CE 461

Irrigation & Flood Control

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Objective of this course

This course contains fundamental information on irrigation and flood control. After completion of this course one should be able to design an irrigation project which includes water requirements, canal design and hydraulic structures for irrigation project. In addition one should be able to design hydraulic structures for flood protection also.

Syllabus

Importance of irrigation, Sources and Quality of irrigation water, Soil Water Relationship, Consumptive Use and Estimation of Irrigation, Methods of Irrigation, Water Requirements, Design of Irrigation, Canal System, Irrigation Structures, Irrigation Pumps, Problems of Irrigation Land. Flood and its Control

Reference Books

- Irrigation Engineering & Hydraulic Structures
  ----- *S K Garg*
- Irrigation Development and Management in Bangladesh
  ----- *M A Sattar*
- Irrigation Engineering
  ----- *N N Basak*
- Irrigation Principles and Practices
  ----- *Vaughn E. Hansen & W. Israelsen*
- Irrigation (Theory & Practice)
  ----- *A M Micheal (2nd Edition)*
- Irrigation and Water Management
  ----- *Dilip Kumar Majumder*
- Irrigation Engineering
  ----- S. K. Mazumder
- Irrigation Engineering
  ----- *R. K. Sharma & T. K. Sharma*

Assessment

Class Grading - 30%

(Attendance - 10%, Class test -20%)

Mid Term Examination - 20%

Final Examination - 50%

Lecture Plan

Chapter	Name of Chapters	No. of Lectures
1	Methods of Irrigation	3
2	Sources & quality of irrigation water	2
3	Consumptive use & estimation of water requirements of crops	3
4	Soil water relationship	2
5	Physical & economical justification for canal	3
	Midterm Examination	
6	System of irrigation canal	2
7	Canal design	3
8	Irrigation structures – 1	2
9	Irrigation structures – 2	4
10	Spillway & irrigation pumps	3
	Final Examination	

Thank You

CHAPTER – 1

Methods of Irrigation

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LECTURE 2

Definition of Irrigation

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

History

- Started in ancient civilizations in arid and semi-arid climatic zones – where the food shortage was acute.
- Old cities excavated in Asia and Africa have disclosed that more than 5000 years ago people in Mesopotamia, Lebanon, China, Indo-Pakistan, Egypt adopted a permanent irrigation canal system. They knew that without digging canals and taking water from rivers through canals, land cultivation was not possible.
- Chinese are known to have practiced irrigation since 2627 BC. Has the longest irrigation canal, known as Grand Canal of 1285 miles, constructed originally by the Sui Empire during 589 to 618 AD and completed 1300 AD. This oldest and longest canal links Beijing and Hanchow by crossing the Great Yangzi and Yellow rivers and still serving 159223 sq. km of area.
- One of the largest irrigation canal of North Kiangsu Province of China which is 170 km long and 127 m wide in bed was dug by 13,00,000 people in 80 days. 69.9 million cum earthwork was done only with shovels and baskets.

Importance of Irrigation

Following are the factors which govern the importance of irrigation:

- Insufficient rainfall
- Uneven distribution of rainfall
- Improvement of perennial crops
- Development of agriculture in desert area

Advantages of Irrigation

- Increase in food production
- Optimum benefits
- Elimination of mixed cropping
- Improvement of cash crops
- Source of revenue
- General prosperity
- Generation of hydroelectric power
- Domestic water supply
- Facilities of communications
- Inland navigation
- Afforestation

Disadvantages of Irrigation

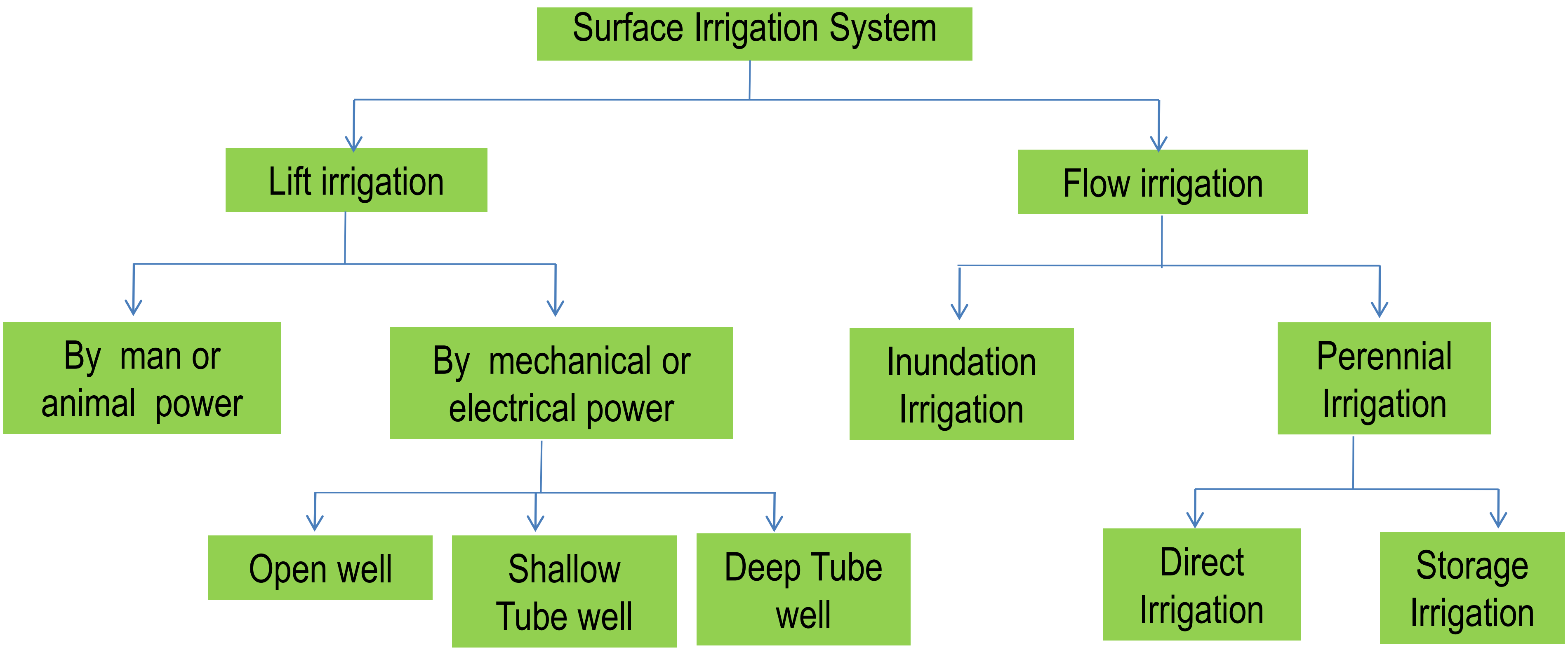
- Rising of water table: water-logging
- Problem of water pollution (nitrates seepage into GW)
- Formation of marshy land
- Dampness in weather
- Loss of valuable lands

Types of Irrigation

Broadly classified into:

- Surface Irrigation, and
- Sub-surface irrigation

System of Irrigation



Lecture 2

Lift Irrigation

Lecture 2

Subsurface irrigation system

Sub-Surface Irrigation System

- Natural Sub-irrigation
- Artificial Sub-irrigation

LECTURE 3

Methods of Irrigation

- Free Flooding
- Border Flooding
- Check Flooding
- Basin Flooding
- Furrow irrigation method
- Sprinkler irrigation method
- Drip irrigation method

Free Flooding or Ordinary Flooding

- Ditches are excavated in the field
- Movement of water is not restricted, it is sometimes called "*wild flooding*"
- It is suitable for close growing crops, pastures etc.
- This method may be used on rolling land (topography irregular) where borders, checks, basins and furrows are not feasible.

Contd…… Free Flooding

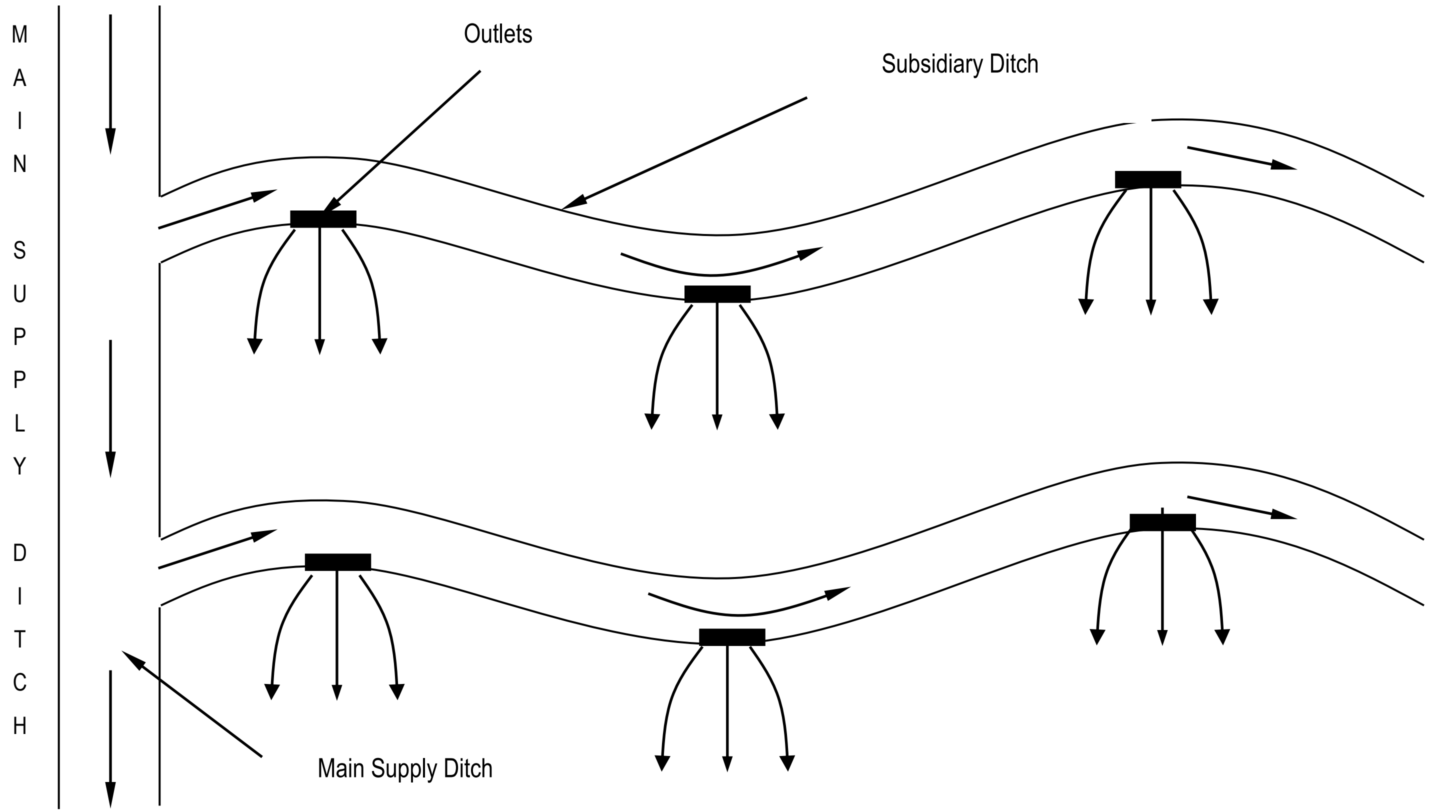


Fig: Free flooding (plan view)

Border Flooding

- Land is divided into a number of strips
- Strips separated by low levees called “borders”
- Strip area: width 10 ~ 20 m and length 100 ~ 400 m
- Ridges between borders should be sufficiently high
- The land should be perpendicular to the flow to prevent water from concentrating on either side of the border

Contd. ……..Border Flooding

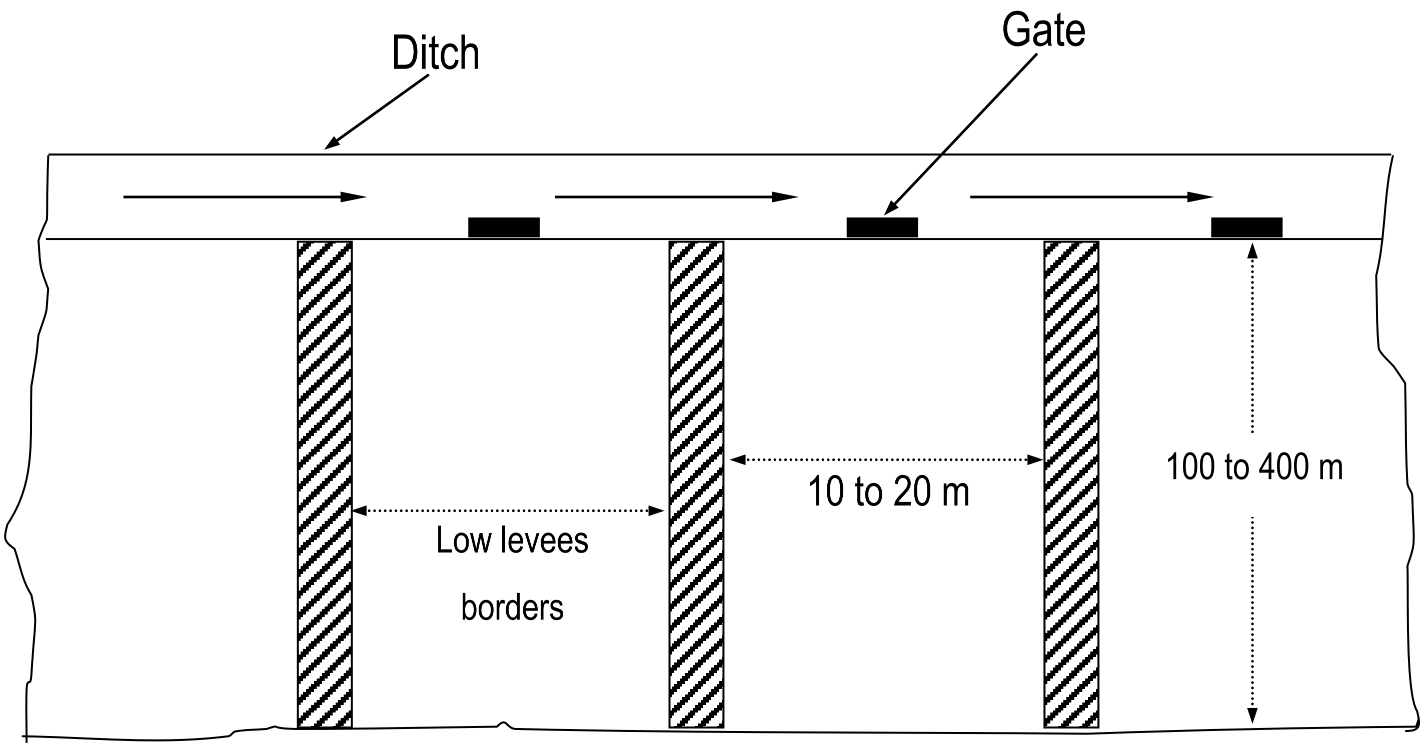
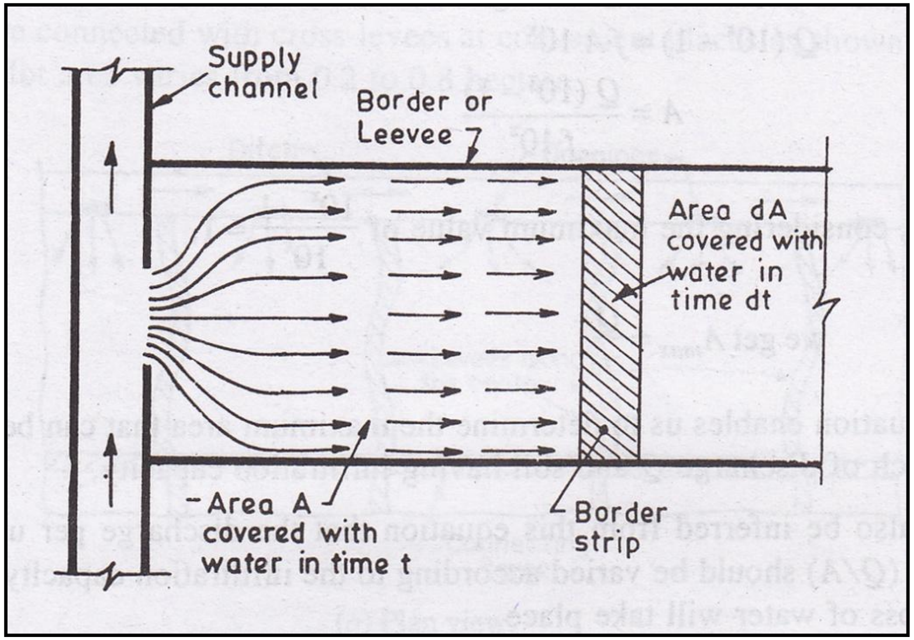


Fig: Border Flooding (plan view)

Time estimation for border flooding



Derivation of Equation:

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

Considering small area, *dA* of the border strip of area (*A*)

Depth of water, *y* over this area (*A*)

Assume that in time *dt*, water advances over this area (*dA*).

Now, the volume of water that flows to cover this area = y.dA ------- (1)

During the same time dt

The volume of water that percolates into the soil over the area (A)

$$A = f.A.dt \text{ ---------------- } (2)$$

The total quantity of water supplied to the strip during time (dt)

$$dt = Q.dt \text{ ------------- } (3)$$

From equation (1), (2) & (3) $\Rightarrow$

$$\therefore\ Q\ .dt = y.\ dA + f.\ A.\ dt$$

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

For getting time required to irrigate the whole land, we have to integrate the above equation and considering y, f, and Q as constants

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

After integrating the equation in the previous slide, we get.

$$t = \frac{y}{f}\ln\left(\frac{Q}{Q-f.A}\right) + C \text{ (Constant)} \qquad \text{---------------} (4)$$

But at, t = 0, A = 0

From equation (4) $\Rightarrow$

$$0 = \frac{y}{f}\ln\left(\frac{Q}{Q-f.0}\right) + C$$

$$\Rightarrow 0 = \frac{y}{f}\ln\left(\frac{Q}{Q}\right) + C = \frac{y}{f}\ln(1) + C = \frac{y}{f} \times 0 + C \qquad \therefore C = 0$$

Finally, $\boxed{\therefore t = \frac{y}{f}\ln\left(\frac{Q}{Q-fA}\right) = 2.3 \times \frac{y}{f}\log\left(\frac{Q}{Q-fA}\right)}$

This above equation can be further written as

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

Now, let $\frac{t.f}{2.3.y}$ = x

$$\Rightarrow x = \log\left(\frac{Q}{Q - fA}\right)$$

$$\Rightarrow 10^x = \left(\frac{Q}{Q - fA}\right) \Rightarrow Q.10^x - f.A.10^x = Q$$

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

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

Further, considering the maximum value of $\frac{10^x - 1}{10^x}$ = 1

We, get $\boxed{A_{max} = \frac{Q}{f}}$

Problem

Determine the time required to irrigate a strip of land of 0.04 hectares in area from a tube-well with a discharge of 0.02 cumec. The infiltration capacity of the soil may be taken as 5 cm/h and the average depth of flow on the field as 10 cm. Also determine the maximum area that can be irrigated from this tube well.

Solution:

Time required for irrigating the strip of land,

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

$$= 2.3 \times \frac{0.10}{0.05} \times \log\left(\frac{72}{72 - 0.05 \times 400}\right)$$

= 0.65 hrs

= 39 minutes

Here,

A = 0.04 hectares

= $0.04 \times 10^4\ m^2 = 400\ m^2$

Q = 0.02 cumec = $0.02\ m^3/s$

= $0.02 \times 60 \times 60\ m^3/hr$

= $72\ m^3/hr$

f = 5 cm/hr = 0.05 m/hr

y = 10 cm = 0.10 m

<u>Maximum area</u> that can be irrigated is given by the equation:

$$A_{max} = 72/0.05\ m^2$$

$$= 1440\ m^2$$

$$= 1440/10^4 \text{ hectares}$$

$$= 0.144 \text{ hectares}$$

LECTURE 4

Cheek Flooding

- Similar to Ordinary flooding
- Water is controlled by surrounding the ***check*** area with low and flat levees
- The check is filled with water at a fairly high rate and allowed to stand until the water infiltrates
- The confined plot area varies from 0.2 to 0.8 hectares

Contd….. Check Flooding

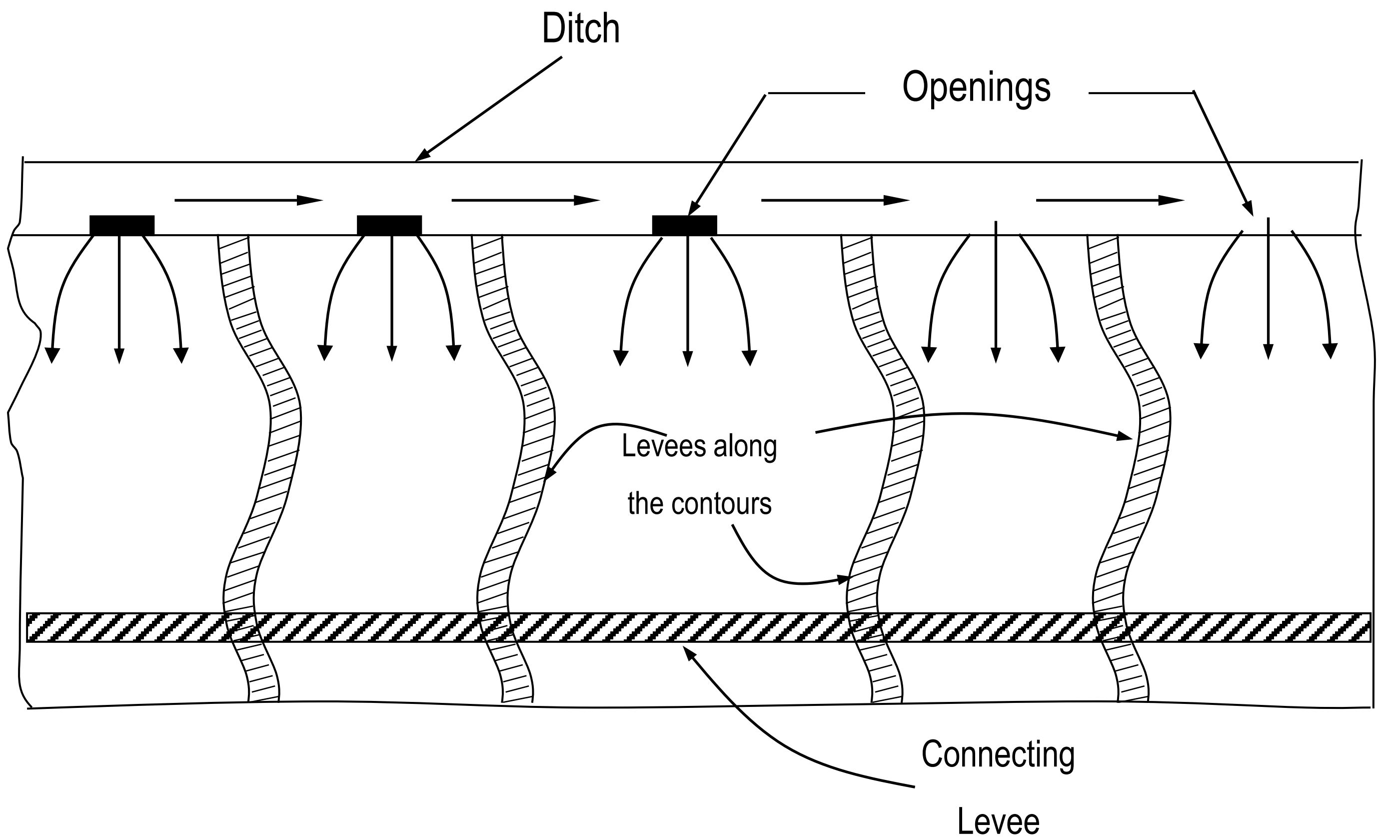


Fig: Check Flooding (plan view)

Basin Flooding

- Special type of check flooding
- Adopted specially for “Orchard trees”
- One or more trees are generally placed in the basin
- Surface is flooded as in check method by ditch water

Contd…….. Basin Flooding

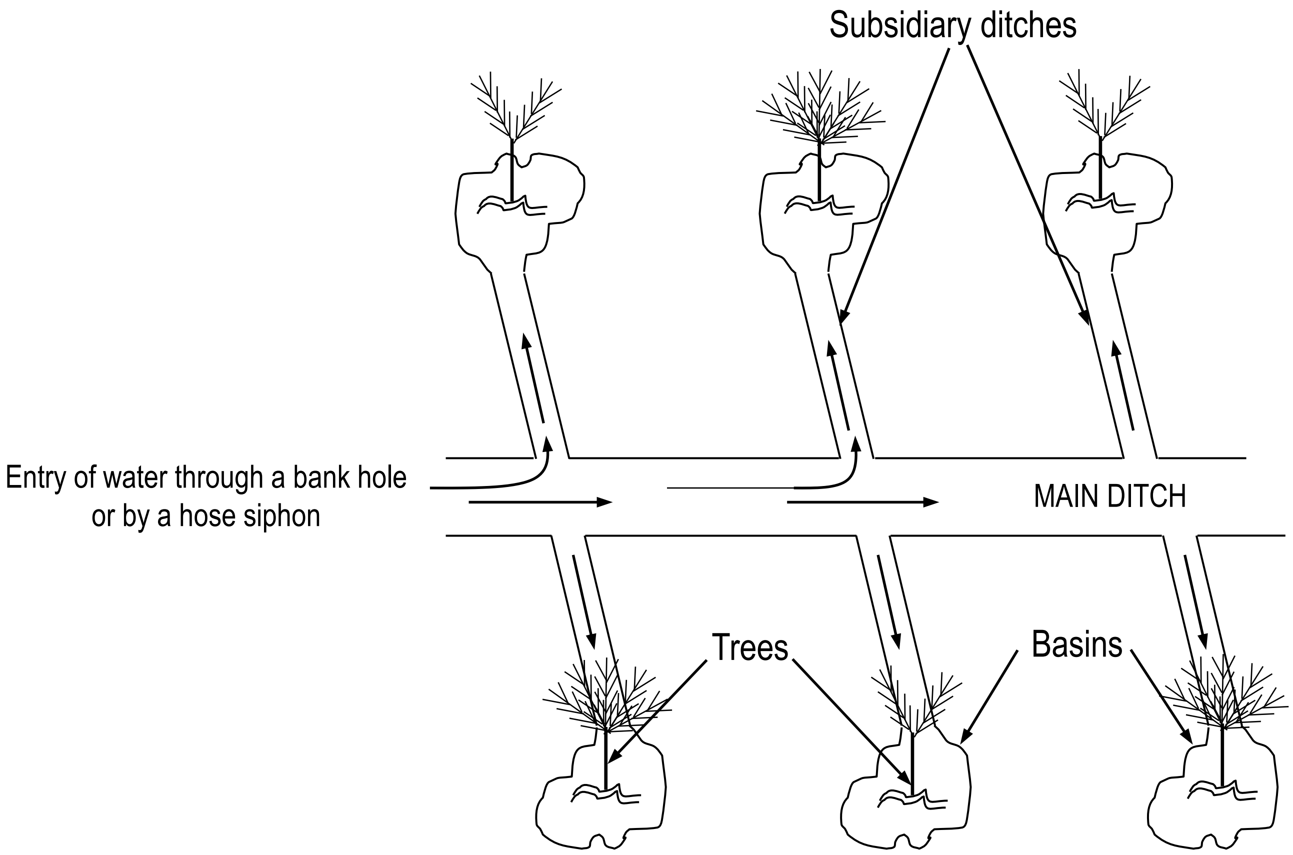
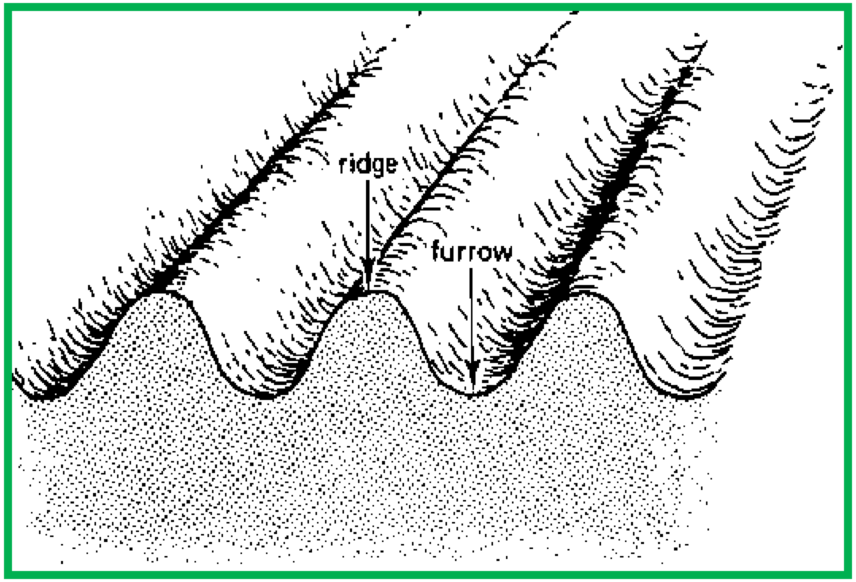


