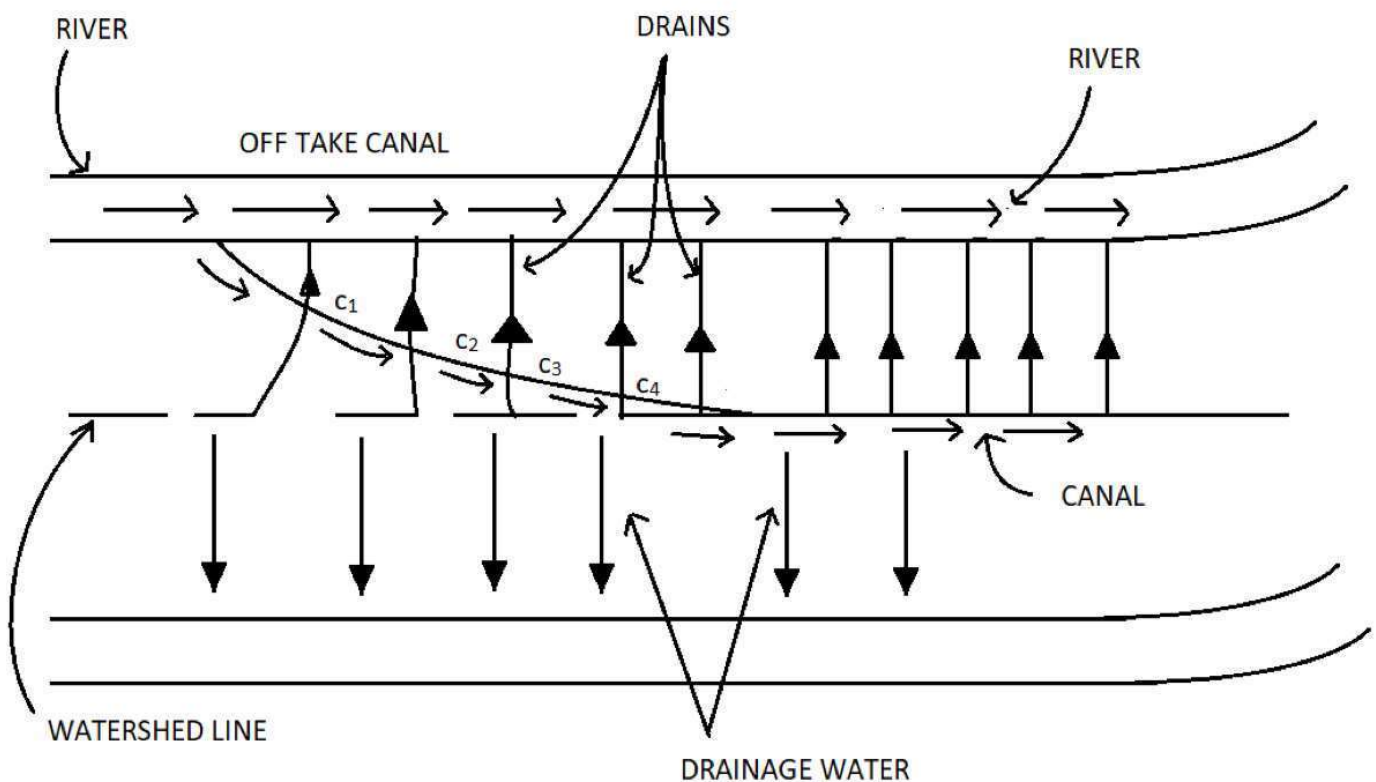
- ⑥ Outlet discharge to the rate of Change of the discharge of the Parent Channel.

$$F = \left[\frac{dq/q}{dQ/Q} \right]$$

MISCELLANEOUS

CROSS DRAINAGE WORK

- A Cross drainage Work is a Structure Which is Constructed at the Crossing of a Canal a natural drain, so as to disposed off drainage water without interrupting the Continuous Canal Supply.
- In Whatever way Canal is aligned, Cross Drainage works are Required.
- Since Cross drainage Work is generally a Costly Construction it must be avoided as possible.
- Cross drainage is required to minimum extent in Watershed ridge line Canal and is Required to maximum extent in Contour Canal.



••TYPES OF CROSS-DRAINAGE WORK

— The Drainage water intercepting the Canal Can be disposed off in either of the following ways —

① By Passing the Canal Over the Drainage

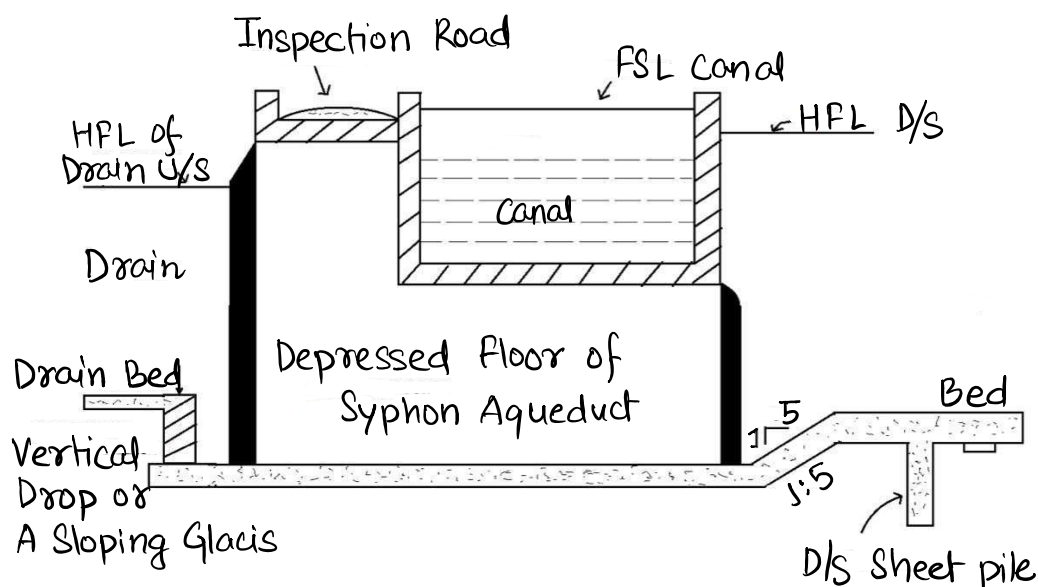
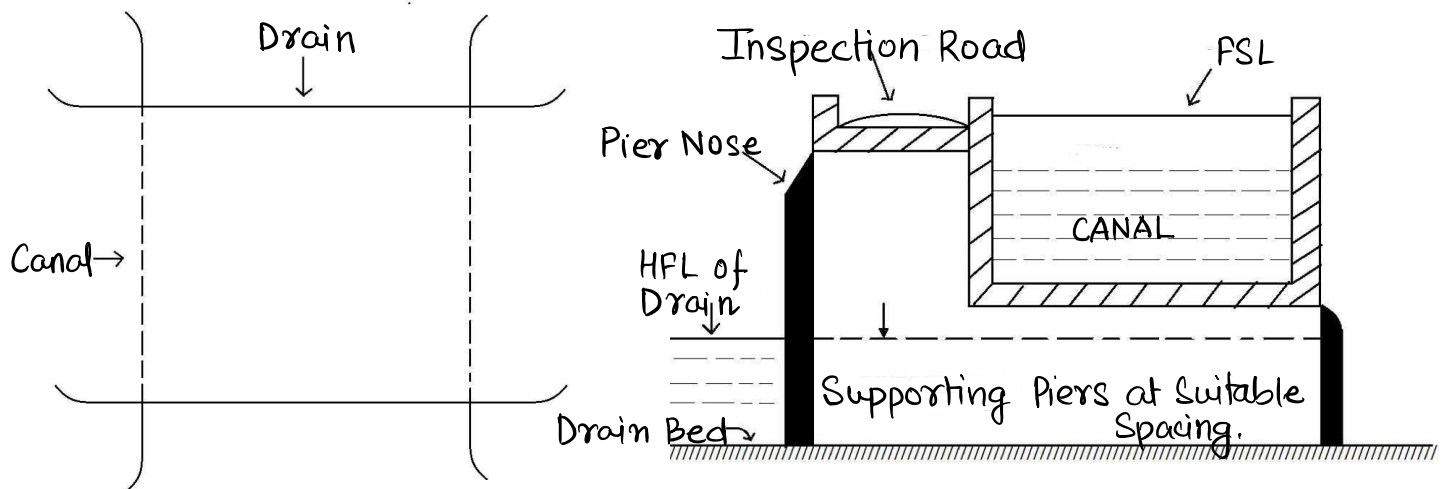
— It Can be achieved by Use of any of following Units.

② AQUEDUCT & SYPHON AQUEDUCT

— In these Works, canal is taken Over the the natural drain, Such that drainage water runs below the Canal either freely Under gravity or Under Syphoning pressure.

— When the HFL of the drain is Sufficiently below the bottom of Canal, So that the drainage can pass freely below it Under Gravity, it is termed as 'AQUEDUCT'.

— But, if the HFL of drain is higher than actual Canal bed and the Water passes through the Aqueduct Under Syphonic action, it is termed as 'SYPHON AQUEDUCT'.



- In these type of work, an inspection road is generally provided along with the Canal.
- In this type of work, the Canal remains Open to inspection throughout & the damage caused by flood is minimum.

