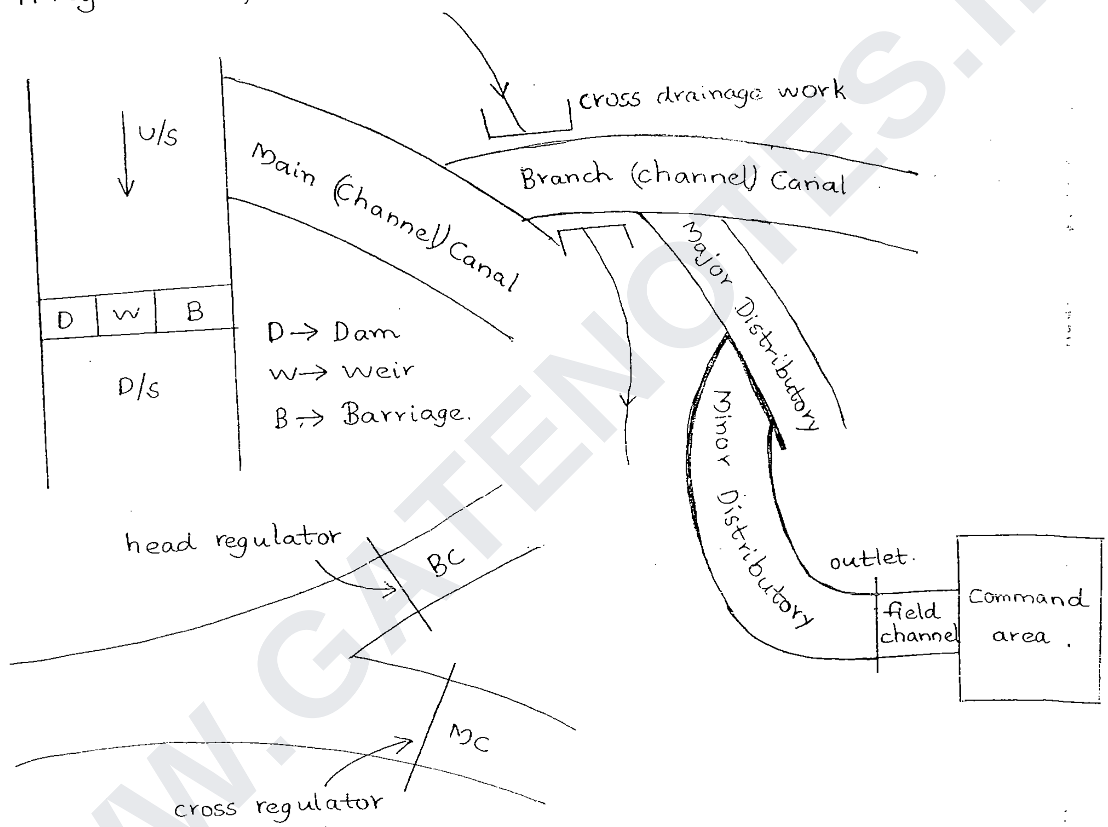
Oct,
DAY

IRRIGATION

Science which deals with artificial supply of water to the crops.

→ Irrigation System :



A regulator controls the flow of water into the canal (weir-like structure)

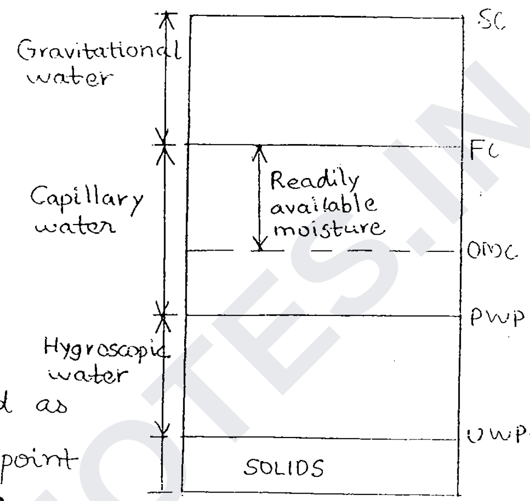
Irrigation system comprises of hydraulic structures like dam, weir, barriage, canal system, cross drainage works (like aquaduct), regulators, canal falls and canal escapes (Surplus escape & Silt escape)

* Hygroscopic water

Thin layer of water surrounding the soil grain which the roots of plants cannot absorb.

→ Types of Water

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



The moisture content expressed as percentage at ultimate wilting point is called 'Hygroscopic Coefficient'

* Hygroscopic Water

It consists of two parts:

(i) Amount of moisture absorbed from atmosphere by the soil grains.

(ii) Thin film of moisture sticking to the soil grain which cannot be extracted by the plant.

* Capillary water.

The amount of moisture stored in capillary pores which can be extracted by the plant is called capillary water of available water.

Saturation Capacity :- a state where all the voids are filled with moisture

Field capacity :- amount of moisture retained in the soil against pull of gravity.

Water stored in capillary pore against gravity pull in the root zone – field capacity

Water stored in the pore spaces of soil grains – saturation capacity

$$ET = \text{Evaporation} + \text{Transpiration}$$

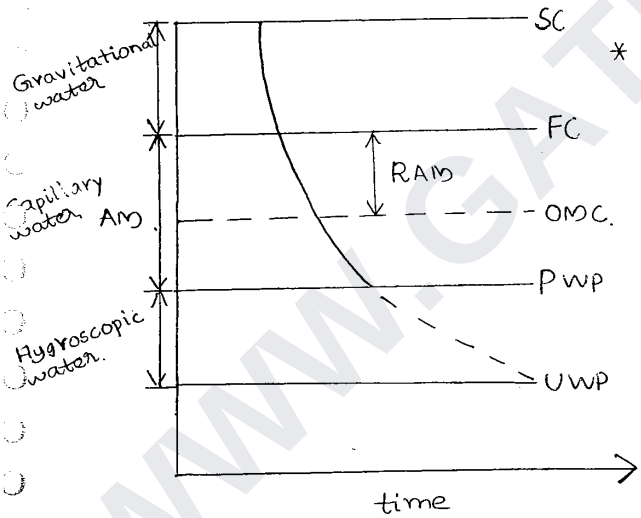
$$CU = \text{Consumptive Use} = E + T + \Delta W$$

↳ for metabolic activity

* Temporary Wilting Point – Plants can make use of water available in the soil.

* Permanent Wilting Point – Plant recovers only with application of water.

* Ultimate Wilting Point – even with application of water, plant cannot recover, ie, plant is dead.



* Available moisture (y):

$$AM, y = FC - PWP.$$

* Readily available moisture $= 75 - 80\%$ AM.

NOTE:

- Yield is less if irrigation is carried out below OMC
- Yield is more if irrigation is carried out above OMC
- Duty of irrigation engineer is to supply water when moisture content reaches OMC

* Gravitational water (or) Unavailable water $= SC - FC$

* Hygroscopic water

Thin layer of water surrounding the soil grains which the roots of plants cannot absorb.

→ Types of Water

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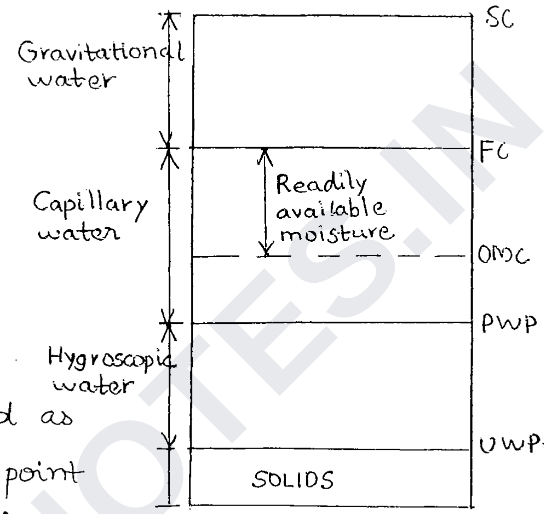

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Saturation Capacity :- a state where all the voids are filled with moisture

Field capacity :- amount of moisture retained in the soil against pull of gravity.

* Field Capacity, $FC = \dfrac{\text{wt. of moisture retained in unit area of soil}}{\text{wt. of soil in unit area.}}$

Let $d \to$ depth of soil.

$\gamma_w \to$ unit weight of water

$\gamma_d \to$ unit weight of dry soil.

$A \to$ area.

$$FC = \frac{\gamma_w * A * \text{depth of water}}{\gamma_d * A * d}.$$

$\therefore$ Depth of water at $FC = \dfrac{\gamma_d}{\gamma_w} * d * Fc.$

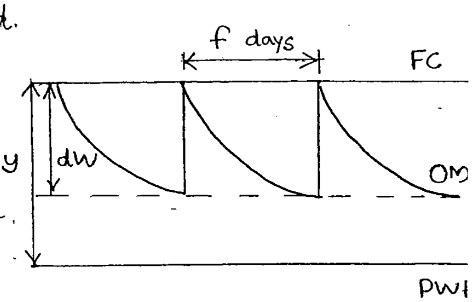
$= S * d * Fc.$

Similarly, depth of water at $PWP = S * d * PWP.$

$$\boxed{\left.\begin{array}{l}\text{Depth of available moisture}\\ \text{or Water holding capacity}\end{array}\right\} y = S * d * (Fc - PWP).}$$

$$\boxed{\text{Depth of moisture at OMC, RAM} = S * d * (Fc - OMC)}$$

→ Frequency (or) Irrigation Period.



$$f = \frac{dw}{CU} \text{ days.}$$

$dw \to$ readily available moisture.

$CU \to$ consumptive use.

Say $dw = 10$ cm & $CU = 1$ cm/day.

$$f = \frac{10}{1} = \underline{\underline{10 \text{ days}}}$$

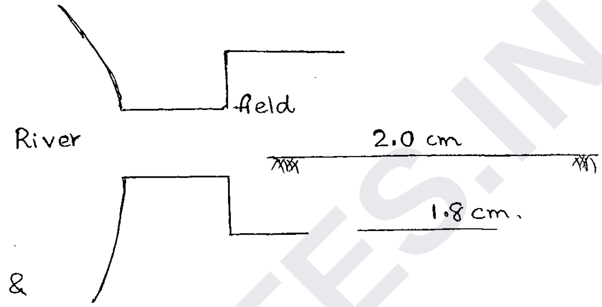
⊙ If $f = 10$ days, it means that once in 10 days irrigation is to be carried out (or) irrigation water is supplied once in 10 days.

→ Application Efficiency (η_a)

$$\eta_a = \frac{\text{Quantity of water stored in root zone}}{\text{Quantity of water delivered to the field}}$$

$$\eta_a = \frac{1.8}{2} \times 100$$

$$= 90\%$$



∴ Loss = 10 % ; due to Tail water run off loss & ~~Deep~~ Percolation loss.

→ Conveyance Efficiency (η_c)

$$\eta_c = \frac{\text{Qty of water delivered to the field}}{\text{Qty of water delivered to canal system.}}$$

$$= \frac{2}{3} \times 100 = 67\%$$

Loss = 33% ; Conveyance loss

8th Oct, aturday → Water use Efficiency (η_w)

$$\eta_w = \frac{\text{Qty of water beneficially used}}{\text{Qty of water delivered to field.}}$$

Quantity of water beneficially used = water used by crop
+
water used for leaching

→ Water Storage Efficiency (η_s)

$$\eta_s = \frac{\text{Qty of water stored in root zone}}{\text{Qty of water required to bring moisture content to field capacity}}$$

Leaching is the supply of additional water to wash away the salts in a saline prone area

→ Water Distribution Efficiency (η_d)

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

D → average depth of penetrations (water) in the soil.
d → average of absolute values of deviations from the mean.
Say the $\eta_d = 90\%$, means that 90% area has received uniform depth of moisture in the root zone. The remaining 10% may receive excess water or less water called as 'over irrigation' or 'under irrigation'

→ Consumptive Irrigation Requirement (CIR)

$$CIR = \text{Consumptive use} + \text{Effective rainfall}.$$

* Net irrigation requirement, NIR = CIR + water require for leaching.

* Field irrigation requirement, $FIR = \dfrac{NIR}{\eta_a}$.

* Gross irrigation requirement, $GIR = \dfrac{FIR}{\eta_c}$

A loamy soil has FC = 25%, PWP = 10%, γ_d of soil = 1.5 g/c if root zone depth = 0.75 m, what is storage capacity of soil. Irrigation water is applied when moisture content = 14%. If $\eta_a = 75\%$, determine the water depth required.

Water holding capacity, $y = S * d * (F_c - PWP)$.

$$= \frac{1.5}{1} * 0.75 * (0.25 - 0.1).$$

$$= \underline{\underline{0.168 \text{ m}}}$$

$\therefore$ Readily available moisture, $d_w = S * d * (F_c - OMC)$.

$$= 1.5 * 0.75 * (0.25 - 0.14).$$

$$= 0.124 \text{ m}.$$

$$\eta_a = \frac{\text{depth of water stored in root zone}}{\text{depth of water applied to field.}}$$

$$0.75 = \frac{0.124}{\text{depth applied.}}$$

$\Rightarrow$ Depth of water applied $= \frac{0.124}{0.75} = \underline{\underline{0.165}}$ m

2 $FC = 29\%$, $PWP = 11\%$, $d = 70$ cm $= 0.7$ m.

Consumptive use, $CU = 12$ mm.

Water holding capacity; $y = \frac{1.3}{1} * 0.7 (0.29 - 0.11)$.

$$= 0.1638 \text{ m}.$$

Readily available moisture, d_w $= 0.75 \times y$.

$$= 0.123 \text{ m}.$$

$\therefore$ Irrigation period (or) frequency of irrigation, $\Big\}$ $f = \frac{d_w}{C_u} = \frac{0.123}{12/1000}$

$$= \underline{\underline{10.25 \text{ days}}}$$

3. Area irrigated $= 50$ ha.; $t = 12$ hours/day

$\eta_c = 85\%$; $y = 20$ cm/m depth.

6 (b)

Average root zone depth, $d = 1$ m.

$\eta_a = 80\%$

Readily available moisture, $d_w = 50\%\ y$.

Consumptive use $= 5$ mm.

$f = ?$ & $Q = ?$ (L/min).

$$d_w = 0.5 \times 20 = 10 \text{ cm/m.} \quad (\text{given } d = 1 \text{ m}).$$

$$f = \frac{d_w}{C_u} = \frac{10}{0.5} = \underline{\underline{20 \text{ days}}}$$

$$Q = \frac{\text{volume of water to be delivered}}{\text{Time of pumping}}$$

$$= \left(\frac{A * d_w}{t}\right) \times \frac{1}{\eta_a \times \eta_c}$$

$$= \frac{50 * 10^4 * 0.1}{0.85 \times 0.8 \times (20 \times 12 \times 60)} = 5.106 \text{ m}^3/\text{min}$$

$$= \underline{\underline{5106}} \text{ L/min}$$

4 $A = 5000$ ha, $F_c = 25\%$, $PWP = 5\%$, $S = 1.4$, $d = 0.8$ m,
$C_U = 1.68$ cm/day, $d_w = 75\%\ y$, $f = ?$, $Q = ?$; $\eta_a = 20\%$

$$y = S * d * (F_c - PWP).$$

$$= 1.4 * 0.8 * (0.25 - 0.05).$$

$$= \underline{\underline{0.224}} \text{ m}$$

$$d_w = 0.75 \times y = 0.75 \times 224 = 0.168 \text{ m}$$

$$f = \frac{d_w}{C_u} = \frac{0.168}{1.68/100} = \underline{\underline{10 \text{ days}}}$$

$$Q = \frac{\text{volume of water delivered}}{t} = \left(\frac{A * d_w}{\eta_a}\right) \times \frac{1}{\text{time}}$$

$$Q = \frac{5000 \times 10^4 \times 0.168}{0.2 \times (10 * 24 * 3600)} = \underline{\underline{48.61}} \ m^3/s$$

Q. A discharge of 130 L/s was delivered from a canal and ~~hum~~ 100 L/s was delivered to the field. An area of 1.6 ha was irrigated in 8 hours. The run off loss in the field is 420 m³. Determine conveyance efficiency and water application efficiency. Depth of root zone is 1.7 m and available moisture holding capacity of soil is, 20 cm/m depth of root zone. Irrigation is started at a moisture extraction level of 50% of available moisture. Determine water storage efficiency.