Fig: Basin Flooding (plan view)

Furrow Irrigation Method

Contd…….. Furrow irrigation method



Sprinkler Irrigation Method

Lecture 4

Contd…….. Sprinkler irrigation method

Drip Irrigation Method

Lecture 4

Irrigation Project Surveying

- Availability of Irrigation Water
- Selection of probable site for Barrage or Dam
- Discharge observation for the river
- Marking of GCA and Cultivable area
- Marking alignment of main canal
- Preliminary location survey
- Final Survey: Final location of Barrage or Dam, Route survey, Longitudinal leveling, Cross-sectional leveling, Data for cross drainage works, soil survey and well observation
- Preparation of drawings
- Office works
- Justification of the selection of final alignment
- Final location survey

Irrigation Project Report

- Introduction: Aim of the project, location, Area, population to be benefited, cost of the project etc.
- Necessity and economic justification
- Report on land acquisition and compensation
- Details of design and drawing of hydraulic structures
- Detailed estimate
- Specification
- Availability of materials and laborers
- Communication
- Maps
- Conclusion and recommendation

End of Chapter – 1

CHAPTER – 2

Sources and Quality of Irrigation Water

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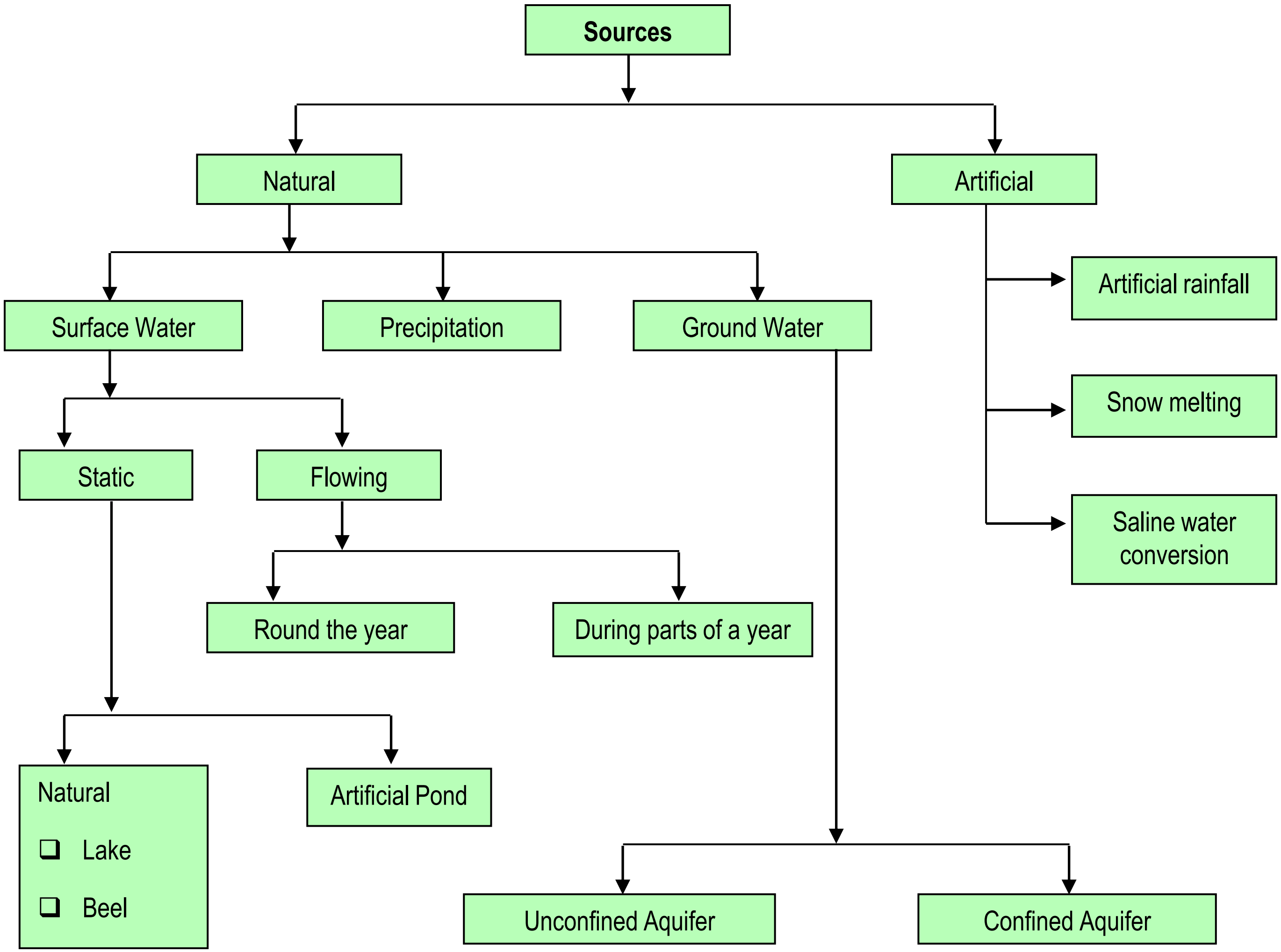
LECTURE 5

Sources of Irrigation water



Lecture 5

Consideration for surface water as a source

- Lowest available water
- Crop water requirement

  Water needed (S_1) = $R_1 - x_1$

  Where, R_1 = Crop water requirement

  x_1 = Effective rainfall
- Water quality (salinity & toxicity)
- Water right – Other users
  - Domestic water supply
  - Navigation
  - Fish culture
  - Industry
  - River morphology
- Control structure
  - Initial cost
  - Operating & maintenance cost

Consideration for ground water as a source

- Crop water requirement
- Availability of surface water source
- Position of ground water table
- Water quality
- Ground water recharge
- Environmental impact

Conjunctive use of ground water & surface water

- Use of both Surface Water (SW) & Ground Water (GW)
- Such a way that Ground Water recharges & draft balances with each other.
- Factors governing the percentage of sharing SW & GW
    - Natural recharge
    - Artificial recharge
    - Aquifer characteristics
    - Availability of surface water
    - Availability of fuel
    - Operation & maintenance cost for pumps
    - Economic consideration

Storage of surface water or rainfall

- Availability of space
- Series of droughts can be easily overcome by GW storage
- Water table control easier
- Ground water is recharged

Quality of Irrigation water

Good quality water is essential for high production

Physical	: Color, odor, silt
Chemical	: Salt, alkaline
Biological	: Coliform

Water quality related problems

- Salinity
- Water infiltration rate
- Toxicity
- Miscellaneous

Lecture 5

Various impurities in irrigation water

Every water may not be suitable for plant life. The quality of suitable irrigation water is very much influenced by the constituents of the soil which is to be irrigated.

The various types of impurities, which make the water unfit for irrigation, are classified as:

1) Sediment concentration in water.
2) Total concentration of soluble salts in water.
3) Concentration of sodium ions to other cations.
4) Concentration of potentially toxic elements present in water.
5) Bicarbonate concentration as related to concentration of Ca plus Mg.
6) Bacterial concentration

(1) Sediment concentration in water

- Organic soils, the fertility is improved.
- Eroded areas sediment; it may reduce the fertility or decrease the soil permeability.
- Sediment water increases the siltation and maintenance costs.

(2) Total concentration of soluble salts

Salinity concentration of the soil solution (C_s)

$$C_s = \frac{CQ}{\left[Q - (C_u - P_{eff})\right]}$$

Where,

Q = Quantity of water applied

C_u = Consumptive use of water, i.e, the total amount of water used by the plant for its growth.

P_{eff} = Useful rainfall

$C_u - P_{eff}$ = Used up irrigation water

C = Concentration of salt in irrigation water

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

Total salt concentration in water is expressed as:

- Parts per million parts of water (ppm)
- Milligram per liter of water (mg/l)
- Electrical Conductivity of water (EC)

(a) **Parts per million (ppm)**

The result of a chemical analysis of water are usually report in parts per million of the various substances present in the sample. One part per million (ppm) means one part in a million parts. As commonly measured and used, parts per million in numerically equivalent to milligrams per liter.

Amount in excess of 700 ppm	Harmful to some plants
More than 2000 ppm	Injuries to all crops

(b) **<u>Milligram per liter (mg/l)</u>**

(c) Electrical Conductivity (EC)

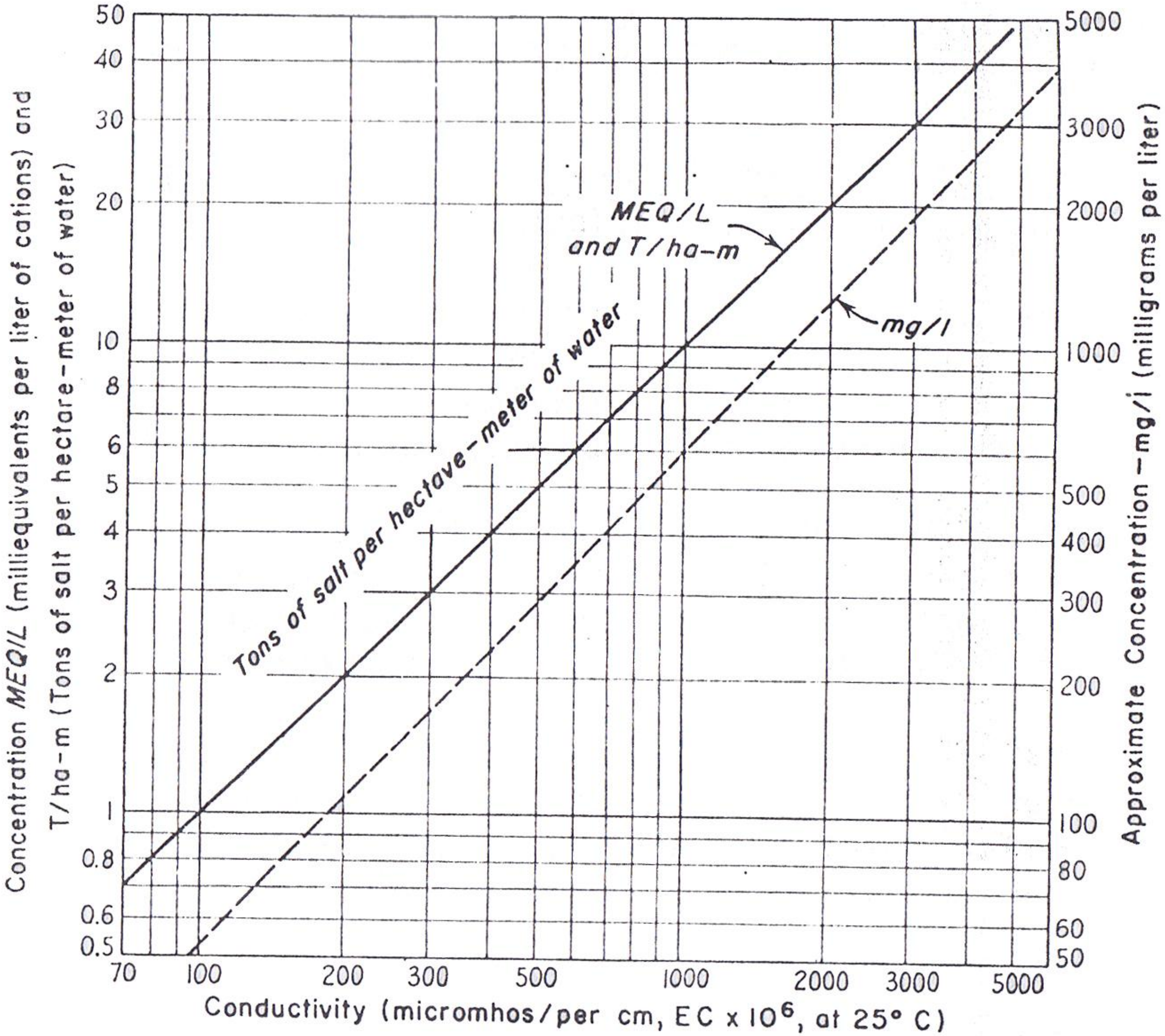
It is the reciprocal of the Electrical resistivity. Quantitively the electrical resistivity is the resistance, in ohms, of a conductor, metallic or electrolytis, which is 1 cm long and has a cross-sectional area of 1 cm^2 at 25^{o}C.

Units:

EC = Reciprocal ohms/cm or mhos/cm.
millimhos/cm (10^{-3} mhos/cm)
micromhos/cm (10^{-6} mhos/cm)

SL	Electrical Conductivity (micro mhos/cm at 25^o C)	Type of water
1	Up to 250	Low Conductivity Water (C1)
2	250 to 750	Medium Conductivity Water (C2)
3	750 to 2250	High Conductivity Water (C3)
4	Above 2250	Very High Conductivity Water (C4)

Cont…..Electrical Conductivity (EC)



(3) Relative proportions of sodium ions

Most of the soils contain Ca^{++} and Mg^{++} ions and small quantities of Na^{+}. The percentage of the Na^{+} is generally less than 5% of the total exchangeable cations. If this percentage increases to about 10% to more, the aggregation of soil grains breaks down. The soil becomes less permeable and of poorer tilth. It starts crusting when dry and its pH increases towards that of an alkaline soil. High sodium soils are, therefore, plastic, sticky when wet, and are prone to form clods, and they crust on drying.

The **methods for determining relative proportion of sodium ions to other cations** are:

a) Sodium absorption ratio (SAR)
b) Exchangeable sodium percentage (ESP)
c) Sodium percentage (SP)

(a) Sodium Adsorption Ration (SAR)

A ratio for soil extracts and irrigation water used to express the relative activity of sodium ions in exchange reaction with soil in which the ionic concentration are expressed in milli-equivalents per liter..

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

SL	SAR	Type of water
1	0 to 10	Low Sodium Water (S1)
2	10 to 18	Medium Sodium Water (S2)
3	18 to 26	High Sodium Water (S3)
4	More than 26	Very High Sodium Water (S4)

(a) <u>Sodium Absorption Ration (SAR)</u>

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(b) Exchangeable Sodium Percentage (ESP)

It is the degree of saturation of the soil exchange complex with sodium and may be calculated by the following formula:

$$ESP = \frac{\text{Exchangeable Sodium (milli-equivalent/100 gm soil)}}{\text{Cation exchange capacity (milli-equivalent/100 gm soil)}}$$

Where, ionic exchange is in milliequivalent per 100 gm soil

Cation Exchange:

Interchange of a cation in solution with another cation on a surface active material.

Cation Exchange Capacity (CEC):

The total quantity of cations which a soil can absorb by cation exchange usually expressed as milli-equivalence per 100 grams. Measured value of the cation exchange capacity depends somewhat on the method used for its determination.

Exchangeable Cation:

A cation that is adsorbed on the exchange complex and which is capable of being exchange with other cations.

Equivalent Weight:

It is the combining capacity of an element or radical with hydrogen. It is the weight in grams of an ion or compound that combine with or replace 1 gm of hydrogen.

Equivalent weight = Atomic weight/valency

For example: Equivalent weight of Na^+ = 23/1 = 23

Equivalent weight of Cu^{++} = 40/2 = 20

Equivalent weight of Cl^- = 35.5/1 = 35.5

Note:

Milliequivalent weight = Equivalent weight/1000

Milli-equivalent (meq): One thousand of an equivalent.

Milli-equivalent per liter (meq/litre): A milli-equivalent of an ion or a compound in 1 liter of solution.

(c) Sodium Percentage (SP)

The moisture percentage of a saturated soil paste expressed on dry wt. basis.

$$SP = \frac{Na^+}{(Na^+ + Ca^{++} + Mg^{++} + K^+)} \times 100$$

Where, ionic concentration is in (me/l)

LECTURE 6

(4) Concentration of potentially toxic elements

- A large number of elements such as B, Se etc. may be toxic to plants.
- B is essential to plant growth
- Even for the most tolerant crops, the [B] $\geq$ 4 ppm.
- B is generally present in various soaps.
- Se even in low concentration, toxic, and must be avoided.

Table: Relative tolerance of some crop to Boron

SL	High	Medium	Low
1	Sugar beat	Cotton	Black walnut
2	Gladiolus	Radish	Nary beam
3	Onion	Field peas	Pear
4	Carrot	Barley	Apple

(5) HCO_3^- concentration as related to concentration of Ca plus Mg

Residual Sodium Carbonate (RSC)

Indicate the residual carbonates in excess of the lime elements.

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

Where, ionic concentrations are in (me/l)

Table: Quality Rating of Ground Water

Water quality		EC (dS/m)	SAR $(mmole/l)^{1/2}$	RSC (me/l)
Good		< 2	< 10	< 2.5
Saline				
(i)	Marginally Saline	2 ~ 4	< 10	< 2.5
(ii)	Saline	4	< 10	< 2.5
(iii)	High SAR saline	4	> 10	< 2.5
Alkaline Water				
(i)	Marginally Alkali	< 4	< 10	2.5 – 4.0
(ii)	Alkali	< 4	< 10	4.0
(iii)	Highly Alkali	Variable	10	4.0

(6) Bacterial contamination

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

- Good Water, EC < 2, and SAR < 10
- Saline Water, EC = 2, and SAR < 10
- High SAR saline water, EC = 4, and SAR > 10
- Alkali water, EC variable, SAR variable, RSC > 2.5

Precautions in saline water use

Use of saline water in irrigation creates many problems. When situation demands its use, the following points should be borne in mind:

- Water should be applied in excess amount than required to meet the water deficit in the crop root zone to leach down the surplus salts.
- Excess salt should be leached down by abundant irrigation particularly before sowing.
- Soil should be lighter in texture, porous and permeable so that the leaching operation is easy. Clay soils do not allow easy leaching and are likely to become saline at a faster rate.
- Irrigation should be frequently applied to avoid shortage of available water to plants and a sudden variation in salt concentration of the salt solution.

- Water table should be lowered to a depth from which there is no reaching of water and salts in the root zone. Low water table encourage a good drainage of the soil.
- Land should be properly graded and leveled as greater salt accumulation occurs in higher part of an uneven field.
- Drainage of the field must be properly maintained to prevent water logging.
- Soil should be maintained in good physical condition with addition of organic matter and by proper tillage.
- Liming of soil may be undertaken if sodium content of irrigation water is likely to cause injury to soil or crops.
- All corrective measures should be undertaken to keep sodium ion concentration in soil as low as possible. Sodium ion concentration should not exceed 12 percent of the total cation exchange capacity or the soil exchange complex.

The foregoing measures would make the use of saline water much safer for irrigation and ensure better crop growth and yield.

Guidelines for using poor quality water

Special consideration:

- Use of gypsum when saline water (having SAR > 20 and/or Mg/Ca ratio > 3 and rich in silica) induce water stagnation during rainy season and crops grown are sensitive to it.
- Leaving the field fallow during the rainy season is helpful when SAR > 20 and water of higher salinity and used in lower rainfall areas.
- Additional phosphorus fertilization is beneficial especially when Cl/SO_4 ratio in water is greater than 20.
- Canal water preferably be used growth stage including pre-sowing irrigation for conjunctive use with saline water.
- If saline water is to be used for seedling of crops 20% extra seed rate and quick post-sowing irrigation (within 2 – 3 days) will ensure better germination

Cont………Guidelines for using poor quality water

- When $EC_{iw} < EC_e$ (0 – 45 cm soil at harvest of rab crops) saline water irrigation just before the onset of monsoon will lower soil salinity and will raise the antecedent soil moisture for greater salt removal by rains.
- Use organic materials in saline environment enhance yields.
- Accumulation of B, NO_3, Fe, Si, F, Se and heavy metals beyond critical limits proves toxic. Expert advice prior to the use of such water may be obtained.
- For soils having (i) shallow water table (within 1.5 m in kharif season) and (ii) hard sub-soil layers, the next lower EC_{iw}/alternative mode of irrigation (canal/saline) is applicable.

Leaching Requirement

Leaching is the process of dissolving the soluble salts and removing the same from the desired soil layers by the downward movement of water.

- ❑ A quantity more than the normal requirements of the crops to avoid accumulation of salts.
- ❑ It is done by ponding water on the soil surface by bunds or borders and allowing a downward monument of water through the soil column
- ❑ The efficiency of leaching depends on **the amount of water applied**, the **uniformity of water distribution** and the **adequacy of drainage** in the field.
- ❑ Fertilizers should be applied only after leaching is completed.

Leaching Requirement may expressed as:

$$LR = D_d/D_{iw} = EC_{iw}/EC_d \quad \text{-------------------} \quad (1)$$

Where,

LR = Leaching requirement, expressed as a ratio or as percent

EC_{iw} = Electrical conductivity of irrigation water, mSiemens/cm

EC_d = Electrical conductivity of drainage water, mSiemens/cm

D_d = Depth of drainage water, cm

D_{iw} = Depth of irrigation water, cm

The leaching requirement is the additional water required to the normal consumptive use of water by crops. Therefore,

$$D_{iw} = D_c + D_d \text{ ------------------------- } (2)$$

Where,

D_c = Consumptive use of water

Using equation (3) to estimate D_d from equation (4)

$$D_{iw} = D_c/(1 - LR) \text{ --------------------- } (3)$$

Again, expressing the leaching requirement in equation (2) as EC ratio of irrigation and drainage waters, equation (3) stands as:

$$D_{iw} = [EC_d/(EC_d - EC_{iw})] \times D_e \text{ --------------- } (4)$$

Another equation for determining leaching requirements in soil

$$LR = \frac{EC_w}{5EC_e - EC_w}$$

Relation between Leaching Requirement (LR), Available Water (AW) and Evapotranspiration (ET) is

$$AW = \frac{ET}{1 - LR}$$

Leaching Requirement of saline soil

$D_{iw}/D_s = d_s/d_w \times SP/100 \times \Delta EC_e/EC_{iw}$ ------------------------- (5)

Where,

D_{iw} = Depth of irrigation water, cm
D_s = Depth of soil, cm
d_s = Density of soil (bulk density), gm/cm^3
d_w = density of irrigation water, gm/cm^3
ΔEC_e = Change in electrical conductivity of saturation extract of the soil
EC_{iw} = Electrical conductivity of irrigation water, μS/cm

Under high water table conditions, evaporation brings up the soluble salts and deposits the salts in upper layers of soil increasing the salinity. The change in salinity of the soil may be determined by the following equation:

$\Delta EC_e = D_g/D_s \times EC_g/SP \times d_w/d_s \times 100$ ------------------------- (6)

Where,

D_g = Depth of ground water evaporated, cm
EC_g = Electrical conductivity of ground water, mS/cm

Leaching Method

- Leaching of soil is done by ponding water on the soil surface by bunds or borders and allowing a downward movement of water through the soil column.
- Rectangular checks and level borders are employed when the soil is level.
- Contour checks can be used when the land slope is more.
- Sprinkler irrigation is usefully employed to leach out salts especially when the soils are cracked and very permeable.
- Intermittent ponding of water is superior to continuous ponding of water for effective leaching.
- The efficiency of leaching depends on the amount of water applied, the uniformity of water distribution and the adequacy of drainage in the field.

- The sensitive crops or the crops with low salt tolerance have higher leaching requirement and require frequent leaching during a growing season. Leaching of salts once or twice in a growing season is enough for salt tolerant crops. It is usually needed to apply little more water than actually required by crops in areas where salinity is a problem. Occasional analysis of soil is required where irrigation water contains salts.

In areas where leaching is practiced for growing crops, fertilizers should be applied only after leaching is completed and in little higher amounts to make up the loss of nutrients during leaching of salts.

The nitrogenous fertilizers are highly soluble and are prone to leaching.

In areas where salinity is a problem and leaching of salt is essential for crop growing, the drainage of land should be good. Usually, a high water table and the soil salinity occur simultaneously.

Planning of irrigation development should also consider the development of drainage in particular region. If the ground water is of good quality, the water the high water table can be pumped out and used for irrigation in the area or in the nearby areas.

Some important equations

- Salt concentration, mg/l or ppm = 640×EC, mmhos/cm
- Total cation concentration, me/l = 10×EC, mmhos/cm
  When EC is measured up to the range of 5 mmhos/cm at 25^{o} C
- Osmotic pressure, atmospheres = 0.36×EC mmhos/cm
- Parts per million (ppm)/equivalent weight = me/l
- Equivalent weight = Atomic weight/valency

Problem – 1

a) What is the classification of irrigation water having the following characteristics: Concentration of Na, Ca and Mg are 22, 3 and 1.5 milli-equivalents per liter respectively, and the electrical conductivity is 200 μmhos/cm at 25^o C?

b) What problems might arise in using this water on fine textured soils?

c) What remedies do you suggest to overcome this trouble?

Solution:

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

- If SAR value is between 10 to 18, then it is classified as Medium Sodium Water and is represented by S2 (**See table**)
- If the value of Electrical Conductivity is between 100 to μS/cm at 25^o C, the water is called of Low Conductivity (C1) (**See table**)

$\therefore$ Given water is classified as **C1-S2 water** (**ans**)

(b) In fine-textured soils, the medium sodium (S2) water may create the following problems:

- Soil becomes less permeable.
- It starts crusting when dry.
- It becomes plastic and sticky when wet.
- Its pH increases towards that of alkaline soil.

(c) Gypsum ($CaSO_4$) addition, either to soil or to water is suggested to overcome sodium hazards posed by the given water.

Problem – 2

Express 8300 ppm of sodium salt concentration in mmhos/cm, μmhos/cm and mhos/cm

Solution:

We know, Salt concentration in ppm or mg/l of water = 640 × EC in mmhos/cm

$$\therefore EC = \text{ppm salt concentration}/640$$
$$= 8300/640$$
$$= 12.97 \text{ mmhos/cm}$$
$$= 12.97 \times 1000 = 12970\ \mu\text{mhos/cm}$$
$$= 12.97/1000 = 0.012 \text{ mhos/cm}$$

End of Chapter – 2

CHAPTER – 3

Consumptive Use and Estimation of water requirements of crops

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LECTURE 7

Definition of Consumptive use

Consumptive use (CU), or Evapotranspiration (ET), is the sum of two terms:

(a) Transpiration:

Water entering plant roots and used to build plant tissue or being passed through leaves of the plant into the atmosphere

(b) Evaporation:

Water evaporating from adjacent soil, water surfaces, and surfaces of leaves of the plant or intercepted precipitation

Factors affecting CU or ET

(a) Evaporation affected by:

- The degree of saturation of soil surface
- Temperature of air and soil
- Humidity
- Wind velocity
- Extent of vegetative cover etc.