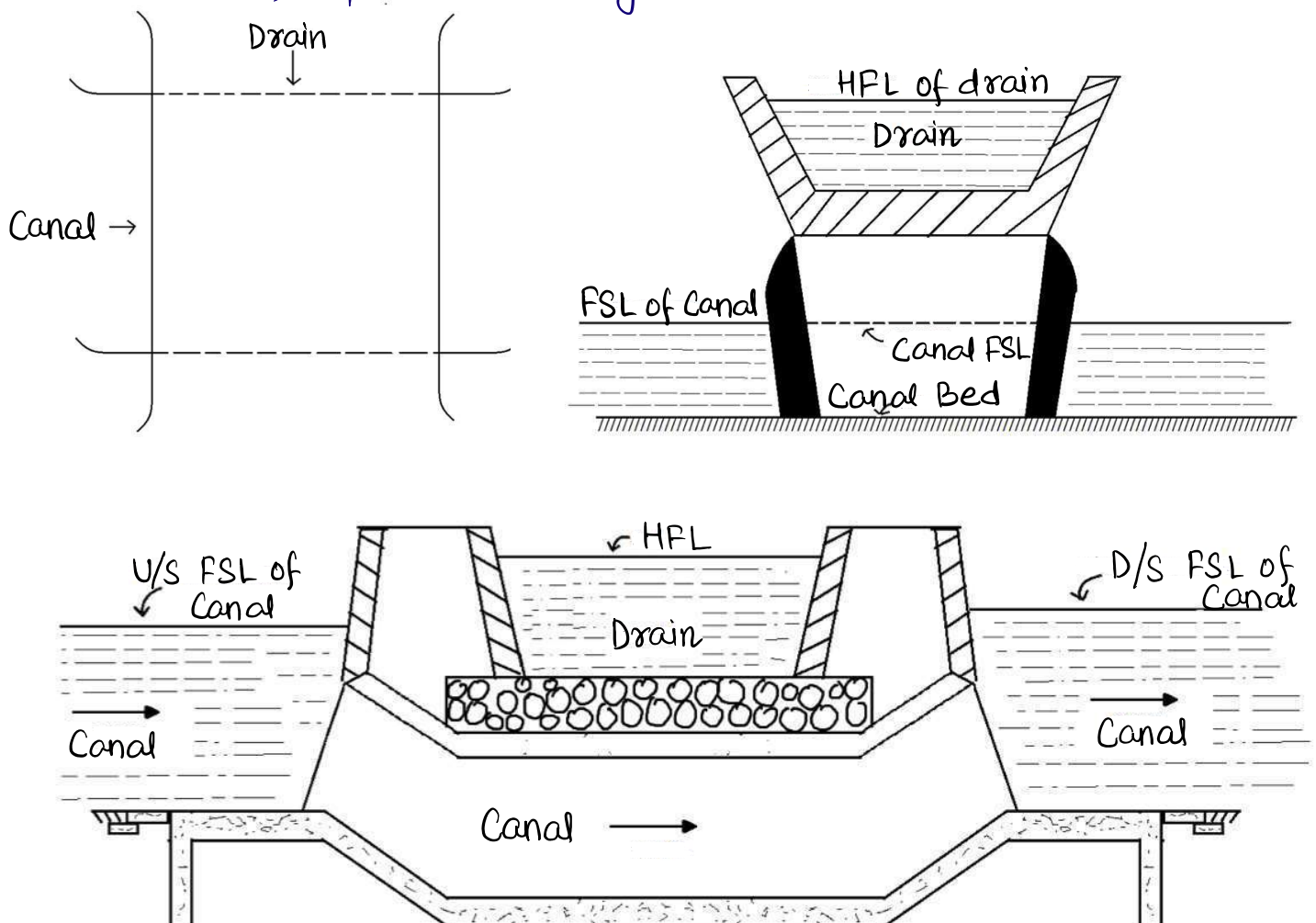


② By Passing the Canal below the Drainage

- It can be achieved by the following Units —

⑥ SUPER PASSAGE & SYPHON

- In these works, drain is taken over the Canal such that if Canal water runs below the drain freely Under Gravity it is termed as 'SUPER PASSAGE'. (When FSL of Canal is below the bed level of drain).
- However, if FSL of Canal is above the drain bed level, water in Canal flows Under Syphonic Pressure, the Structure is termed as 'CANAL SYPHON OR SYPHON'.
- In this Case, Inspection road along the Canal Cannot be Provided.



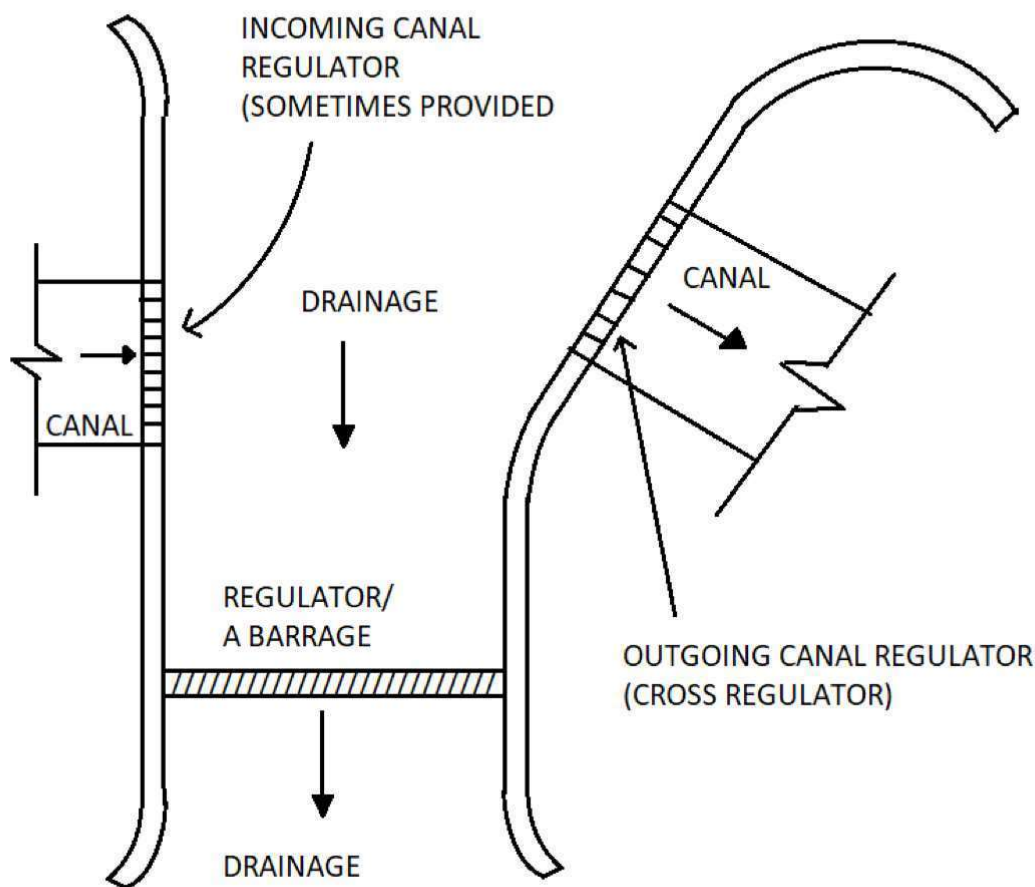
(iii) By Passing the drain Through Canal



— It Can be Achieved by Any of the following Units —

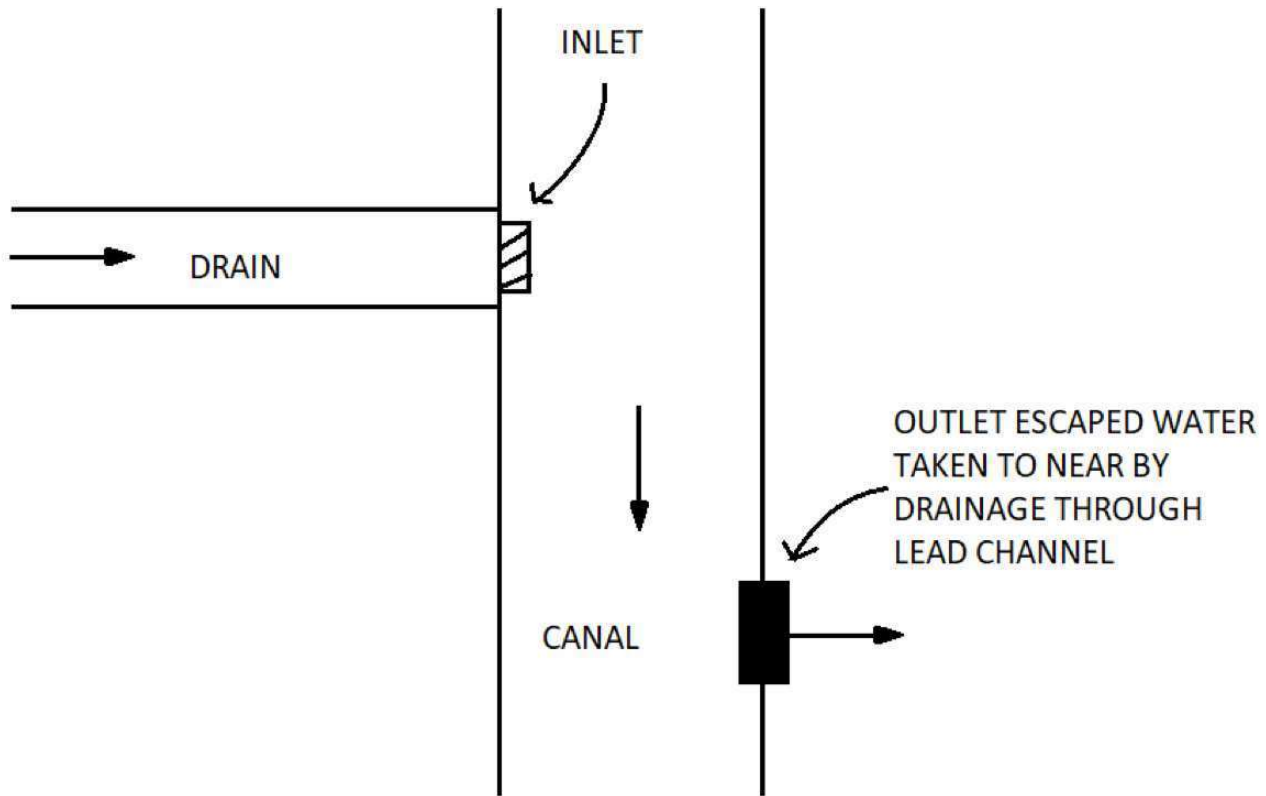
© LEVEL CROSSING

- In this type of CD Work, the Canal Water is allowed to mix with drain Water.
- A level Crossing is generally provided When a large Canal meets large drainage at Same level.