Discharge into canal = 130 L/s.

Discharge delivered to field = 100 L/s.

$$\eta_c = \frac{100}{130} \times 100 = 76.92\%.$$

$$\eta_a = \frac{\text{vol. of water stored in root zone}}{\text{vol. of water delivered to field.}}$$

Volume of water stored = vol. applied − run off loss.

$$= 0.1 \times 8 \times 3600 - 420$$

$$= 2880 - 420 = 2460 \ m^3.$$

$$\eta_a = \frac{2460}{2880} \times 100 = \underline{\underline{85.4\%}}$$

Given,

$d = 1.7$ m, $y = 20$ cm/m depth.

Total water holding capacity in 1.7 m = $20 \times 1.7 = 34$ cm.

$$d_w = 0.5 \times y = \underline{\underline{17 \text{ cm}}}$$

$\therefore$ Volume of water required to bring moisture content to field capacity $=$ Area $\times d_w$

$$= 1.6 \times 10^4 \times 0.17 = \underline{2720}\ m^3$$

$\Rightarrow$ water storage efficiency, $\eta_s = \frac{2460}{2720} \times 100 = \underline{\underline{90.4\%}}$

Q. The depths of penetration along the lengths of an irrigation field at 130m apart are observed as follows:
2m, 1.9 m, 1.8 m, 1.6 m, 1.5 m. Compute distribution efficiency.

$$D = \frac{2+1.9+1.8+1.6+1.5}{5} = \underline{\underline{1.76\ m}}$$

Deviations from mean:

$2-1.76 = 0.24$ $\quad 1.8-1.76 = 0.04$ $\quad |1.5-1.76| = 0.26$

$1.9-1.76 = 0.14$ $\quad |1.6-1.76| = 0.16.$

$$d = \frac{0.24+0.14+0.04+0.16+0.26}{5} = 0.168\ m$$

$$\Rightarrow \eta_d = \left(1 - \frac{d}{D}\right) \times 100 = \left(1 - \frac{0.168}{1.76}\right) \times 100$$

$$= \underline{\underline{90.45\%}}$$

$\therefore$ 90.45% of command area receives equal depth of moisture in the root zone.

19th Oct,
SUNDAY

* Crop Period:

Time elapsed b/w sowing and harvesting.

* Base Period:

Time elapsed b/w 1st watering and last watering before harvesting.

Crop Period > Base period.

* Duty of water:

It establishes a relation b/w Q & area.
It is defined as the area of land in hectares which can be irrigated for growing any crop if $1\ m^3/s$ is continuously supplied throughout the base period.

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

If duty = 1000 ha/cumecs, it means that 1000 ha can be cultivated if $1\ m^3/s$ is supplied continuously for the entire base period B.

— Variation of Duty along the Canal System:

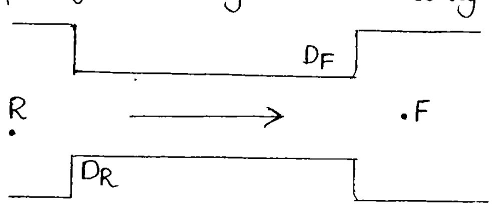
$$\boxed{D_F = \frac{D_R}{\eta_c}}$$

$D_F \rightarrow$ duty on the field.

$D_R \rightarrow$ duty at the head of the canal.

$\eta_c \rightarrow$ conveyance effeciency.

Unless specified, duty means duty on the field.



* Delta (Δ) :

The depth of flow applied during entire base perioa

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

$\Delta \rightarrow$ depth of flow (in m)

$B \rightarrow$ base period. (in days)

$D \rightarrow$ duty (in. ha/cumecs).

– Volumetric Units :

1. m^3

2. ~~ha-cumec~~ ~~or~~ cumec day

It is the total volume of water supplied at the rate of $1\ m^3/s$ in a day.

$$1 \text{ cumec day} = \frac{1\,m^3}{s} \times 24 \times 3600$$

$$\boxed{1 \text{ cumec day} = 8.64 \text{ ha-m}}$$

3. TMC

1 TMC = Thousand Million Cusecs.

$$1 \text{ cusec} = 1\ ft^3/s$$

$$1 \text{ cumec} = 1\ m^3/s$$

To represent inflows and outflows from a reservoir, generally TMC is used.

→ Paleo irrigation :

Watering done for land preparation, ie, first watering is called as Paleo irrigation.

→ Kor watering :

Watering done or irrigation carried out when the plants are at a younger stage, ie, the height of the plant is few cm. The demang for water during this period is more.

- Kor Period

The period of Kor watering is Kor period, which is around 2-3 weeks.

- Kor Depth

The total depth applied during Kor period is called as Kor depth.

→ Gross Command Area (GCA)

The total area under a canal system

* Culturable Command Area (CCA)

CCA = GCA − uncultivated area.

Uncultivated area includes :-

a) Habitants
b) Paved areas.

* Culturable Cultivated Area.

Culturable Cultivated Area = CCA − Culturable Uncultivated Area.
= % CCA

* Culturable Uncultivated Area.

(i) Water logging

(ii) Fallow land — land which is left idle for a few crop seasons to allow replenishment of nutrients.

(iii) Lack of Resources.

* Intensity of Irrigation.

Intensity of irrigation = % of CCA proposed to be irrigated annually

* Capacity Factor (CF)

$$CF = \frac{\text{mean discharge}}{\text{max. discharge}}$$

* Time Factor (TF)

$$TF = \frac{\text{No. of days canal has run}}{\text{No. of days canal is supposed to run}}$$

$$\text{Also } TF = \frac{\text{Actual discharge}}{\text{Design discharge}}$$

$$\therefore \boxed{\text{Design discharge of canal} = \frac{\text{Mean discharge}}{CF * TF}}$$

→ Kharif Season (monsoon season).

April to September (6 months).

Crops during this period are called Kharif crops.

Eg: Paddy, Maize.

→ Rabi Crops (winter Crops).

October to March (6 months).

Crops grown during this period are called Rabi crops.

Eg: Wheat.

* Perennial Crops. – crop grown throughout the year.

Eg: Sugar cane.

* 8 month crops

May to Dec / January.

Eg: Cotton.

* Hot weather crops

February – June.

* Dry Crops. – no irrigation required.

* wet crops – irrigation needed.

6. $\Delta = D_F = 800$ ha/cumecs.

$\eta_c = 80\%$ (given loss is 20%)

$D_F = \frac{D_R}{\eta_c} \Rightarrow D_R = 800 \times 0.8 = \underline{\underline{640}}$ ha/cumecs

7. For rice :

$B = 140$ days

$\Delta_R = 1.34$ m.

$D_R = 8.64 \frac{B}{\Delta}$.

$= 8.64 \times \frac{140}{1.34}$

$= 902.686$ ha/cumecs

For wheat :

$B = 120$ days

$\Delta_w = 0.52$ m

$D_w = 8.64 \times \frac{120}{0.52}$

$= 1993.846$ ha/cumecs.

Area irrigated under rice , $A_R = 1200$ ha.

$$D_R = \frac{A_R}{Q}.$$

$$\therefore \; Q = \frac{1200}{902.686} = \underline{\underline{1.33}}\ m^3/s$$

Area irrigated under wheat $= Q \times D_w$.

$= 1.33 \times 1993.846$

$= \underline{\underline{2650.55}}$ ha

$\frac{ha}{cumec}$

8 Volume of tank $= 10\ Mm^3 = 10 \times 10^6\ m^3$.

Loss $= 10\%$

Available storage $= 90\%\ 10\ Mm^3 = 9\ Mm^3$.

$B_w = 120$ days

$\Delta = 0.4$ m

Volume $=$ Area $*$ Δ.

$9 \times 10^6\ m^3 =$ Area $*$ 0.4.

$\therefore \; A = 2250 \times 10^4\ m^2 = \underline{\underline{2250}}$ ha

9. $B = 15$ days

$\Delta = 60$ cm $= 0.6$ m.

Useful rain $= 8$ cm.

$D_F = ?$ & $D_R = ?$

$$D_F = 8.64 \times \frac{B}{\Delta} = \frac{8.64 \times 15}{(0.6 - 0.08)} = 246 \text{ ha/cumecs.}$$

$$D_R = D_F \times \eta_c$$

$= 249 \times 0.75 = \underline{\underline{186.92}}$ ha/cumecs

10.

Crop	Base period	Duty (D_F)	Intensity of irrigation	Area
Wheat	120	1800	20	8000
Sugar Cane	360	1700	20	8000
Cotton	180	1400	10	4000
Rice	120	800	15	6000
Vegetables.	120	700	15	6000

CCA = 40,000 ha.

$\Delta = 8.64 \frac{B}{D} \Rightarrow \Delta_{wheat} = 0.576 \quad \Delta_{sugar} = 1.83$

$\Delta_{cotton} = 1.11 \quad \Delta_{rice} = 1.296$

$\Delta_{vegetables} = 1.48$

$\Rightarrow$ Volume $= 8000 \times 0.576 + 8000 \times 1.83 + 4000 \times 1.11 + 6000 \times 1.296 + 6000 \times 1.48$

$= 40344$ ha-m.

$\therefore$ Reservoir capacity $= \frac{40354}{0.8 \times 0.9} = \underline{56047}.22$ ha-m

11. $\eta_p = \frac{o/p}{i/p}$

Output of pump, $o/p = \frac{\gamma Q H}{75}$ hp.

$D = \frac{A}{Q}$.

$\therefore Q = \frac{2.5}{860} = \frac{2.907}{1000}$ cumecs

$o/p = \frac{\overset{1000}{\cancel{9810}} \times 2.907 \times 10^{-3} \times (107.5 - 100)}{75} = \overset{0.29}{\cancel{2.85}}$ hp

$i/p = \frac{o/p}{\eta_p} = \frac{\overset{0.29}{\cancel{2.85}}}{0.5} = \underline{\underline{\overset{0.57}{\cancel{5.7}}}}$ hp

$1\ kw = 1.34\ hp.$

$1\ hp = 0.746\ kW.$ (British hp).

$= 0.736\ kw$ (Metric hp).

$$o/p = \gamma Q H = 9.81 \times \frac{2.5}{860} \times 7.5$$

$$= 0.214\ kw$$

$$i/p = \frac{o/p}{\eta_p} = \frac{0.214}{0.5} = 0.427\ kw = \underline{\underline{0.5736}}\ hp$$

12.

Crop	B(days)	A(in ha)	Duty	$Q = \frac{A}{D}$.
Rice	120	4000	1500	2.667
wheat	120	3500	2000	1.75
Sugar Cane	310	3000.	1200	2.5

Discharge required during Kharif season $= Q_{Rice} + Q_{S.C}$

$= \underline{\underline{5.167}}$ cumecs

Discharge required during Rabi season $= 1.75 + 2.5$

$= \underline{\underline{4.25}}$ cumecs

$\therefore$ Discharge = greater of above two values.

$= \underline{\underline{5.167}}$ cumecs

$$\text{Design discharge} = \frac{5.17}{\eta_c \times TF} = \frac{5.17}{0.8 \times 0.75} = \underline{\underline{8.611}}\ \text{cumecs}$$

Q The main canal taking of from a storage reservoir irrigates the following crops. Necessary data is given as follows.

Crop	Crop Period	A	D.
Sugar Cane	280	360	640
Overlap in SC in hot weather	100	70	640
Jowar (rabi)	120	4860	1700
Bajri (monsoon)	120	6480	2860
Vegetables (hot weather)	120	360	700.

7.125

Find the discharge required at the head of the main canal taking $\frac{6}{10}$ as time factor and 0.8 as capacity factor for the main canal.

→ Crop Ratio

$$\text{Crop ratio} = \frac{\text{Area under Rabi}}{\text{Area under Kharif}}$$

Q Wheat crop requires 55cm of water during 120 days of base period. The total rainfall during this period is 100 mm.

Assume irrigation efficiency as 60%. The area is hectares of land which can be irrigated with a canal flow of 0.01 m^3/s is —— ?

$\Delta = 55$ cm , Total rainfall $= 10$ cm.

$B = 120d$, $\eta = 60\%$

Net $\Delta = 55 - 10 = 45 \text{ cm} = 0.45 \text{ m}$.

Duty on field, $D_F = 8.64 \frac{B}{\Delta} = 8.64 \times \frac{120}{0.45} = \underline{\underline{2304}}$ ha/cumec

$$D_F = \frac{D_R}{\eta_c}$$

Duty on canal, $D_R = D_F \times \eta_c = 2304 \times 0.6$

$= 1382.4$ ha/cumecs.

1 cumecs irrigates $= 1382.4$ ha.

0.01 cumec irrigates $= 1382.4 \times 0.01$

$= \underline{\underline{13.824}}$ ha

Q 10 m^3/s of water is delivered to a 32 ha field for 4 hours Soil after irrigation showed that 0.3 m of water had been stored in root zone. Application efficiency is a) 96%

b) 66.67% c) 48% d) 24%.

$$\eta_a = \frac{\text{vol. stored in root zone}}{\text{vol. of water applied.}}$$

$$= \frac{32 \times 10^4 \times 0.3}{10 \times 4 \times 3600} = 0.666 \times 10 = \underline{\underline{66.67\%}}$$

Q. In an irrigation project, in a certain year, 70% and 46% of CCA in Kharif and Rabi remained without water and the rest of the area got irrigation water. The intensity of irrigation in that period for the project wa —— ?

13

Area which received water during Kharif season

$$= 100 - 70 = 30\%$$

Area which received water during Rabi season

$$= 100 - 46 = 54\%$$

$\therefore$ Intensity for that year $= 54 + 30 = 84\%$

Q. The CCA for a distributed channel is 20,000 ha. Wheat is grown in the entire area and intensity of irrigation is 50%. The Korr period is 30 days, korr depth = 120 mm. The outlet discharge for the distributory is ____

Area under wheat = 10,000 ha.

$$D = \frac{8.64 \times 30}{0.12} = 2160 \text{ ha/cumecs.}$$

$$\text{Outlet discharge} = \frac{A}{D} = \frac{10000}{2160} = \underline{\underline{4.63 \text{ cumecs}}}$$

9th Oct,
SUNDAY

02. METHODS OF IRRIGATION

1. Surface Irrigation Methods.
   (i) Flooding Method.
      a) Uncontrolled Flooding. Eg: Inundation irrigation
      b) Controlled Flooding
         - Field Channel method.
         - Border irrigation method.
         - Check or leave method.
         - Basin method.
   (ii) Furrow method.
   (iii) Contour Farming.