(b) Transpiration affected by:

Climate factors:

- Temperature
- Humidity
- Wind speed
- Duration & intensity of light
- Atmospheric vapor pressure

Soil factors:

- Texture
- Structure
- Moisture content
- Hydraulic conductivity

Plant factors:

- Efficiency of root systems in moisture absorption
- The leaf are
- Leaf arrangement and structure
- Stomatal behavior

Direct Measurement of CU/ET

(a) Tank or Lysimeter experiments:

Lysimeter experiments involve the growing of crops in large containers (lysimeters) and measuring their water and grains.

Limitations:

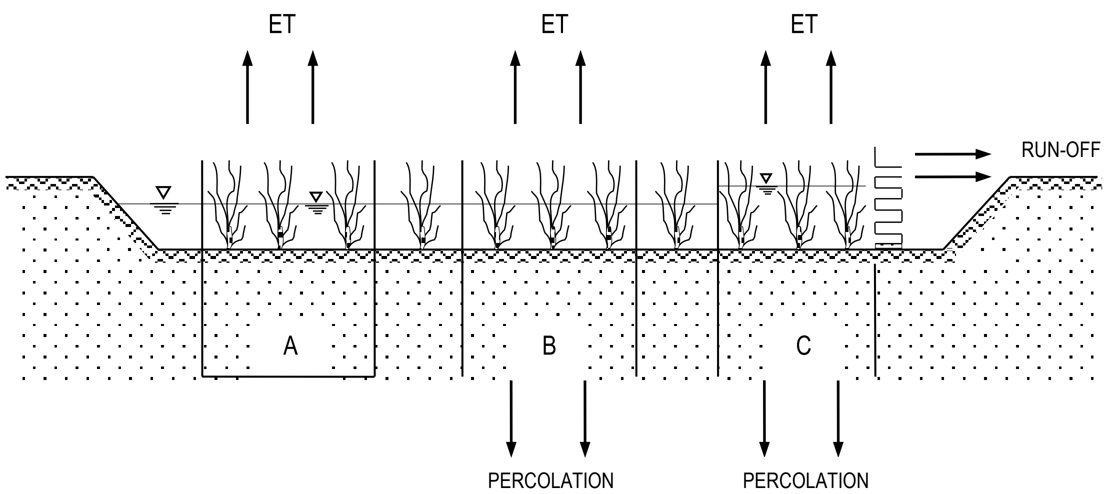
Reproduction of physical conditions such as temperature, water table, soil texture, density etc.

Types of Lysimeters

There are types of lysimeters:

- Non-weighing constant water table type:
- Non-weighing percolation type:
- Weighing type:

Non-weighing constant water table type

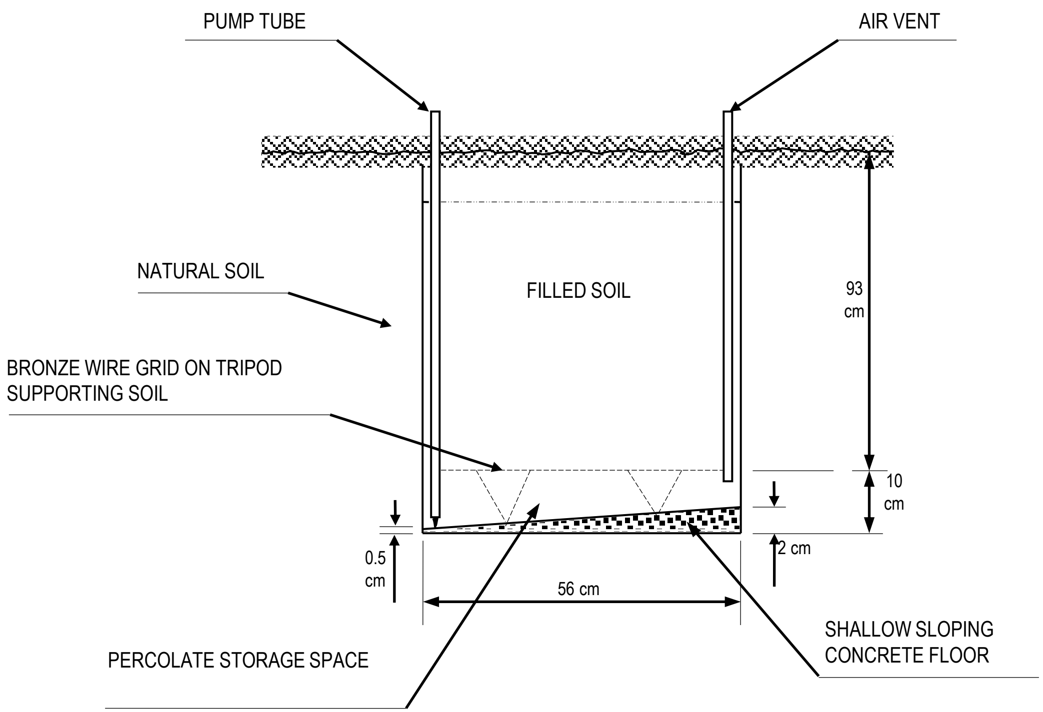


- Constant water level is maintained by applying water
- Effective rainfall (R_e) and irrigation (I) are measured by rain-gauges and calibrated container
- The overflow (R) and deep percolation (D_r), if any, are measured.

$$ET = I + R_e - R - D_r$$

- R_e, R, D_r, may be zero depending on site condition
- This method is applicable where high water table in soil exists

Non-weighing percolation type



Consumptive Use (CU) is computed by adding measured quantities of irrigation water, the effective rainfall received during the season and the contribution of moisture from the soil

Continue…….. Non-weighing percolation type

$$ET = I + R_e - D_r + \sum_{i=1}^{n}\left[\frac{M_{bi} - M_{ei}}{100}\right] \times A_i D_i$$

Where,

ET = Evapotranspiration

I = Total irrigation water applied (mm)

R_e = Effective rainfall (mm)

M_{bi} = Moisture content at the beginning of the season in the i^{th} layer of the soil

M_{ei} = Moisture content at the end of the season in the i^{th} layer of the soil

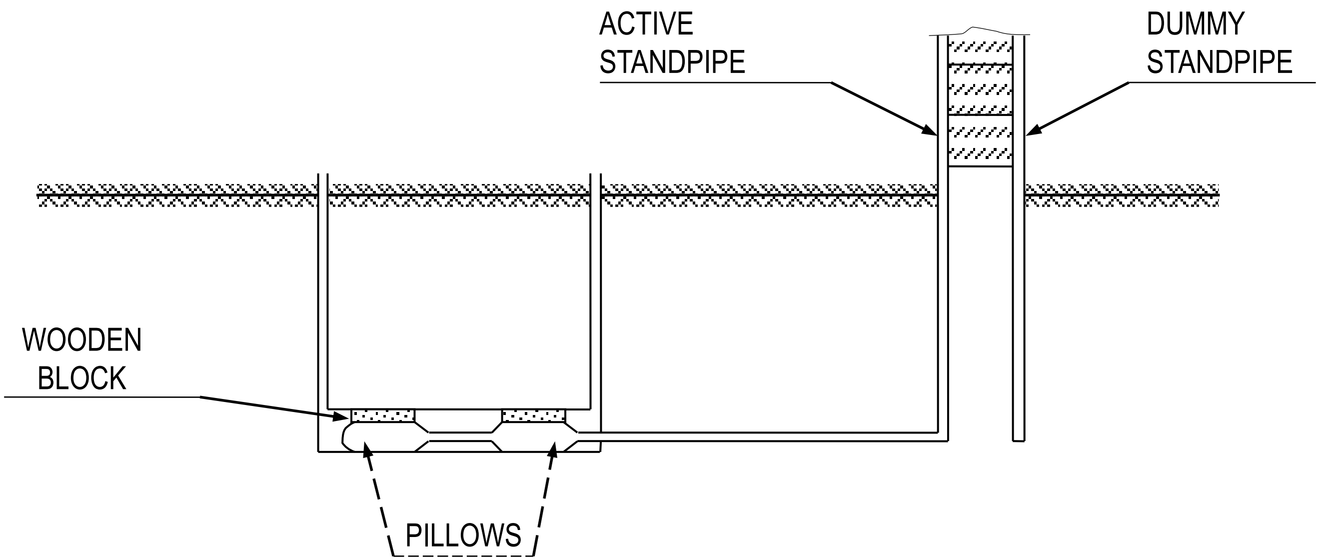
A_i = Apparent specific gravity of the i^{th} layer of soil

D_i = Depth of the i^{th} layer of the soil with root zone (mm)

n = No. of soil layers in the root zone

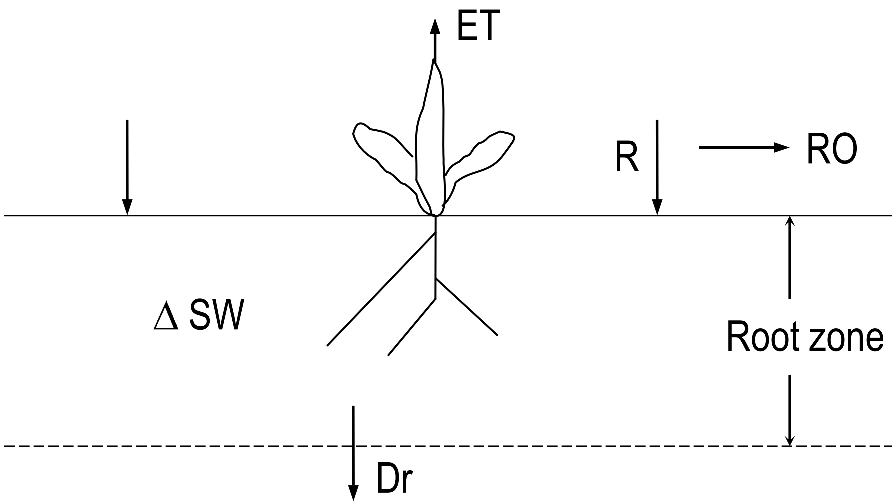
- Applicable for areas having high precipitation
- Special arrangements are made to drain and measure the water percolating through the soil mass

Weighing type



- ET is determined by taking the weight of the tank and making adjustment for any rain
- Provides the most accurate data for short time periods

Soil moisture depletion studies



- The soil is sampled 2 to 4 days after irrigation and again 7 to 15 days later or just before the next irrigation
- Only those sampling periods are considered in which rainfall is light. This is done to minimize drainage and percolation errors
- The depth to ground water should be such that it will not influence the soil moisture fluctuation within the root zone.
- It cannot be applied where water table is high

Continue……. Soil moisture depletion studies

$ET = I + R - R_o - D_r + \Delta SW$

Where, $\Delta SW = \sum_{i=1}^{n} \left[\frac{M_{1i} - M_{2i}}{100} \right] \times A_i D_i$

M_{1i} = moisture content at the time of 1st sampling in the ith layer

M_{2i} = moisture content at the time of 2nd sampling in the ith layer

Some important definitions

Effective rainfall:

Precipitation falling during the growing period of a crop that is available to meet the evapotranspiration needs of the crop is called effective rainfall. It is that part of rainfall which is available to meet ET needs of the crop

$$R_e = R - R_r - D_r$$

Where, R = Precipitation

R_r = Surface runoff

D_r = Deep percolation

Factors affecting R_e:

- Rainfall characteristics (intensity, frequency and duration)
- Land slope
- Soil characteristics
- Ground water level
- Crop characteristics (ET rate, root depth, stage of growth, ground cover)
- Land management practices (bunding, terracing, mulching reduce runoff and increase R_e)
- Carryover of soil moisture (from previous season)
- Surface and sub-surface in and out flows
- Deep percolation etc.

Generally a percentage of total rainfall is taken as effective rainfall

Consumptive Irrigation Requirement (CIR):

Irrigation water required in order to meet the evapo-transpiration needs of the crop during its full growth.

$$CIR = (C_u) - (R_e)$$

Net Irrigation Requirement (NIR):

It is the amount of irrigation water required in order to meet the evapo-transpiration need of the crop as well as the other needs.

$NIR = (C_u) - (R_e)$ + Water lost as percolation in satisfying other needs such as leaching.

Field Irrigation Requirement (FIR):

It is the amount of water required to be applied to the field

FIR = NIR + water application losses

$= NIR/E_a$ Where, E_a = Water application efficiency

Gross Irrigation Requirement (GIR):

It is the amount of water required at the head of a canal

GIR = FIR + conveyance loss

$= FIR/E_c$ Where, E_c = Conveyance efficiency

Estimation of ET using empirical equation

(a) Blaney – Criddle Equation:

$$C_u = (k.p)/40\ [1.8t + 32]$$

C_u = Monthly consumptive use in cm.

k = Crop factor, determined by experiments

t = Mean monthly temperature in ^{0}C

p = Monthly percent of annual day light hours that occur during the period

If (p/40)[1.8t + 32] is represented by f, we get

$$C_u = k.\ f$$

Problem

Wheat has to be grown at a certain place, the useful climatological conditions of which are tabulated below. Determine the evapo-transpiration and consumptive irrigation requirement of wheat crop. Also determine the field irrigation requirement if the water application efficiency is 80%. Use Blaney-Criddle equation and a crop factor is 0.8.

Month	Monthly temperature (^oC) averaged over the last 5 years	Monthly percent of day time hour of the year computed from the Sun-shine	Useful rainfall in cm averaged over the last 5 years
Nove	18.0	7.20	1.7
Dec	15.0	7.15	1.42
Jan	13.5	7.30	3.01
Feb	14.5	7.10	2.75

Solution:

Blaney – Criddle Equation is $C_u = k \frac{P}{40} [1.8\, t + 32]$

$$= k.\, \Sigma f$$

Month	**t (°C)**	**p (hr)**	**R_e (cm)**	**f = P/40(1.8t + 32) (cm)**
Nov	18.0	7.20	1.7	11.6
Dec	15.0	7.15	1.42	10.5
Jan	13.5	7.30	3.01	10.3
Feb	14.5	7.10	2.75	10.3
			$\Sigma R_e = 8.38$	$\Sigma f = 42.7$

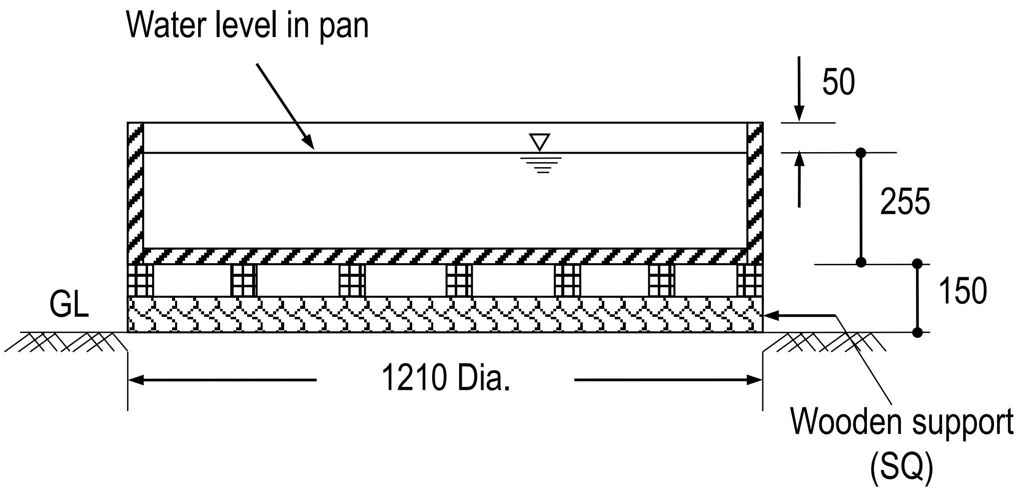
$C_u = k.\, \Sigma f = 0.8 \times 42.7 = 34.16$ cm

Hence, Consumptive use, $C_u = 34.16$ cm

Consumptive irrigation requirement, C.I.R $= C_u - R_e = 34.16 - 8.38 = 25.78$ cm

Field irrigation requirement, F.I.R $=$ C.I.R/ $\eta_a = 25.78/0.8 = 32.225$ cm

(b) Hargreaves class A pan evaporation method:



- The quantity of water (E_p) evaporated from the standard class A evaporation pan is measured.
- The pan is 1.2 m in diameter, 25 cm deep, and bottom is raised 15 cm above the ground surface.
- The depth of water is maintained such that the water surface is at least 5 cm, and never more than 7.5 cm, below the top of the pan.

Continue….. Hargreaves class A pan evaporation method:

- Evapotranspiration is related to pan evaporation by a constant k, called consumptive use co-efficient.

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

Or. $\mathbf{E_t}$ **or** $\mathbf{C_u = k \quad E_p}$

- Consumptive use co-efficient, k varies with crop type, crop growth etc. values of k are found from Table 2.9 (S. K Garg).

(b) FAO Penman-Monteith equation :

$$ET_o = \frac{0.408 \times \Delta \times (R_n - G) + \gamma \times \left(\frac{900}{T+273}\right) \times u_2 \times (e_s - e_a)}{\Delta + \lambda \times (1 + 0.34 \times u_2)}$$

Where,

ET_o = Reference crop (green grass) evapotranspiration (mm/day)

Δ = Slope of saturation vapor pressure vs temperature curve at mean air temperature, kPa per ^oC

R_n = Net radiation, MJ/m^2 per day, can be calculated from actual sunshine hour and other weather data

G = Soil heat flux, MJ/m^2 per day

γ = Psychometric constant, kPa per ^oC

T = Mean air temperature, ^oC

u_2 = Wind speed at 2 m height (m/s)

e_s = Saturation vapor pressure of the evaporating surface at mean air temperature, kPa

e_a = Actual vapor pressure, kPa

For monthly value, G = 0.14 $(T_i – T_{i-1})$ Where, Ti = Mean air temperature for the month (^oC)

T_{i-1} = Mean air temperature for the previous month (^oC)

G = 0 for 10 days or short period

LECTURE 8

Irrigation Efficiencies

- **Efficiency of water-conveyance (η_c)**: It is the ratio of the water delivered into the fields from the outlet point of the channel, to the water pumped into the channel at the starting point.
- **Efficiency of water application (η_a)**: It is the ratio of the quantity of water stored into the root zone of the crops to the quantity of water actually delivered into the field.
- **Efficiency of water-storage (η_s)**: It is the ratio of the water stored in the root zone during irrigation to the water needed in the root zone prior to irrigation.
- **Efficiency of water use (η_u)**: It is the ratio of the water beneficially used including leaching water, to the quantity of water delivered.
- **Uniformity coefficient or water distribution efficiency (η_d)**: The effectiveness of irrigation may also be measured by its water distribution efficiency, which is defined below:

$$\eta_d = (1 - d/D)$$

where,

η_d = Water distribution efficiency

D = Mean depth of water stored during irrigation

d = Average of the absolute values of deviations from the mean

Problem-1: The depths of penetrations along the length of a boarder strip at points 30 meters apart were measured. Their values are 2.0, 1.9, 1.8, 1.6 and 1.5 meters. Compute the distribution efficiency.

***Solution*:**

Mean depth, D = (2.0 + 1.9 + 1.8 + 1.6 + 1.5)/5 = 1.76 m

Values of deviations from the mean are (2.0 – 1.76), (1.9 – 1.76), (1.8 – 1.76), (1.6 – 1.76), (1.5 – 1.76) = 0.24, 0.14, 0.04, – 0.16, – 0.26

The absolute values of these deviations from the mean are 0.24, 0.14, 0.04, 0.16, and 0.26

The average of these absolute values of deviations from the mean,

d = (0.24 + 0.14 + 0.04 + 0.16 + 0.26)/5 = 0.168 m

$\therefore$ The water distribution efficiency, $\eta_d = \left(1 - \frac{d}{D}\right) = \left(1 - \frac{0.168}{1.76}\right)$

$= 0.905 \times 100 = 90.5\%$

Problem (Home Work)

A stream of 130 liters per second was diverted from a canal and 100 liters per second were delivered to the field. An area of 1.6 hectares was irrigated in 8 hours. The effective depth of root zone was 1.7 m. The runoff loss in the field was 420 m^3. The depth of water penetration varied linearly from 1.7 m at the head end of the field to 1.1 m at the tail end. Available moisture holding capacity of the soil is 20 cm per meter depth of soil. It is required to determine the

(a) water conveyance efficiency,

(b) water application efficiency,

(c) water storage efficiency and

(d) water distribution efficiency.

Irrigation was started at a moisture extraction level of 50% of the available moisture.

***Solution*:**

(a) Water conveyance efficiency (η_c)

$$\eta_c = \frac{\text{Water delivered to the fields}}{\text{Water supplied into the canal at the head}} \quad 100 = \frac{100}{130} \quad 100 = 77\%$$

(b) Water application efficiency (η_a)

$$\eta_a = \frac{\text{Water stored in the root zone during irrigation}}{\text{Water delivered to the field}} \quad 100$$

Water supplied to field during 8 hours @ 100 liters per second

= 100 8 60 60 liters = 2.88 10^6 liters

= 2.88 $10^6/10^3$ m^3 = 2880 m^3

Runoff loss in the field = 420 m^3

∴ The water stored in the root zone = 2880 – 420 m^3 = 2460 m^3

∴ Water application efficiency (η_a) = $\frac{2460}{2880}$ 100 = 85.4%

(c) Water storage efficiency (η_s)

$$\eta_s = \frac{\text{Water stored in the root zone during irrigation}}{\text{Water needed in the root zone prior to irrigation}} \quad 100$$

Moisture holding capacity of soil = 20 cm per m length × 1.7 m height of root zone
= 34 cm

Moisture already available in root zone at the time of start of irrigation = $\frac{50}{100} \times 34$ = 17 cm

Additional water required in root zone = 34 – 17 = 17 cm

Amount of water required in root zone = Depth × Plot area = $\frac{17}{100}$ × (1.6×10^4) m^3 = 2720 m^3

But actual water stored in root zone = 2460 m^3

$\therefore$ Water storage efficiency (η_s) = $\frac{2460}{2720}$ × 100 = 90% (say)

(d) Water distribution efficiency, η_d = $\left(1 - \frac{d}{D}\right)$

Mean depth of water stored in the root zone, D = (1.7 + 1.1)/2 = 1.4 m

Average of the absolute values of deviations from the mean, d = $\frac{|1.7 - 1.4| + |1.1 - 1.4|}{2}$

Average of the absolute values of deviations from the mean,

$$d = \frac{|1.7 - 1.4| + |1.1 - 1.4|}{2} = \frac{0.3 + 0.3}{2} = 0.3 \text{ m}$$

$\therefore$ Water distribution efficiency, $\eta_d = \left(1 - \frac{d}{D}\right)$

$$= \left(1 - \frac{0.30}{1.4}\right)$$

$$= 0.786 \quad 100 = 78.6\%$$

Irrigation Scheduling

Irrigation schedule is a decision making process involving:

- When to irrigate?
- How much water to apply each time?
- How to apply (method of irrigation)?

<u>Available Water (AW)</u>:

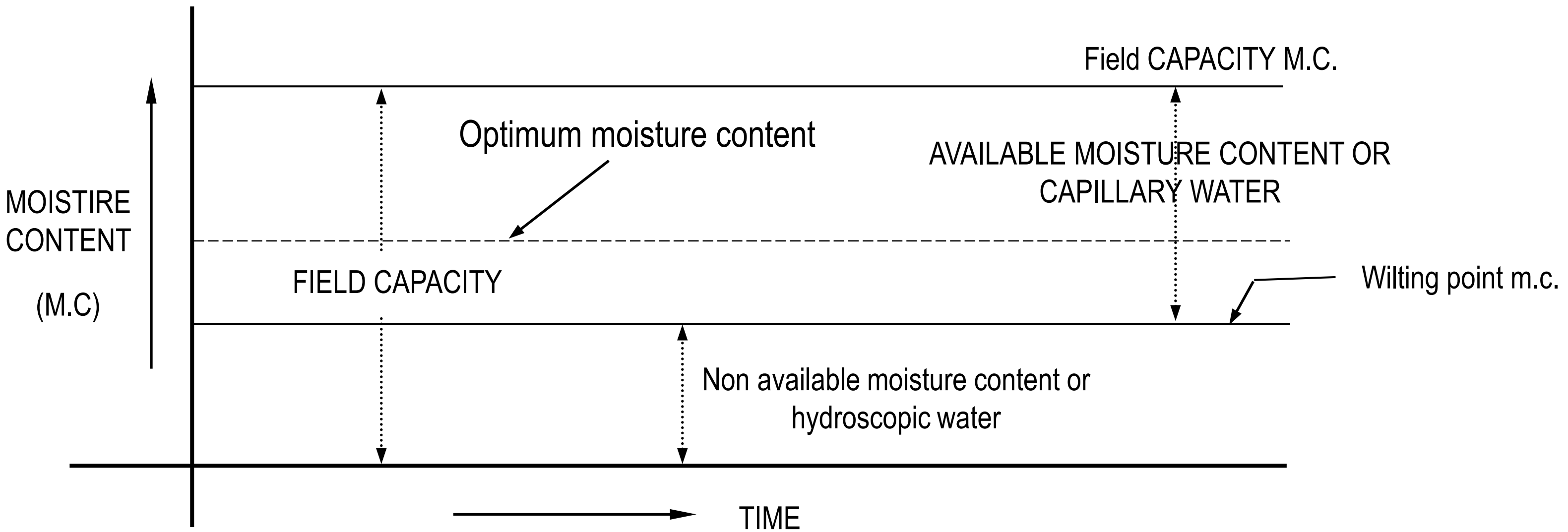
The water contained in the soil between FC and PWP is known as the available water. (**Fig. in the next slide)**

<u>Total Available Water (TAW)</u>:

The amount of water which will be available for plants in root zone is known as Total Available Water (TAW). It is the difference in volumetric moisture content at FC and that at PWP, multiplied by root zone depth. (**Fig. in the next slide)**

<u>Available Water (AW)</u>:

The water contained in the soil between FC and PWP is known as the available water.



Management Allowable Depletion (MAD):

MAD is the degree, to which water in the soil is allowed to be depleted by management decision and expressed as,

$$MAD = f \quad TAW \qquad \text{Where, } f = \text{Allowable depletion (\%)}$$

Reference crop Evapotranspiration (ET_o):

The rate of evapotranspiration from an extensive surface of 8 ~ 15 cm tall, green grass cover of uniform height, actively growing, completely shading the ground and not short of water is known as reference crop evapotranspiration (ET_o)

Crop Evapotranspiration (ET_c):

The depth of water need to meet the water loss through evapotranspiration of a disease free crop, growing in large fields under non-restricting soil conditions including water and fertility and achieving full production potential under the given growing environment.

Crop Co-efficient (k_c):

The ratio of crop evapotranspiration (ET_c) to the reference evapotranspiration (ET_o) is called Crop co-efficient (k_c).

$$\therefore k_c = ET_c/ET_o$$

Water Requirements of a Crop

- Arid Region and Semi Arid Region
- Water requirements of a crop means the total quantity and the way in which a crop requires water from the time it is sown to the time it is harvested. Water requirements depends on: **water table, crop, ground slope, intensity of irrigation, method of application of water, place, climate, type of soil, method of cultivation and useful rainfall**.

Crop Period or Base Period

- The time period that elapses from the instant of its sowing to the instant of its harvesting is called the **crop period**.
- The time between the first watering of a crop at the time of its sowing to its last watering before harvesting is called the **base period**.

Duty and Delta of a Crop

Delta: The total quantity of water required by the crop for its full growth may be expressed in hectare-meter or simply as depth to which water would stand on the irrigated area if the total quantity supplied were to stand above the surface without percolation or evaporation. This total depth of water is called delta (Δ).

Problem: If rice requires about 10 cm depth of water at an average interval of about 10 days, and the crop period for rice is 120 days, find out the delta for rice.

Solution: No. of watering required = 120/10 = 12

Total depth of water required in 120 days = 10 × 12 = 120 cm

Δ for rice = 120 cm

Duty: The area (in hectares) of land can be irrigated for a crop period, B (in days) using one cubic meter of water.