④ INLET & OUTLET

- An Inlet is a Structure Constructed in Order to allow the drainage Water to enter the Canal & get mixed with Canal water & thus to help in increasing Canal Supplies.
- When the drainage discharge is high or if the Canal is Small, so that the Canal Section Cannot take entire drainage, an outlet may Sometimes be Provided to escape out additional Water.
- It is not necessary that the Number of Outlets must be equal to Number of Inlets.

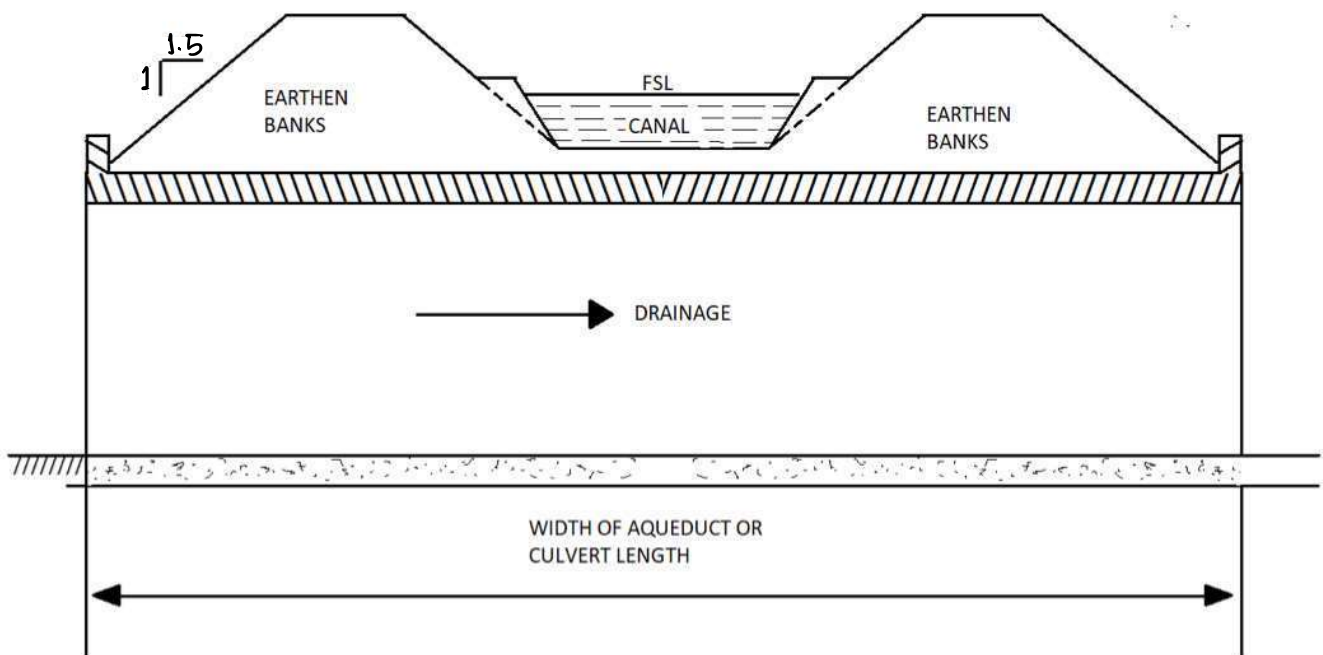


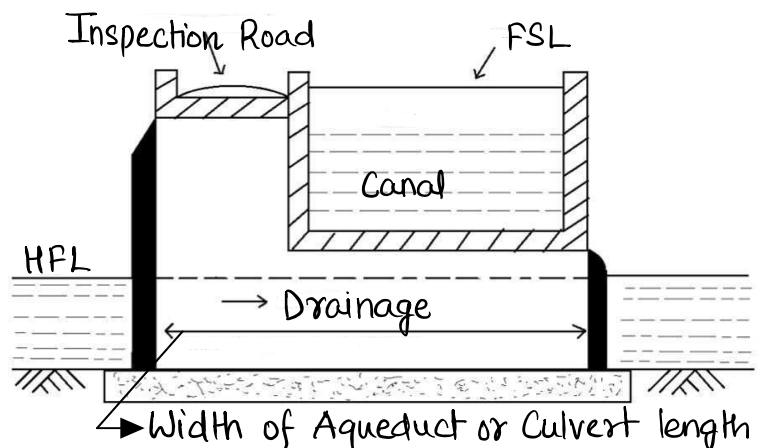
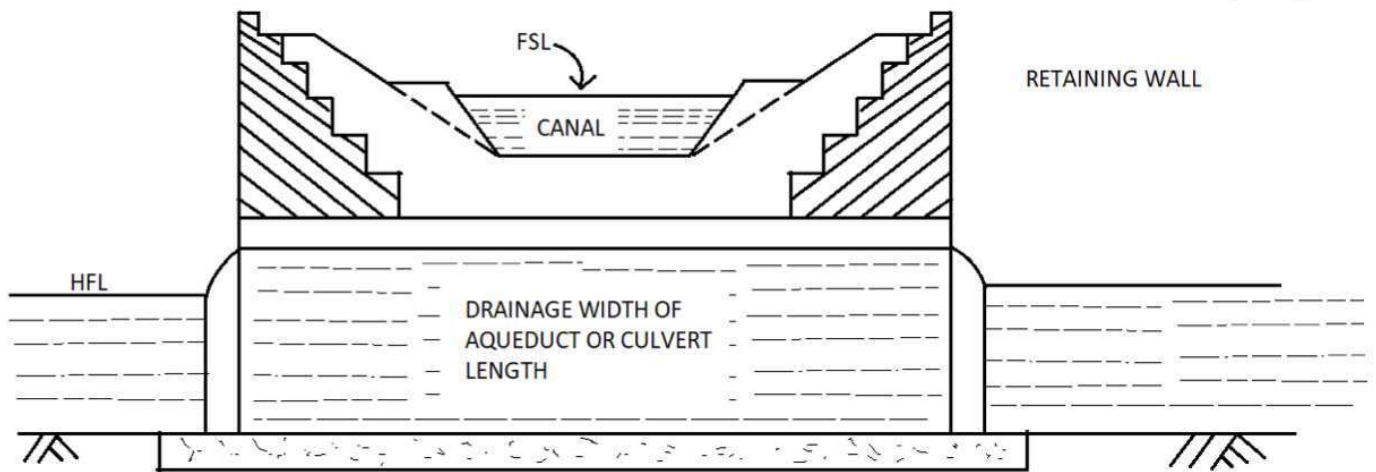
Note : In level Crossing of inlets & outlets, Sluice drainage is allowed to mix with Canal Water, they are considered to be inferior to aqueduct or Super-passage.



•• TYPE OF AQUEDUCTS AND SYPHON AQUEDUCTS

— These are also classified on the basic Nature of their Banks.





Note : It can be considered that the choice of a Particular type of Aqueduct depends upon the length of Aqueduct { i.e. width of the drainage } in Relation to the Size of Canal.

— On a Very small drain, type III is Most economical & on a very wide drainage type I is economical.

•• FLUMING OF CANAL

— The Contraction in the Waterway of the Canal is termed as FLUMING OF CANAL.

— It will reduce the length of barrels or width of the Aqueduct.

— It may induce economy in certain type work. { Type - III Aqueduct }.

— This is also required in cases where Canal approaches rock terrain in front of it.

— It can be achieved by any of the following methods.

① MITRA'S METHOD OF DESIGN OF TRANSITION



— In the first Case, hyperbolic Transition is provided of which is given by —

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

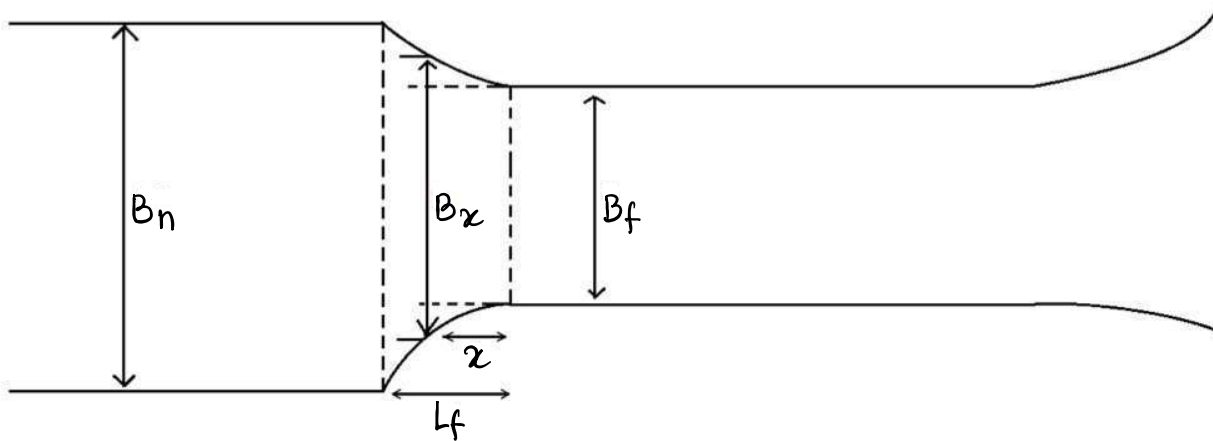
Where,

B_n = Bed Width of Normal Channel Section.

B_f = Bed Width of flummed Channel Section.

B_x = Bed width at any distance x from the flummed Section.

L_f = length of transition.