2. Subsurface Irrigation Method.
   (i) Natural Subsurface
   (ii) Artificial Subsurface.
   (iii) Drip Irrigation method.

3. Sprinkler Irrigation Method.

* Border Strip Method.

$$\boxed{t = \frac{y}{I} \log_e\left(\frac{q}{q - IA}\right)}$$

where $y \rightarrow$ average depth of sheet of flowing water.
$t \rightarrow$ time required for water to cover an area
$q \rightarrow$ discharge of a stream
$A \rightarrow$ area of the strip covered at any time
$I \rightarrow$ rate of infiltration.

$$\boxed{A_{max} = \frac{q}{I}}$$

A_{max} → max area that can be irrigated.

- Slope should be along the length of the border.

→ Infiltration Oppurtunity Time (IOT) = Advance time − Recession time.

Using IOT, deep percolation loss is calculated from infiltration profiles. Knowing the deep percolation loss and tail water run off loss, application efficiency (η_a) is calculate

Advance time — time at which the water reaches a particular section.

Recession time — time at which the water siezes from that section.

* Furrow Method.

Furrows are small channels with mild slope which carry water with low velocities. Due to lateral seepage of water, the crops are grown. The water is directly supplied to the root zone

Depth of flow is 10 cm

Width of furrow is 25 cm with mild slope.

Eg: Maize crop.

* Sprinkler Method.

- Application efficiency, $\eta_a = 80\%$
- used when water is scarce.
- Tail water run off loss and DP loss are less
- No erosion.
- No channels, no bunds, no land preparation.

- high initial investment and power requirement.
- distribution efficiency gets affected by wind action.

* Drip Irrigation

- $\eta_a = 90\%$ (highest)

P-11

Q.1 $A = 100\,m \times 10\,m$.

$q = 0.03\ m^3/s$

$y = 7.5\ cm = 0.075\ m$.

$I = 5\ cm/hr = \dfrac{5}{100 \times 3600}\ m/s$.

$$t = \frac{y}{I} \log_e \left(\frac{q}{q - IA} \right).$$

$$= \frac{\frac{7.5}{100}}{\frac{5}{100 \times 3600}} \log_e \left(\frac{0.03}{0.03 - \frac{5 \times 100 \times 10}{100 \times 3600}} \right)$$

$$= 3357\ s = \underline{\underline{55.95\ min}}$$

$$A_{max} = \frac{q}{I} = \frac{0.03}{\frac{5}{100 \times 3600}} = \underline{\underline{2160\ m^2}}$$

9th Oct, SUNDAY

15

06. DESIGN OF UNLINED ALLUVIAL CANALS

Lined	Unlined.
(i) Shape is regular.	(i) Shape varies
(ii) More velocities.	(ii) Less velocities
(iii) To avoid silting & scouring	(iii) Silting and scouring occur
(iv) Constant cross section. (Rigid channel).	(iv) C/S varies, ie, mobile chann
(v) $V = \frac{1}{n} R^{2/3} S^{1/2}$ $V = C\sqrt{RS}$	(v) Kutter's equation. Kennedy's theory. Lacey's theory.

→ Kennedy's Theory

- stable channel (non silting & non scouring)
- Upper Bari Doab Canal System (Punjab, now in PAK).
- $\boxed{V_0 = 0.55 D^{0.64}}$ (For upper Bari).

where $V_0 \rightarrow$ critical velocity (non silting, non scouring velocity)

- For reaches other than Upper Bari Doab.

$$\boxed{V_K = 0.55\, m\, D^{0.64}}$$

where $m = \frac{V_K}{V_0}$; critical velocity ratio.

- m is a function of type of silt.

$V > V_0$; Scouring

$V < V_0$; Siltation

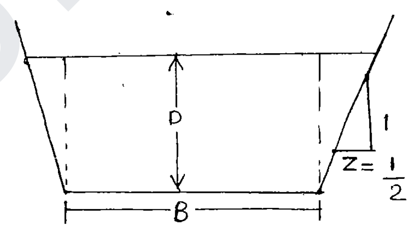
$V \approx V_0$; Stable.

* Drawbacks.

- No procedure to find 'm' value.
- Silt grade, silt concentration not considered.
- Drawbacks in Kutter's equation are included.
- Trial and error procedure.
- No slope equation was available.

Therefore basic theory of Kennedy's theory is to design a channel which is free from silting & scouring.

Case 1: Data given are $Q, m, n, \frac{B}{D}$



$$A = BD + 2\left(\tfrac{1}{2} \times ZD \times D\right).$$
$$= BD + ZD^2$$
$$\Rightarrow \boxed{A = BD + \frac{D^2}{2}}$$

$$P = B + 2D\sqrt{1+z^2} \qquad \left(z = \tfrac{1}{2}\right)$$
$$\boxed{P = B + D\sqrt{5}}$$

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

$$A = D^2\left(\frac{B}{D} + 0.5\right). \Rightarrow A = f(D^2).$$
$$V = 0.55\, m\, D^{0.64} \Rightarrow V = f(D).$$
$$Q = AV \Rightarrow Q = f(D).$$

$\therefore$ D & hence B can be obtained.

Case 2: Q, m, n, S are given

1. Assume trial 'D'.
2. $V = 0.55\, m\, D^{0.64} \rightarrow (1)$
3 $A = \frac{Q}{V}$

4. But $A = BD + \frac{D^2}{2}$

From this, determine B.

5. Determine V from Kutter's equation. $\rightarrow$ ②

If ① = ②, then trial D is correct.

NOTE :

- Garret's Diagrams are used to design a channel by Kennedy's theory using Kutter's equation.
- Side slope should be $\frac{1}{2}$ H : 1 V

→ Lacey's Theory

* Regime Channel :

It is defined as a channel which has undergo or silting and scouring and attained stability.

Lacey gave two Regime conditions :

(i) Initial Regime → Depth = constant & Slope = constant

(ii) Final Regime → Bed width = constant, Depth = constant
Slope = constant

(iii) A channel which has passed initial and final regime conditions is called a Regime Channel.

* True Regime Conditions : (theoretical conditions)

(i) Channel flows continuously in unlimited incoherent alluvium

(ii) Silt grade = constant. (ϕ_{silt} = const)

(iii) Silt concentration = constant. (min load of silt from active bed = const.)

(iv) Q = constant.

But all those conditions are practically impossible; ie it is hypothetical.

Based on these conditions, he assumed that the final regime has <u>semi-elliptical</u> cross section.

* Lacey's Regime Equations.

(i) Silt factor, $f = 1.76\sqrt{d}$;

$d \rightarrow$ min silt particle size (in mm).

(ii) Velocity, $V = \left(\dfrac{Qf^2}{140}\right)^{1/6}$

$$V = \sqrt{\left(\frac{2}{5}\right) fR}$$

(iii) Wetted Perimeter, $P = 4.75\sqrt{Q}$

- used in length of waterway.

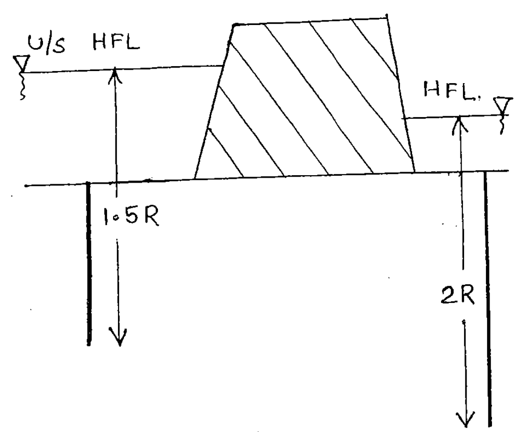
(iv) Scour depth, $R = 1.35\left(\dfrac{q^2}{f}\right)^{1/3}$

where $q \rightarrow$ discharge per m width. $\left(\dfrac{Q}{B}\right)$

(v) Slope $S = \dfrac{f^{3/2}}{4980\sqrt{R}}$

$$R = 0.47\left(\frac{Q}{f}\right)^{1/3}.$$

(vi) $S = \dfrac{f^{5/3}}{3340\ Q^{1/6}}.$



Design of a channel by Lacey's theory can be carried out by using Lacey's diagrams.

* Drawbacks:

- All his equations consist of silt factor
- True regime conditions are only theoretical
- Silt grade & silt concentration. are not defined properly.
- Emperical equations.

⊙ Lacey's theory is valid for side slopes $\frac{1}{2}H : 1V$

$$A = BD + \frac{D^2}{2}$$

$$P = B + D\sqrt{5}.$$

P-29.

1. $Q = 5\ m^3/s$; $n = 0.0225$.

$m = 1$; $\frac{B}{D} = 3.24$.

Assume side slopes as $\frac{1}{2}H : 1V$.

$$A = BD + \frac{D^2}{2}$$

$$= D^2\left(\frac{B}{D} + 0.5\right) = D^2(3.24 + 0.5)$$

$$A = 3.74\ D^2$$

$$V = 0.55\ m\ D^{0.64}$$

$$= 0.55 \times 1 \times D^{0.64}$$

$$Q = AV$$

$$5 = 3.74\ D^2 \times 0.55\ D^{0.64}$$

$$\therefore D = 1.64\ m.$$

$$\frac{B}{D} = 3.24\ m \Rightarrow \underline{\underline{B = 4.53\ m}}$$

02. $B = ?$; $Q = 5\ m^3/s$; $\frac{1}{2}H : 1V$

$$m = CVR = 0.8$$

$$D = 0.71\ m.$$

$$S = \frac{0.2}{1\ km} = \frac{0.2}{1000} = \frac{1}{5000}$$

$$V = 0.55\ m\ D^{0.64}.$$

$$= 0.55 \times 0.8 \times (0.71)^{0.64}.$$

$$= \underline{\underline{0.35}}\ m/s$$

$$A = \frac{Q}{V} = \frac{5}{0.35}$$

$$= \underline{\underline{14.28\ m^2}}$$

But $A = BD + \frac{D^2}{2}$

$$14.28 = B \times 0.71 + \frac{0.71^2}{2}$$

$$\Rightarrow B = \underline{\underline{19.76}}\,m$$

03. $S = \frac{1}{4000}$; $f = 0.9$;

$$S = \frac{f^{5/3}}{3340\, Q^{1/6}}.$$

$$\frac{1}{4000} = \frac{0.9^{5/3}}{3340\, Q^{1/6}} \Rightarrow Q = 1.029\ m^3/s$$

$$P = 4.75\sqrt{Q} = 4.82\ m.$$

$$V = \left(\frac{Qf^2}{140}\right)^{1/6} = \left(\frac{1.029 \times 0.9^2}{140}\right)^{1/6} = 0.426\ m/s$$

$$A = \frac{Q}{V} = \frac{1.029}{0.426} = 2.416\ m^2.$$

$$\Rightarrow A = BD + \frac{D^2}{2} = 2.416.\ m^2.$$

$$P = B + D\sqrt{5} = 4.82\ m$$

$$\therefore (4.82 - D\sqrt{5})\, D + \frac{D^2}{2} = 2.416.$$

$$4.82D - D^2\sqrt{5} + \frac{D^2}{2} = 2.416.$$

0.656
2.12.

$$D = \underline{\underline{0.656}}\,m \ \&\ 2.12\ m$$

$$\Rightarrow B = \underline{\underline{3.35}}\ m \ \&\ 0.07\ m$$

05. DIVERSION HEADWORKS

→ Types of Headworks :

(i) Storage Headwork – to serve various purposes.

Eg: Dam.

(ii) Diversion Headwork – irrigation

Eg: Weirs and Barriages. – allows rise in water level.

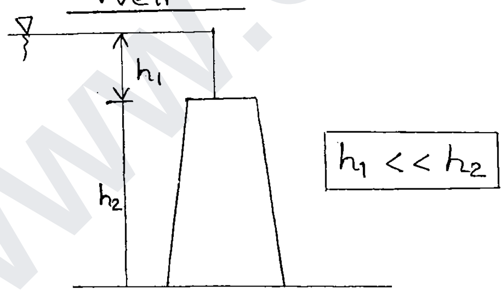
Water is allowed to divert into off taking canal system.

→ Stages of a River :

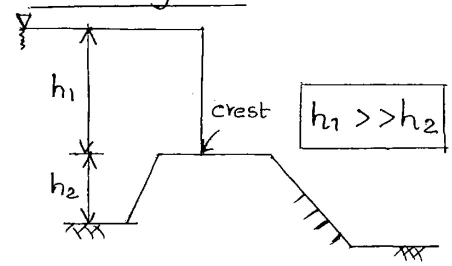
(i) Rocky Stage
(ii) Boulder stage
(iii) Alluvial stage
(iv) Delta stage.

DHW schemes are constructed at Boulder stage & alluvial stage.

Weir



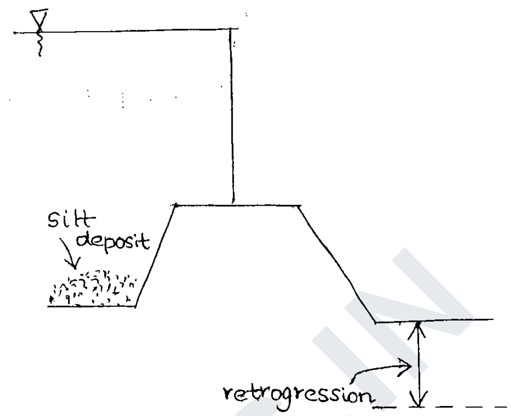
Barrages.



Weir	Barrages
1. Rise is due to masonry structure	1. Rise is due to gate
2. Afflux – increase in HFL due to weir. Afflux is more	2. Afflux is less.
Small roadway	3. Large roadway.
Cost is less	4. More cost.

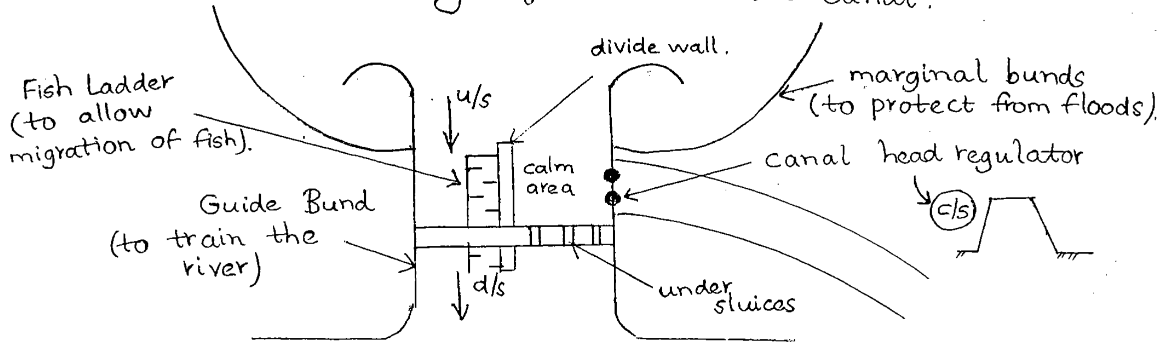
* Retrogression :

Decrease in bed level in d/s side (to attain silt carrying capacity) is called Retrogression.