Relation Between Duty and Delta

Let there be a crop of base period B days.

Let one cumec of water be applied to this crop on the field for B days.

Now , the volume of water applied to this crop during B days

$= V = (1 \times 60 \times 60 \times 24 \times B)\ m^3$

$= 86\,400\ B$ (cubic meter)

This quantity of water (V) matures D hectares of land or 10^4 D sq. m of area

The depth of water applied on this land

$=$ Volume/Area $= 86{,}400\ B/10^4\ D$

$= 8.64\ B/D$ meters

By definition, this total depth of water is called delta (Δ)

$\Delta = 8.64\ B / D$ meters

$\Delta = 864\ B / D$ cm

Problem:

Find the delta for a crop when its duty is 864 hectares/cumec on the field, the base period of this crop is 120 days.

***Solution*:**

In this question, B = 120 days and D = 864 hectares/ cumec

$$\therefore \Delta = 864B/D = 864 \times 120/864 = 120 \text{ cm}$$

Importance of Duty

- It helps us in designing an efficient canal irrigation system. Knowing the total available water at the head of a main canal, and the overall duty for all the crops required to be irrigated in different seasons of the year, the area which can be irrigated can be worked out.

  Inversely, if we know the crops area required to be irrigated and their duties, we can work out the discharge required for designing the channel.

LECTURE 9

Crop Season & Cash Crop

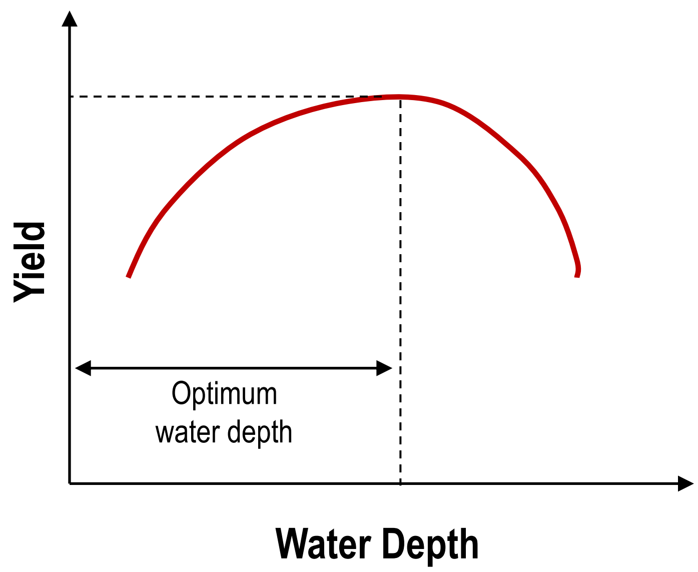
Crop Season

Rabi (October to March) and Kharif (April to September)

Cash Crop

A cash crop may be defined as a crop which has to be en-cashed in the market for processing as it cannot be consumed directly by the cultivators. All non food crops are thus included in cash crops. Examples: Jute, Tea, Cotton, Tobacco etc.

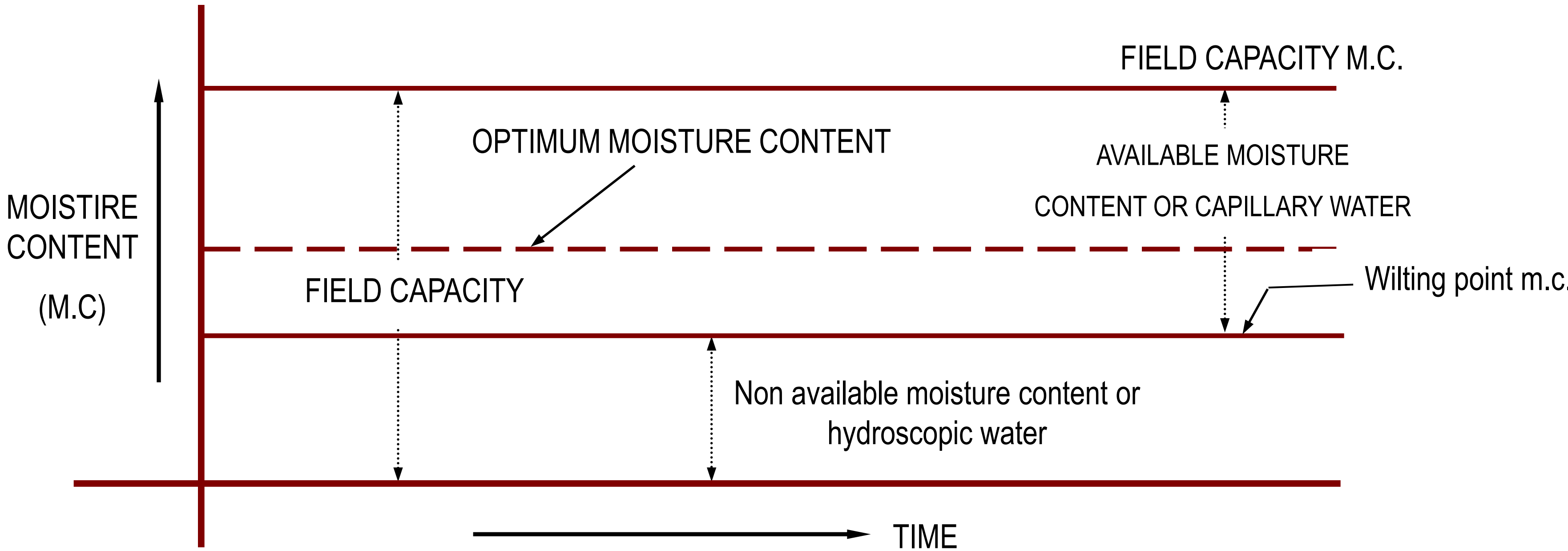
Optimum Utilization of Irrigation Water



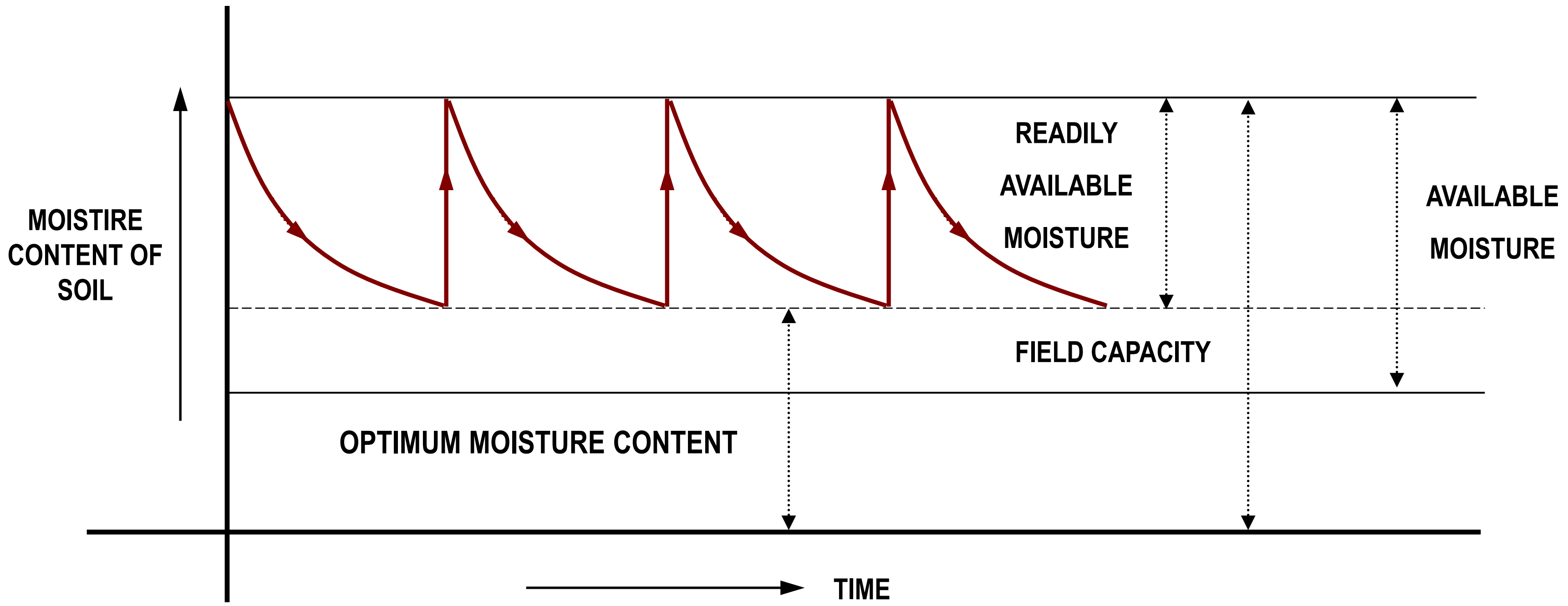
In an identical situation, yield is going to vary with the application of different quantities of water. The yield increases with water, reaches maximum value and then falls down.

The quantity of water at which the yield is maximum, is called the optimum water depth.

Estimating depth and frequency of irrigation on the basis of soil moisture regime concept



Estimating depth and frequency of irrigation on the basis of soil moisture regime concept



Field Capacity

The field capacity water (i.e. the quantity of water which any soil can retain indefinitely against gravity) is expressed as the ratio of the weight of water contained in the soil to the weight of the dry soil retaining that water: i.e.

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

If we consider 1 m^2 area of soil and d meter depth of root zone,

$\therefore$ The volume of soil = $d \times 1 = d$ m^3

If the dry unit wt. of soil = γ_d kN/m^3

$\therefore$ Wt. of d m^3 of soil = $\gamma_d \times d$ kN

If F is the field capacity,

$$\therefore F = \frac{\text{Wt. of water retained in unit area of soil}}{\gamma_{\mathrm{d}} \times d}$$

$\Rightarrow$ Wt. of water retained in unit area of soil = $(\gamma_d \times d \times F)$ kN/m^2

$\Rightarrow$ Volume of water stored in unit area of soil × γ_w = $(\gamma_d \times d \times F)$ kN/m^2

$\Rightarrow$ Volume of water stored in unit area of soil = $\dfrac{\gamma_d \times d \times F}{\gamma_w}$ meters

$\therefore$ Total water storage capacity of soil (m depth of water) = $\dfrac{\gamma_d \times d \times F}{\gamma_w}$ meters

Hence, the depth of water stored in the root zone in filling the soil up to field capacity = $\dfrac{\gamma_d \times d \times F}{\gamma_w}$ meters

Problem

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

- Field capacity of the soil = 28%
- Permanent wilting point = 13%
- Dry density of soil = 1.3 gm/cc
- Effective depth of root zone = 70 cm
- Daily consumptive use of water for the given crop = 12 mm.

Solution:

We know, by definition of available moisture, that

Available moisture = Field Capacity – Permanent wilting point

= 28 – 13 = 15 %

Let us assume that the readily available moisture or the optimum soil moisture level is 80 % of available moisture

i.e. Readily available moisture = 0.80   15 % = 12 %

$\therefore$ Optimum moisture = 28 – 12 = 16 %

It means that the moisture will be filled by irrigation between 16 % and 28 %. Depth of water stored in root zone between these two limits

$$= \frac{\gamma_d \times d}{\gamma_w} \text{ [FC – OMC] -------------- (i)}$$

Now,

$$\frac{\gamma_d}{\gamma_w} = \frac{\rho_d \times g}{\rho_w \times g} = \frac{\rho_d}{\rho_w} = \frac{1.3}{1} \qquad [\ \because \rho_w = 1 \text{ gm/cc}]$$

= 1.3 gm/cc

From equation (i) $\Rightarrow$ Depth of water = $\dfrac{\gamma_d \times d}{\gamma_w}$ [FC – OMC]

= 1.3 0.7 [0.28 – 0.16]

= 0.1092 m = 10.92 cm

Hence, water available for evapo-transpiration = 10.92 cm

1.2 cm of water is utilized by the plant in 1 day
10.92 cm of water will be utilized by the plant in = 1 10.92/1.2 days

= 9.1 days

≈ 9 days

Hence, after **9 days**, water should be supplied to the given crop

End of Chapter – 3

CHAPTER – 4

Soil Water Relationship

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LECTURE 10

Introduction

Root zone (depth of soil penetrated by roots) soil provides the storage reservoir which needs to be periodically recharged.

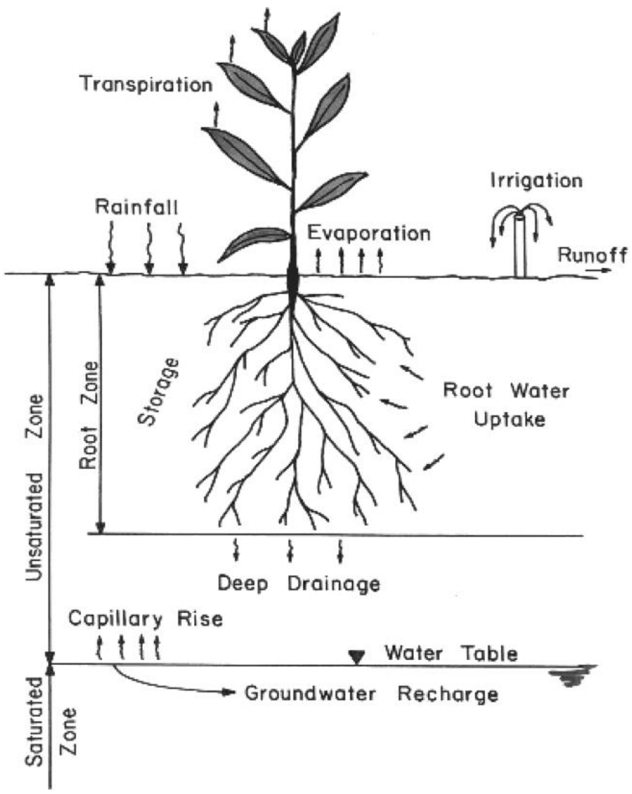
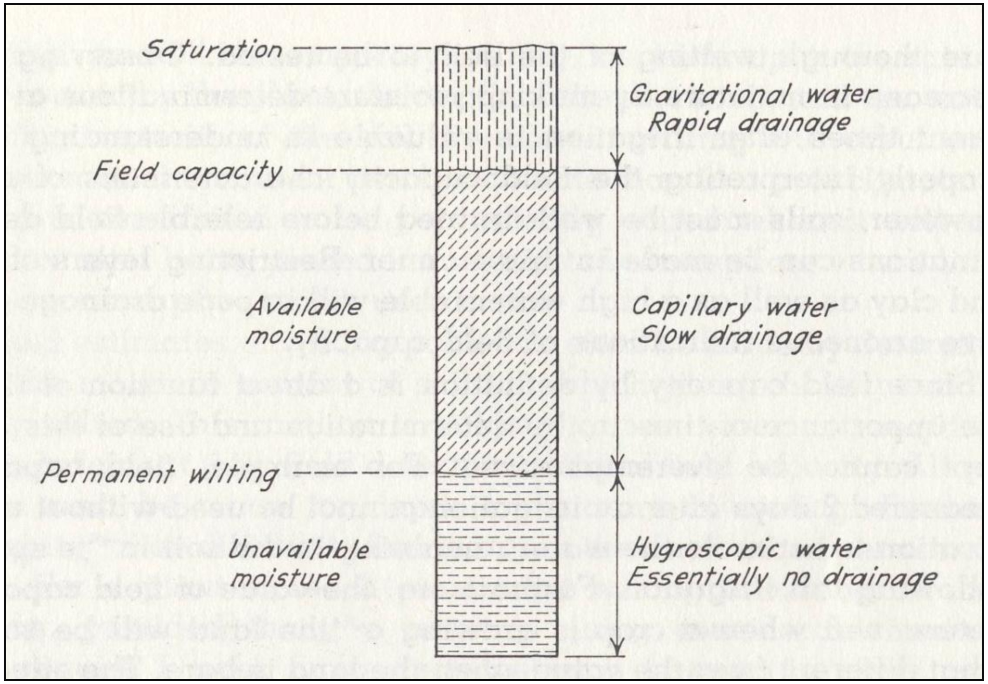


Fig: Schematic diagram of the sub-processes linked to field irrigation system

Classes and availability of soil water

- **Gravitational water:** Water moves freely in response to gravity.
- **Capillary water:** Water held by surface tension in the pore spaces.
- **Hygroscopic water:** Water held tightly to the surface of the grains by adsorption.



Field Capacity (FC):

The water content of the soil when gravitational water has been removed

- It represents the upper limit of available soil water range
- It is determined two days after an irrigation or thorough wetting. Limitations are: restricting layers, high water table, surface evaporation, consumptive use by crops
- Water content corresponding to a soil-moisture of 1/10 atm for sandy soil to 1/3 atm for silty or clay soil

Permanent Wilting Point (PWP):

The moisture content at which plants permanently wilt

- Wilting depends on the rate of water use, depth of root zone and water holding capacity of soil
- It is the lower end of available moisture range
- Water content corresponding to soil-moisture tension of 15 atm
- As an approximation, PWP can be estimated from:

$$PWP = \frac{FC}{(2 \text{ to } 2.4)}$$

Available Water (AW):

The difference of water content of the soil between field capacity and permanent wilting point

$$AW = FC - PWP$$

- It represents the moisture which can be stored in the soil for subsequent use by plants

- The moisture near the wilting point is not readily available to the plant. The portion of the available moisture which is most easily extracted by plants is termed as readily available moisture.

- Irrigation water should be supplied as soon as the moisture falls upto optimum level. The optimum level represents the maximum deficiency upto which the soil moisture may be allowed without any fall in crop yields.

- The amount of irrigation should be just enough to bring the moisture content upto its field capacity making allowance for application losses

Soil moisture content

(a) Moisture content by mass: $\theta_m = \dfrac{\text{Mass of water}}{\text{Mass of dry soil}} = \dfrac{M_w}{M_s}$

(a) Moisture content by volume: $\theta_v = \dfrac{\text{Volume of water}}{\text{Bulk volume of soil}} = \dfrac{V_w}{V_b}$

θ_v is more useful, since it represents the equivalent depth of water per unit depth of soil

$$\theta_v = \frac{d_w}{D_s} \Rightarrow d_w = \theta_v \times D_s \text{ ------------------------ (i)}$$

Again, $\theta_v = \dfrac{V_w}{V_b} = \dfrac{\dfrac{M_w}{\rho_w}}{\dfrac{M_s}{\rho_b}} = \theta_m \times \dfrac{\rho_b}{\rho_w} = \theta_m \times A_s$ ------------ (ii)

From equation (i) $\Rightarrow$

$$d_w = \theta_m \times A_s \times D_s$$

Soil moisture tension

- In saturated soils, water is held in the soil matrix under negative pressure due to attraction of the soil matrix for water

- Instead of referring to this negative pressure the water is said to be subjected to a tension exerted by the soil matrix

- The tension with which the water is held in unsaturated soil is termed as soil-moisture tension, soil-moisture suction. It is usually expressed in atmospheres, the average air pressure at sea level. Other pressure units like cm of water or cm or mm of mercury are also often used.

(1 atmosphere = 1023 cm of water = 76 cm Hg)

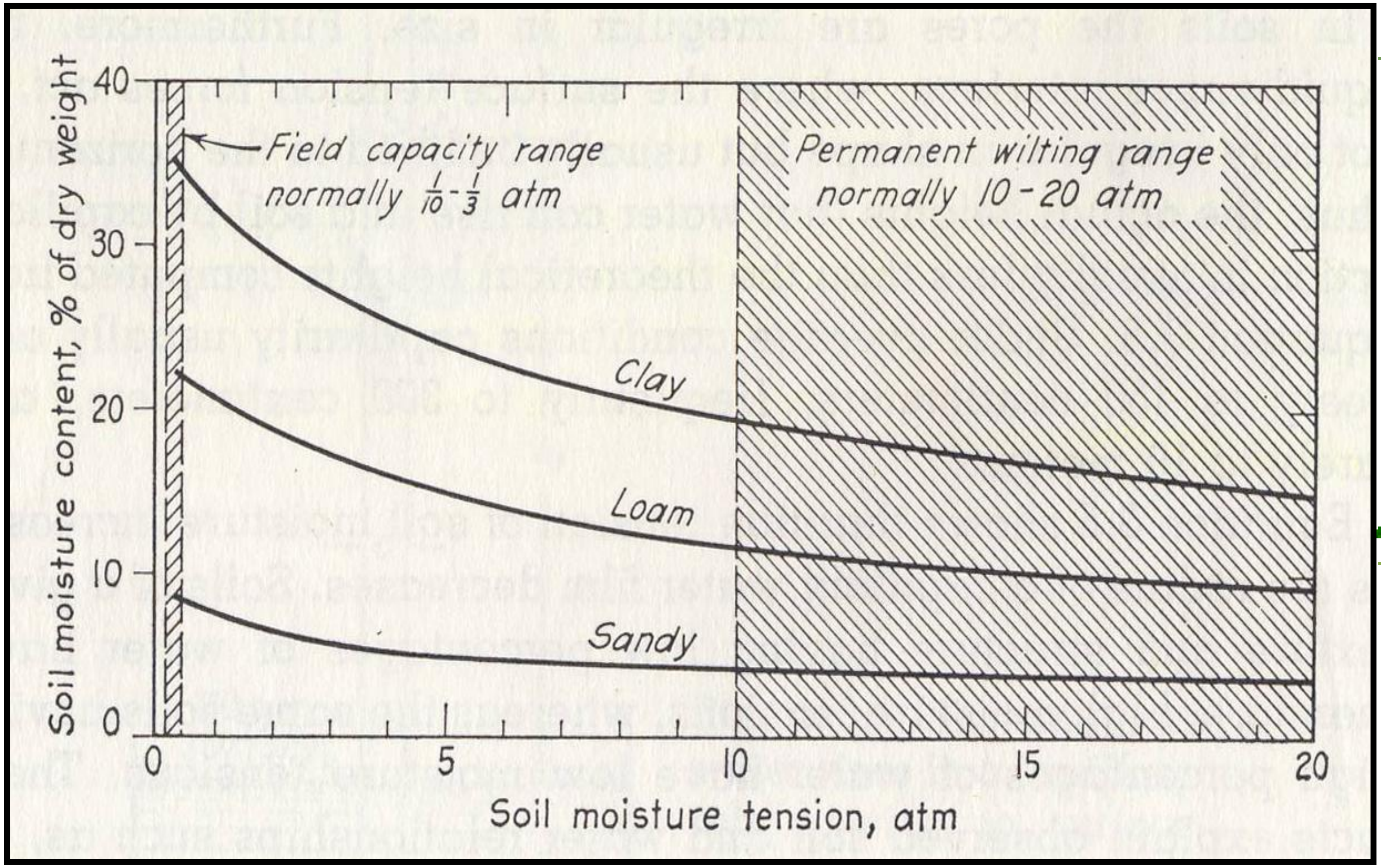
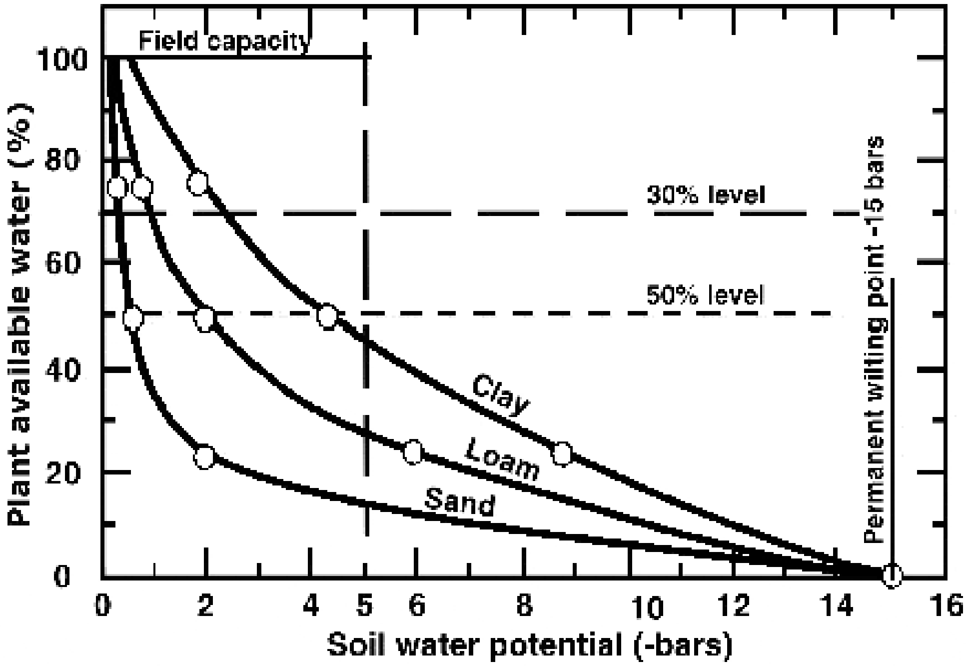


Fig: Typical curves of soil moisture variation with tension

Soil moisture characteristics

Moisture extraction curves, also called moisture characteristics curves, which are plots of moisture content versus moisture tension, show the amount of moisture a given soil holds at various tensions.



A knowledge of the amount of water held by the soil at various tensions is required in order to understand the amount of water that is available to plants, the water that can be taken up by the soil

Soil moisture stress

- In many irrigation soils, the soil solution contains an appreciable amount salts. The osmotic pressure developed by the soil solution retards the uptake of water by plants.
- Plant growth is a function of the soil moisture stress which is the sum of the soil moisture tension and osmotic pressure of soil solution.
- For successful crop production in soils having appreciable salts, the osmotic pressure of the soil solution must be maintained as low as possible by controlled leaching and the soil moisture tension is the root zone is maintained in a range that will provide adequate moisture to the crop.

Measurement of soil moisture

Objective or importance:

- To determine the time and amount of irrigation
- To estimate evapotranspiration/use rate

Methods:

(a) Appearance and feel method
(b) Gravimetric Method
(c) Electro-resistance blocks
(d) Tensiometer
(e) Neutron method

Appearance and feel method

- Using the soil auger, soil samples throughout the root zone are collected.
- By looking and feeling the sample, soil moisture deficiency is determined using guideline
- Not precise and it requires experience and judgment
- Simple, quick and it requires no equipment except soil auger
- In many applications, greater accuracy is not needed, nor is it justified economically.

Gravimetric Method

- This method is used for primary measurement
- It involves weighting a sample of moist soil, drying to a constant weight at a temperature of $105^o \sim 110^oC$, and re-weighting. Usually 24 hours are required for drying.
- Most accurate and direct method
- Destructive, labor intensive and time consuming; several samples are required to obtain a satisfactory representative indication of moisture content.

Electro-resistance blocks

- The porous blocks (gypsum) are calibrated against a range of moisture. The blocks containing desired electrical elements are placed in the field of at required depth.
- As the moisture content of the blocks changes, the electrical resistance also changes
- The gypsum blocks are soluble and deteriorate in one to three seasons of use.
- Normally there is considerable variation between blocks and considerable changes occur in the calibration during the season

Tensiometer

- A porous ceramic cup filled with water is attached to a vacuum gauge or mercury manometer.

- A hole is bored or dug to a desired depth; a handful of loose soil is placed into the hole, and the cup pushed firmly into the soil. The water inside the cup comes into hydraulic contact through the pores in the cup. When initially placed in the soil, water contained in the tensiometer is generally at atmospheric pressure. Soil water, being generally at sub-atmospheric pressure, exercises a suction which draws out a certain amount of water within the tensiometer, thus causing a drop in its hydrostatic pressure. This pressure is indicated by the manometer or vacuum gauge.

- Tensiometer is effective upto a tension of 0.8 bar. At this pressure air enters the closed system through the pores of the cup and makes the unit inoperative.