② CHATURVEDI'S METHOD OF DESIGN

— In this Case, Semi-Cubical Parabolic Transition is provided and its equation is given by —

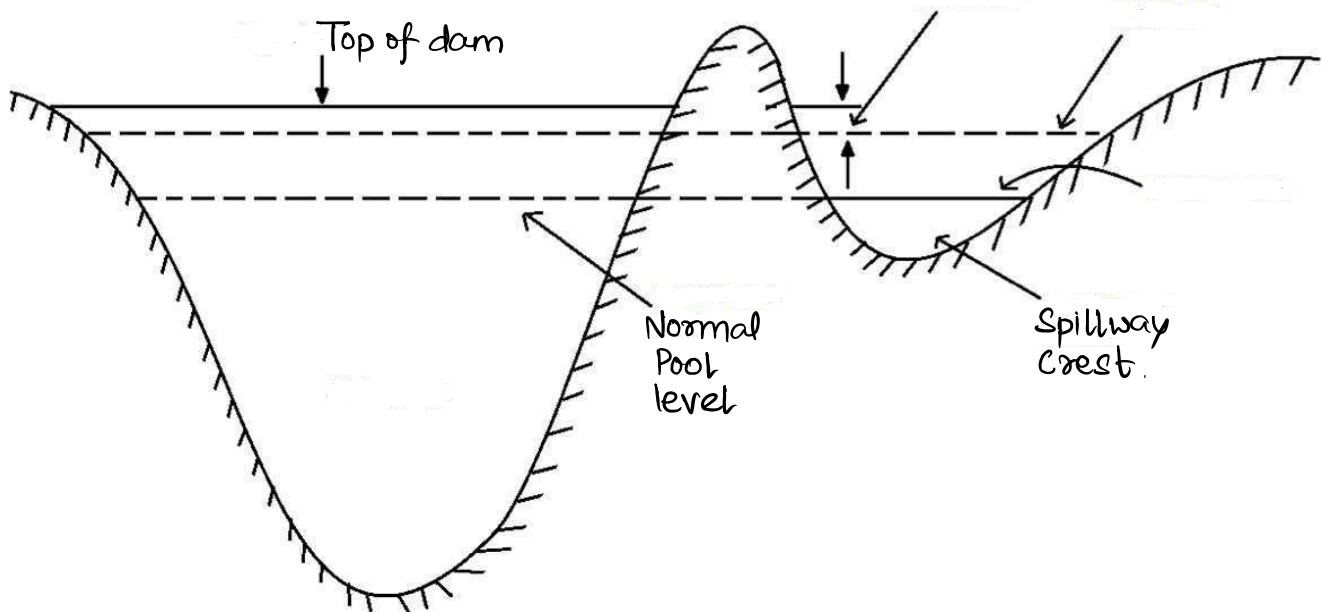
$$x = \frac{L \cdot B_n^{3/2}}{B_n^{3/2} - B_f^{3/2}} \left[1 - \left\{ \frac{B_f}{B_x} \right\}^{3/2} \right]$$



SPILLWAYS AND SPILLWAY GATES



- A Spillway is a Structure Constructed on a dam Site, for effective disposing of the Surplus water from Upstream to Downstream.
- If this excess water is not being disposed, it would led to failure of the dam hence Spillway acts as a Safety Valve of the dam.
- A Spillway Can be located either within the body of the dam, or at the end of it or entirely away from it independently in a Saddle or flank.
- If a deep Narrow gorge with Steep banks, Separated from a flank by a hill lock with its level above the top of the dam is available, the Spillway Can be best built independently of the dam.



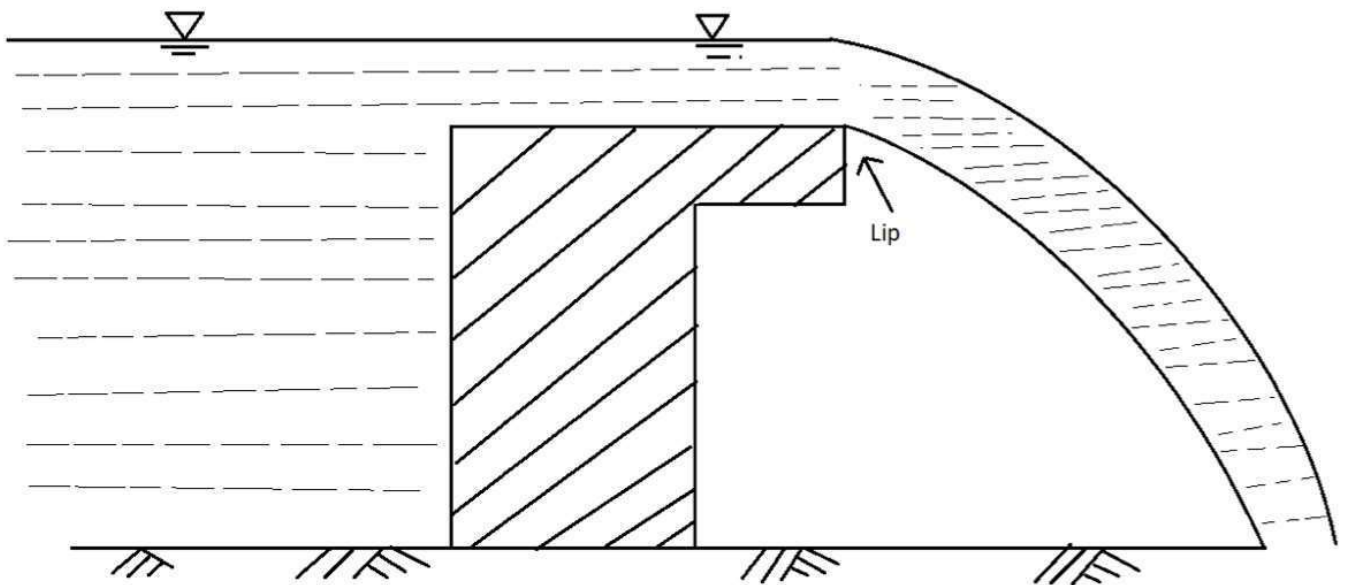
Note: If the flow of water Over a Spillway is Controlled, it is termed as Controlled Spillway and if it is not Controlled it is termed as Uncontrolled Spillway.

- Depending Upon the type of Structure Constructed for disposing Surplus water, Spillways are of following types —

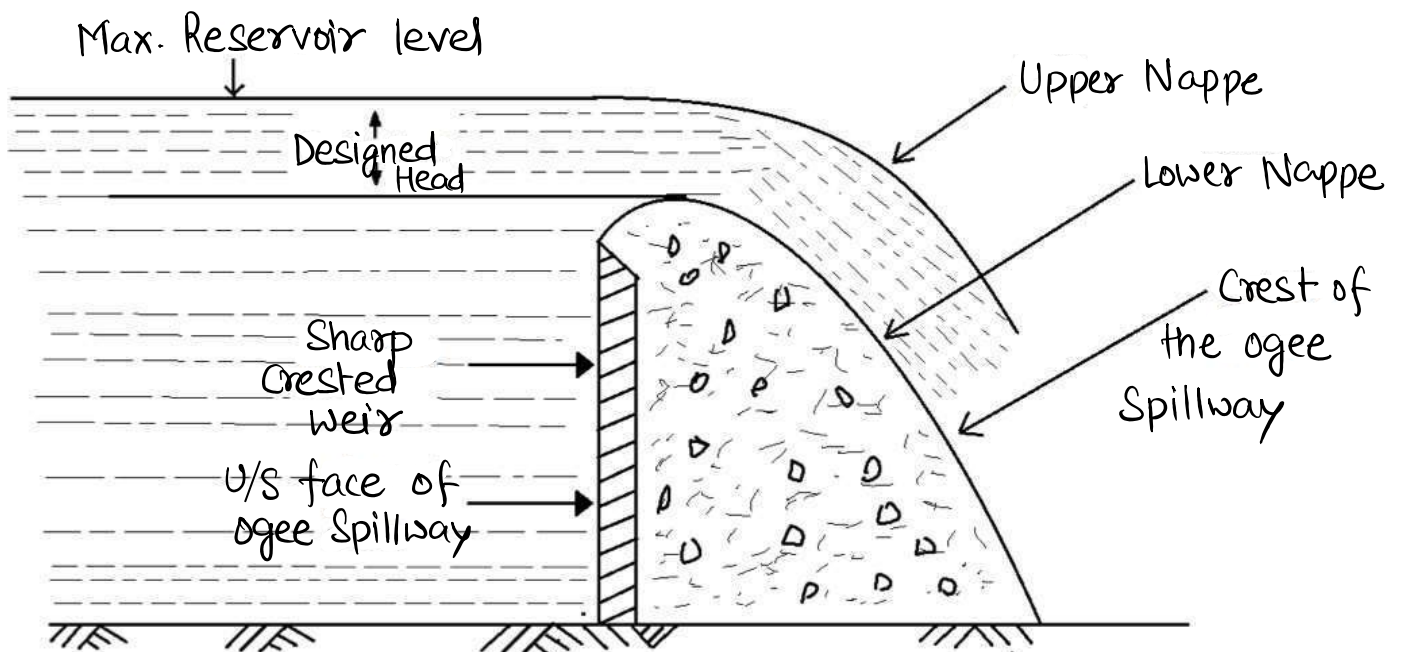
① STRAIGHT DROP SPILLWAY

- This is the Simplest type of Spillway and may be Constructed on Small Embankment or on thin arch dam.

— It is a low weir and may have Vertical or inclined walls and is also provided with 'LIP' at the Crest in some cases.