→ Components of DHW

1. Weir or Barrage
2. Canal system
3. Under sluices or Scouring sluices.
4. Divide wall.
5. Fish ladder.
6. River training works
   a) Guide Bund
   b). Marginal Bund.
   c) Spur or Gyroyne.
7. Silt excluding devices
   a) Silt excluder
   b) Silt extractor / ejector.
8. Canal Head regulator.
   (i) To regulate flow of water
   (ii) Controls entry of flood water
   (iii) Prevents entry of silt into the canal.



* Functions of Under Sluices :
  (i) To allow silt scouring and convey to d/s side.
  (ii) To release dry weather flows.

* Discharge capacity of Under Sluices :
  It is the greater of —
  (i) 2 times the discharging capacity of channel.
  (ii) Max winter discharge
  (iii) 10–15% of max flood discharge

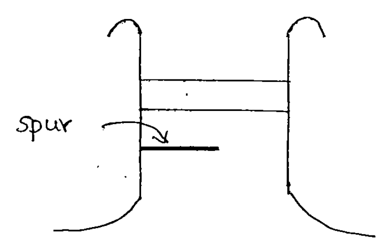
* Regulation Methods :
  (i) Still Pond regulation system.
  (ii) Semi open flow regulation system.

* Functions of Divide Wall.
  (i) To create a calm area near canal system so that velocities are reduced to settle silts.
  (ii) To separate under sluices

* Silt excluder
  - Diaphragm slab
  - provided before the canal head regulator.
  - to exclude silt entry into the canal.

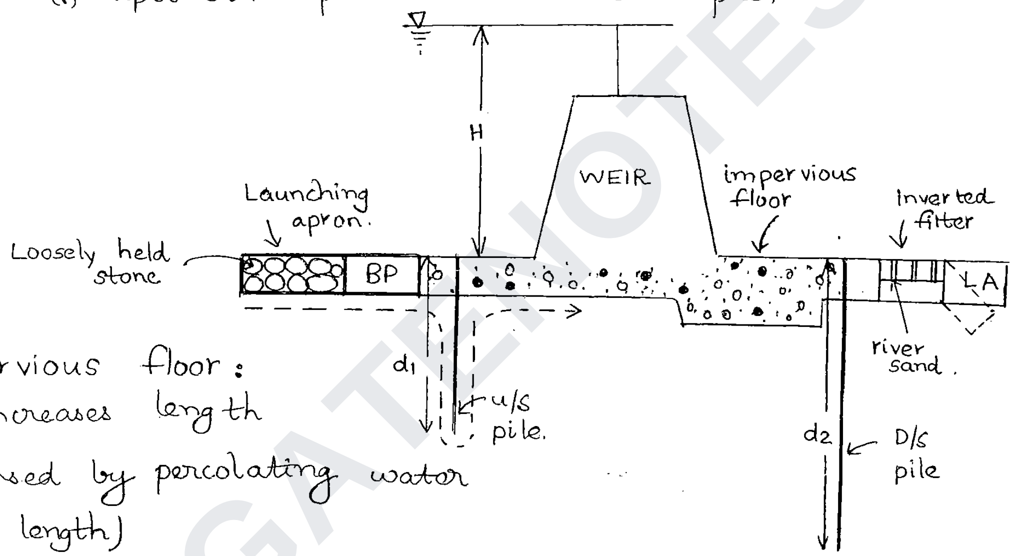
* Silt ejector / extractor.
  - provided on the canal, near to the head of canal.
  - to collect silt and disperse off.

* Spur
  - earthen embankment
  - to deflect water away from that area



→ Weir

* Components of a Weir
  (i) Impervious floor on u/s & d/s
  (ii) Block protection on u/s
  (iii) Launching Aprons on u/s & d/s.
  (iv) Inverted Filter on d/s
  (v) Upstream pile & Downstream pile.



Impervious floor :
– increases length traversed by percolating water (creep length)

Upstream Pile :
– To control uplift pressure.

Downstream Pile :
– To prevent piping and decrease $\frac{H}{L}$ value.

Inverted filter : ($L = 1 - 1.5\ d_2$)
– To relieve uplift pressure.

Launching Apron ($L = 1 - 1.5\ d$)
– To prevent scour.

Length of Block Protection $= d_1$
– Block Protection prevent scouring of impervious floor.

- Provide nominal thickness of impervious floor on u/s side because the standing water resist the uplift pressure.

- Downstream imp. floor thickness is to be designed because the self weight of the impervious floor has to resist the uplift pressure. The impervious floor is to be designed for max. percolating (or) seepage head. ie, water on the u/s side and no water on the d/s side.

- More creep length (more length of impervious floor) is to be provided to u/s side and less length on d/s side. Also less thickness on u/s side and more floor thickness on d/s side.

→ CREEP THEORIES

* Failures of Weirs on Permeable Foundations:

(i) Surface Failures.

a) due to suction, by formation of hydraulic jum

b) Scouring.

(ii) Subsurface Failure

a) Piping or undermining

b) Uplift pressure.

* Remedial measures to Prevent Piping:

a) Increase the creep length by providing impervious floor, u/s pile and d/s pile.

* Measures to control Uplift Pressure:

- Provide u/s pile & suitable thickness of impervious floor.
- u/s nominal thickness & d/s design thickness.

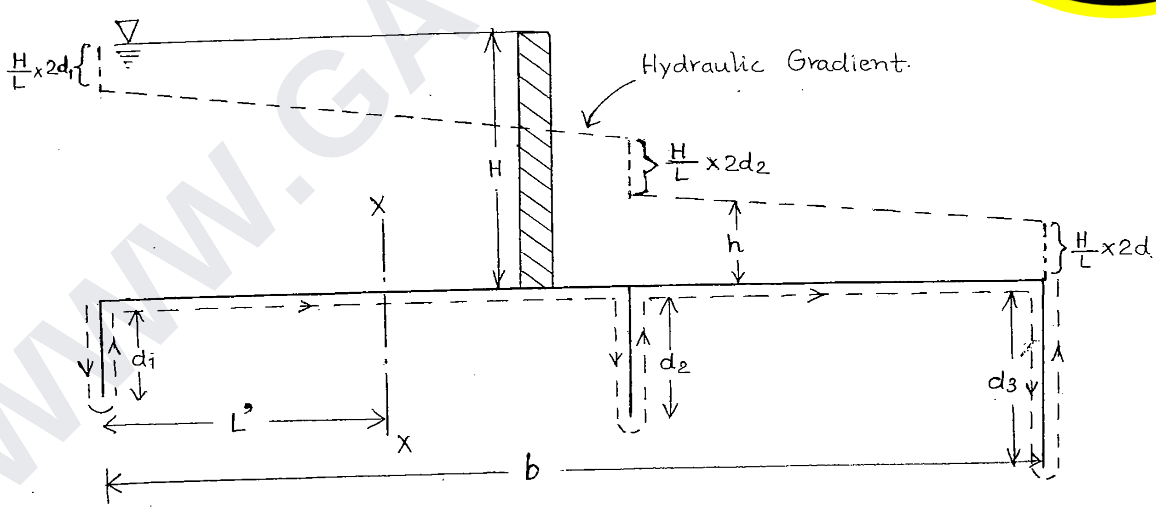
- Upstream pile : to prevent uplift pressure.
- Downstream pile : to prevent piping.

* Surface Failures :

- Scouring :- provide launching aprons on upstream downstream sides.
- Rupture of floor due to hydraulic jump :- provide suitable thickness of floor.

⊙ Bligh's Creep Theory.
⊙ Lanes Weighted Creep Theory.
⊙ Khosla's Theory.

→ Bligh's Creep Theory



Creep Length : Path traversed by percolating water.

Assumptions :

(i) Creep takes place along the base profile of the structure.
(ii) Hydraulic gradient is constant.

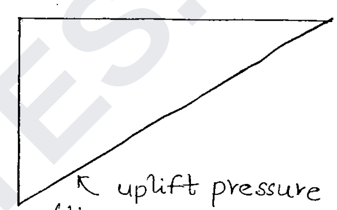
$$\frac{H}{L} = \text{constant}$$

ie $H \propto L$

where L = creep length = $b + 2d_1 + 2d_2 + 2d_3$

Consider a section X–X :

Creep length upto XX, $L'' = 2d_1 + L'$



$L \longrightarrow H$

$L' \longrightarrow \frac{H}{L} \times L'$

$\therefore$ Residual head on Balance uplift head $= H - \frac{H}{L} \times L'$

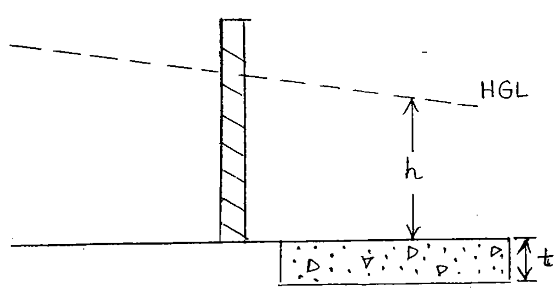
⊙ Safety against Piping :

$$\boxed{\frac{H}{L} \leq \frac{1}{C}}$$

$C \rightarrow$ Bligh's Creep Coeffecient (Function of type of soil).

$$\boxed{L = CH}$$

⊙ Safety against Uplift Pressure :



$h \rightarrow$ residual uplift pressure head.

$$A(h+t)\gamma_w = \text{Weight of floor.}$$

$$A\gamma_w(h+t) = A\gamma_c t.$$

$$\gamma_w(h+t) = G_c\gamma_w t$$

$$t_{min} = \frac{h}{G_c - 1} \quad ; \; G_c \rightarrow \text{sp. gravity of materi.}$$

$$\boxed{\text{Design thickness, } t = \frac{4}{3}\left(\frac{h}{G-1}\right)}$$

where h measured from top of floor.

- If h is measured from bottom of floor,

$$A * h * \gamma_w = A * \gamma_c * t.$$

$$t = \frac{h}{G_c}$$

$$t_{min} = \frac{h}{G_c}$$; h measured from bottom of floor.

* Drawbacks of Bligh's Theory:

(i) No distinction b/w horizontal and vertical creep ($V_c > H_c$)

(ii) No distinction b/w effectiveness of outer and inner faces (more loss on outer side)

(iii) Linear distribution of pressure.

(iv) No significance for D/s pile or exit gradient condition is not considered.

→ Lane's Weighted Creep Theory :

$$\boxed{L_w = \frac{1}{3}H + V}$$

H → sum of all horizontal contacts and inclined contacts $< 45^\circ$ with horizontal.

V → sum of all vertical contacts and inclined contacts $> 45^\circ$ with horizontal.

⊙ To prevent piping :

$$\frac{H}{L_w} \leq \frac{1}{C_1}$$

C_1 → Lane's creep coeffecient = f (type of soil)

→ Khosla's Theory

Theory of Seepage :-

$$q = kH\frac{N_f}{N_d}.$$

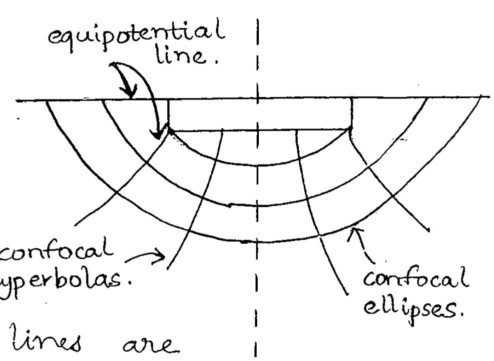
N_f → No: of potential drops.

N_d → No of flow channels.

K → coeffecient of permeability.

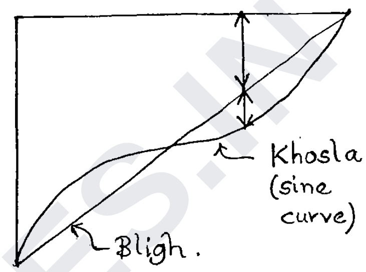
$$\frac{\partial^2\phi}{\partial x^2} + \frac{\partial^2\phi}{\partial y^2} = 0$$

Flow potential, $\phi = KH$



⊙ Streamlines ~~ar~~ and equipotential lines are confocal ellipses and confocal hyperbolas

- outer faces are effective than the inner ones.
- D/s pile is essential to prevent piping.
- pressure variation is not linear, but follows a sine curve



- exit gradient is significant

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

where $H \rightarrow$ percolating head.

$d \rightarrow$ depth of d/s pile

$\lambda \rightarrow \dfrac{1 + \sqrt{1+\alpha^2}}{2}$

$\alpha \rightarrow \dfrac{b}{d}$; $b \rightarrow$ horizontal floor length.

- Critical hydraulic gradient, $i_c = \dfrac{G_c - 1}{1+e}$

where $G_c \rightarrow$ sp. gr of material.

$e \rightarrow$ void ratio.

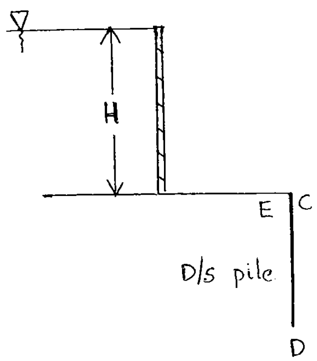
$$i_c = (1-n)(G-1) \ ; \quad n \rightarrow \text{porosity.}$$

⊙ For safety against piping,

$$i_c > G_E$$

$$\boxed{\text{FOS against piping} = \frac{i_c}{G_E}}$$

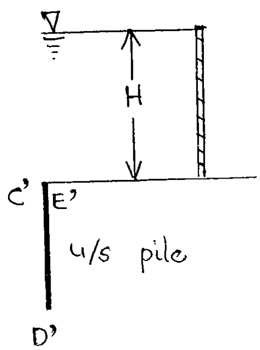
* To determine residual uplift pressure head.



$$h_E = \frac{H}{\pi} \cos^{-1}\left(\frac{\lambda-2}{\lambda}\right)\frac{\pi}{180}$$

$$h_D = \frac{H}{\pi} \cos^{-1}\left(\frac{\lambda-1}{\lambda}\right)\frac{\pi}{180}$$

$$h_C = 0.$$



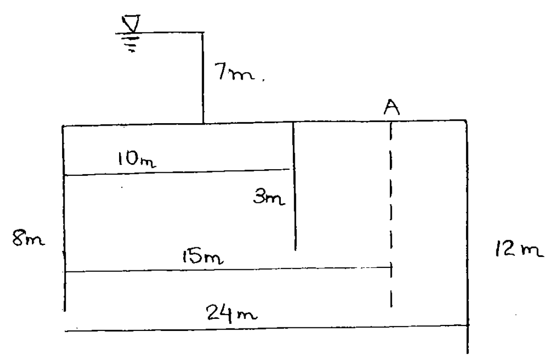
$$h'_C = H$$

$$h'_D = H - h_D$$

$$h'_E = H - h_E$$

P-26.

01. a) $G = 2.4$



Creep length $= 24 + 2(8+3+12)$

$= 70$ m.

$H = 7$m.

Hydraulic gradient $= \frac{H}{L} = \frac{7}{70} = \underline{\underline{0.1}}$

b). $L = CH.$

$70 = C \times 7.$

$\therefore$ Bligh's creep coeffecient, $c = \underline{\underline{10}}$

c) Creep length upto A $= 15 + 2(8+3) = \underline{\underline{37\text{ m}}}$

$L \longrightarrow H$ is lost.