Tensiometer….. …continue

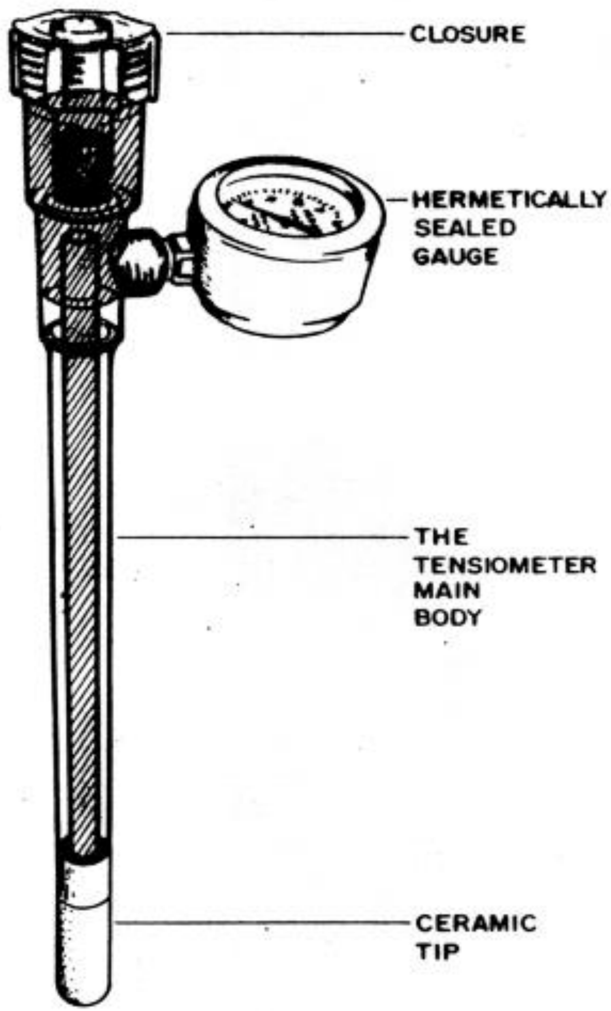


Fig: The essential parts of a tensiometer

- Tensiometer readings are useful in deciding when to irrigate, but they do not indicate how much water should be applied. A special moisture-characteristic curve for the particular soil is needed to convert moisture tension measurements into available moisture percentage.

- Tensiometers are less well suited to use in fine-textured soils in which only a small part of the available moisture is held at a tension of less than 1 atmosphere.

- Since the unit operates satisfactorily only up to tensions of 0.8 atm, they are most useful in sandy soil, where this represents a major portion of the available water.

- Because of its narrow range of application the tensiometer is used for moist and resistance blocks for dryer soil conditions. Sometimes a combination of tensiometer and resistance blocks is used.

LECTURE 11

Neutron Method

- A hole is dug with an auger, and a metal tube is driven into the hole to retain the soil. The neutron source and counting device are lowered to the desired depth.
- Fast neutrons emitted from the source and slowed down by water in the surrounding soil. The resulting slow neutrons which reach the counting tube are recorded. Fast neutrons are not registered by the counter.
- The greater the water content of the soil, the greater is the number of slow neutrons reaching the counting tube.

- There exists a good correlation between moisture content and the number of slowed down neutron reaching the counter.

$$\theta_v = (a + b) \quad \frac{R_s}{R_{st}}$$

Where, a and b are calibration coefficient

R_s = Count rate in the soil

R_{st} = Standard count rate

It measures θ_v directly

It is expensive, can not be used to measure near the surface because of boundary effect and possible radiation hazard, and needs calibration

Flow of water through soil

Energy in fluid is in two forms:

- Kinetic Energy
- Potential Energy consisting of
- Energy resulting from pressure difference
- Energy resulting from elevation difference

Widely used Bernoulli's Energy Equation, showing energy per unit mass of fluid:

$$H = z + P/\gamma + V^2/2g$$

Since the velocity through soil is very small, the term $V^2/2g$ can be ignored. Thus,

$H = z + P/\gamma$ is called the piezometric head or hydraulic head

Flow of water in soil occurs in the direction of decreasing piezometric head.

Pressure head is due to adsorptive and capillary forces in unsaturated soil; elevation head due to gravitational potential.

Darcy's law relates velocity to head loss:

$$V = k \quad (h_L/L)$$

Where,

V = Flow velocity

k = Co-efficient of permeability or hydraulic conductivity

h_L/L = Hydraulic gradient or slope of H.G.L

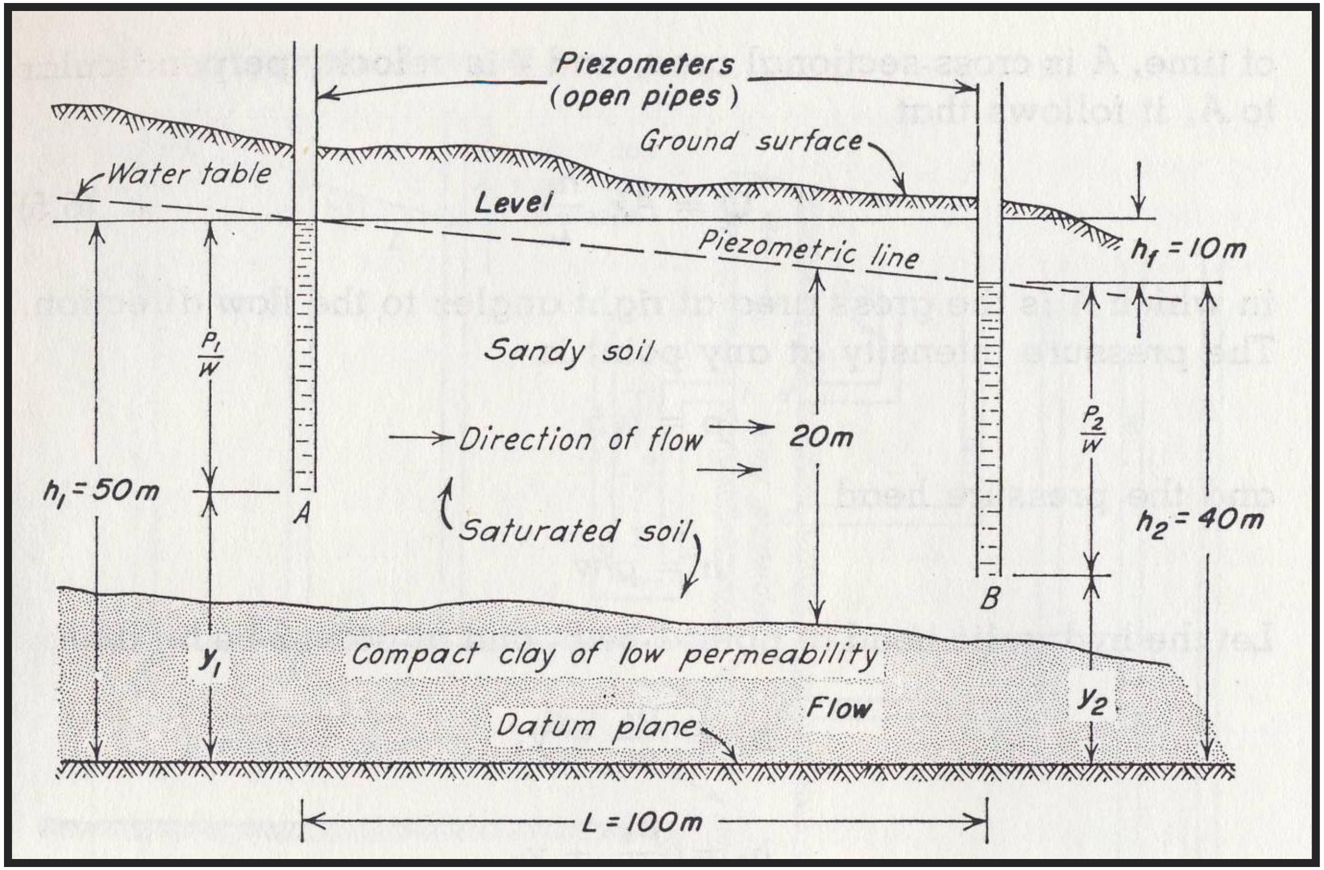
The hydraulic head h can be measured by piezometer in saturated soil and by tensiometer in unsaturated soil.

The quantity of flow,

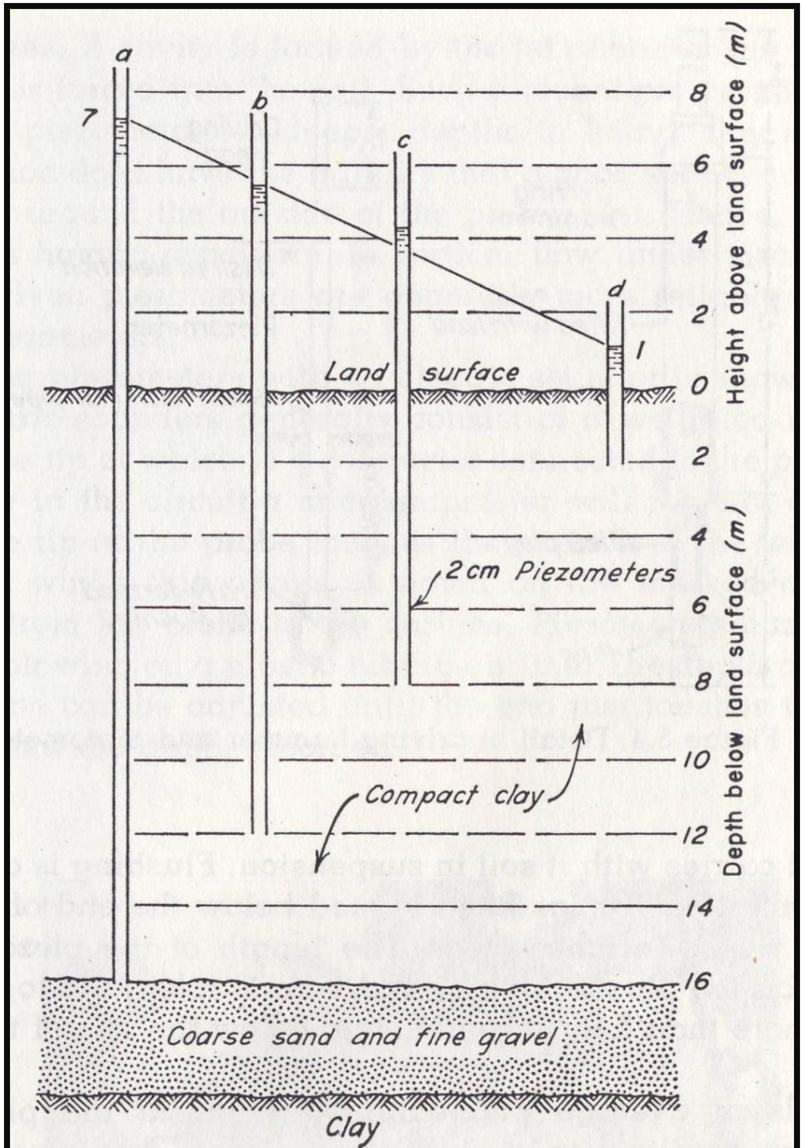
$$Q = AV$$
$$= A \quad k \quad (h_L/L)$$

Where, A = Gross area at right angles to flow direction

Flow of unconfined ground water:



Flow of confined ground water:



$$\frac{h_L}{L} = \frac{h_a - h_d}{L} = \frac{(6+7) - (4+3)}{16-2}$$

= 6/14 = 0.43

For unsaturated soil, the hydraulic conductivity decreases many folds as the moisture content decreases. Moreover, it is difficult to measure h because tensiometer becomes inoperative when the tension exceeds 0.8 atm. Furthermore, flow occurs in both liquid and vapor phases.

Infiltration/intake characteristics of soil

Infiltration is the time rate of entry of water into soil. Whenever the soil surface configuration influences the rate of entry, the term intake is used.

It has great practical importance: design and operation of water application system, intake rate of fine-textured soil is very low.

Factors influencing infiltration are:

- ❑ Initial moisture content
- ❑ Condition of soil surface
- ❑ Hydraulic conductivity of soil profile
- ❑ Depth of water on the surface
- ❑ Viscosity/temperature of water
- ❑ Soil texture

Infiltration/intake characteristics of soil

The intake rate plotted against time on a logarithmic scale gives a straight line.

$$I = a \quad T^n$$

Where,

I = Intake rate

a = Constant (ordinate at T = 1)

n = slope of the line

When the observation of intake extends over long periods, a better representation of the data can be obtained by:

$$I = b + aT^n$$

Since, n is negative, I decreases with an increase in T. I approaches a constant value b as time increases. This value is called final intake rate.

$$I = k \frac{\delta h}{\delta s}$$

Initially I is high because of large difference in tension in addition to gravity, after several hours difference in tension becomes zero and hydraulic gradient equals to unity and I approaches to Ks.

Measuring Intake Rate

- 25 cm diameter cylinder is driven upto 15 cm below soil surface
- Water is applied at surface. The radial flow at bottom of cylinder causes great change in intake rate.
- Two concentric cylinders having same water level are used to create buffer ponds
- Depth of water for inner cylinder is recorded with time

Cylinder No. 3					
Time (hr)			Intake (mm)		
Watch	Difference	Cumulative	Depth	Difference	Cumulative
4:15			260		
4:16	1	1	249	11	11
4:18	2	3	242	7	18
4:22	4	7	234	8	26
Refill			271		
4:30	8	15	260	11	37
4:48	18	33	248	14	61
Refill			270		
5:02	14	47	252	8	59
5:29	27	74	248	14	73
6:00	31	105	238	10	83
Refill			280		
6:29	29	134	269	11	94
5:57	28	162	260	9	103
7:23	26	188	253	7	110
7:43	20	208	248	5	115

Intake rate, $I = aT^n$

Accumulated intake,

$$D = \int (I \times dT)$$

$$= \frac{a}{n+1} \times T^{n+1}$$

$$= C \times T^N$$

Average Infiltration rate,

$$I_{avg} = \frac{D}{T} = CT^{N-1}$$

Instantaneous Infiltration rate,

$$I_{inst} = \frac{dD}{dT} = CNT^{N-1}$$

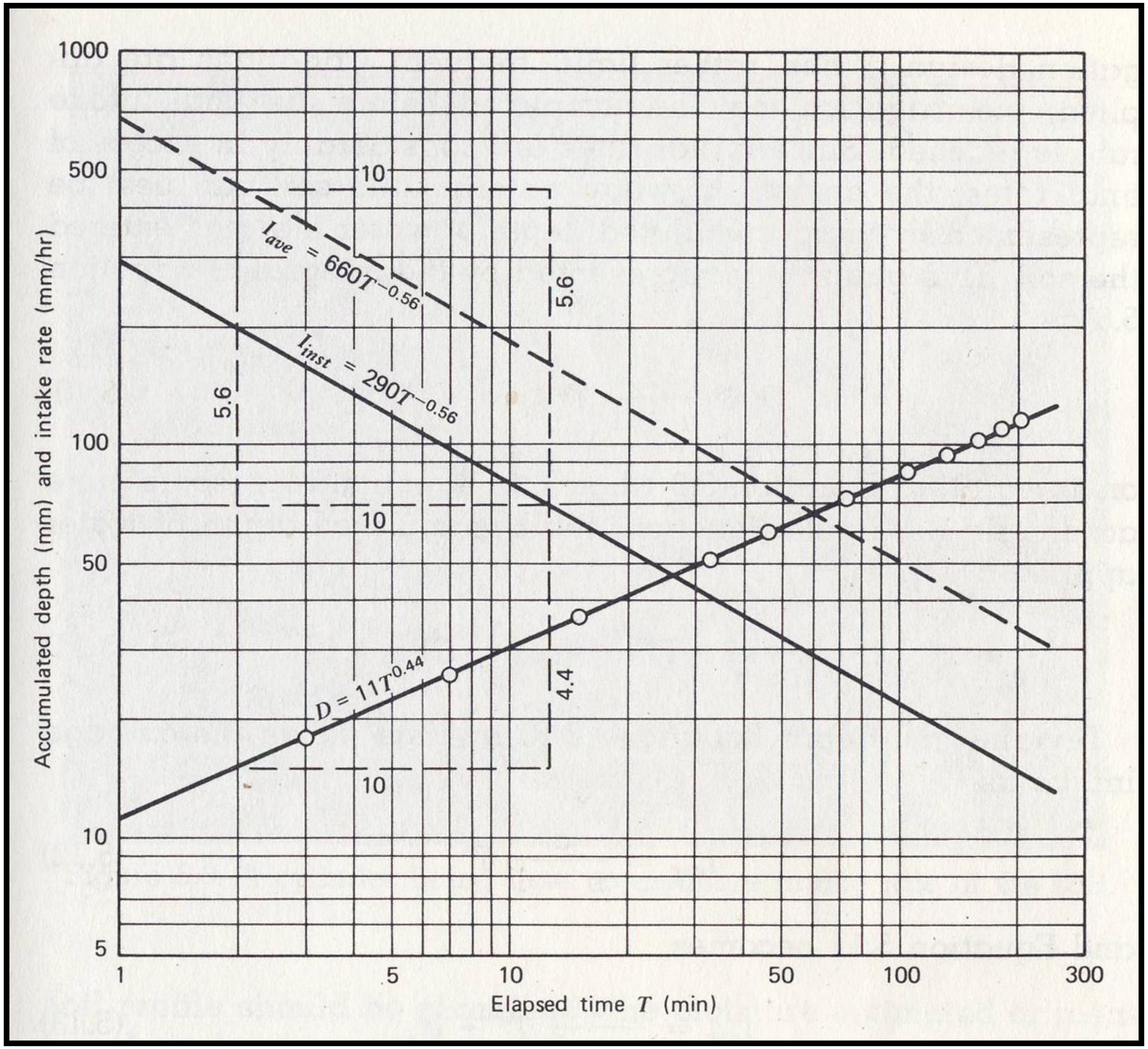


Fig: Typical Intake curves

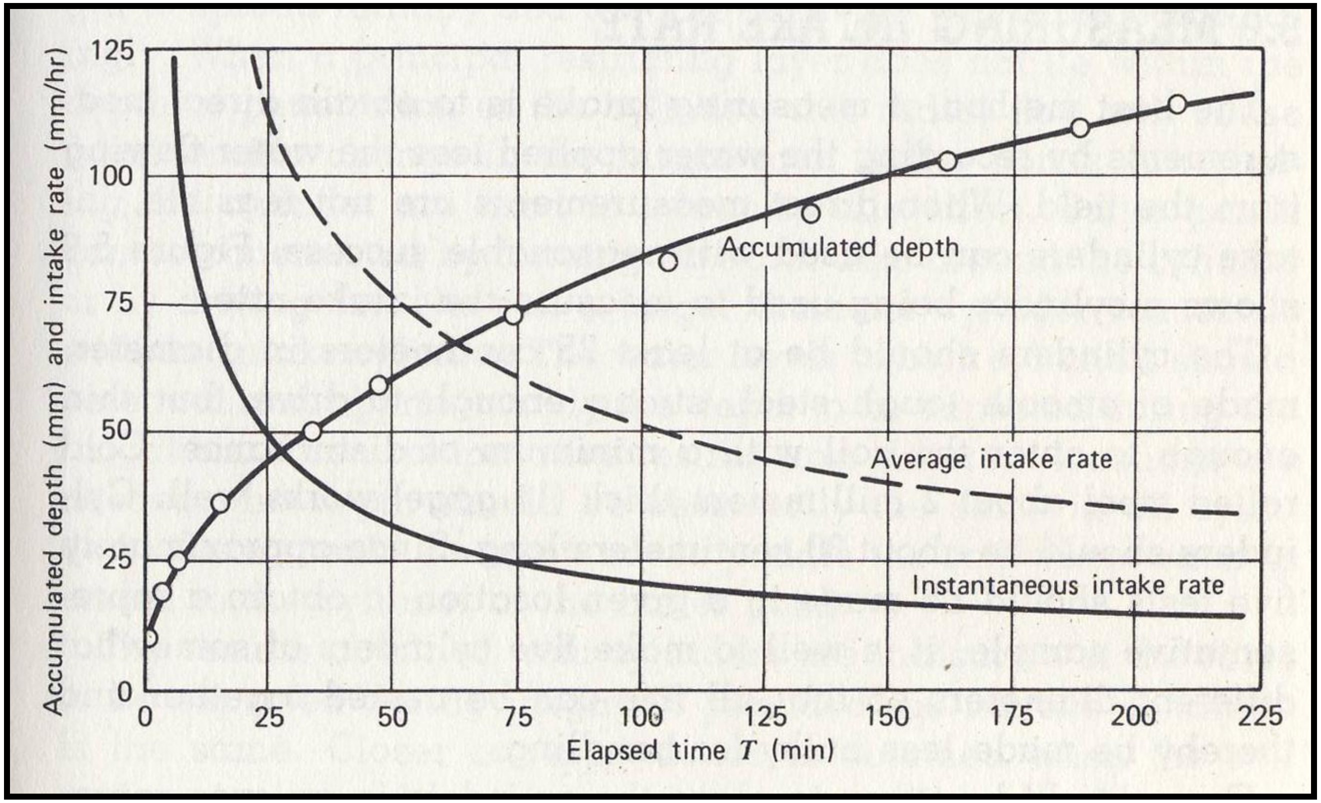
The slope of the line on log-log plot,

$$N = 0.44$$

Hence, $D = 11 \quad T^{0.44}$

$$I_{avg} = CT$$
$$= (11 \quad 60) \quad T^{(0.44-1)}$$
$$= 660 \quad T^{-0.56}$$

$$I_{inst} = CN\ T^{N-1}$$
$$= (11 \quad 60) \quad 0.44 \quad T^{(0.44-1)}$$
$$= 290 \quad T^{-0.56}$$



At T = 1 min, D = 11 mm,

so , $D = CT^N$

$\Rightarrow 11 = C\times(1)^N$

$\therefore\ C = 11$

End of Chapter – 4

CHAPTER – 5

Economical & Physical Justification For Canals

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LECTURE 12

Lining

Advantages of Lining:

- ✓ Seepage Control
- ✓ Prevention of Water-Logging
- ✓ Increase in Channel Capacity
- ✓ Increase in Commanded Area
- ✓ Reduction in Maintenance Costs
- ✓ Elimination of Flood Dangers

Selection of Suitable Type of Lining

- Low cost
- Impermeability
- Hydraulic efficiency (i.e. reduction in rugosity coefficient)
- Durability
- Resistance to erosion
- Repairability
- Structural stability

Financial Justification & Economics of Canal Lining

Annual benefits:

(a) *Saved seepage water by lining*:

Let, the rate of water is sold to the cultivators = Tk. R_1/cumec

If *m* cumecs of water is saved by lining the canal annually, then the money saved by lining = Tk. $m R_1$

(b) *Saving in maintenance cost*:

Let, the average cost of annual upkeep of unlined channel = Tk. R_2

If *p* is the percentage fraction of the saving achieved in maintenance cost by lining the canal, then the amount saved = pR_2 Tk.

$\therefore$ The total annual benefits = $mR_1 + p\,R_2$

Annual costs:

Let, the capital expenditure is *C* Tk. & the lining has a life of *Y* years

$\therefore$ Annual depreciation charges = *C*/*Y* Tk.

$\therefore$ Interest of the capital *C* = C(r/100) [*r* = percent of the rate of annual interest]

$\therefore$ Average annual interest = C/2(r/100) Tk.

[Since the capital value of the asset decreases from C to zero in Y years]

$\therefore$ The total annual costs of lining = C/Y + C/2(r/100)

$$\therefore \text{Benefit cost ratio} = \frac{\text{Annual Benefits}}{\text{Annual Costs}} = \frac{mR_1 + pR_2}{\frac{C}{Y} + \frac{C}{2} \times \frac{r}{100}}$$

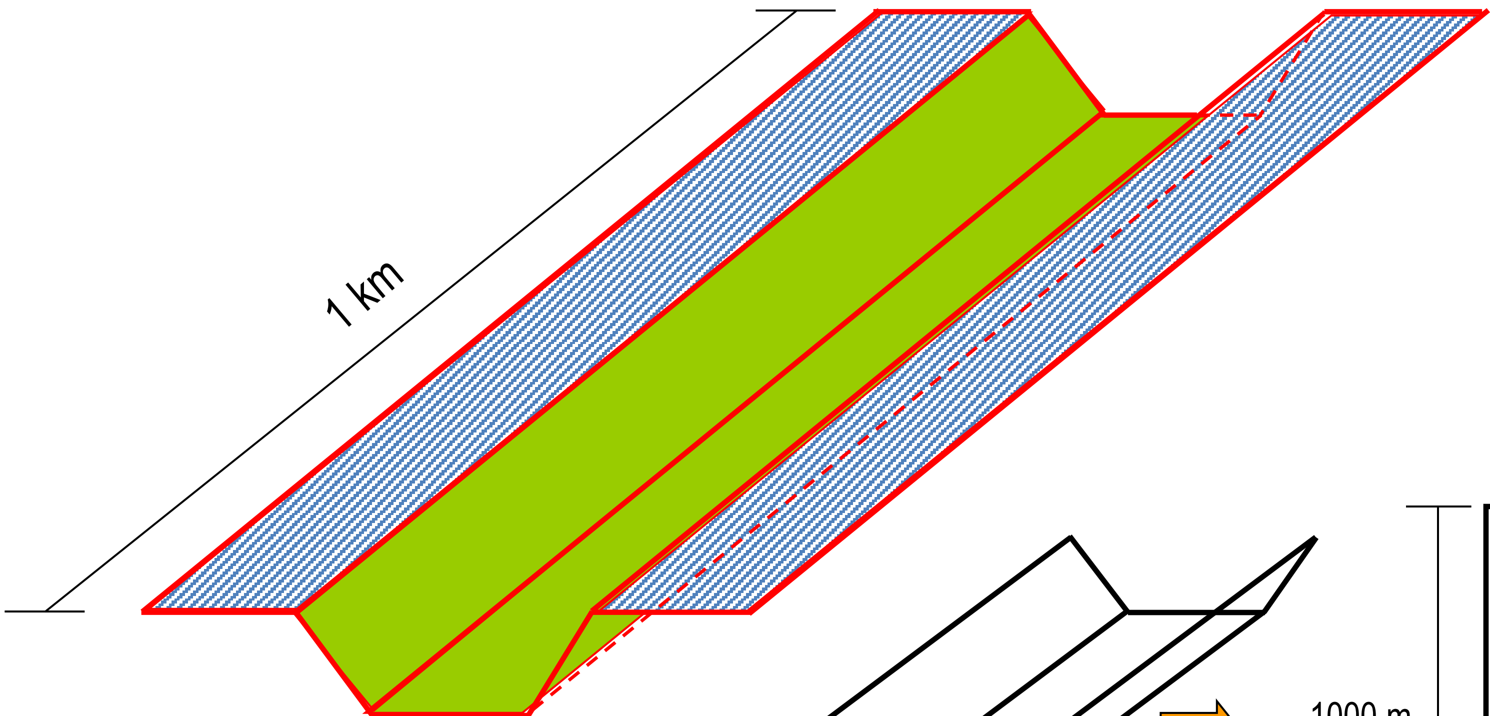
If p is taken as 0.4, then

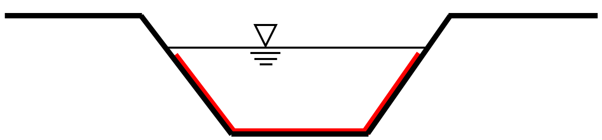
$$\therefore \text{Benefit cost ratio} = \frac{mR_1 + 0.4R_2}{\frac{C}{Y} + \frac{C}{2} \times \frac{r}{100}}$$

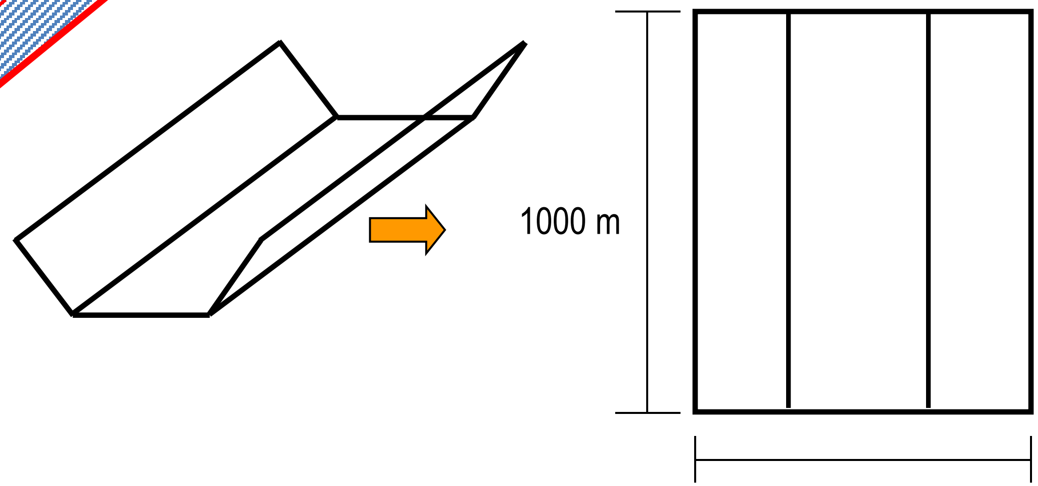
Problem

An unlined canal giving a seepage loss of 3.3 cumec per million square meters of wetted area is proposed to be lined with 10 cm thick cement concrete lining, which costs Tk. 180 per 10 square meters. Given the following data, work out the economics of lining and benefit cost ratio.