② OGEE SPILLWAY OR OVERFLOW SPILLWAY

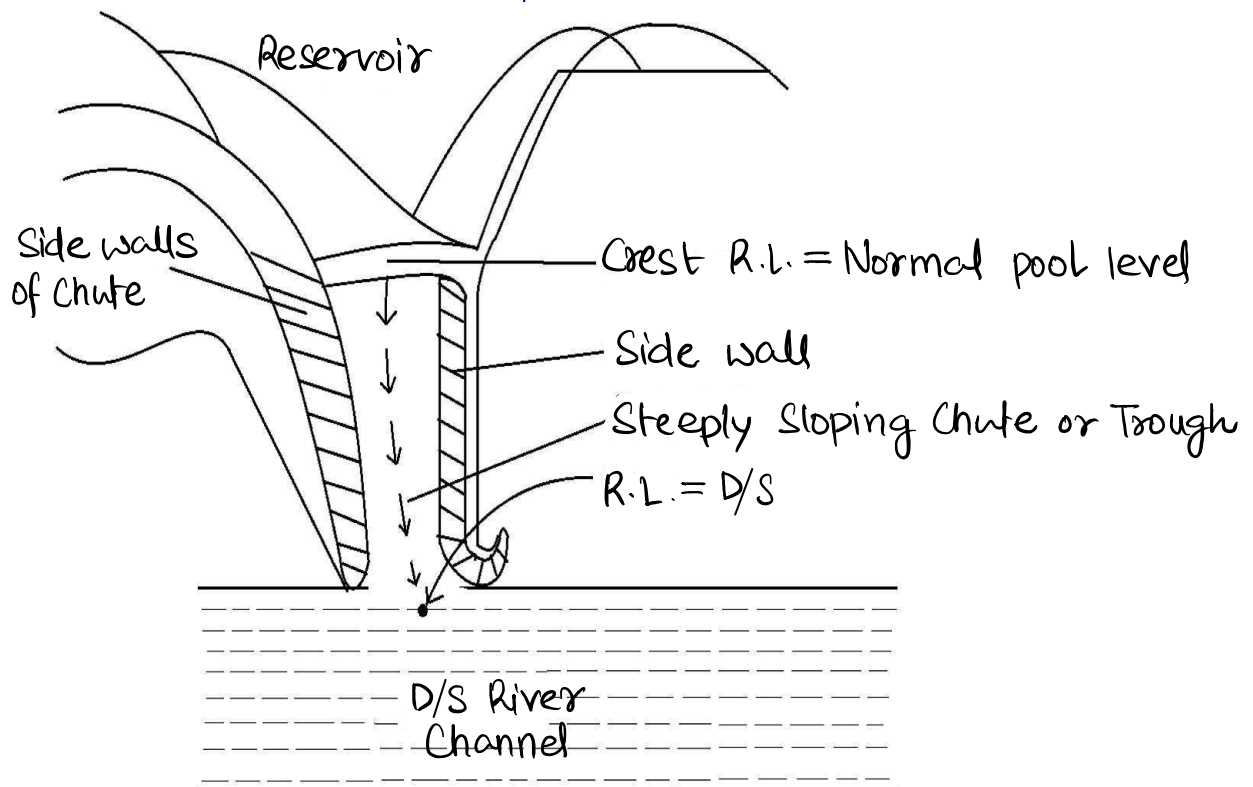


- It is an improvement Upon the free Overfall Spillway and is widely Used Concrete, Masonary arch and buttress dam.
- Such a Spillway can be easily Used on Valleys Where the width of river is Sufficient to provide the Required Crest length and River bed below Can be Protected from Scouring.
- It follows the profile lower Nappe.



(iii) CHUTE SPILLWAY / TROUGH SPILLWAY

- An Ogee Spillway is mostly Suitable for Concrete gravity dam When the Spillway is located in the body of dam in Same Valley.
- But the earthen and rockfill dam Cannot be provided with Ogee Spillway in Such Case a Seperate Spillway is generally Constructed, Where Chute type Spillway Can be Provided.
- It is the Simplest type of Spillway and can be easily provided at low Cost. It may also be termed as Work weir / Saddle weir.

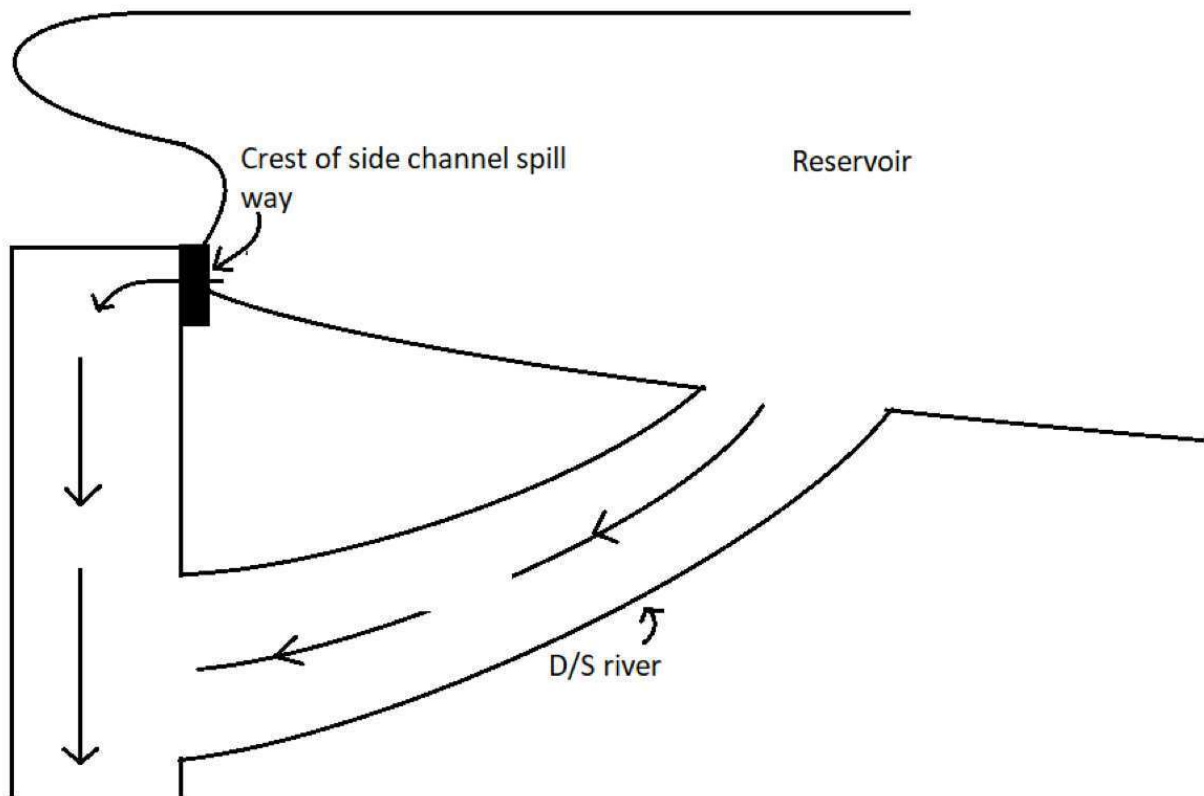


(iv) SIDE CHANNEL SPILLWAY

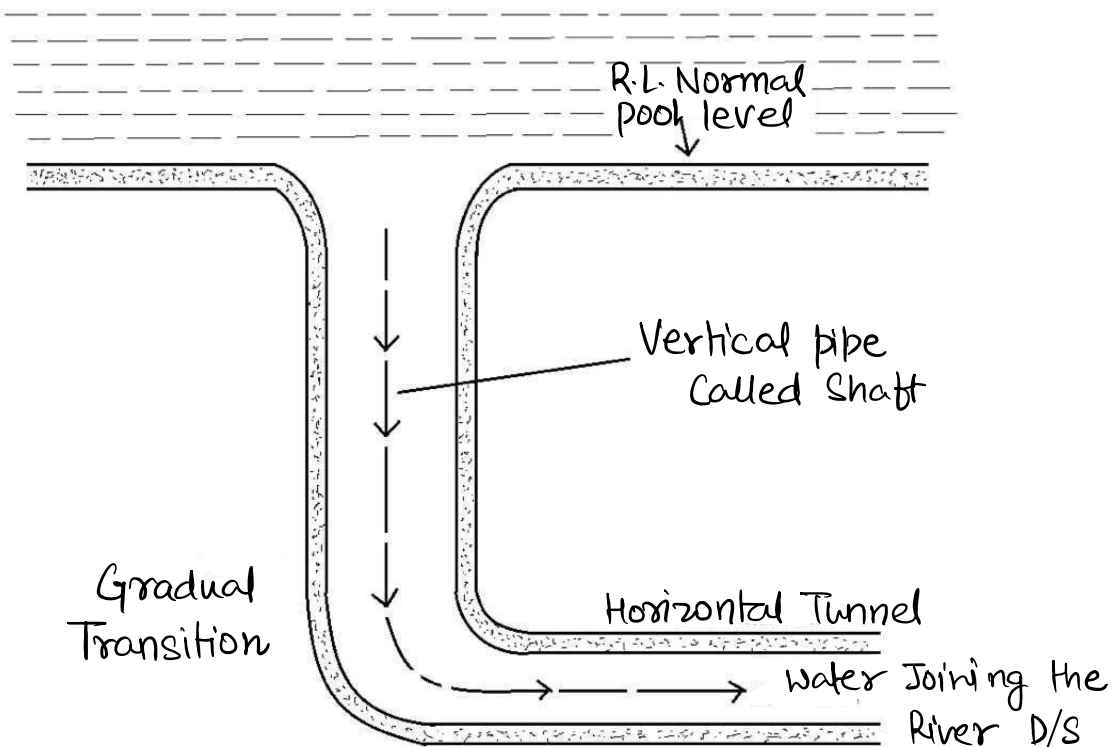
- It differs from Chute Spillway in the sense that while in Chute Spillway, the Water flows at right Angles to the Weir Crest after Spilling Over it.

Whereas, in this Case the flow of Water after Spilling Over the Crest is turned by 90° Such that its flow parallel to weir.

- It is provided in Narrow Valley Where No side flank of Sufficient width is available and front length Available is also not Sufficient Which will Require heavy Cutting in these type of Spillway.