$37 \longrightarrow 37 \times \frac{H}{L}$

$\therefore$ Head lost $= \frac{H}{L} \times 37 = \frac{7}{70} \times 37 = \underline{\underline{3.7\,m}}$

$\therefore$ Residual pressure head $(h_A) = 7 - 3.7$

$= \underline{\underline{3.3\,m}}$

d). $h_A = \frac{P_A}{\gamma}$

$\therefore P_A = 3.3 \times \gamma_w = 3.3 \times 9.81$

$= \underline{\underline{32.37}}$ kPa

e). Floor thickness required at A $= \frac{4}{3} \frac{h_a}{(G-1)}$.

$= \frac{4}{3} \times \frac{3.3}{(2.4-1)} = \underline{\underline{3.14\,m}}$

Q. Determine the required values for above problem using Lane's theory.

a) Lanes Creep Length $= \frac{1}{3} H + V$.

$= \frac{24}{3} + 2(8+3+12)$.

$= \underline{\underline{54\,m}}$

$H = 7\,m$

Hydraulic gradient $= \underline{\underline{\frac{7}{54}}}$

b) $L = CH$.

$\therefore C = \frac{L}{H} = \frac{54}{7} = \underline{\underline{7.7}}$

c) Lanes creep length upto A $= \frac{15}{3} + 2(8+3)$.

$= \underline{\underline{27\ m}}$

Head lost $= \frac{7}{54} \times 27 = \underline{\underline{3.5\ m}}$

Residual head at A $= 7 - 3.5 = \underline{3.5m}$

d) Pressure head at A $= 3.5 \times 9.81 = \underline{\underline{34.324}}$ kPa

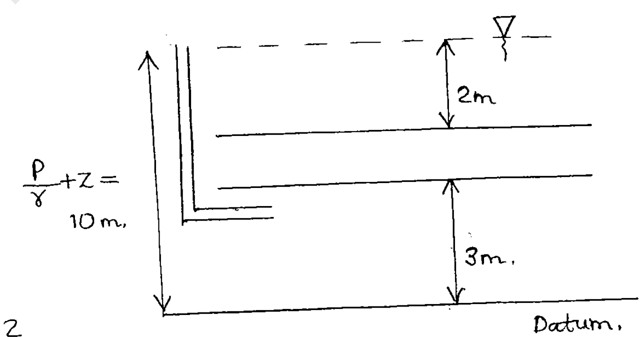
e) Floor thickness, $t_A = \frac{4}{3} \frac{h_A}{G-1} = \frac{4}{3} \times \frac{3.5}{(2.4-1)}$

$= \underline{\underline{3.33\ m}}$

03. $\frac{P}{\gamma} + Z = 10$

$\frac{P}{\gamma} + 3 = 10$

$\therefore \frac{P}{\gamma} = 7\ m$



Balance uplift head $= 7 - 2$
$= \underline{\underline{5m}}$

$\therefore t = \frac{h}{G} = \frac{5}{2.5} = \underline{\underline{2m}}$

OR

Downward pr. = Upward pressure.

$2\gamma_w \times A + tA \times \gamma_c = 7\gamma_w \times A$.

$t = \frac{5\gamma_w}{\gamma_c} = \frac{5}{G} = \underline{\underline{2m}}$

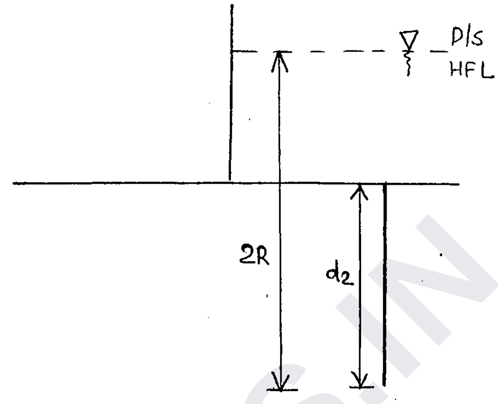
04. $q = 6.5\ m^3/s/m$

Lacey's scour depth,

$$R = 1.35\left(\frac{q^2}{f}\right)^{1/3}.$$

$$= 1.35 \times \left(\frac{6.5^2}{1}\right)^{1/3}.$$

$$= \underline{\underline{4.7\ m}}$$



Depth of d/s pile, $d_2 = 2\times4.7 - 4.4$

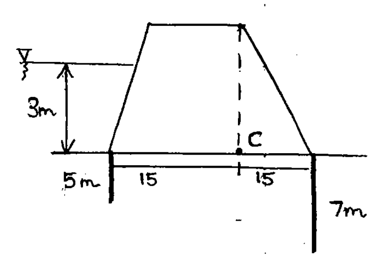
$= \underline{\underline{5.004\ m}}$

Length of launching apron, $L = 1$ to $1.5\,d_2$

$= \underline{\underline{5\ m}}$ to $7.5m$

Q c/s of a weir is shown in the fig. Relative density $= 2.5$. Match the following:

LIST I	LIST II.
A. Uplift pressure head (Bligh's Theory)	0.67 m.
B. As per Lane's theory.	0.645 m.
C. Thickness of floor. (as per Bligh's theory).	1.676 m.
D. As per Lane's theory	1.612 m.



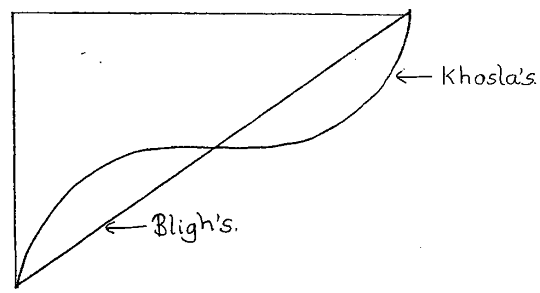
-29

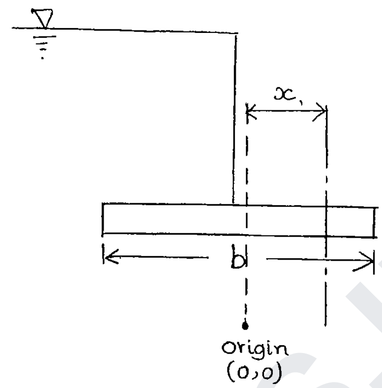
04

(i) Width of waterway, $P = 4.75\sqrt{Q}$.

(iii) Depth of d/s pile (D): $\Rightarrow$ ~~NB~~ $R = 1.35\left(\frac{q^2}{f}\right)^{1/3}$; $G_E = \frac{H}{d}\ \frac{\pi}{\sqrt{\lambda}}$.

(iv) Barrage floor thickness depends on uplift pr. head.





$$p = \frac{H}{\pi} \cos^{-1}\left(\frac{2x}{b}\right).$$

or $$\phi = \frac{p}{H} = \frac{1}{\pi} \cos^{-1}\left(\frac{2x}{b}\right).$$

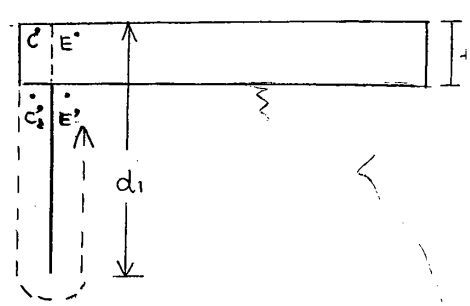
where $x \rightarrow$ length of floor where uplift is to be determined.

$b \rightarrow$ length of impervious floor.

→ Corrections to be Applied :

1. Thickness Correction. (+ve or −ve)

Assuming linear variation b/w C, D.



$$\text{Correction at } C' = \left(\frac{\phi_C - \phi_D}{d_1}\right) * t$$

$$\phi_{C_1} = \phi_C - \left(\frac{\phi_C - \phi_D}{d_1}\right) * t$$

2. Mutual Interference Correction. (+ve or −ve)

The effect of outer pile on intermediate pile is considered when:

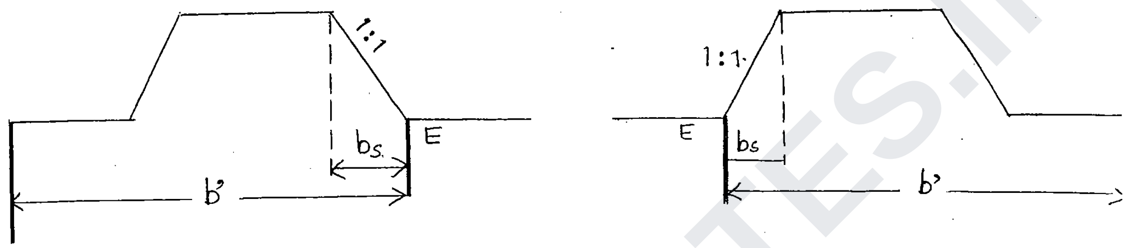
(i) Intermediate pile length > Outer pile length.

(ii) Distance b/w 2 piles > 2 times outer pile length

3. Slope Correction.

$$\text{Correction @ } E = \% \text{ correction} * \frac{b_s}{b'}$$

$$\text{Final } \phi_E = \text{Initial } \phi_E + \text{Correction}$$



1. For upward slope, correction is −ve and for downward slope correction is +ve.

2. Correction are to be made to the keypoint facing the slope.

3. If there is no pile at the slope end, no correction is allowed

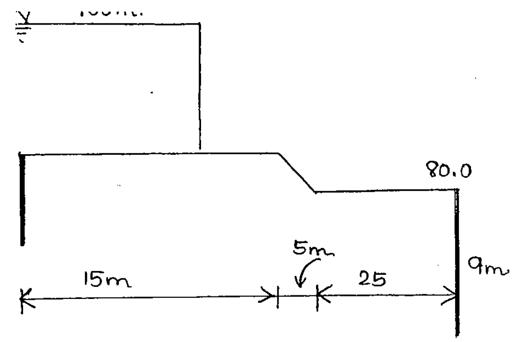
Q. A flow net for seepage under a sheet pile wall has $N_f = 4$, $N_d = 8$ and permeability of soil in horizontal and vertical direction are $K_h = 8 \times 10^{-5}$ m/s, $K_v = 2 \times 10^{-5}$ m/s. If the head loss through the soil = 2 m, the quantity of seepage per m length of wall is ___ ?

$$K = \sqrt{K_x K_y}$$

$$= \sqrt{8 \times 2 \times (10^{-5})^2} = 4 \times 10^{-5} \text{ m/s.}$$

$$q = kH \frac{N_f}{N_d} = 4 \times 10^{-5} \times 2 \times \frac{4}{8} = \underline{\underline{4 \times 10^{-5}}} \text{ m}^2/\text{s}$$

Q. Check the safety of barrage shown in fig against piping ~~agains~~ action. Safe exit gradient $= \frac{1}{5}$.



By Khosla's theory,

$$G_E, \text{ exit gradient} = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} \quad ; \quad \alpha = \frac{b}{d} = \frac{45}{9} = 5.$$

$$= \frac{1+\sqrt{1+25}}{2} = 3.04.$$

$$\therefore \quad G_E = \frac{(100-80)}{9} \times \frac{1}{\pi\sqrt{3.04}} = 0.405.$$

$i_c = \frac{1}{5}$ (given)
$= 0.2$

$\Rightarrow G_E > i_c$

$\therefore$ Unsafe

Q Consider the following:
Garrets diagram for the design of irrigation channels is based on :
a) Kennedy's Theory b) Lacey's Theory c) Kutter's Formula
d). Manning's eqn.
Which of these are correct ?
a) a & b b) a & c c) b & d d) b & c

Q. In a river, silt excluder and silt ejector are constructed
a). at a location after the head regulator and head of canal
b) at head of canal and at a location after head regulator.

c) At same location

d) At specific locations depending upon diverse factors and their respective locations do not follow a set pattern.

Silt excluder is provided before the regulator to prevent entry of silt into the canal.

Silt ejector is provided on the canal, ie after the head regulator to collect the silt entering into the canal.

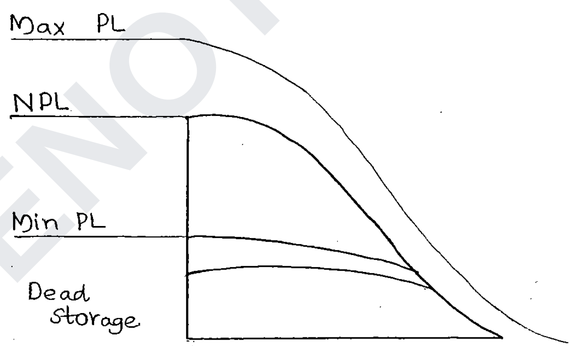
22nd Oct,
WEDNESDAY

(22)

03. GRAVITY DAMS

→ Zones of Storage :

1. Dead storage – volume below min pool level.
2. Live storage – volume b/w min. pool level & normal PL
3. Surplus storage – volume b/w NPL & max. pool level.
4. Bank storage.
5. Valley storage.
   – subsoil flow.



→ Forces Acting on Gravity Dam.

1. Water pressure (normal)
2. Uplift pressure. (due to seepage)
3. Earthquake forces (seismic zone)
4. Silt pressure. (due to silt deposited u/s).
5. Wave pressure.
6. Wt. of dam.
7. Wind pressure.
8. Ice pressure.

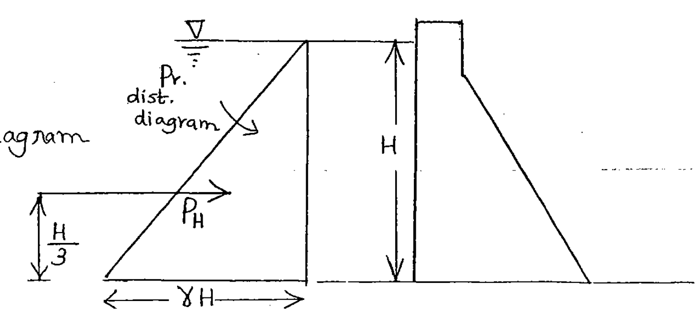
Gravity Dam is one in which all external forces are resisted by weight of dam (high sp. gravity)

→ Water Pressure.

P_H = area of pr. dist. diagram

$$\boxed{P_H = \frac{1}{2}\gamma H^2}$$

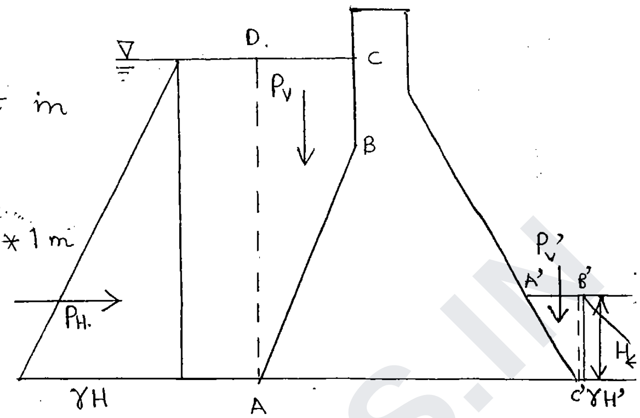
@ $\frac{H}{3}$ from base



$P_H = \frac{1}{2}\gamma H^2$

P_v = wt. of water present in prism ABCD.