Annual revenue per cumec of water from all crops	Tk. 3.5 lakhs
Discharge in the channel	83.5 cumecs
Area of the channel	40.8 m^2
Wetted perimeter of the channel	18.8 m
Wetted perimeter of the lining	18.5 m
Annual maintenance cost of unlined channel per 10 square meter	Tk. 1.0







Discharge in the canal = 83.5 cumec

Area of the canal = 40.8 m^2

Wetted Perimeter of the canal = 18.8 m

Wetted Perimeter of the lined canal = 18.5 m

***Solution*:**

Let us consider 1 km (= 1000 m) reach of canal. Therefore,

the wetted surface per km = 18.8×1000 = 18,800 m^2

(i) Annual Benefits

(a) *Seepage loss*

Seepage loss in unlined canal @ 3.3 cumec per million sq. m

= $(3.3/10^6)\times 18,800$ cumec/km = $62,040\times 10^{-6}$ cumec/km

Assume, seepage loss in lined channel at 0.01 cumec per million square meter of wetted perimeter

$\therefore$ Seepage loss in unlined canal = $(0.01/10^6)\times 18,800 = 188\times 10^{-6}$ cumecs/km

Net saving = $(62,040\times 10^{-6} - 188\times 10^{-6})$ cumec/km = 0.06185 cumec/km

Annual revenue saved per km of channel = (0.06185×3.5) lakhs

= 0.21648 lakhs = 21,648 Tk.

(b) *Saving in maintenance*

Annual maintenance cost of unlined channel for 10 m^2 = Tk.1

Total wetted perimeter per 1 km length = 18,800 m^2

$\therefore$ Annual maintenance charge for unlined channel/ km = Tk.1,880

Assume that 40% of this is saved in lined channel

Annual saving in maintenance charges = Tk. (0.4×1880) = Tk.752

$\therefore$ Total annual benefits per km = Tk. (21,648 + 752) = Tk.22,400

(ii) Annual Costs

Area of lining per km of channel = 18.5×1000 = 18500 m^2

Cost of lining per km of channel @ Tk. 180 per 10 m^2

= (18500×180/10) Tk. = 333000 Tk.

Assume, life of lining as 40 years

Depreciation cost per year = Tk. (3,33,000/40) = Tk. 8325

Assume 5% rate of interest

Average annual interest = C/2 (r/100) = 3,33,000/2×(5/100) = Tk. 8325

$\therefore$ Total annual cost = Tk (8325 + 8325) = Tk. 16,650

Benefit cost ratio = Annual benefits/Annual costs = 22,400/16,650 = 1.35

Benefit cost ratio is **<u>more than unity</u>**, and hence, the lining is justified.

LECTURE 13

Causes of failure of weir or barrage on permeable foundation

- ***Failure due to Subsurface Flow***
  - (a) Failure by Piping or Undermining
  - (b) Failure by Direct Uplift
- ***Failure by Surface Flow***
  - (a) By Hydraulic Jump
  - (b) By Scouring

(a) Failure by Piping or undermining

The water from the upstream side continuously percolates through the bottom of the foundation and emerges at the downstream end of the weir or barrage floor. The force of percolating water removes the soil particles by scouring at the point of emergence.

(b) Failure by Direct uplift

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

(a) Failure by Hydraulic Jump

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

(b) Failure By Scouring

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

Bligh's creep theory for seepage flow

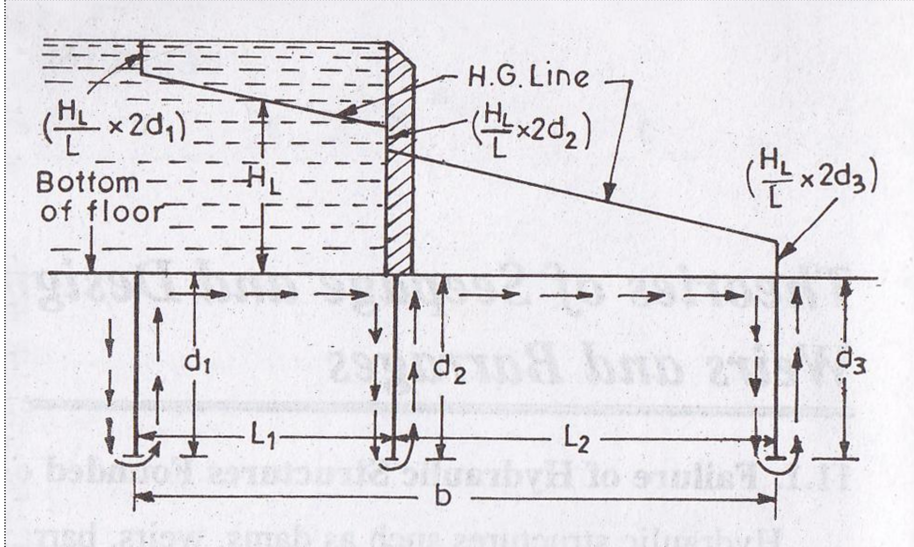


Fig-1: Bligh's Creep

Head losses equal to $\left(\frac{H_L}{L}\times 2d_1\right)$ $\left(\frac{H_L}{L}\times 2d_2\right)\left(\frac{H_L}{L}\times 2d_3\right)$;will occur respectively, in the planes of three vertical cut offs. The hydraulic gradient line (H.G. Line) can then be drawn as shown in figure aside.

$$L = d_1 + d_1 + L_1 + d_2 + d_2 + L_2 + d_3 + d_3 = (L_1 + L_2) + 2(d_1 + d_2 + d_3)$$
$$= b + 2(d_1 + d_2 + d_3)$$

Head loss per unit length or hydraulic jump = $\left[\frac{H_L}{b+2(d_1+d_2+d_3)}\right] = \frac{H_L}{L}$

(i) Safety against piping or undermining

By providing sufficient creep length, given by $L = C \quad H_L$

Where C is the Bligh's Coefficient for the soil.

Different values of C for different types of soils are tabulated in the table below:

SL	Type of Soil	Value of C	Safe Hydraulic gradient should be less than
1	Fine Sand	15	1/15
2	Coarse Grained Sand	12	1/12
3	Sand mixed with boulder and gravel, and for loam soil	5 to 9	1/5 to 1/9
4	Light Sand and Mud	8	1/8

(i) Safety against uplift pressure

The ordinates of the H.G line above the bottom of the floor represent the residual uplift water head at each point. Say for example, if at any point, the ordinate of H.G line above the bottom of the floor is 1 m, then 1 m head of water will act as uplift at that point. If h' meters is this ordinate, then water pressure equal to h' meters will act at this point, and has to be counterbalanced by the weight of the floor of thickness say t.

$\therefore$ Uplift pressure = $\gamma_w \quad h'$

[where γ_w is the unit weight of water]

Downward pressure = $(\gamma_w \quad G).t$

[Where G is the specific gravity of the floor material]

For equilibrium,

$$\gamma_w \quad h' = (\gamma_w \quad G).\ t$$

$$\Rightarrow h' = G \quad t$$

Subtracting t on both sides, we get

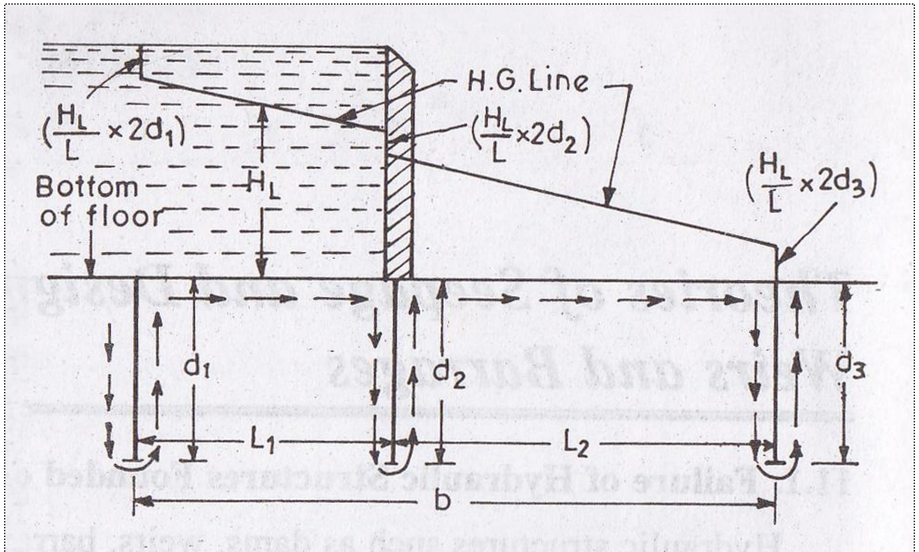
$$(h' - t) = (G\ \ t - t) = t\,(G - 1)$$

$$\Rightarrow t = \left(\frac{h' - t}{G - 1}\right) = \left(\frac{h}{G - 1}\right)$$

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

G – 1 = Submerged specific gravity of the floor material

Lane's weighted creep theory



ig-1: Bligh's Creep

<u>Weightage factor</u> of 1/3 for the horizontal creep, as against 1.0 for the vertical creep

$$L_l = (d_1 + d_1) + (1/3)\,L_1 + (d_2 + d_2) + (1/3)\,L_2 + (d_3 + d_3)$$
$$= (1/3)\,(L_1 + L_2) + 2(d_1 + d_2 + d_3)$$
$$= (1/3)\,b + 2(d_1 + d_2 + d_3)$$

Khosla's theory and concept of flow

<u>Main principles of Khosla's theory</u>:

- **Stream Lines:** The streamlines represent the paths along which the water flows through the sub-soil.
- Every particle entering the soil at a given point upstream of the work, will trace out its own path and will represent a streamline.
- The first streamline follows the bottom contour of the works and is the same as Bligh's path of creep.
- The remaining streamlines follows smooth curves transiting slowly from the outline of the foundation to a semi-ellipse, as shown below.

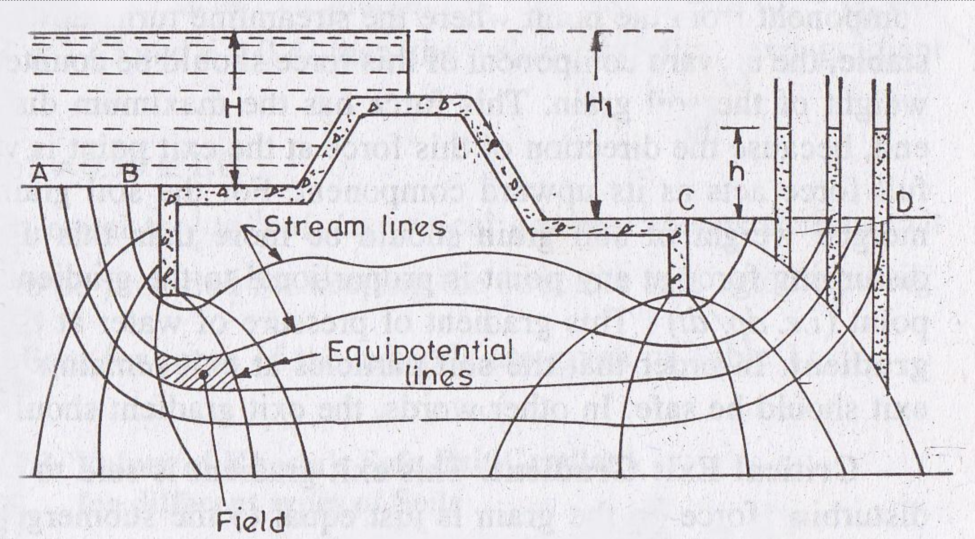


Fig: Khosla's Flow Net

Khosla's method of independent variables

(*For determination of pressures and exit gradient*)

- To know the seepage below the foundation of a hydraulic structure, it is necessary to plot the flow net.
- In other words, we must solve the Laplacian equations.
- This can be accomplished either by
  - (i) Mathematical solution of Laplacian equations,
  - (ii) Electrical analogy method,
  - (iii) Graphical sketching

These are **complicated methods and are time consuming**.

For designing hydraulic structures such as weirs or barrage or pervious foundations, Khosla has evolved a simple, quick and an accurate approach, called **Method of Independent Variables.**

The simple profiles which hare most useful are:

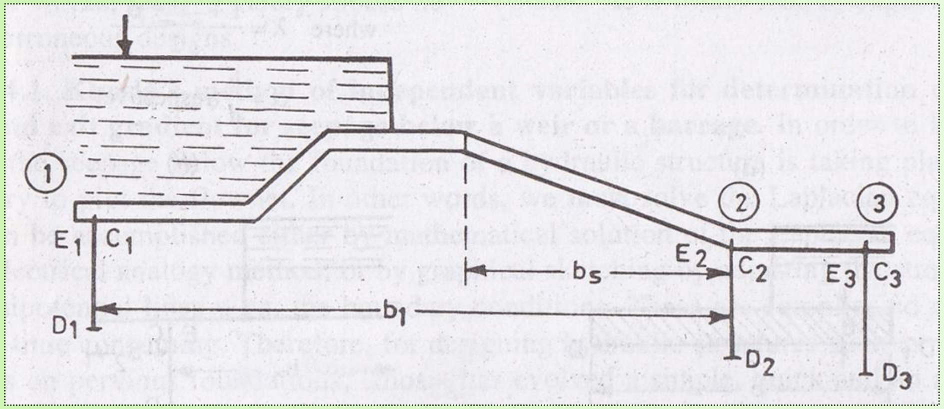
- ✓ A straight horizontal floor of negligible thickness with a sheet pile line on the upstream end and downstream end.
- ✓ A straight horizontal floor depressed below the bed but without any vertical cut-offs.
- ✓ A straight horizontal floor of negligible thickness with a sheet pile line at some intermediate point.

Three corrections in Khosla's theory

a) Correction for the Mutual interference of Piles
b) Correction for the thickness of floor
c) Correction for the slope of the floor

(a) Correction for the Mutual Interference of Piles:

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



Where,

- *b*' = The distance between two pile lines
- *D* = The depth of the pile line, the influence of which has to be determined on the neighboring pile of depth *d*. (*D* is to be measured below the level at which interference is desired.)
- *d* = The depth of the pile on which the effect is considered
- *b* = Total floor length

Suppose in the above figure, we are considering the influence of the pile No. (2) on pile No. (1) for correcting the pressure at C_1. Since the point C_1 is in the rear, this correction shall be **positive**. While the correction to be applied to E_2 due to pile No. (1) shall be negative, since the point E_2 is in the forward direction of flow. Similarly, the correction at C_2 due to pile No. (3) is positive, and the correction at E_2 due to pile No. (2) is **negative**

(b) **Correction for thickness of floor**:

- The corrected pressure at E_1 should be less than the calculated pressure at E_1'
- The correction to be applied for the joint E_1 shall be **negative**.
- The pressure calculated C_1' is less than the corrected pressure at C_1
- The correction to be applied at point C_1 is **positive**.

(c) <u>Correction for the slope of the floor</u>:

The correction factor given in the table below is to be multiplied by the ***horizontal length of the slope*** and divided by the ***distance between the two pile lines*** between which the sloping floor is located.

This correction is applicable only to the key points of the pile line fixed at the start or the end of the slope.

- Positive for down slope
- Negative for up slope

Slope (H : V)	Correction Factor
1 : 1	11.2
2 : 1	6.5
3 : 1	4.5
4 : 1	3.3
5 : 1	2.8
6 : 1	2.5
7 : 1	2.3
8 : 1	2.0

Exit gradient (G_E)

Gradient at the exit end is called exit gradient (G_E) which is determined from the equation below:

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

Where, $\lambda = \dfrac{1+\sqrt{1+\alpha^2}}{2}$

$\alpha = b/d$

H = Max. Seepage Head

Where, $\dfrac{1}{\pi\sqrt{\lambda}}$ is determined from the Plate No. 2 which is given in the next slide

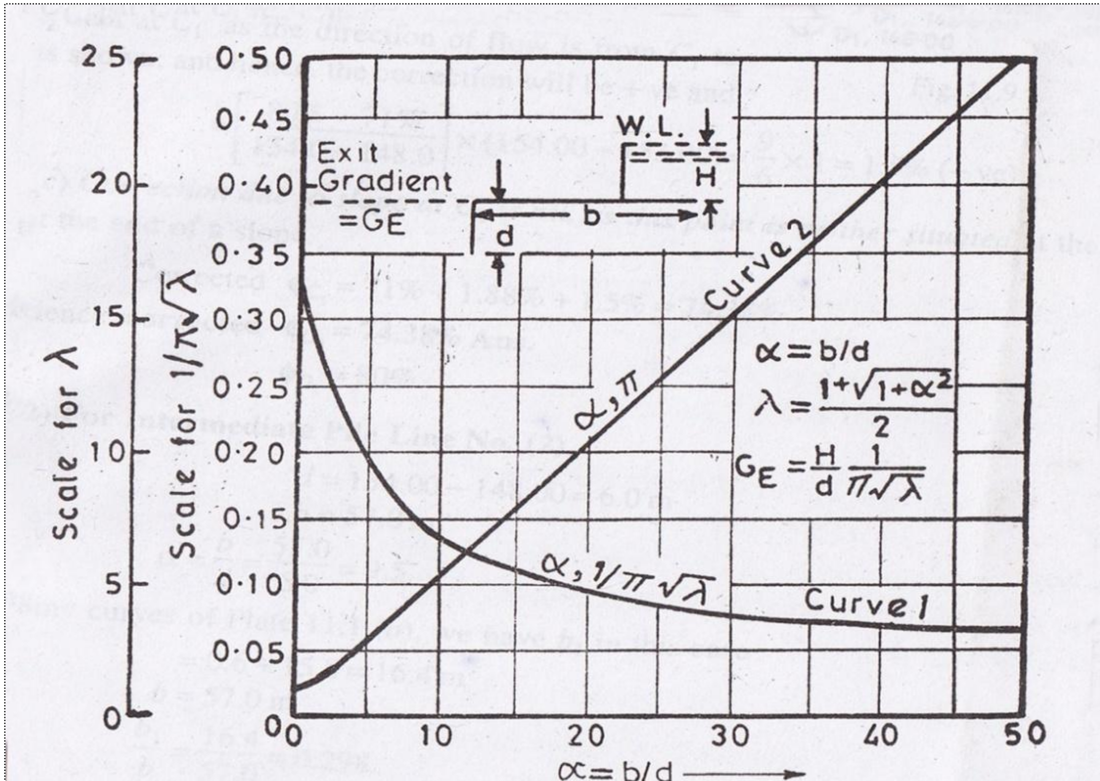


Plate 11.2

Type of Soil	Safe exit gradient
Shingle	1/4 to 1/5
Coarse Sand	1/5 to 1/6
Fine Sand	1/6 to 1/7

LECTURE 14

Problem: Determine the percentage pressures at various key points in figure below. Also determine the exit gradient and plot the hydraulic gradient line for pond level on upstream and no flow on downstream

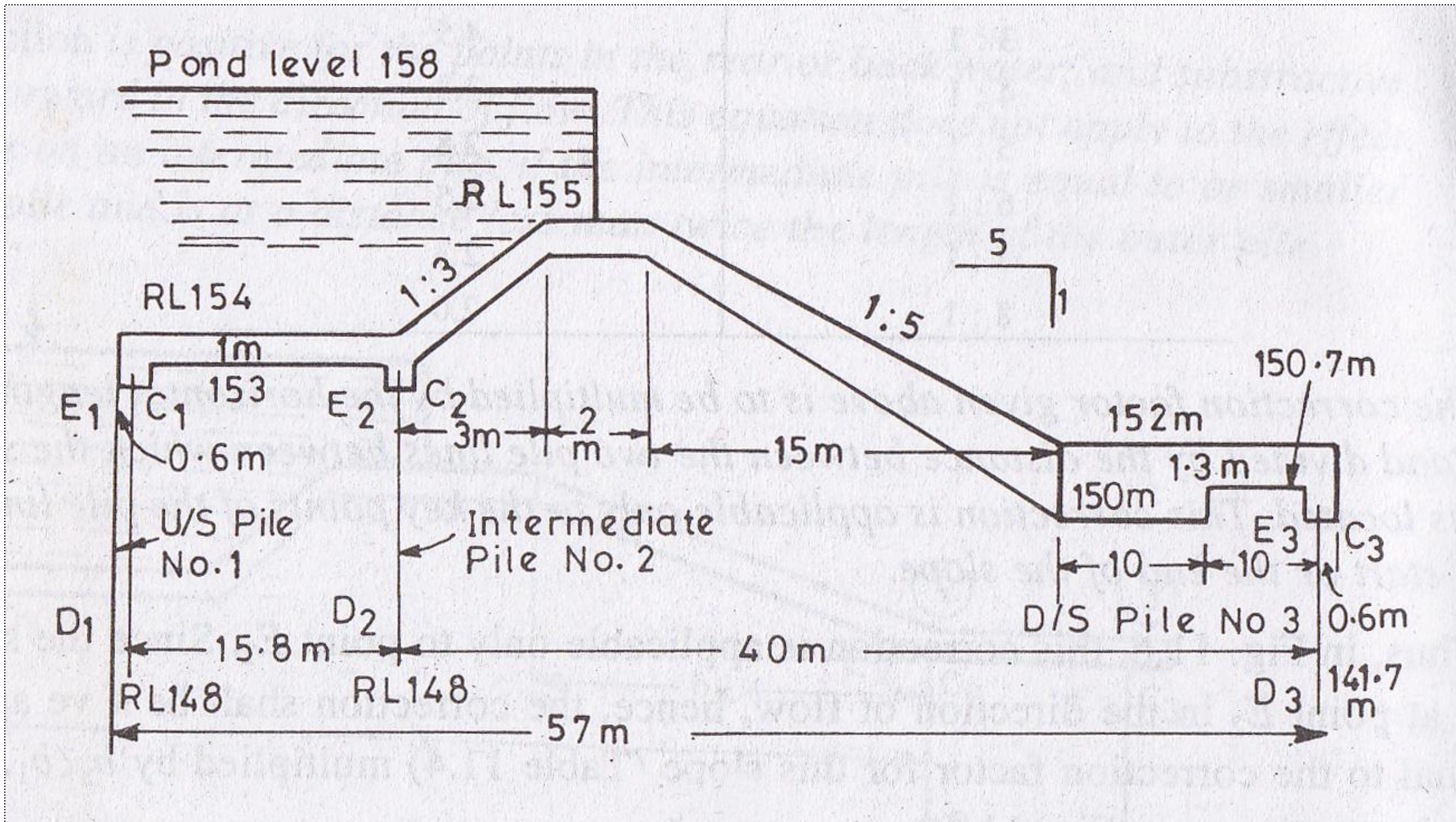
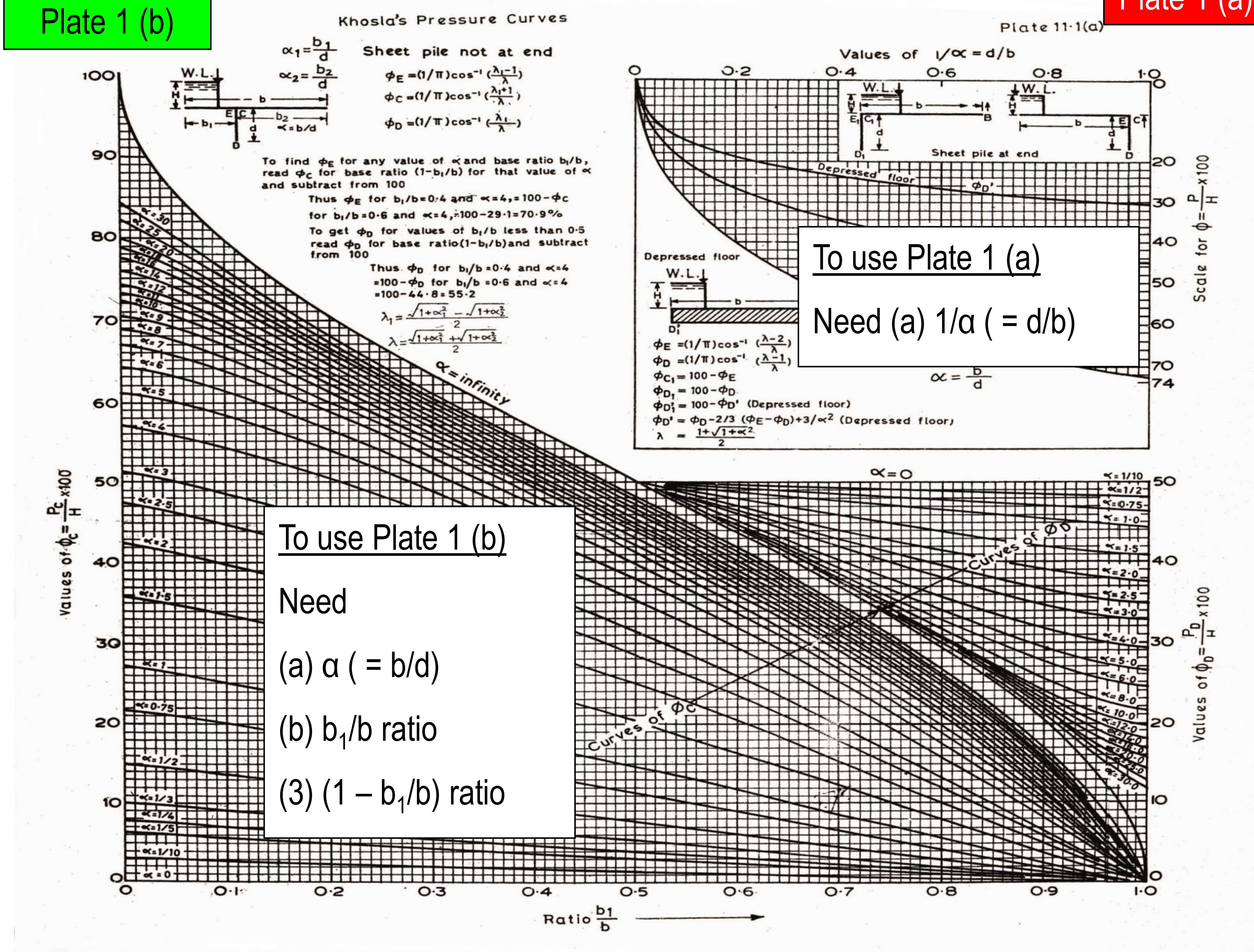


Plate 1

Plate 1 (b)

Plate 1 (a)



To use Plate 1 (a)

Need (a) $1/\alpha$ (= d/b)

To use Plate 1 (b)