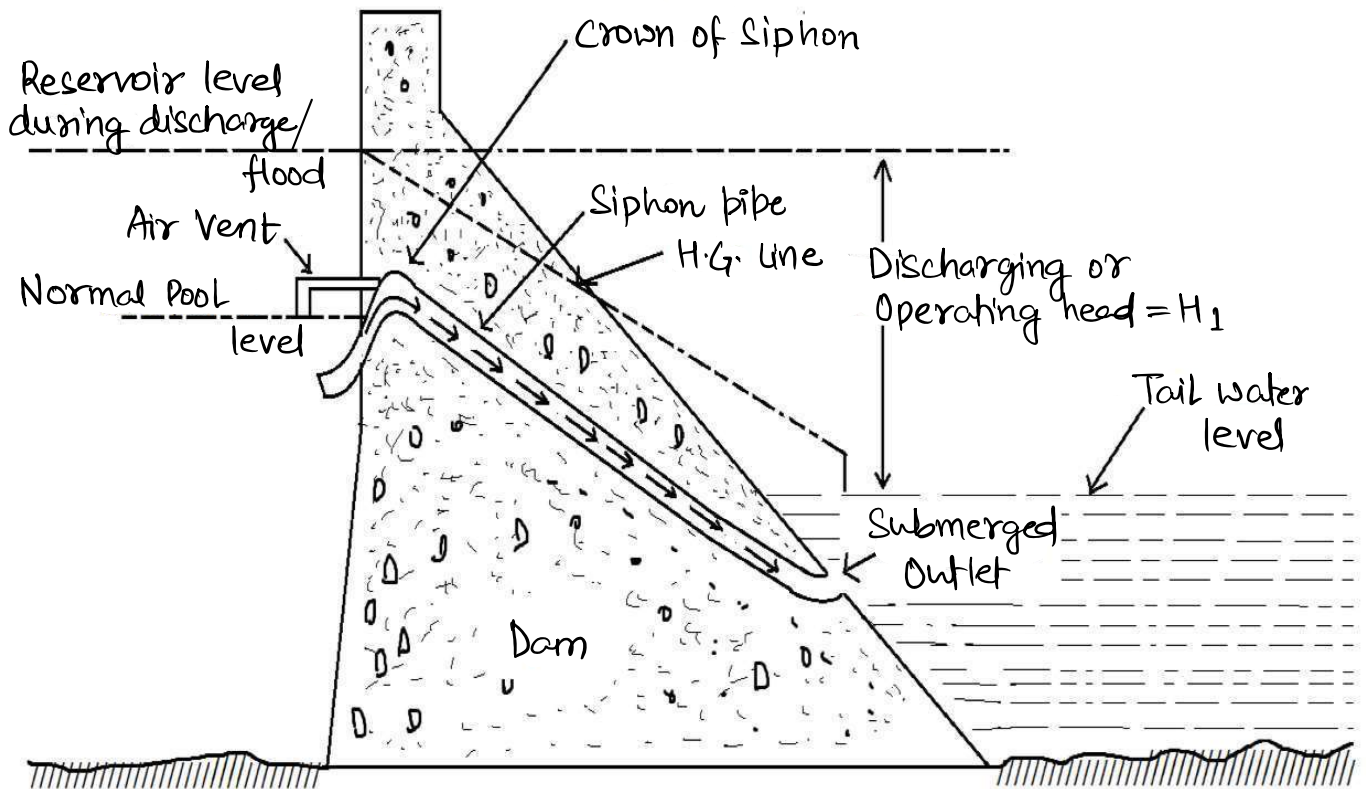


⑤ SHAFT SPILLWAY



- In this the water from the Reservoir enter into a Vertical Shaft which Conveys this Water into a horizontal tunnel and finally discharges the Water into the River downstream,
- Sometimes, the Vertical Shaft may be Excavated through some natural rock Island Which Increases its Cost.

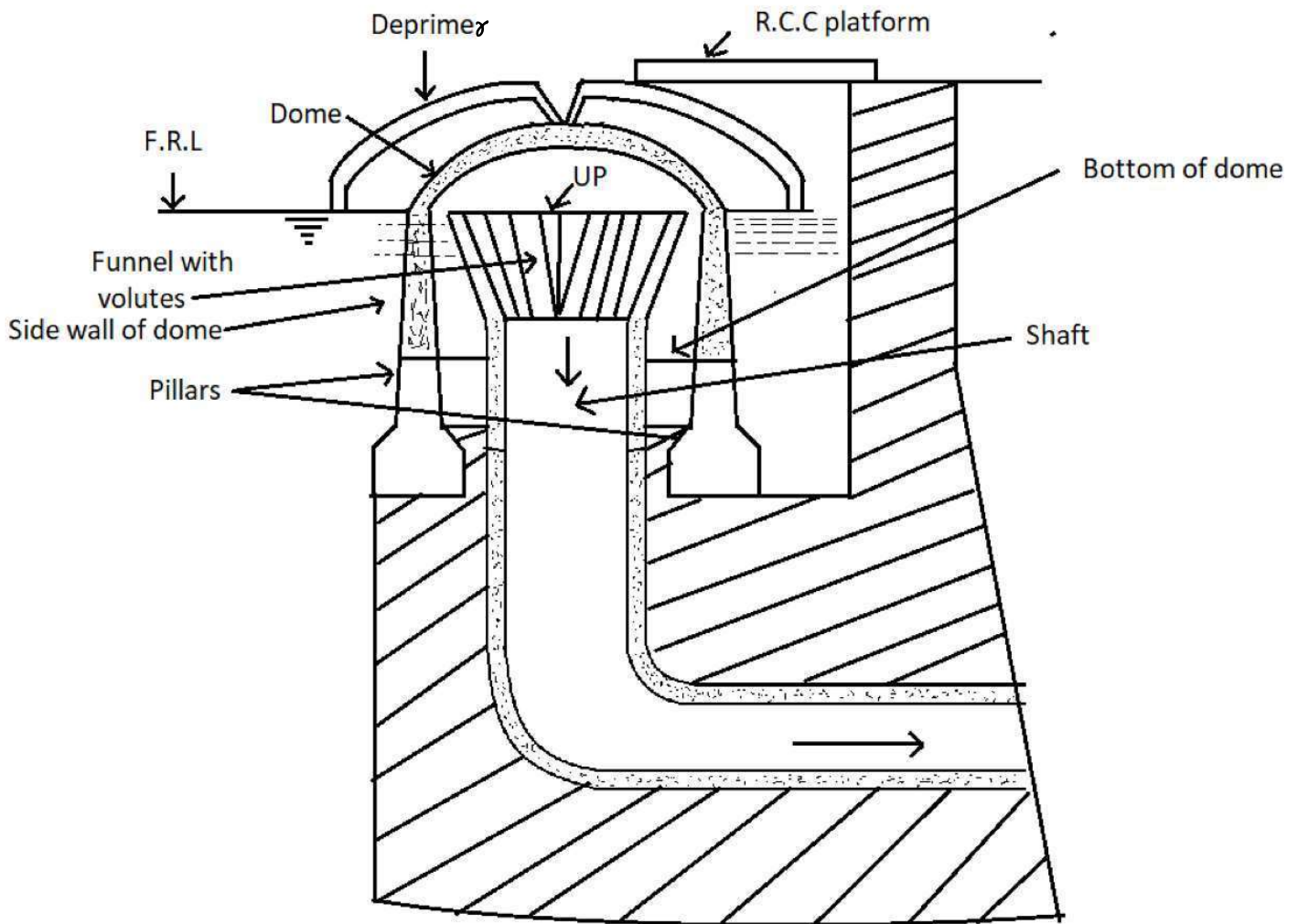
(VI) SYPHON SPILLWAY



- In this Case, the Siphon pipe has been Installed within the body of dam to drain the Surplus water from the Reservoir Under Syphonic action.