$= \gamma_w * \text{Area of prism} * 1\,m$



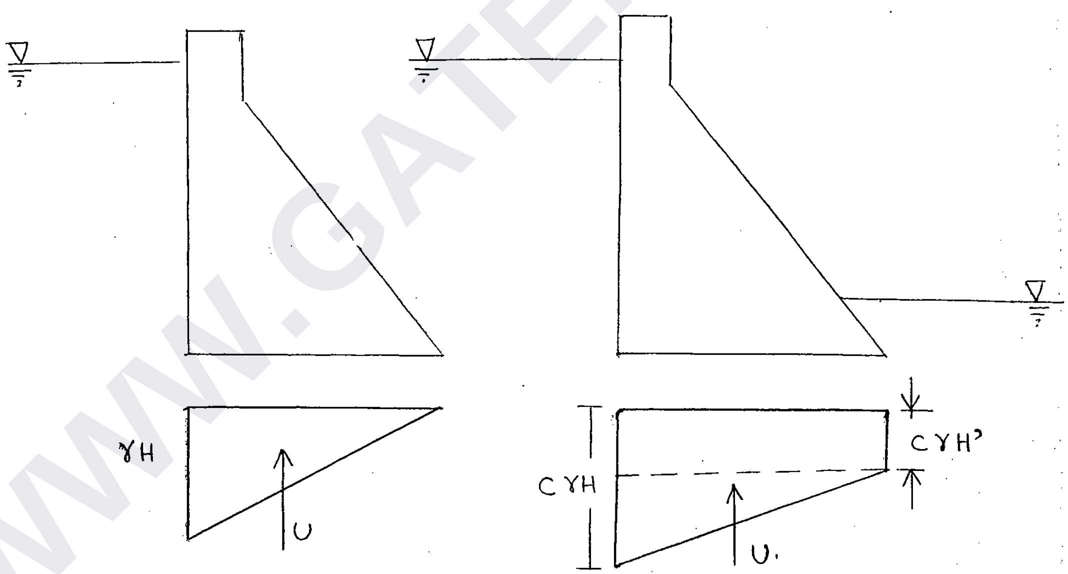
On downstream side (due to tail water of depth H').

$P_{H'} = \frac{1}{2}\gamma (H')^2$

$P_{v'}$ = wt. of water in A'B'C'

Net horizontal $= P_H - P_{H'}$

→ Uplift Pressure.



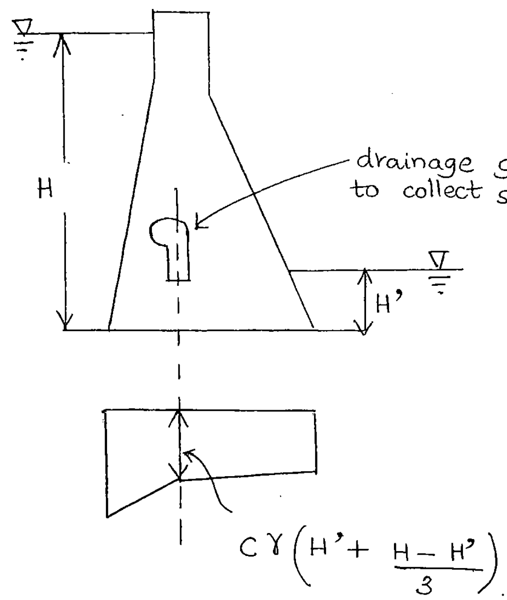
U → Area of uplift pressure distribution diagram.

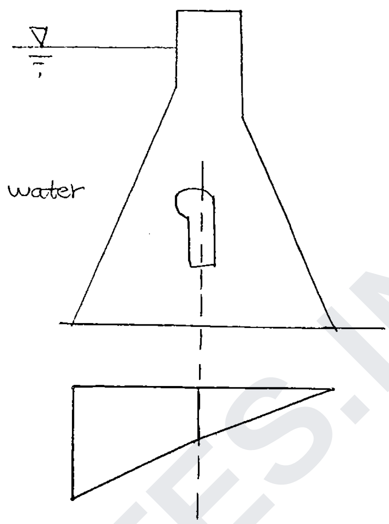
C → Uplift pressure coeffecient (or) Uplift intensity factor

C = 0 ; for hard rocks with seepage.

C = 1 ; for pervious nature of rock

$\boxed{0 \leqslant C \leqslant 1}$



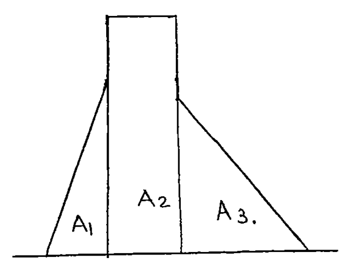


Uplift pressure = Area of uplift pressure dist. diagram.

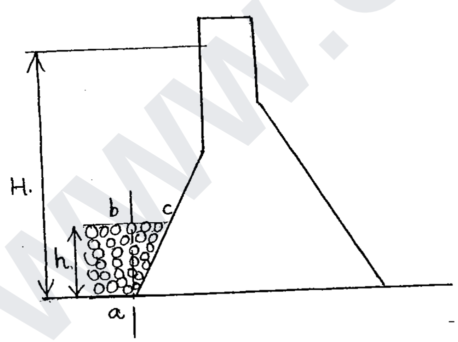
→ Weight of Dam.

Weight of dam = 1m * Area of c/s * γ_c.

Area of c/s = $A_1 + A_2 + A_3$.



→ Silt Pressure.



Horizontal silt pressure $= \frac{1}{2} \gamma_s * K_a * h^2$.

where γ_s → submerged unit wt. of silt

K_a → active earth pressure coeffecient. $\left(\frac{1 - \sin\phi}{1 + \sin\phi}\right)$.

h → depth of silt.

Vertical silt pressure, P_{sv} = Wt of silt in abc on inclined u/s portion

$\gamma_{mixture}$ = Water + Silt.
= 1360 kg/m^3 (Horizontal)
= 1920 kg/m^3 (Vertical).

24th Oct,
FRIDAY → Earthquake force (or) Seismic force.

- Primary Waves
- Secondary Waves.

} propogate in different direction.

Analysis is carried out in horizontal, vertical direction.

* Effect of Horizontal Acceleration.
  (i) Inertia Force.
  (ii) Hydrodynamic pressure.

* Critical Case
  - acceleration from D/s to U/s
  - inertia acts from U/s to D/s.

(i) Inertia Force, F_i = mass * acceleration.
$= m * \alpha_f g.$

α_f → seismic coeffecient in horizontal direction.
W → wt. of dam.

$$\Rightarrow F_i = W * \alpha_f.$$

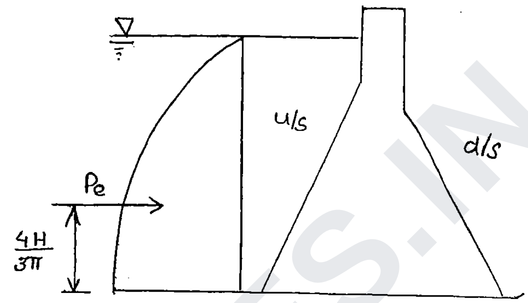
- when the earth quake acceloration is from D/s to U/s, the dam movement is resisted due to the inertia possessed by it.

(ii) Hydrodynamic pressure, $P_e = 0.55\, \alpha_h\, \gamma_w\, H^2$

Von Kerman Equation: @ $\frac{4}{3}\frac{H}{\pi}$.

$$M_e = P_e \times \frac{4}{3\pi} \times H$$

Hydrodynamic pressure is in addition to water pressure.



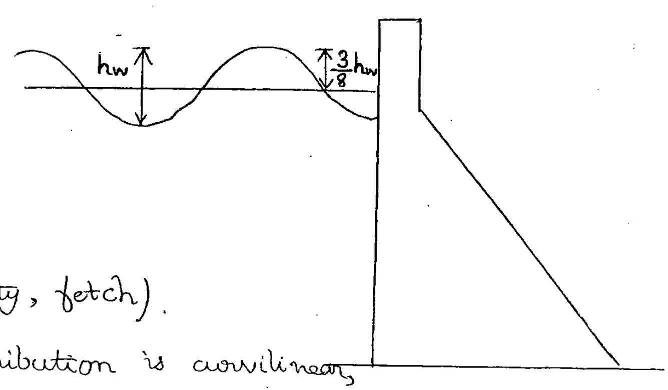
▫ hydrodynamic pressure distribution.

a) When earthquake acceleration is from d/s to u/s, the movement of the dam is resisted by the water in the reservoir due to its inertia. In this process, there is a momentary increase in pressure of water called as hydrodynamic pressure.

→ Wave Pressure

$p_w = 2.4\, \gamma_w\, h_w$

$P_w = 2.4\, \gamma_w\, h_w^2$



$P_{wave} = f(h_w)$

$h_w = f$ (wind velocity, fetch).

Wave pressure distribution is curvilinear, but for convenience

Fetch is defined as the longest straight distance from the dam upto the farthest point of reservoir

→ Wind Pressure $h_w = 0.0322\sqrt{VF} + 0.763 - 0.271\,(F)^{1/4}$;

Wind $F < 32$ km.

$h_w = 0.0322\sqrt{VF}$; $F > 32$ km (D-Molitor's formula)

h_w : wave height (m).

V : wind velocity (km/hr).

F : Fetch (km).

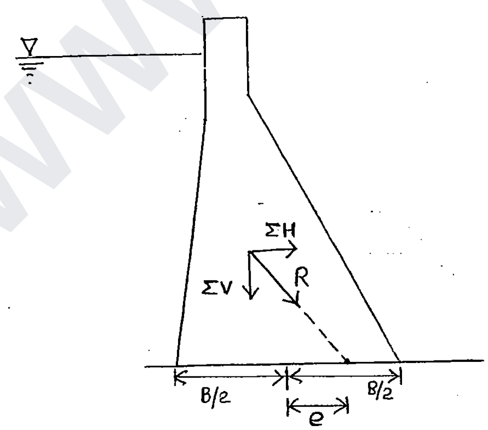
→ Wind Pressure

Wind pressure = 1 to 1.5 kN/m²

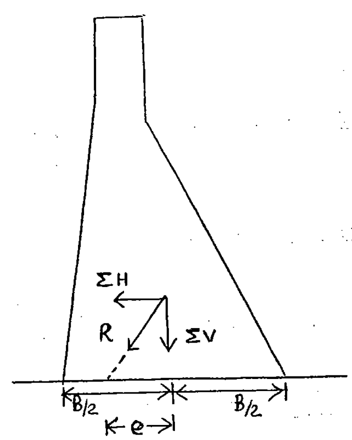
→ Ice Pressure

Ice pressure = 500 kN/m²

→ Normal Stresses



▫ Reservoir Full



▫ Reservoir empty

Reservoir Full:

$$(P_n)_{toe} = \frac{\Sigma V}{B}\left(1+\frac{6e}{B}\right)$$

$$(P_n)_{heel} = \frac{\Sigma V}{B}\left(1-\frac{6e}{B}\right)$$

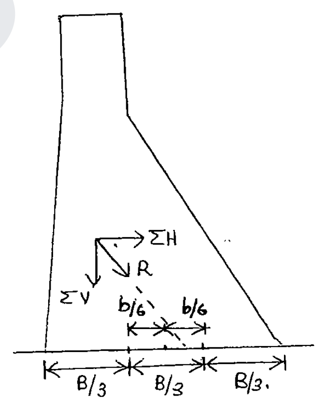
For tension to occur, $(P_n)_{min} = 0$.

$$\frac{\Sigma V}{B}\left(1-\frac{6e}{B}\right) = 0$$

$\therefore\ e = \frac{B}{6}$ or $e \leq \frac{B}{6}$ (middle third rule).

ie to avoid tension, $e \leq \frac{B}{6}$

Reservoir Empty:

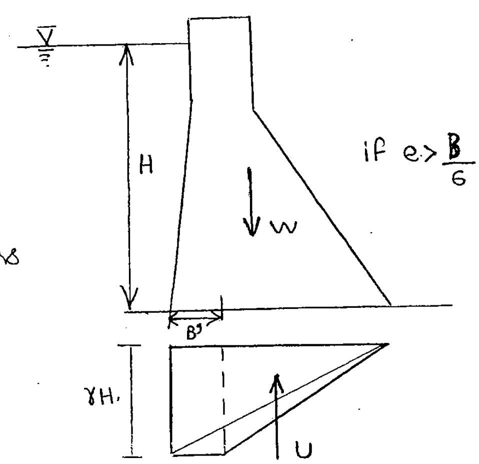


$$(P_n)_{max} \text{ @ heel} = \frac{\Sigma V}{B}\left(1+\frac{6e}{B}\right).$$

$$(P_n)_{min} \text{ @ toe} = \frac{\Sigma V}{B}\left(1-\frac{6e}{B}\right).$$

NOTE:

- When $e > \frac{B}{6}$, tension may develop. For reservoir full case tension develops at the heel.
- When tension prevails, cracks develop near the heal and uplift pr. distribution gets modified, ie, increases, reducing the net stabilising force.

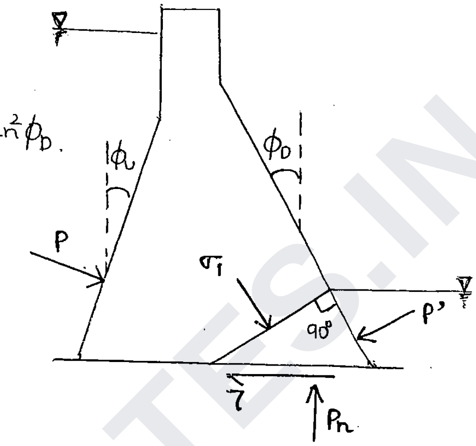


→ Principle Stress & Shear Stress.

- $(P_n)_{max}$ is not the max stress in dam
- Principal Plane → plane on which shear stress is zero

$\therefore$ u/s & d/s faces are principal planes

σ_1 is the principal stress



- $(\sigma_1)_{toe} = (P_n)_{toe} \sec^2\phi_D - p'\tan^2\phi_D$.

where $p' \rightarrow$ tail water pressure.

For no tailwater, $p' = 0$.

$\therefore \quad (\sigma_1)_{toe} = (P_n)_{toe} \sec^2\phi_D$

$\sec^2\phi_D > 1$ (always)

$\Rightarrow$ $(\sigma_1)_{toe}$ is max

$(\sigma_1)_{heel} = (P_n)_{heel} \sec^2\phi_U - p\tan^2\phi_U$

- $(\tau)_{toe} = \left[(P_n)_{toe} - p'\right] \tan\phi_D$.

  $(\tau)_{heel} = \left[(P_n)_{heel} - p\right] \tan\phi_U$

→ Factor of Safety

(i) Factor of Safety against Overturning.

$$FOS = \frac{\Sigma M_R}{\Sigma M_O} \qquad (1.5 - 2.5)$$

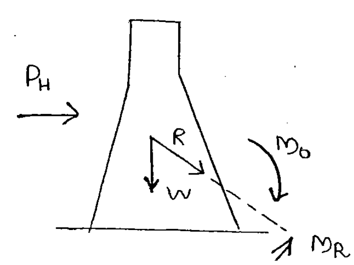
$M_R \rightarrow$ restraining moment (due to weight)

$M_O \rightarrow$ overturning moment (due to external forces)

$M_R > M_O$; Safe

$M_R = M_O$; Critical

$M_R < M_O$; Unsafe



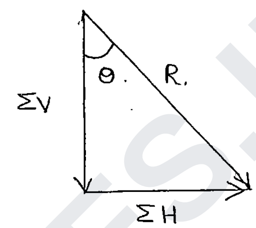
(ii) Factor of Safety against Sliding

$$\mu \Sigma V > \Sigma H.$$

μ : coeffecient of friction $(= 0.65 \text{ to } 0.75)$

For safe condition,

$$\frac{\mu \Sigma V}{\Sigma H} > 1$$



$$\tan\theta = \frac{\Sigma H}{\Sigma V}.$$

$$\Rightarrow \frac{\mu}{\tan\theta} > 1$$

where $\tan\theta \rightarrow$ sliding factor

$\frac{\mu}{\tan\theta} = 1$; critical

$\frac{\mu}{\tan\theta} < 1$; unsafe

NOTE:

- For low dams, safety against sliding should be checked for only friction. For high dams, the shear strength of joint must be considered. Generally horizontal joints are provided to have a better bond b/w two surfaces.