Need

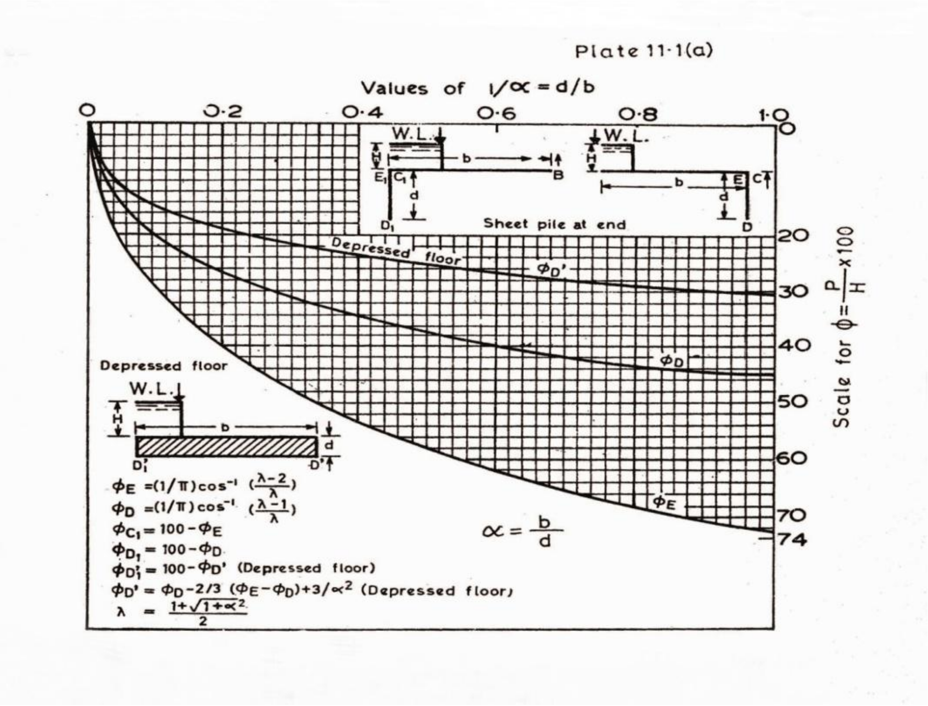
(a) α (= b/d)

(b) b_1/b ratio

(3) $(1 - b_1/b)$ ratio

For calculating **intermediate pile pressure**, use plate 1 (a)

Plate 1 (a)

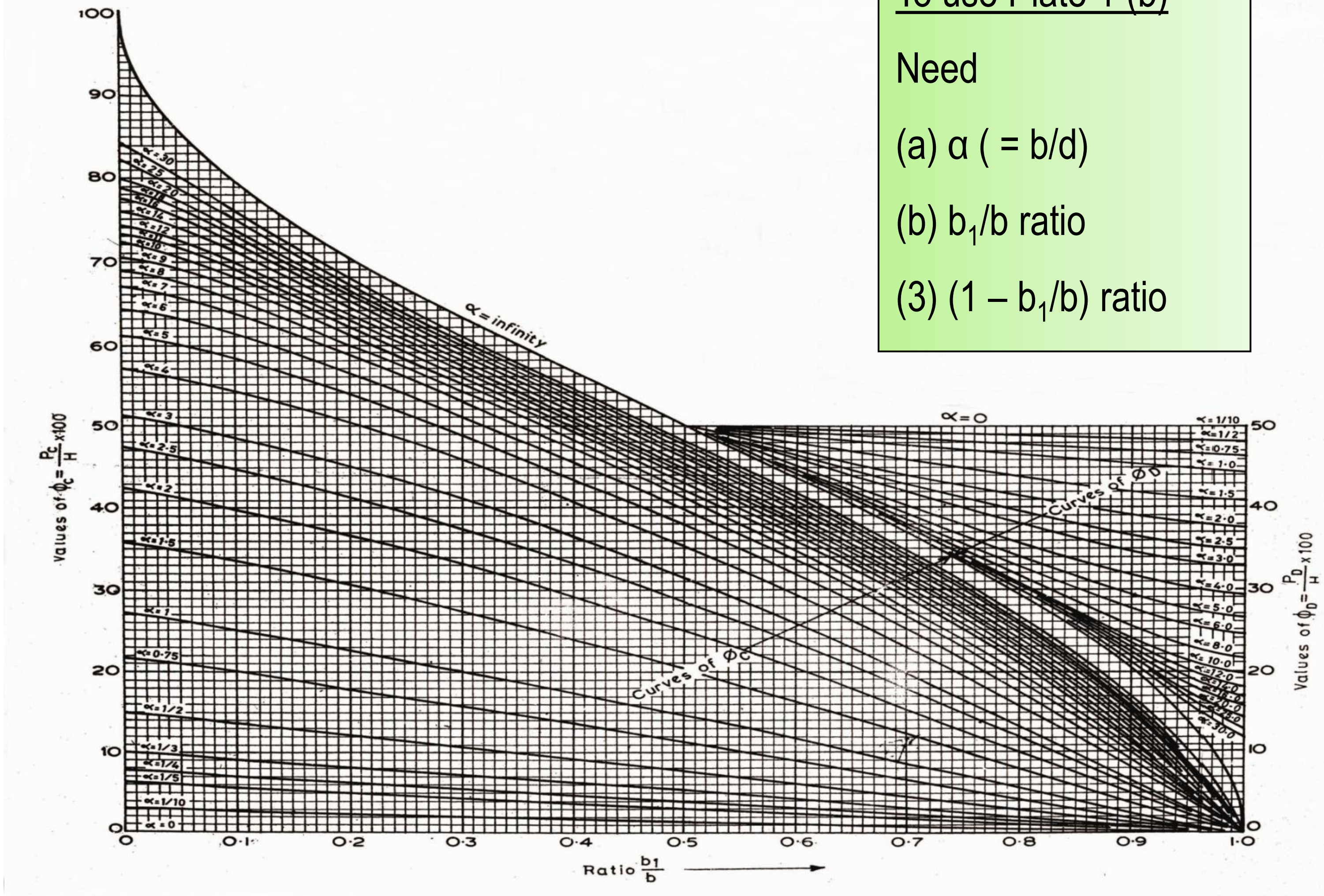


To use Plate 1 (a)

Need (a) $1/\alpha$ (= d/b)

For calculating **starting or end pile pressure**, use plate 1 (a)

Plate 1 (b)



To use Plate 1 (b)

Need

(a) α (= b/d)

(b) b_1/b ratio

(3) (1 – b_1/b) ratio

For calculating **exit gradient** , use Plate 2

Plate 2

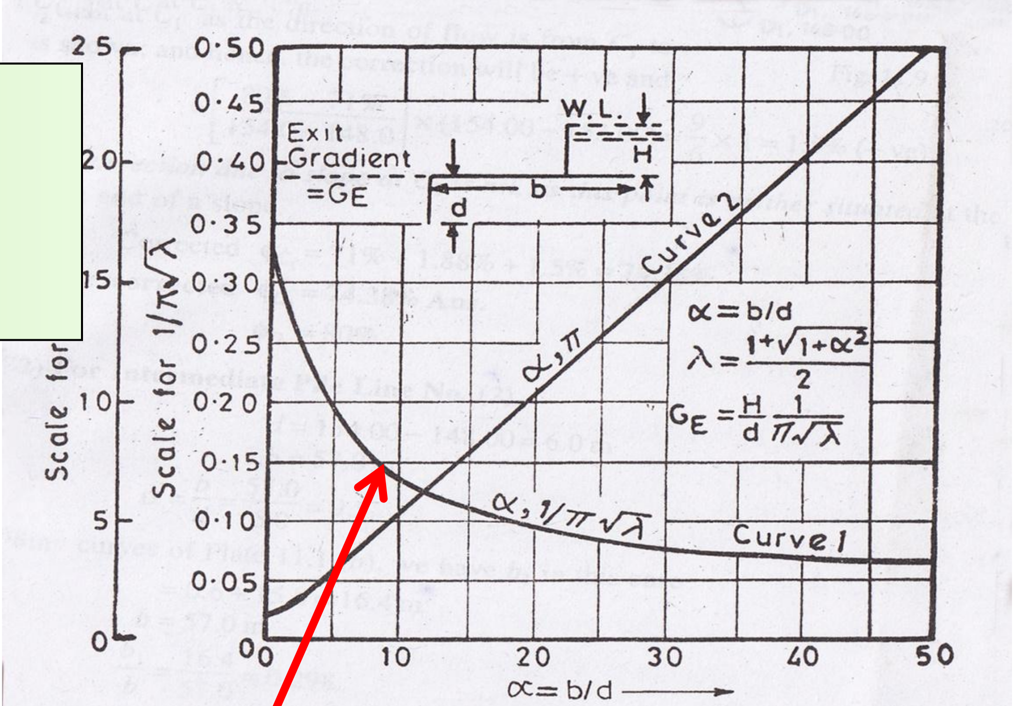


Plate 11.2

<u>To use Plate 2</u>

Need

α (= b/d) value and read curve 1 (1/π√λ)

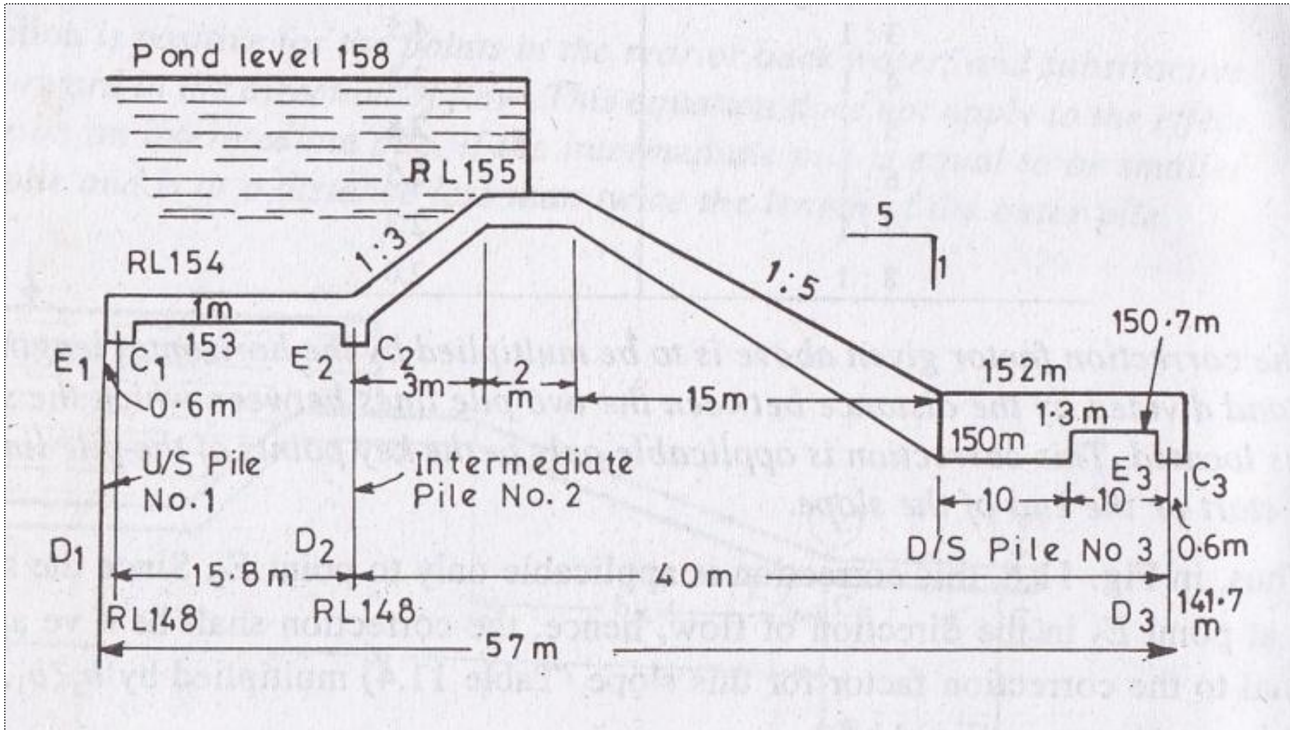
Exit Gradient:

$G_E = H/d \times 1/\pi\sqrt{\lambda}$

Curve 1

Plate 2

Lecture 14



Solution:

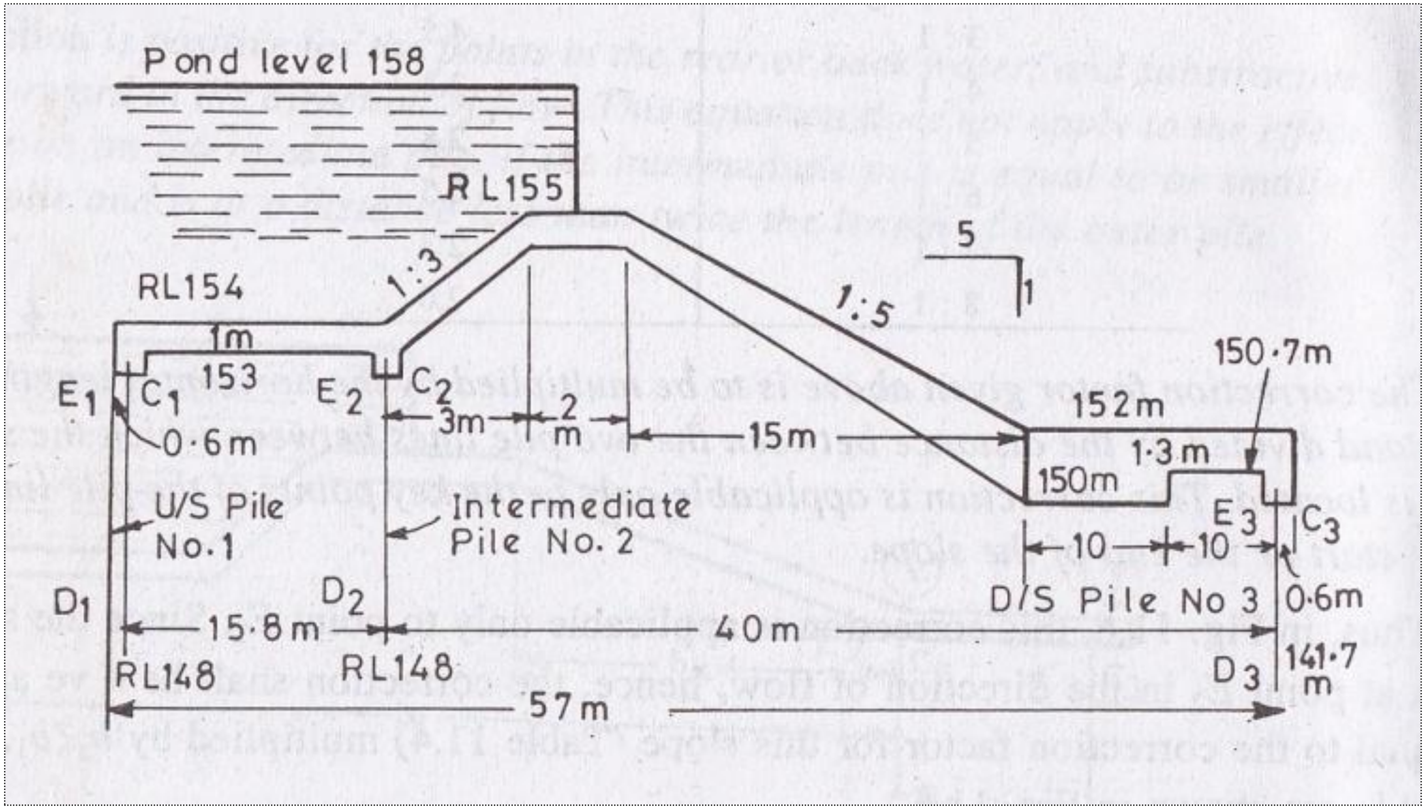
(1) For upstream Pile No. 1

Total length of the floor, b = 57.0 m

Depth of u/s pile line, d = 154 – 148 = 6 m

$\alpha = b/d = 57/6 = 9.5$

$1/\alpha = 1/9.5 = 0.105$



Formula for determining key points pressure at Pile 1:

$$\varphi_{E1} = 0$$

$$\varphi_{C1} = 100 - \varphi_E$$

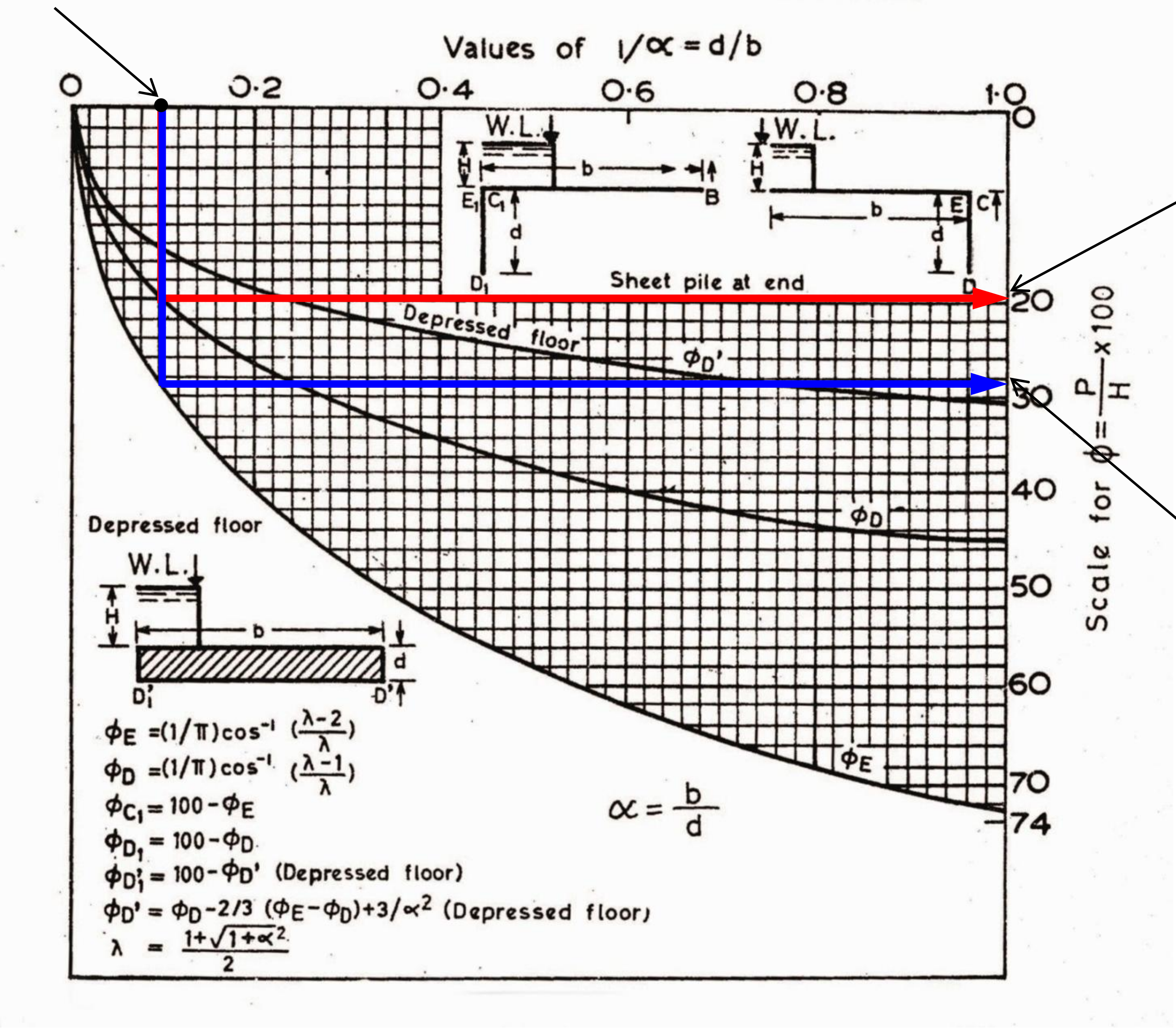
$$\varphi_{D1} = 100 - \varphi_D$$

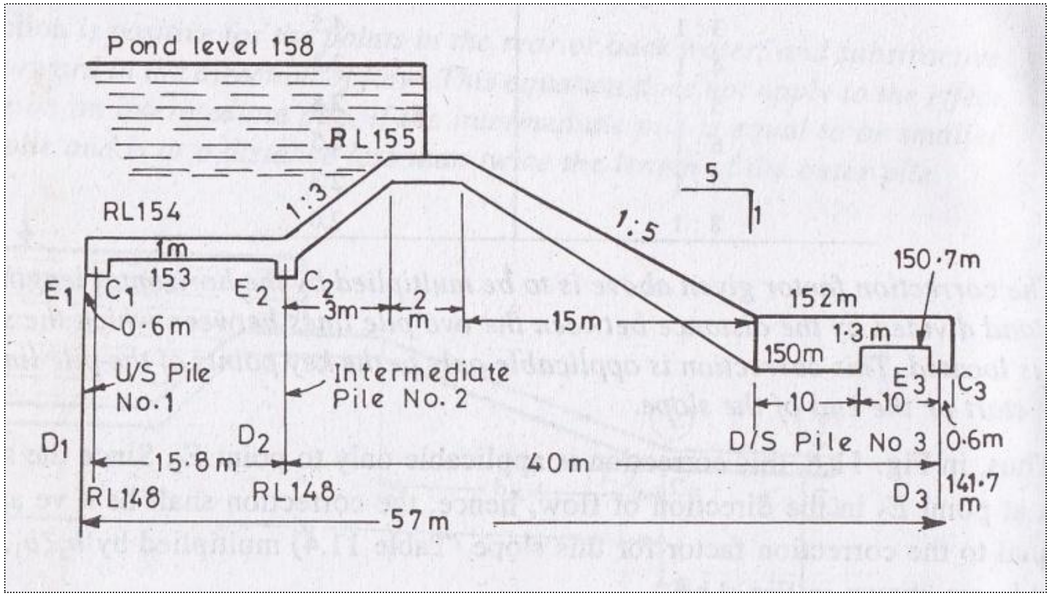
Plate 1 (a)

0.105

φ_D = 20 %

φ_E = 29 %





From plate **1 (a)**

$\varphi_E = 29\ \%$

$\varphi_D = 20\ \%$

Now,

$\varphi_{C1} = 100 - \varphi_E = 100 - 29 = 71\ \%$

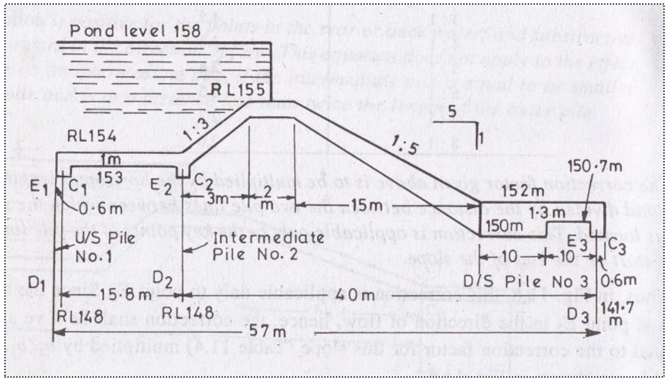
$\varphi_{D1} = 100 - \varphi_D = 100 - 20 = 80\ \%$

Now

Corrections

At point **C1** only

Corrections for φ_{C1}



(a) Mutual Interference of Piles

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

$$= 19\sqrt{\frac{5}{15.8}}\times\left(\frac{5+5}{57}\right)$$

$$= 1.88\ \%$$

Where, D = Depth of pile No.2

= 153 – 148 = 5 m

d = Depth of pile No. 1 = 153 – 148 = 5 m

b' = Distance between two piles = 15.8 m

b = Total floor length = 57 m

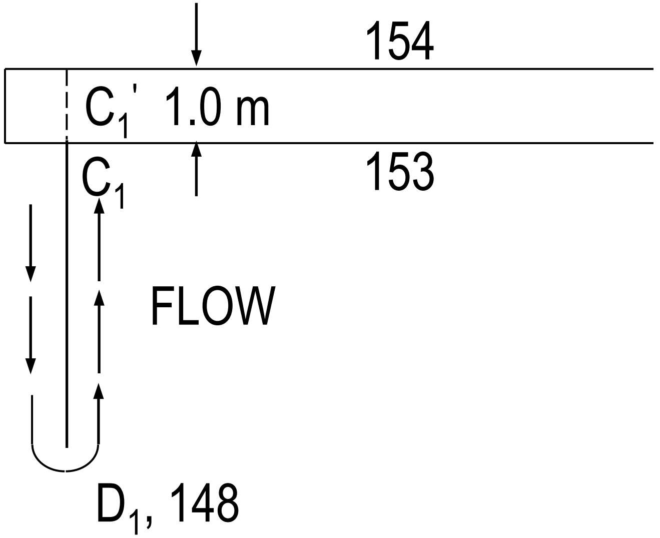
$\therefore$ Correction due to pile interference on C_1 = 1.88 % (+ ve)

(b) Correction at C_1 due to thickness of floor:

$$= \left[\frac{80\% - 71\%}{154 - 148}\right] (154 - 153)$$

$$= \left[\frac{9}{6}\right] 1$$

= 1.5% (+ ve)