(VII) VOLUTE SYPHON SPILLWAY



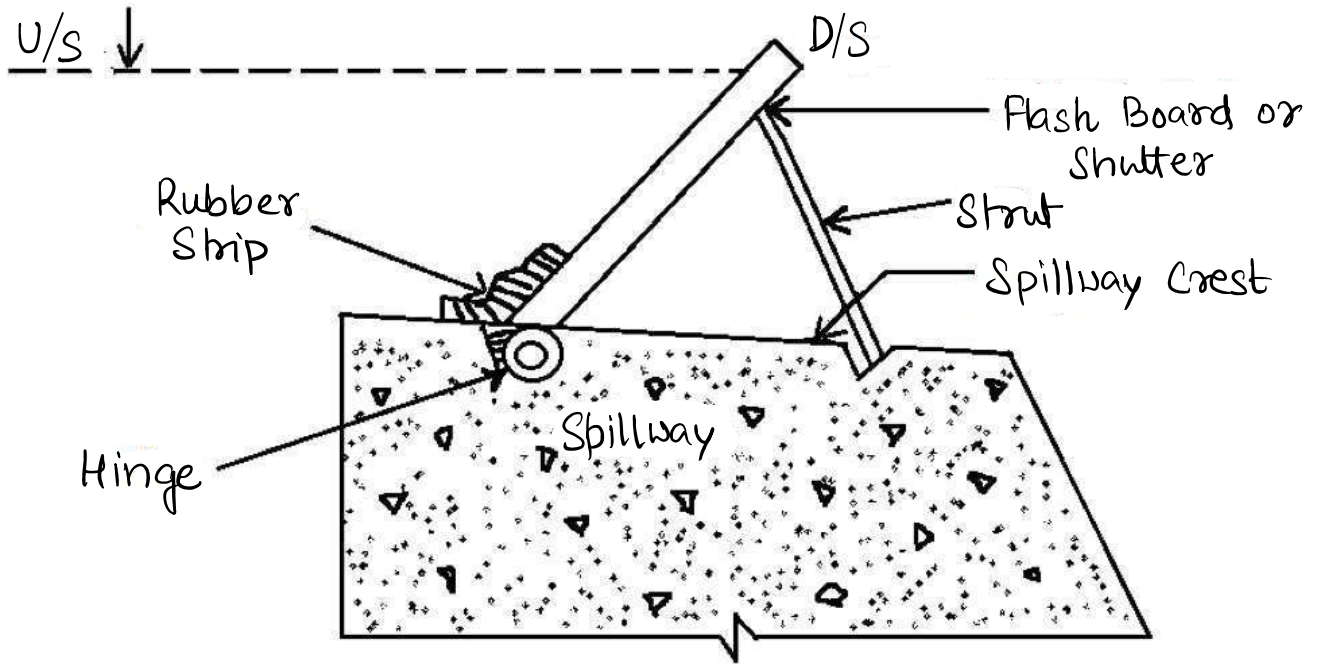


•• SPILLWAY CREST GATES

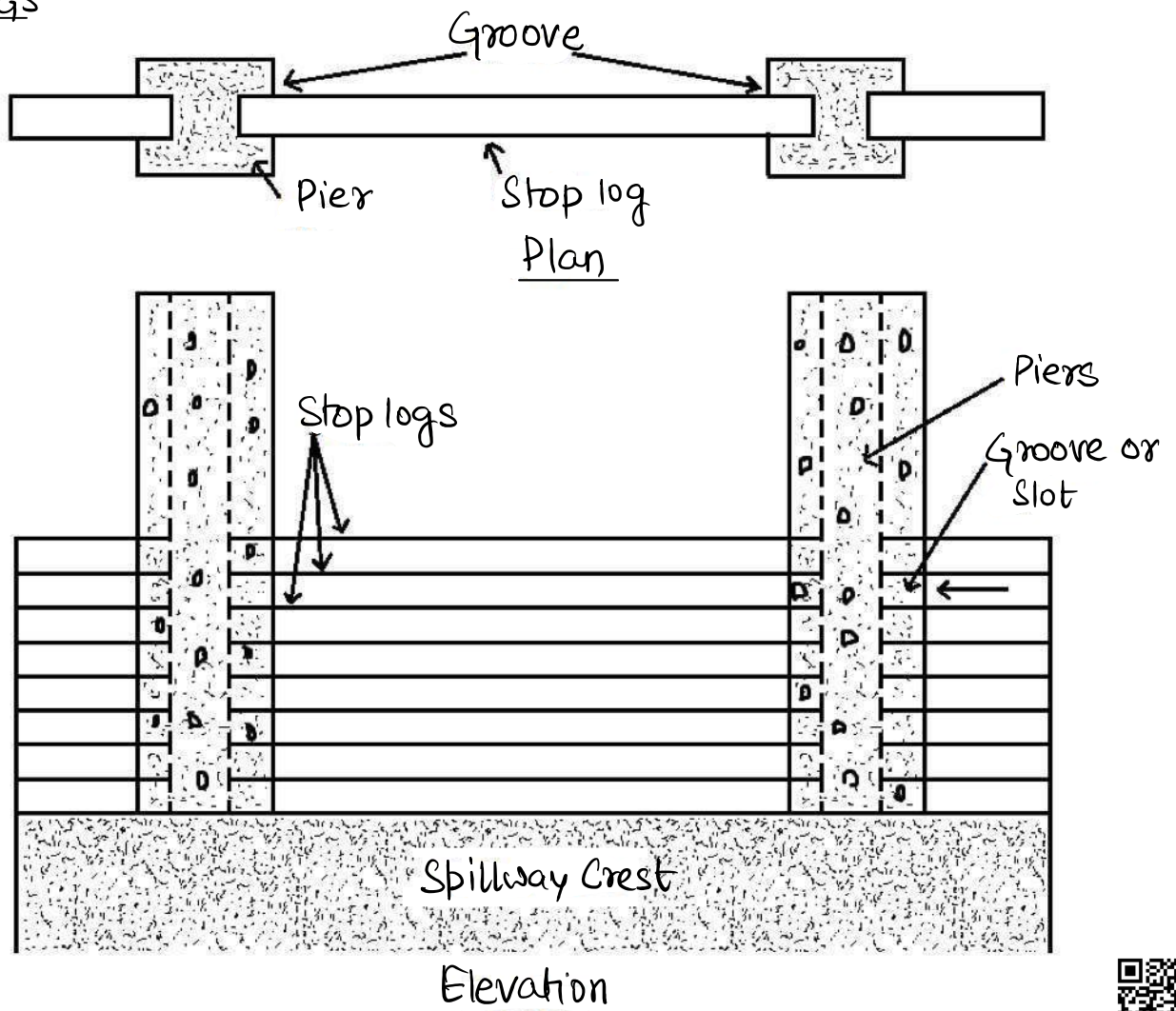
- If a temporary barrier can be installed over the Permanent Raised Crest of a Spillway additional water can be stored in it or the height of the Gate can be Reduced.
- The temporary barrier are termed as Gates.
- These Gates are of following types—
 - ① Dropping Shutter/Permanent Flash Board.
 - ② Stop logs and Needles.
 - ③ Vertical Lift/Rectangular
 - ④ Radial Gates/Tainter Gates
 - ⑤ Drag Gates.