- (iii) Shear Friction Factor (SFF)

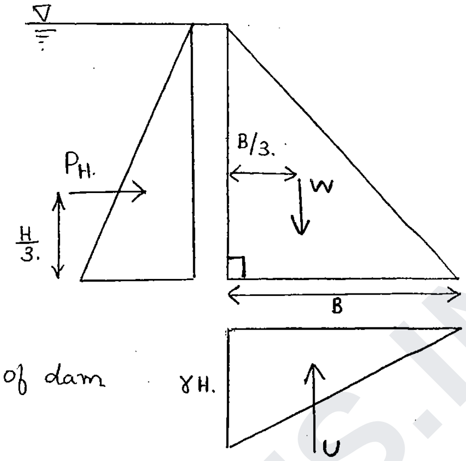
$$SFF = \frac{\mu \Sigma V + Bq}{\Sigma H.}$$

$q \rightarrow$ shear strength of material.

→ Elementary Profile of a Gravity Dam

Elementary profile is a profile with no top width and no free board.

* Base width of elementary profile for no tension case :



$$B = \frac{H}{\sqrt{G-C}}$$

G → sp. gravity of material of dam

C → uplift pr. coeffecient.

If $C = 0$,

$$B = \frac{H}{\sqrt{G}}$$

* Base width for no sliding case :

$$B = \frac{H}{\mu(G-C)}$$

μ → coeffecient of friction.

$$B = \frac{H}{\mu G} \quad ; \quad \text{if } C = 0$$

The highest among the two base width values is to be selected.

* Principal & Shear Stresses in Elementary Profile

– For reservoir full condition

$$(P_n)_{toe} = \gamma H(S-C).$$

$$(P_n)_{heel} = 0$$

$$(\sigma_1)_{toe} = \gamma H(S-C+1).$$

$$\tau_{(toe)} = \gamma H\sqrt{S-C}$$

* Limiting Height of Elementary Profile.

$$\sigma_1 \not> f$$

At limiting condition,

$$\sigma_1 = f$$

$$\gamma H (s-c+1) = f$$

$$\Rightarrow H = \frac{f}{\gamma_w (s-c+1)} \rightarrow (1)$$

If $c = 0$,

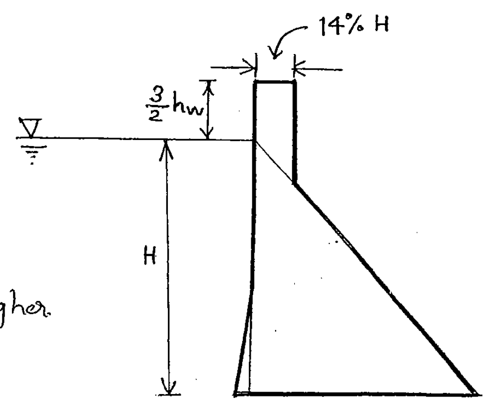
$$H = \frac{f}{\gamma_w (s+1)}$$

where $f \rightarrow$ max. allowable compressive stress.

⊙ A dam is called as 'high dam' if the height of elementary profile is more than the value computed from eqn (1). In such cases, the dam may fail by crushing. If the height of dam is less than the H value computed from eqn (1), it is called as 'Low Dam'

→ Practical Profile.

Practical profile has top width and freeboard.

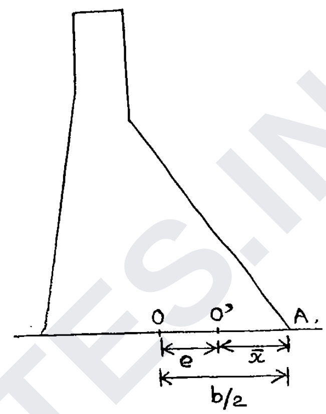


Freeboard = $\frac{3}{2}$ hw (or) 0.9m whichever is higher

→ Stability Analysis.

1. Compute all horizontal and vertical component of forces.

2. $e = \frac{b}{2} - \bar{x}$ $\quad (\bar{x} = O'A)$.



3. $\bar{x} = \frac{\Sigma M}{\Sigma V}$.

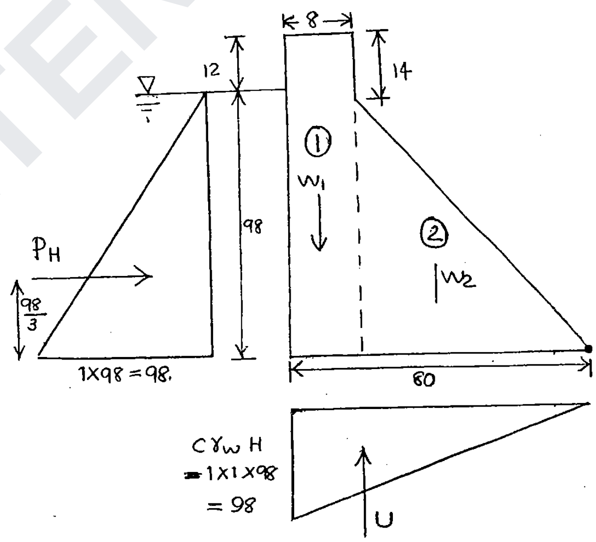
4. Compute $(P_n)_{max}$ & $(P_n)_{min}$.

5. Compute σ_1 & τ.

6. Factor of safeties against overturning, sliding & SFF.

P-16

01. $\gamma_c = 2.4$ t/m³.
$\gamma_w = 1$ t/m³.
$\mu = 0.75$
$q = 140$ t/m²



Item.	Vertical force	Horizontal force.	Lever Arm	Resisting moment	Overturning moment.
W_1 ↓	$110 \times 8 \times 1 \times 2.4$ = ~~528~~ 2112		$72 + \frac{8}{2} = 76$ m.	2112×76 = 160512	
W_2 ↓	$\frac{1}{2} \times 72 \times 96 \times 2.4$ = ~~8467.2~~. 8294.4.		$\frac{2}{3} \times 72 = 48$ m.	8294.4×48 = 398131.2	
U ↑	$\frac{1}{2} \times 80 \times 98$ = 3920		$\frac{2}{3} \times 80 = 53.3$ m.		3920×53.3 = 209053.6
P_H. →		$\frac{1}{2} \times 98 \times 98$ = 4802	$\frac{98}{3} = 32.66$.		4802×32.66 = 156833.32

$$\Sigma M_R = 558643.2 \ \text{tm}$$

$$\Sigma M_0 = 365886.92 \ \text{t.m}$$

$$\Sigma V = W_1 + W_2 - U = 6486.4 \ \text{t}$$

$$\Sigma M = \Sigma M_R - \Sigma M_0 = 192756.28 \ \text{t.m.}$$

$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{192756.28}{6486.4} = 29.71 \ \text{m.}$$

$$e = \frac{b}{2} - \bar{x} = 40 - 29.71 = 10.29 \ \text{m.}$$

$$\frac{b}{6} = \frac{80}{6} = 13.33 \ \text{m} \Rightarrow e < \frac{b}{6}$$ (no tension at heel).

c) Pressure on foundation soil :

$$P_{max} = \frac{\Sigma V}{b}\left(1 + \frac{6e}{b}\right)$$

$$= \frac{6486.4}{80}\left(1 + \frac{10.29}{13.33}\right) = 143.6 \ \text{t/m}^2 \ (+ve).$$

$$P_{min} = \frac{\Sigma V}{b}\left(1 - \frac{6e}{b}\right) = \frac{6486.4}{80}\left(1 - \frac{10.29}{13.33}\right)$$

$$= 18.5 \ \text{t/m}^2 \ (+ve).$$

d) Max compressive stress, $(\sigma_1)_{toe} = (P_n \sec^2 \phi_D)$; $p' = 0$.

$$\phi_D = \tan^{-1}\left(\frac{72}{96}\right) = 36.87^\circ$$

$$\sigma_1 (toe) = 1.5625 \times 143.6 = \underline{\underline{224.375}} \ \text{t/m}^2$$

a) FOS against overturning $= \dfrac{\Sigma M_R}{\Sigma M_0} = \dfrac{558643.2}{365886.92} = \underline{\underline{1.526}}$

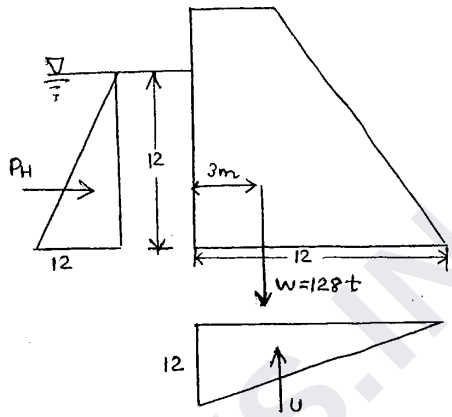
b) FOS against sliding $= \dfrac{\mu \Sigma V}{\Sigma H} = \dfrac{0.75 \times 6486.4}{4802} = \underline{\underline{1.013}}$

c) $SFF = \dfrac{\mu \Sigma V + bq}{\Sigma H} = \dfrac{0.75 \times 6486.4 + 80 \times 140}{4802} = \underline{\underline{3.345}}$

04 $W = 128t$ @ 9 m from toe.

$U = \frac{1}{2} \times 12 \times 12 = 72$ @ 8m from toe

$P_H = \frac{1}{2} \times 12 \times 12 = 72$. @ 4m from toe.



FOS against overturning $= \frac{\Sigma M_R}{\Sigma M_o}$

$M_R = W \times 9 = 128 \times 9$ tm.

$M_o = 72 \times 8 + 72 \times 4 = 864$ tm.

$$FOS = \frac{128 \times 9}{864} = \underline{\underline{1.33}}$$

02. $\Sigma M_o = 3 \times 10^5$ tm.

$\Sigma M_R = 4 \times 10^5$ tm $\Rightarrow \Sigma M = 1 \times 10^5$ tm.

$W = 6000$ t $(\downarrow)$

$U = 1000$ t $(\uparrow)$ $\Rightarrow \Sigma V = 5000$ t.

$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{10^5}{5000} = 20 \text{ m}.$$

$$e = \frac{b}{2} - \bar{x} = \frac{50}{2} - 20 = \underline{\underline{5 \text{ m.}}} < \frac{b}{6} (= 8.33 \text{ m})$$

$$P_{max} = \frac{\Sigma V}{b}\left(1 + \frac{6e}{b}\right) = \frac{5000}{50}\left(1 + \frac{6 \times 5}{50}\right)$$

$$= \underline{\underline{160}} \text{ t/m}^2$$

$$P_{min} = \frac{\Sigma V}{b}\left(1 - \frac{6e}{b}\right) = \frac{5000}{50}\left(1 - \frac{6 \times 5}{50}\right). = \underline{\underline{40}} \text{ t/m}^2$$

06. $$h = \frac{f}{\gamma_w (s+1)} = \frac{7.5 \times 10^3}{9.81 (2.4+1)} = 224.94 \text{ m} \approx \underline{\underline{225 \text{ m}}}$$

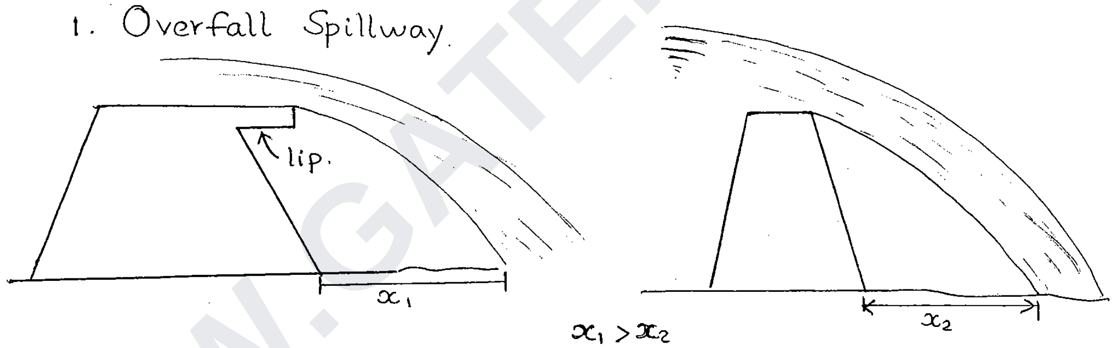
04. SPILLWAYS

Spillways are used to release excess flow entering into the reservoir.

- It is a safety ~~wall~~ valve for the dam.
- It is located along the length of dam or can be seperated.
- It can be provided along rim of a reservoir or in a saddle. or in an abutment of a dam.

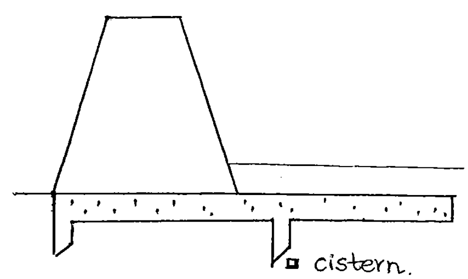
→ Types of Spillways.

1. Overfall Spillway.



In a spillway with lip, jet of water away from spillway.

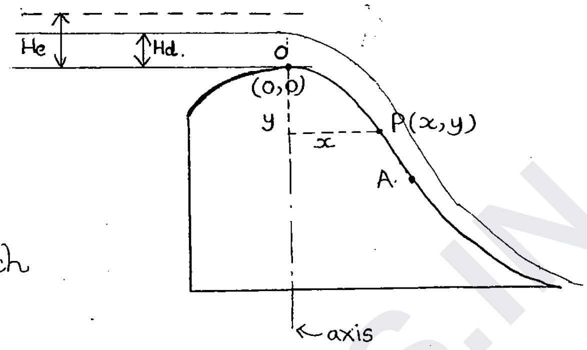
In a spillway without lip, erosion of soil occurs due to impact of water on soil



Erosion is prevented in an overfall spillway by providing a cistern.

|imp|

2. Overflow Spillway or Ogee Spillway



H_d = design head.

$$H_e = H_d + \frac{V_a^2}{2g}$$

$\frac{V_a^2}{2g} \rightarrow$ velocity of approach

Case 1 : $H = H_d$

- max. discharge.
- jet of water flows along the d/s face.
- atmospheric pressure

Case 2 : $H < H_d$

- less discharge
- depressed nappe is formed.
- subject to +ve hydrostatic pressure (> atm. pressure)

Case 3 : $H > H_d$

- seperation of flow occurs
- flow loses its contact with surface.
- (-ve) pressure develops.
- water boils, bubbles form. Bubbles are carried by water to a high pressure zone and collapse.
- cavitation occurs

* Equation of d/s profile for OA portion :

$$x^n = K H_d^{(n-1)} y.$$

For u/s vertical, $n = 1.85$ & $K = 2$.