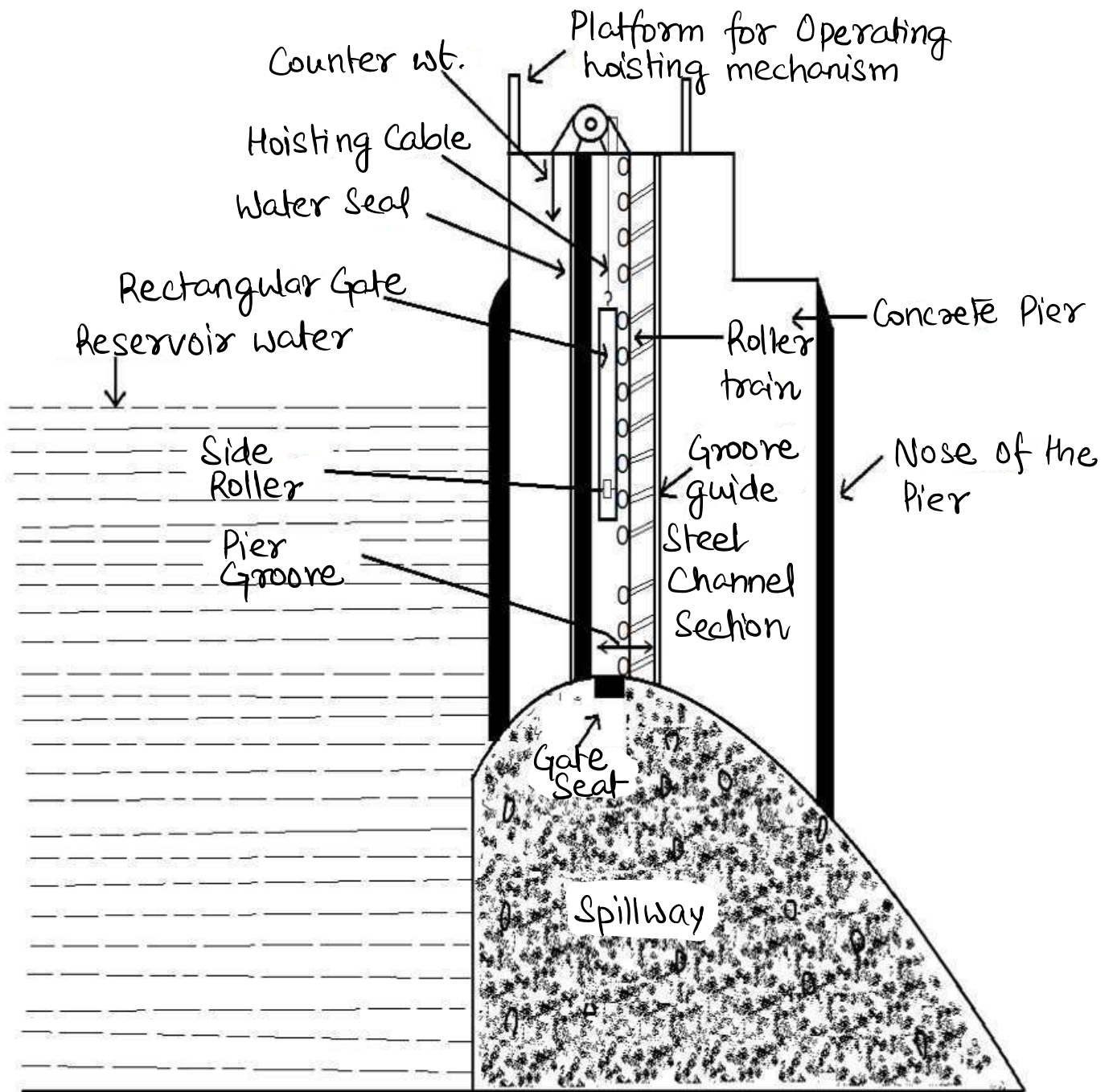
① DROPPING SHUTTERS / _____



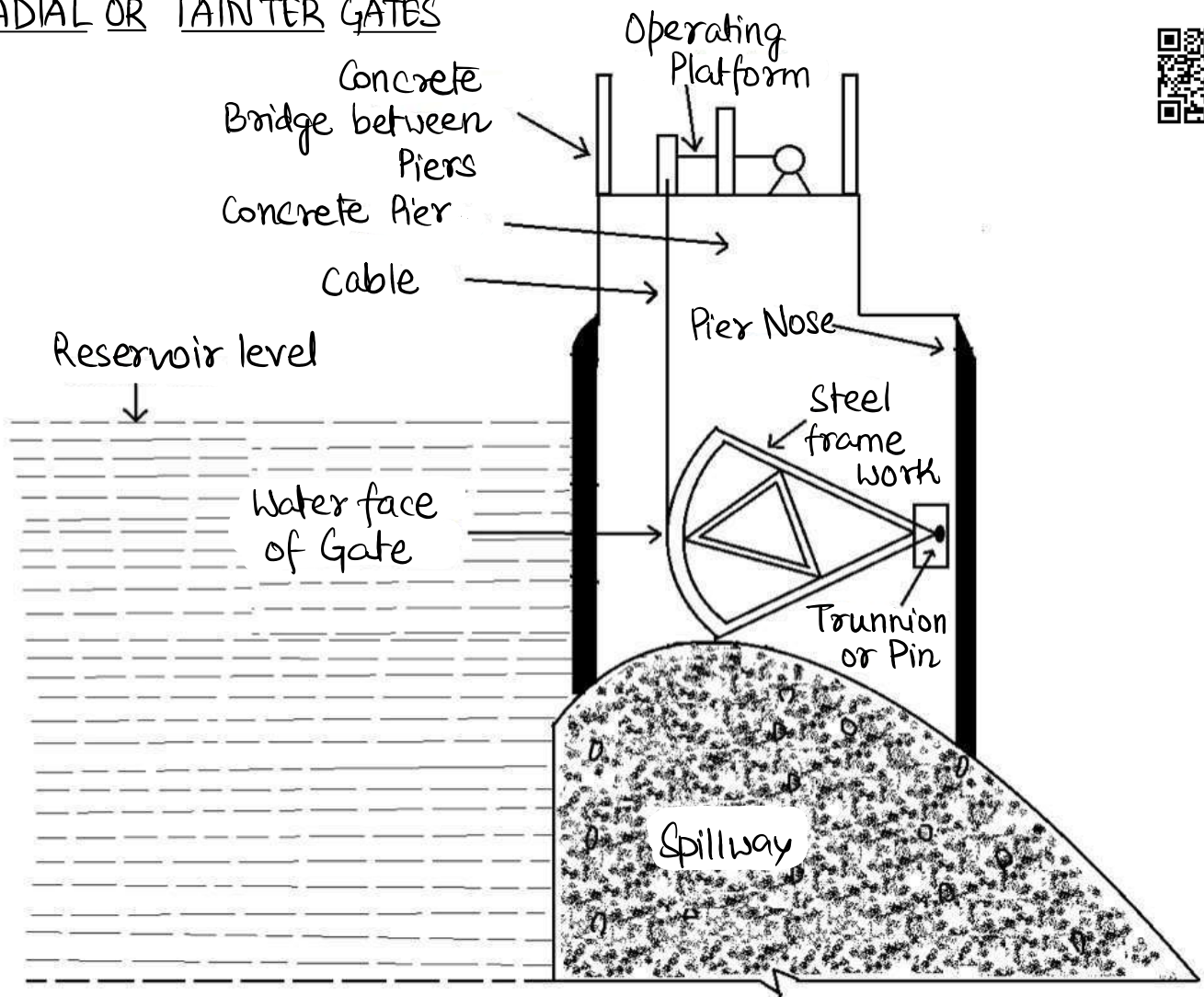
② STOP LOGS



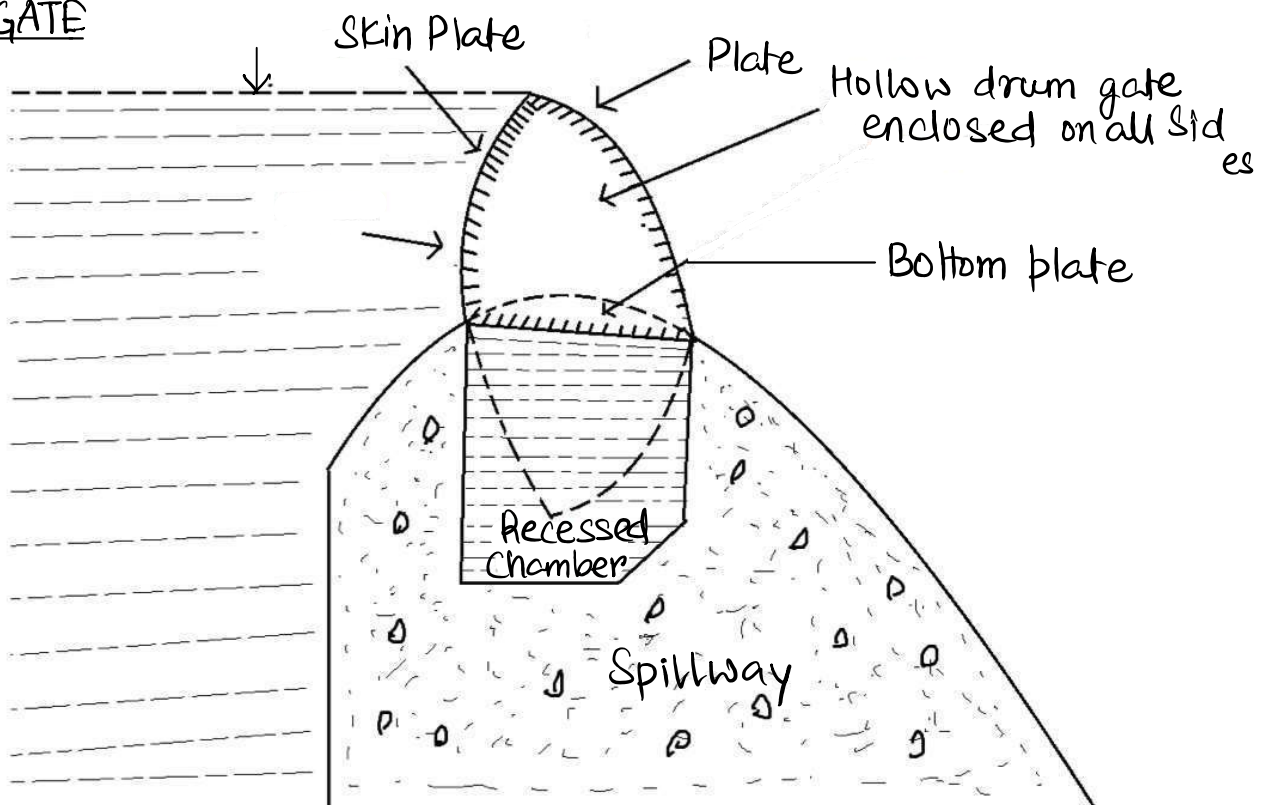
③ VERTICAL LIFT OR RECTANGULAR GATES



④ RADIAL OR Tainter GATES



⑤ DRUM GATE



PREVIOUS YEAR QUESTIONS

ESE

Q. Consider the following data while designing an expansion transition for a Canal by Mitra's method :

Length of flume = 16 m

Width of Throat = 9 m

width of Canal = 15 m

If B_x is the width at any distance x from the flumed section, the values of B_x at $x = 8\text{ m}$ and at $x = 16\text{ m}$ are nearly

- (a) 10.8 m & 15 m
- (b) 11.3 m & 15 m
- (c) 10.8 m & 13 m.
- (d) 11.3 m & 13 m.

[2019]

Ans:

(b) 11.3 m and 15 m.

— According to Mitra, channel at any section, at a distance x from the flumed section is given by,

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

Where $B_n = 15\text{ m}$, width of Canal

$B_f = 9\text{ m}$, width of throat

$L_f = 16\text{ m}$, length of flume.

B_x = width at any distance x from flumed section.

at $x = 8\text{ m}$,

$$B_x = \frac{15 \times 9 \times 16}{\{16 \times 15\} - \{(15 - 9) \times 8\}}$$
$$= 11.25 \cong 11.3\text{ m.}$$

at $x = 16\text{ m}$,

$$B_x = \frac{15 \times 9 \times 16}{16 \times 15 - (15 - 9) \times 16}$$
$$= 15\text{ m.}$$

Q. In a Syphon aqueduct, the Worst Condition of Uplift on the floor occurs when

- (a) The Canal is full and the discharge is empty with water table at drainage bed level.
- (b) The Canal is empty and the drainage is full, with water table at drainage bed level.
- (c) Both the Canal and the drainage are full.
- (d) The Canal is empty and the drainage is full with water table below the floor.

[2018]

Ans:

- ① The Canal is full and the discharge is empty with water table at drainage bed level.
— In Case of Syphon Aqueduct drain flows below the Canal Under Syphonic action.
The maximum Uplift Under the worst Condition would Occur when there is no water flowing in the drain and the water table has risen upto drainage bed. The maximum net Uplift in Such a Case would be equal to the difference in level between drainage bed and bottom of floor.

Q. Consider the following Statements :

1. In a Super passage, the drain runs over the Canal.
2. In a Syphon, the drain runs below the Canal.
3. In a Syphon aqueduct, type-II, the Canal banks are made of RCC walls.

Which of the Statements are Correct ?

- ① 1 Only
② 1 and 2 Only.
③ 2 and 3 Only.
④ 1 and 3 Only.

[2011]

Ans:

- ② 1 and 2 Only.

— A Syphon is a Cross drainage Structure similar to an aqueduct except that the Stream bed is depressed locally where it passes Under the through the Canal and the Barrels discharges the Stream flow Under Pressure.
In Syphon Aqueduct, type II the natural drain is not Carried Over the Canal. So no need for RCC Canal Walls.

GATE

- Q. If the Path of an irrigation Canal is below the level of a natural Stream, the type of Cross drainage Structure provided is
- ① Aqueduct
② Super Passage
③ Sluice gate
④ Level Crossing.

[2019-Shift 1]

Ans:

- ② Super passage

— If the path of an Irrigation Canal is below the bed level of a natural Stream, the type of Cross drainage work provided is Super passage.

- Q. Group I Contains three broad Classes of Irrigation Supply Canal Outlets. Group II Presents hydraulic performance attributes.

Group - I

P. Non-Modular Outlet

Q. Semi-Modular Outlet

R. Modular Outlet

Group - II

1. Outlet discharge depends on the water levels in both the supply canal as well as the receiving water course.

2. Outlet discharge is fixed and is independent of the water levels in both the supply canal as well as the receiving water course.

3. Outlet discharge depends only on the water level in the supply canal.

The correct match of the items in Group I with the items in Group II is

- (a) P-1 ; Q-2 ; R-3
- (b) P-3 ; Q-1 ; R-2
- (c) P-2 ; Q-3 ; R-1
- (d) P-1 ; Q-3 ; R-2

[2017-Shift 1]

Ans :

- (d) P-1 ; Q-3 ; R-2

— Non Modular Outlet : These are outlets through which the discharge depends upon the difference of head between the distributary and the water course.

Semi Modular Outlet : These are outlets through which the discharge is independent of the watercourse so long as a minimum working head is available and depends upon distributary water surface level.

Modular Outlet : These are outlets through which 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.

Q. Which one of the following equations represents the downstream profile of Ogee Spillway with vertical upstream face? (x, y) are the coordinates of the point on the downstream 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) $\frac{y}{H_d} = -2.0 \left\{ \frac{x}{H_d} \right\}^{1/1.85}$

[2005]

Ans:

$$\textcircled{a} \frac{y}{H_d} = -0.5 \left\{ \frac{x}{H_d} \right\}^{1.85}$$

— For a Spillway having a Vertical Upstream face, the downstream crest is given by the equation.

$$x^{1.85} = -2 \cdot H_d^{0.85} \cdot y$$

$$\Rightarrow x^{1.85} = -2 \cdot \frac{H_d^{1.85}}{H_d} \cdot y$$

$$\Rightarrow \frac{y}{H_d} = -0.5 \left\{ \frac{x}{H_d} \right\}^{1.85}$$

SSC-JE

Q. Which of the following Spillways is least suitable for an earth dam?

- Ⓐ Chute Spillway
- Ⓑ Side Channel Spillway
- Ⓒ Shaft Spillway
- Ⓓ Ogee Spillway.

[2011]

Ans:

Ⓓ Ogee Spillway

— Ogee Spillway is an improvement upon the free Overall Spillway and is widely used with concrete, masonry, arch and buttress dam.