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

From model studies, it has been observed that

if $H > 33.5\%\ H_d$, $-ve$ pressure $= 0.35\ H_d$ (at axis).

$-ve$ pressure $= 0.5\ H_d$ (at a distance of $0.2\ H_d$ from axis towards u/s)

* $\boxed{\text{Discharge, } Q = C L_e H_e^{3/2}}$

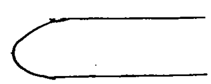
* where L_e, effective length $= L - 2(k_p N + k_a) H_e$.

$L \rightarrow$ clear length of spillway.

L = total length − no. of piers * thickness of each pier.

L_e is used to account for end contractions that occurs due to pier, abutments etc.

$K_p \rightarrow$ pier contraction coeffecient $= f$(shape of pier).



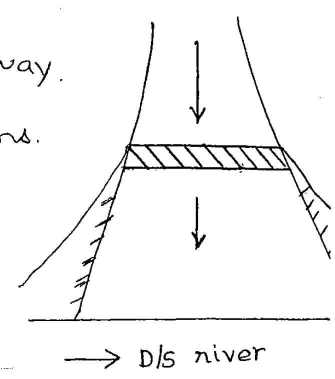
- cut water nose pier (less energy loss).
- blund nose pier (more losses).

$N \rightarrow$ no. of piers.

$K_a \rightarrow$ abutment contraction coeffecient.

$C \rightarrow$ coeffecient of discharge. (≈ 2.2)

3. Chute / Trough / Open Channel Spillway.



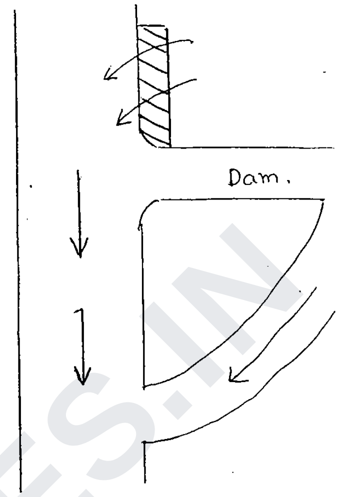
- preferrable for rockfill or earthen dams.
- erodable nature of bed of river.
- abutment of dam.

4. Side Channel Spillway

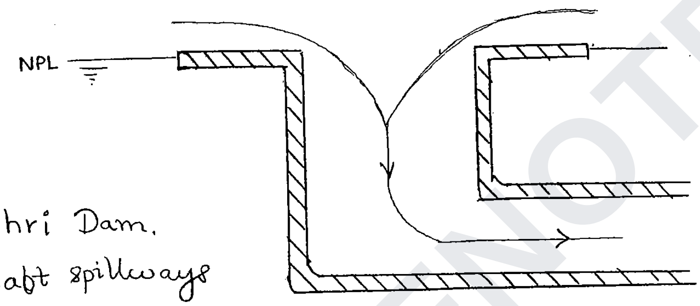
- wherever chute spillway is not possible.

5. Shaft Spillway

- provided in the reservoir
- whenever space is not available for spillway, shaft spillway is preferred.
- It has a shaft connected to a horizontal section embedded in the foundation of a dam, which is then connected to diversion tunnels.

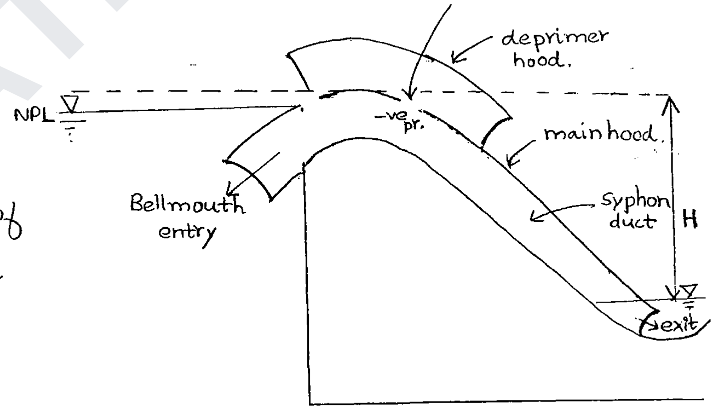


▫ side channel spillway.



Eg: Tehri Dam.
(4 shaft spillways
one chute spillway)

6. Syphon Spillway



Bellmouth entry:
To prevent entry of floating matter into the syphon passage.

Deprimer hood:
To prime the passage

Exit:
to prevent entry of air.

* $\boxed{\text{Discharge, } Q = C_d A \sqrt{2gH}}$

where ~~A~~ L → length of spillway ; b → throat depth.

$A = L \times b$.

- used when less space is available

P-19

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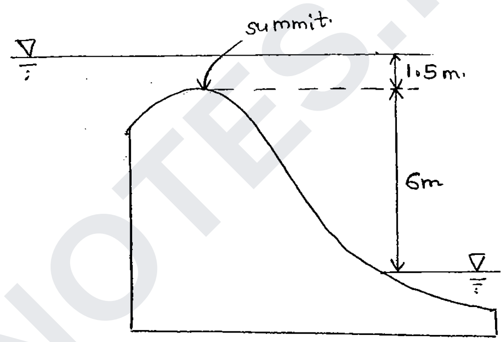
01. $C_d = 2.4$, $H_d = 2$, $L = 100$ m.

$$Q = C_d L H^{3/2}$$

$$= 2.4 \times 100 \times 2^{3/2}$$

$$= \underline{\underline{678 \ m^3/s}}$$

02. $H = 6 + 1.5 = 7.5$ m.



$A = 4 \times 1 = 4 \ m^2$.

$C_d = 0.6$

$$Q = C_d A \sqrt{2gH}$$

$$= 0.6 \times 4 \sqrt{2g \times 7.5}$$

$$= \underline{\underline{29.11 \ m^3/s}}$$

Q. A discharge of 72 m^3/s is to be allowed through syphon spillways of 2m width and 75 cm depth, with working head of 8m. The no. of spillways to be provided will be ___ (Assume $C_d = 0.64$).

$$\text{Discharge}, \ Q = C_d A \sqrt{2gH}$$

$$= 0.64 \times 2 \times 0.75 \sqrt{2g \times 8}$$

$$= 12.025 \ m^3/s$$

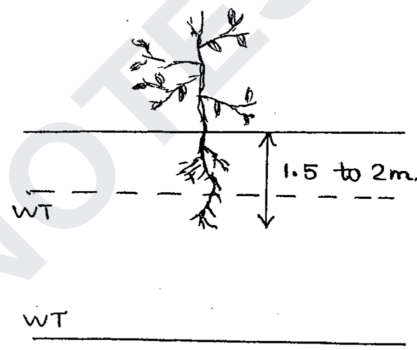
$$\text{No. of spillways} = \frac{\text{Total discharge}}{\text{Discharge per spillway}} = \frac{72}{12.025} \approx \underline{\underline{6 \text{ no.s}}}$$

6th Oct,
JNDAY

07 WATER LOGGING

Due to various recharge condition, the water table may reach the root zone of crop. Due to this, the air circulation in the root is cut off. This phenomenon is called as Water logging.

Eg: Punjab I - Indo Gangetic plains.



→ Causes

(i) Seepage from canal system
(ii) Seepage from reservoir
(iii) Heavy rainfall.
(iv). Improper surface drainage system.
(v) Flat. topography (vi) Over irrigation.

→ Effects

(i) Yield of crop decreases.
(ii) Aerobic bacteria is lost. (nitrification process is halted)
(iii) Climatic damp (Low temperature).
(iv) Low temperature leads to delay in germination of seed, ripening, restricted root growth.
(v). Plant diseases.
(vi) Land preparation becomes difficult.
(vii) Soil turns into saline.

→ Control measures

(i) Lined canals.

(ii) Provide proper drainage system.

(iii) Training the cultivators.

(iv) Conjunctive use of water. (use surface water & subsurface water)

(v) Prevent over irrigation.

(vi) Volumetric assessment of water.

(vii) Cultivate some percentage of CCA only.

(viii) Select crops which require less amount of water.

(ix) Crop rotation.

(x) Light irrigation.

26th Oct,
SUNDAY

08. QUALITY OF IRRIGATION WATER

→ Electric Conductivity

Units : millimhos/cm
or
micromhos/cm

- Irrigation water contains. Na, K, Ca & Mg ions. which represents quality of irrigation water.

- concentration of salt = m. eq/L or mg/L or ppm.

$$m.eq/L = \frac{mg/L}{\text{Equivalent wt.}}$$

→ Coagulated clod.

- assemblage of soil particles.
- occurs due to cementing properties imparted by Ca, Mg silicates and aluminates.
- when water with Na content is added to above, following reactions take place.

$$Na + \begin{matrix} Ca \\ Mg \end{matrix}\ \text{silicates} \longrightarrow Na\ \text{silicates} + Ca, Mg$$

- when this happens cementing property is lost. As a result, chumbling and disintegnation of clod occurs.
- Silica settles down and fills the voids, thus reducing the permeability of soil.

Sodium hazard of water is assessed in terms of Exchangeable Sodium Ions. (

* Exchangeable Sodium Ratio, $ESR = \dfrac{Na^+}{Ca^{++} + Mg^{++} + K^+}$

If sodium hazard is represented as percentage, called as Exchangeable Sodium Percentage (ESP),

$$ESP = \left(\frac{ESR}{1+ESR}\right) * 100 = \frac{Na^+}{Ca^{++} + Mg^{++} + K^+ + Na^+}$$

→ Sodium Adsorption Ratio (SAR).

$$SAR = \frac{Na^+}{\sqrt{\dfrac{Ca^{++} + Mg^{++}}{2}}} \; ; \; (m.eq/L)$$

Irrigation water rich in bicarbonate content tend to precipitate Ca & Mg in soil as their carbonates which are insoluble.

$$Ca + 2HCO_3 \rightleftharpoons \underset{(ppt)}{CaCO_3}\downarrow + H_2O + CO_2$$

$$Mg + 2HCO_3 \rightleftharpoons \underset{(ppt)}{MgCO_3}\downarrow + H_2O + CO_2$$

∴ SAR increases.

* Residual Sodium Carbonate (RSC)

$$RSC = (CO_3^{--} + HCO_3^-) - (Ca^{++} + Mg^{++}).$$

RSC < 1.25 ⇒ fit for irrigation water.

Irrigation water must have low conc. of dissolved salts or low salinity. High conc. of dissolved salts may result in dehydration of plants due to osmotic effects.

→ Saline Soil.

- presence of NaCl.

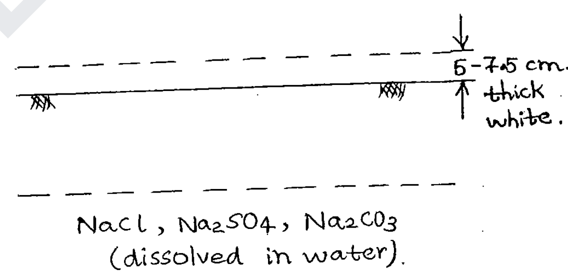
→ Sodic Soil / Alkaline Soil.

- presence of Na_2CO_3.

- Saline soil is reclaimed by leaching (not difficult).
- Sodic soil is difficult to be reclaimed. Sulphur + gypsum is added first and then leaching is done.

→ Salt Efflorescence.

When water with dissolved salts of NaCl, Na_2SO_4, Na_2CO_3



→ Standards of Irrigation Water

(i) Electrical Conductivity, EC = 0 – 1000 micromhos/cm

(ii) TDS = 0 – 700 ppm

(iii) ESP = 0 – 60 ppm

(iv) Chloride = 0 – 142

(v) Sulphate = 0 – 192

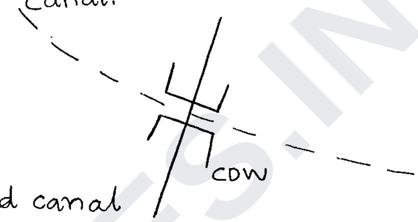
(vi) Boron = 0 – 0.5

(vii)

CROSS DRAINAGE WORKS

Type I : Canal above the drain.

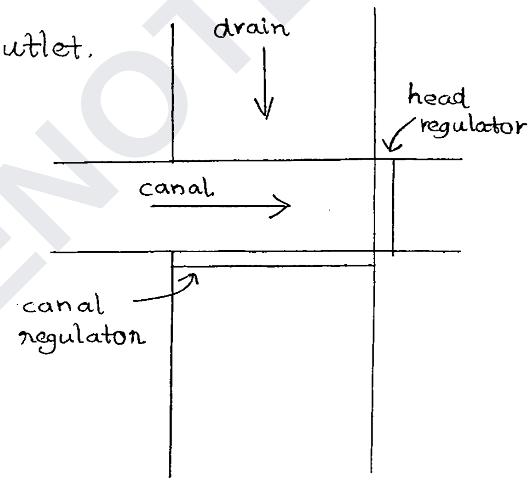
Eg: Aquaduct, Syphon aquaduct.



Type II : Drain above the canal.

Type III : Bed level of the drain and canal meet at the same level.

Eg: Level crossing inlet and outlet.

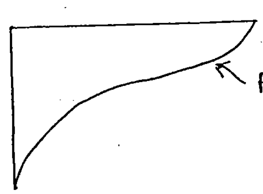


Q. When seepage takes place below a horizontal floor without any sheet piles, streamlines and equipotential lines are ____

a) Hyperbolas & Parabolas. c) Parabola & Ellipse

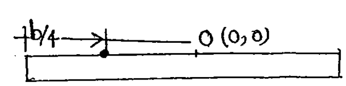
b) Ellipses & Hyperbolas d) Circles & Ellipse.

Q. When seepage of flow takes place below a flat floor without any sheet piles, the potential ϕ_x below the floor at one-fourth the floor length from u/s end is ____



$$p = \frac{H}{\pi}\cos^{-1}\left(\frac{2x}{b}\right).$$

$$\phi = \frac{P}{H} = \frac{1}{\pi}\cos^{-1}\left(\frac{2x}{b}\right).$$



Put $x = -\frac{b}{4}$

$$\phi_x = \frac{1}{\pi} \cos^{-1}\left(\frac{-\frac{2b}{4}}{b}\right)$$

$$\Rightarrow \phi_x = 66.6\ \%$$

Q. At a certain point in the floor of the weir, the uplift pressure head due to seepage is 3. The relative density of concrete is 2.5. The min. thickness of floor is

Q. In an unlined canal, if the depth of floor changes from 1 m to 1.2 m, find the corresponding % change in non-silting, non scouring velocity of flow.

As per Kennedy's theory,

$$V_0 = 0.55\, D^{0.64}$$

$$\frac{V_{01}}{V_{02}} = \left(\frac{D_1}{D_2}\right)^{0.64}.$$

$$V_{02} = V_{01} \times \left(\frac{1.2}{1}\right)^{0.64} = 1.124\, V_{01}$$

$\Rightarrow$ % change in velocity $= \underline{\underline{12.4\%}}$

Q. A non cohesive soil has a porosity of 30%, and relative density of soil particles 2.7. Value of critical exit gradient is ——

$$i_c = (1-n)(G-1) = (1-0.3)(2.7-1) = \underline{\underline{1.19}}$$

Q. Assertion : Duty of water decreases as the point of its measurement moves away from field of application. 40

Reason : Duty depends on soil characteristics.