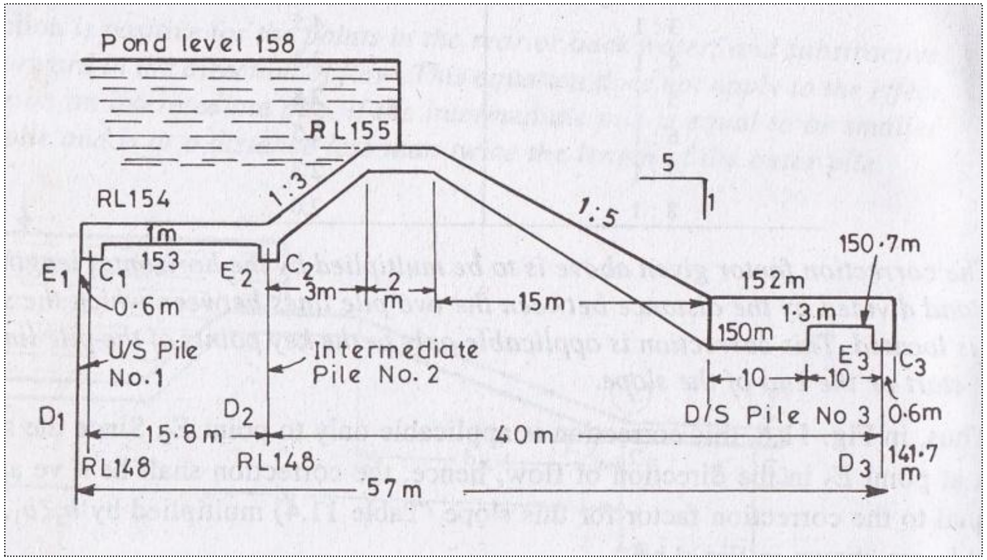


Fig: 5.1

(c) Correction due to slope at C_1 is nil

$$\therefore \text{ Corrected } (\varphi_{C1}) = 71\ \% + 1.88\ \% + 1.5\ \%$$

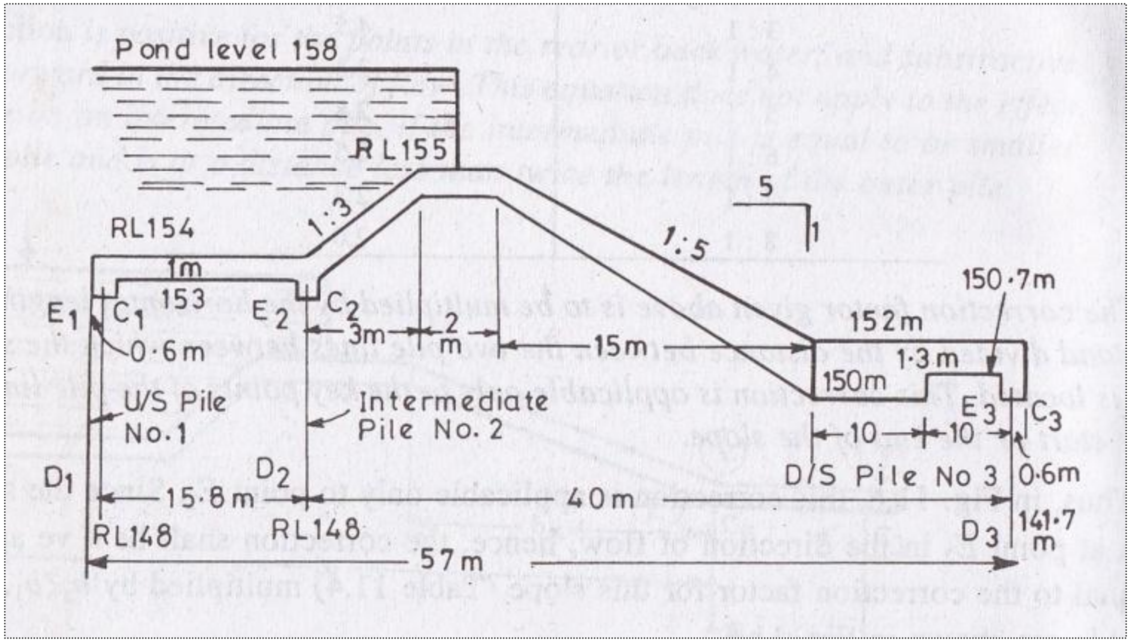
$$= 74.38\ \%$$

After Corrections (For Pile No.1)

$$\therefore\ \varphi_{E1} = 100\ \%$$

$$\therefore\ \varphi_{D1} = 80\ \%$$

$$\therefore\ \varphi_{C1} = 74.38\ \%$$



(2) For upstream Pile No. 2

b = 57.0 m

d = 154 – 148 = 6 m

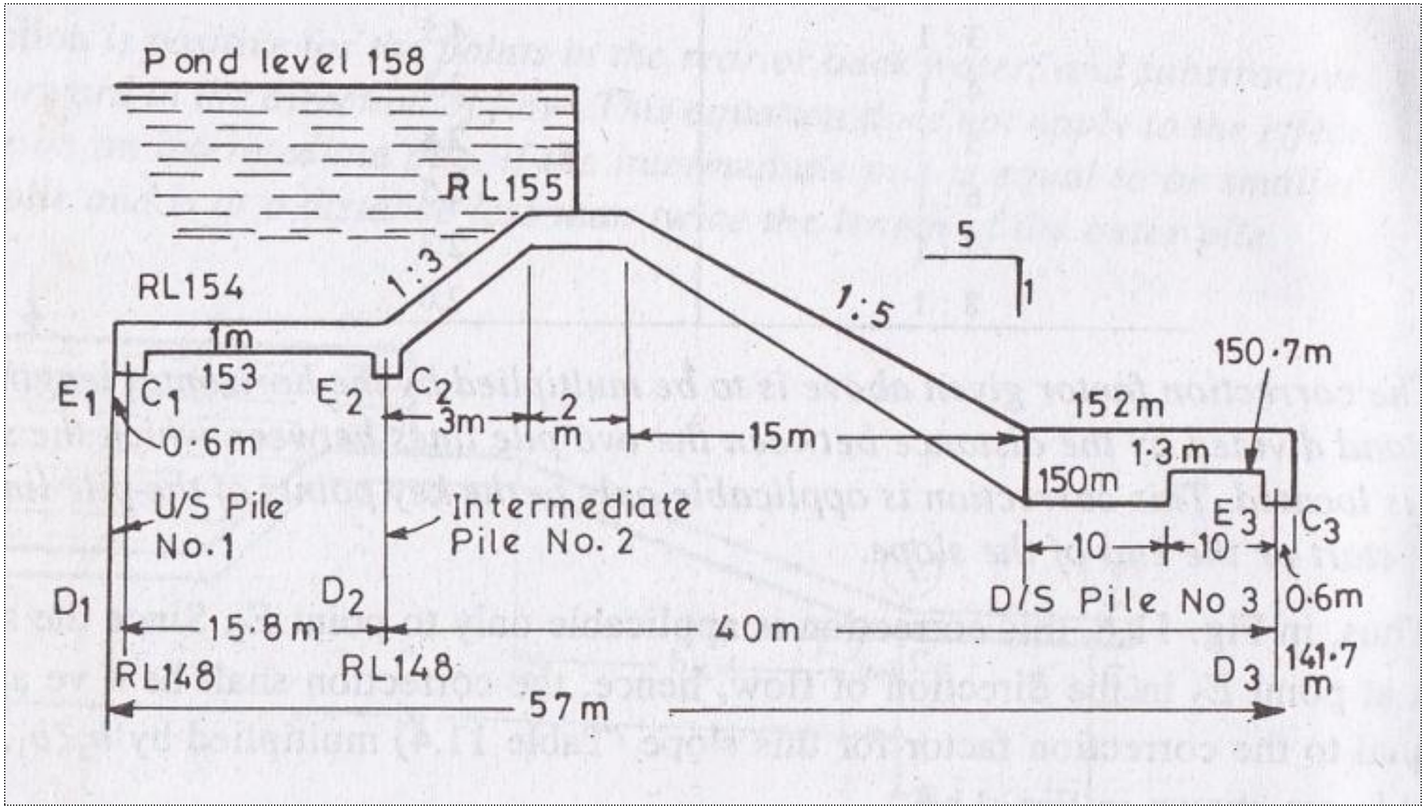
α = b/d = 57/6 = 9.5

b_1 = 0.6 + 15.8 = 16.4

b = 57 m

∴ b_1/b = 16.4/57 = 0.298 (b_1/b = base ratio)

1 – b_1/b = 1 – 0.298 = 0.702



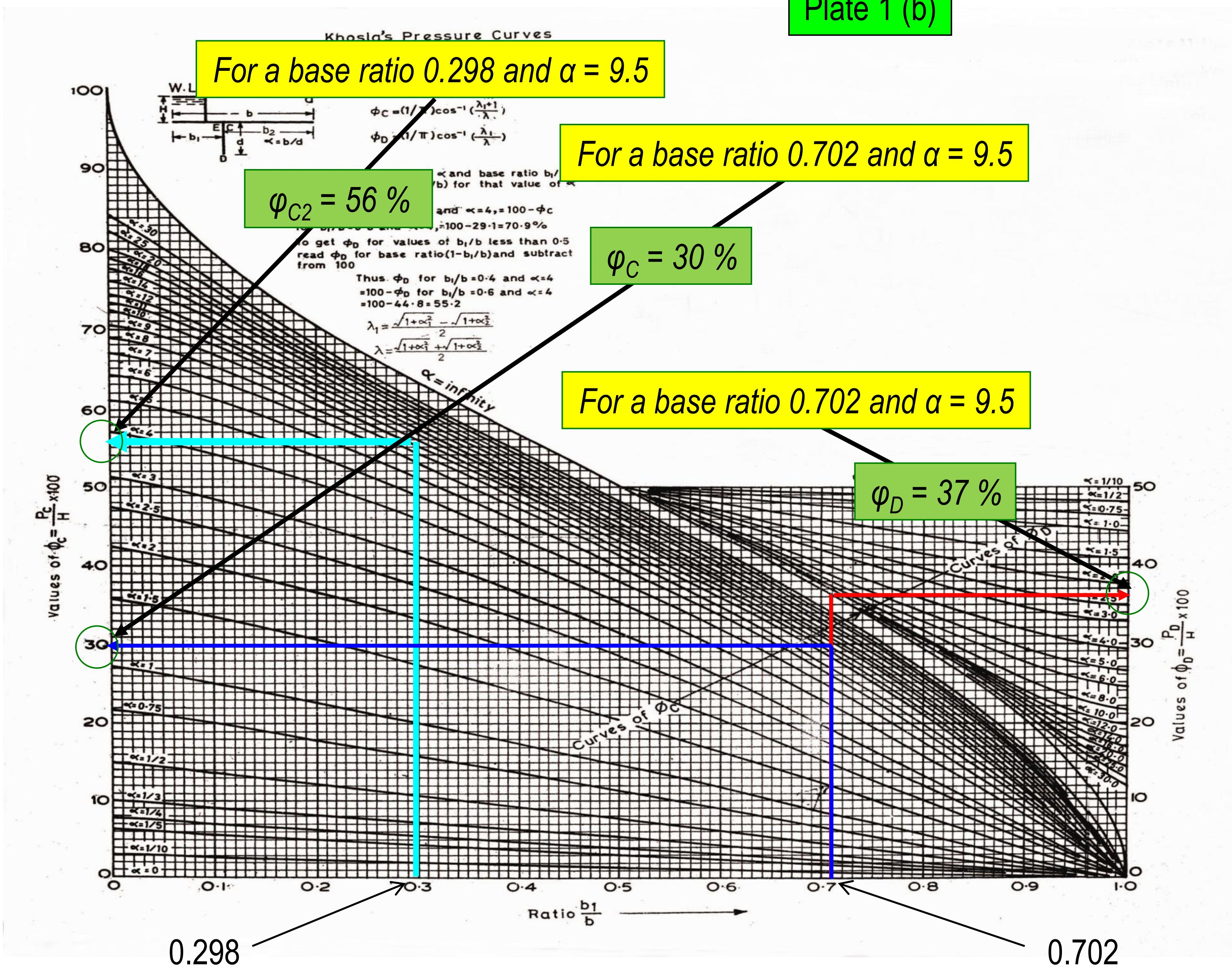
Formula for determining key points pressure at Pile 2:

φ_{E2} = 100 – φ_C **(1 – b_1/b value & α)**

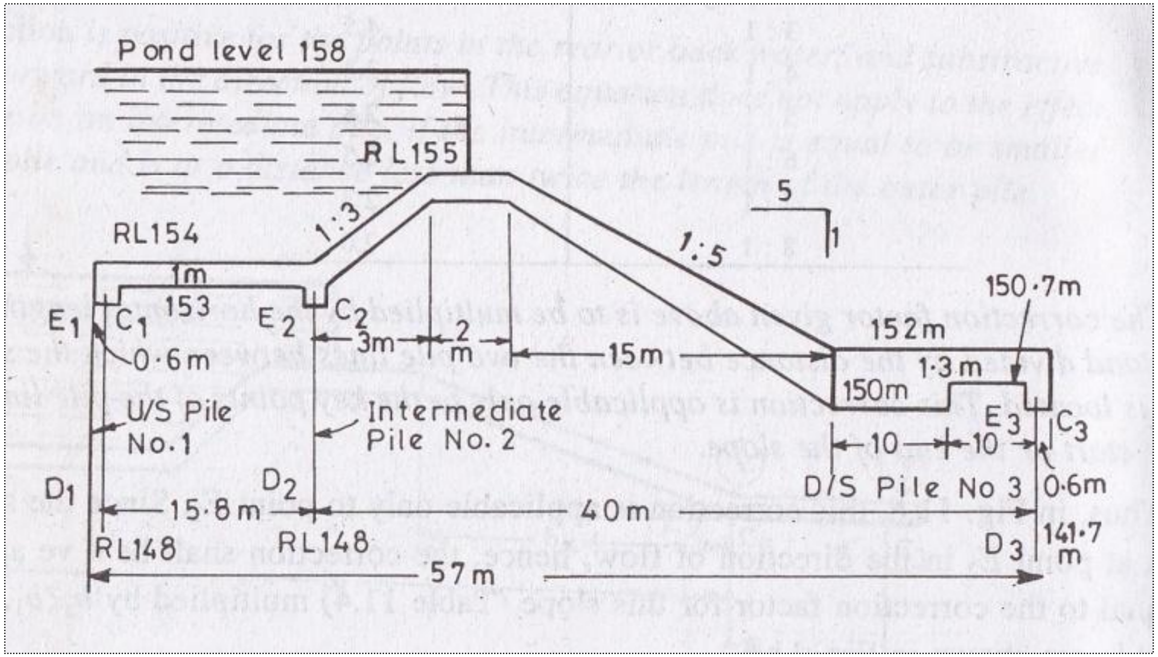
φ_{C2} = Direct value from chart **(b_1/b value & α)**

φ_{D2} = 100 – φ_D **(1 – b_1/b value & α)**

Plate 1 (b)



Khosla's Pressure Curves
For a base ratio 0.298 and α = 9.5
φ_{C2} = 56 %
For a base ratio 0.702 and α = 9.5
φ_C = 30 %
For a base ratio 0.702 and α = 9.5
φ_D = 37 %
To get ϕ_D for values of b_1/b less than 0·5 read ϕ_D for base ratio $(1-b_1/b)$ and subtract from 100
Thus ϕ_D for b_1/b = 0·4 and α = 4 = 100 − ϕ_D for b_1/b = 0·6 and α = 4 = 100 − 44·8 = 55·2
α = infinity
Curves of ϕ_C
Curves of ϕ_D
Values of $\phi_C = \frac{P_C}{H} \times 100$
Values of $\phi_D = \frac{P_D}{H} \times 100$
Ratio $\frac{b_1}{b}$
0.298
0.702



From plate **1 (b)**

$\varphi_C = 30\ \%$

$\varphi_D = 37\ \%$

Now,

$\varphi_{C1} = 100 - \varphi_C = 100 - 30 = 70\ \%$

$\varphi_{C2} = 56\%$

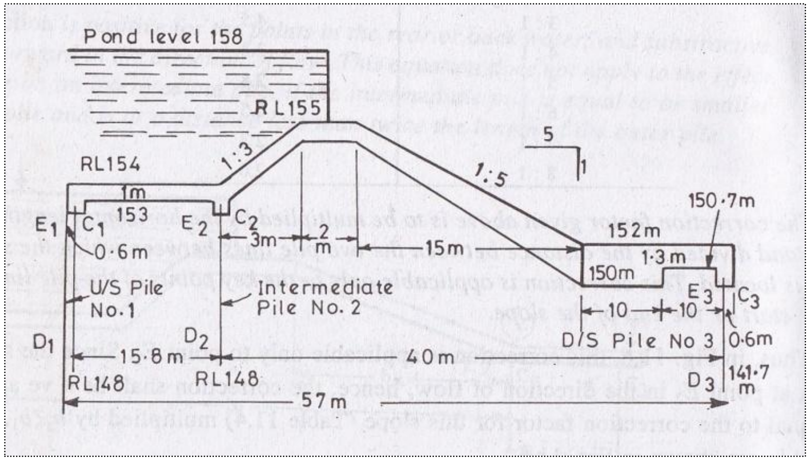
$\varphi_{D1} = 100 - \varphi_D = 100 - 37 = 63\ \%$

Now

Corrections

At points **E2** & **C2**

Corrections for φ_{E2}



(a) Mutual Interference of Piles

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

$$= 19\sqrt{\frac{5}{15.7}}\times\left(\frac{5+5}{57}\right)$$

= 1.88 % (-) ve

Where, D = Depth of pile No.1, the effect of which is considered

= 153 – 148 = 5 m

d = Depth of pile No. 2, the effect on which is considered

= 153 – 148 = 5 m

b' = Distance between two piles

= 15.8 m

b = Total floor length = 57 m

(b) Thickness correction (φ_{E2})

$$= \frac{\text{Obs } \varphi_{E2} \text{ - Obs } \varphi_{D2}}{\text{Distance between } E_2D_2} \text{ Thickness of floor}$$

$$= \left[\frac{70\% - 63\%}{154 - 148}\right] 1.0 = (7/6) \;\; 1.0 = 1.17 \;\%$$

This correction is **negative,**

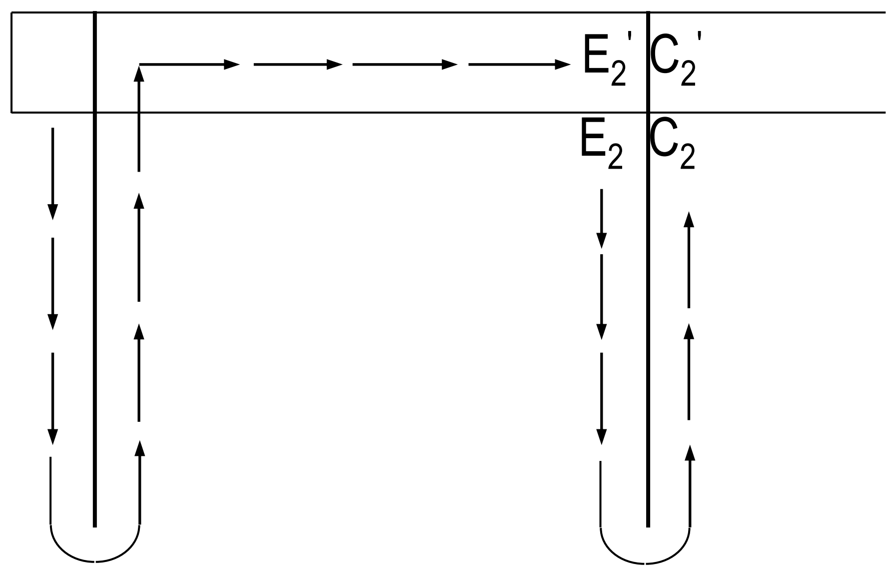


Fig: 5.2

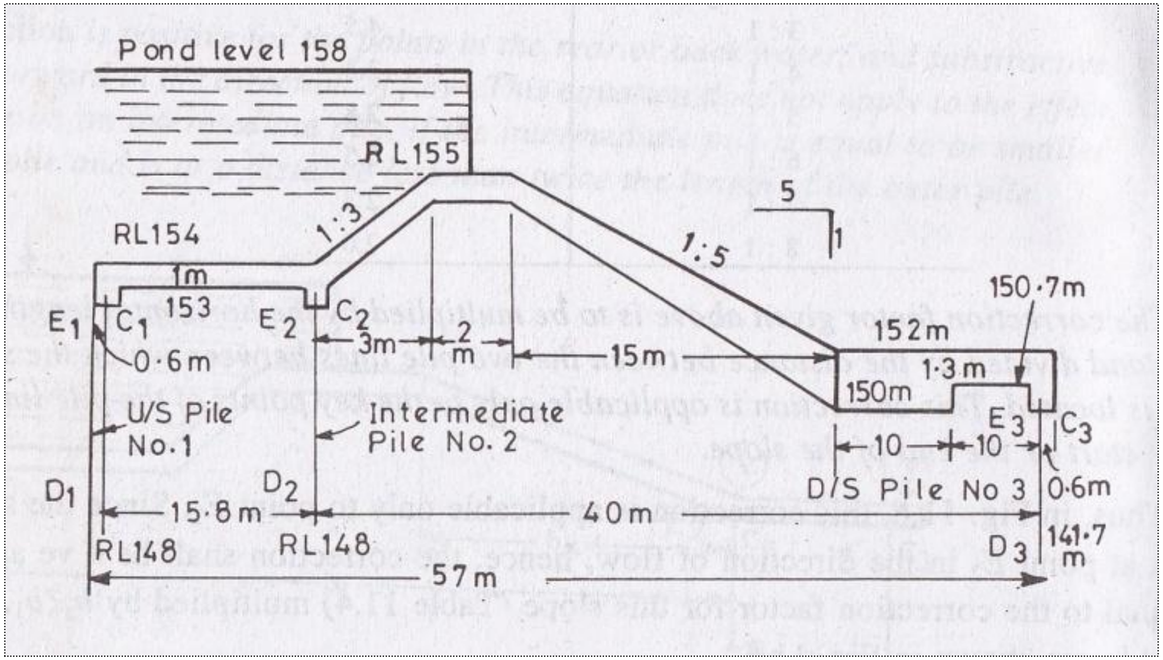
(c) Correction due to slope

Slope correction at E_2 due to slope is nil

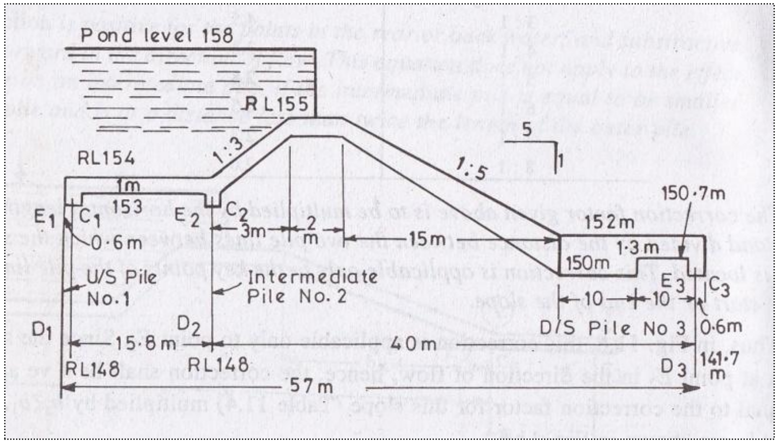
Hence, corrected percentage pressure at E_2

= Corrected φ_{E2}

= (70 – 1.88 – 1.17) % = 66.95 %



Corrections for φ_{C2}



(a) Mutual Interference of Piles

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

$$= 19\sqrt{\frac{11}{40}}\times\left(\frac{11+5}{57}\right)$$

= 2.89 % (+) ve

Where, D = Depth of pile No.3, the effect of which is considered

= 153 – 141.7 = 11.3 m

d = Depth of pile No. 2, the effect on which is considered

= 153 – 148 = 5 m

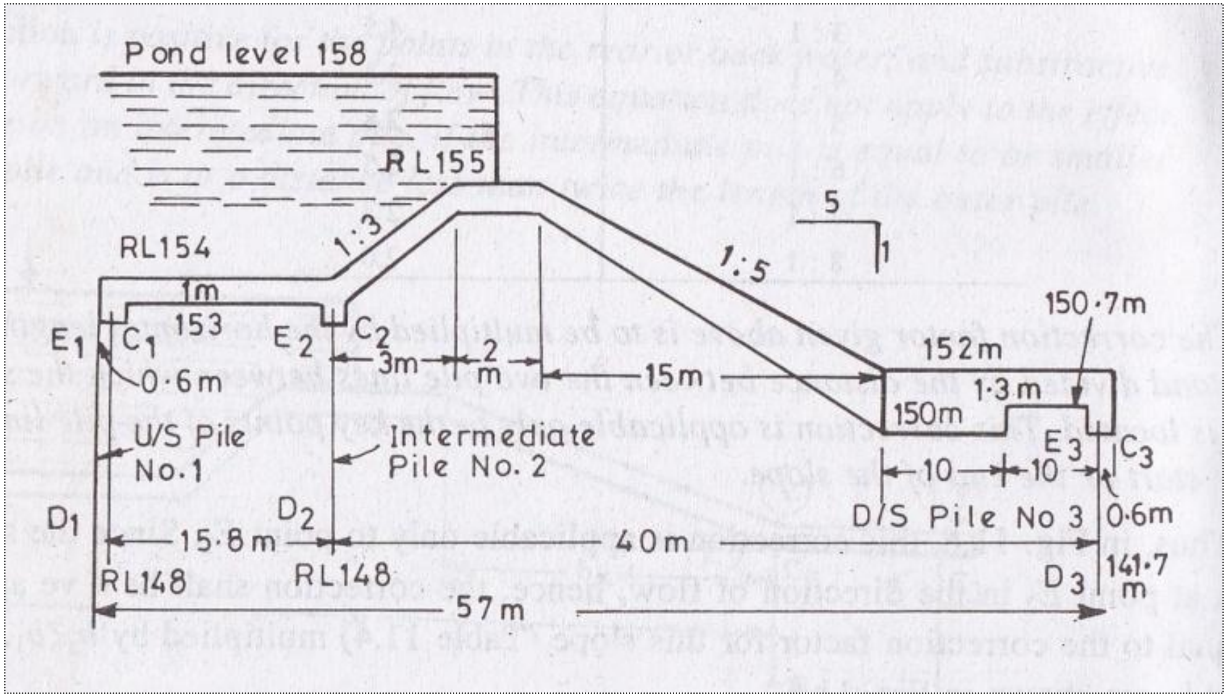
b' = Distance between two piles (2 &3)

= 40 m

b = Total floor length = 57 m

(b) Correction at C_2 due to floor thickness.

Correction at C_2 due to floor thickness = 1.17 % (+ ve)



(c) Correction at C_2 due to slope.

Correction factor for 3:1 slope from Table 5.3 = 4.5

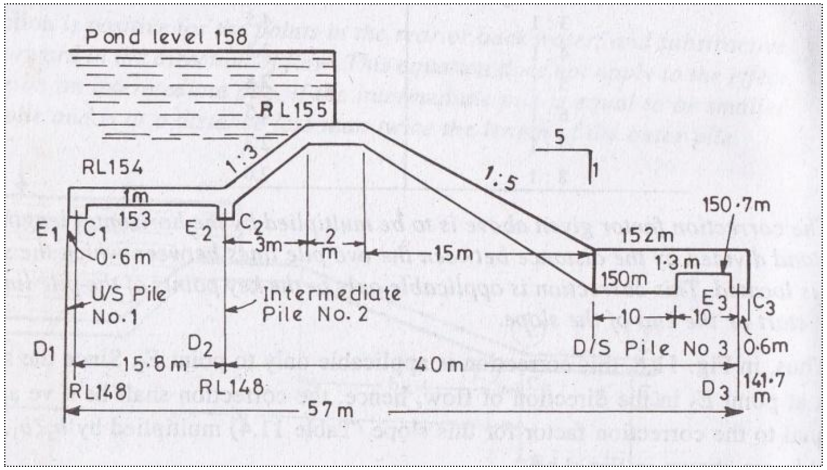
Horizontal length of the slope = 3 m

Distance between two pile lines between which the sloping floor is located = 40 m

$\therefore$ Actual correction = 4.5 (3/40) = 0.34 % (- ve)

Hence, corrected φ_{C2} = (56 + 2.89 + 1.17 – 0.34) %

= 59.72 %

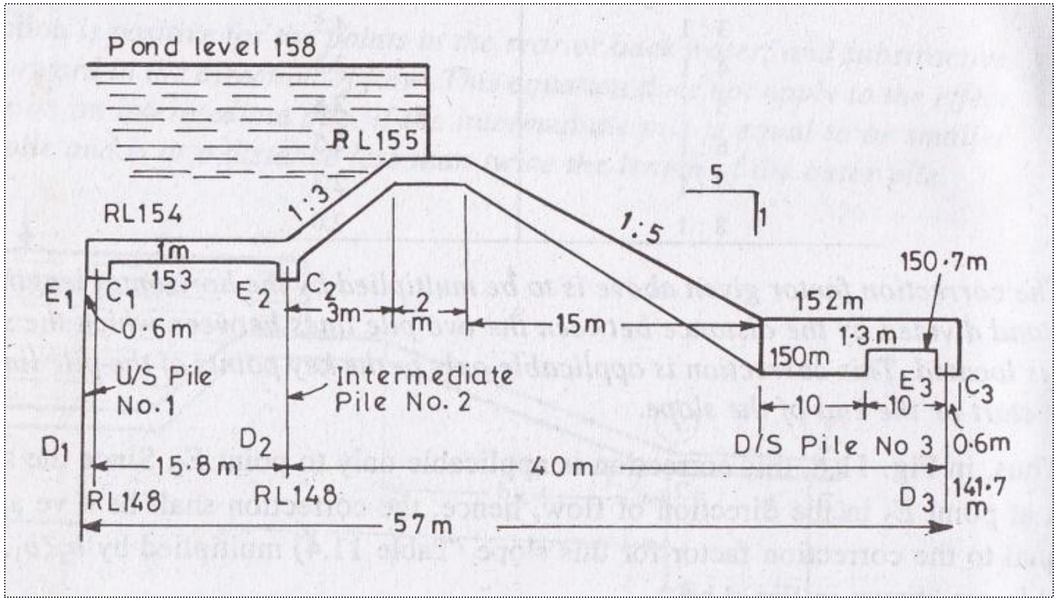


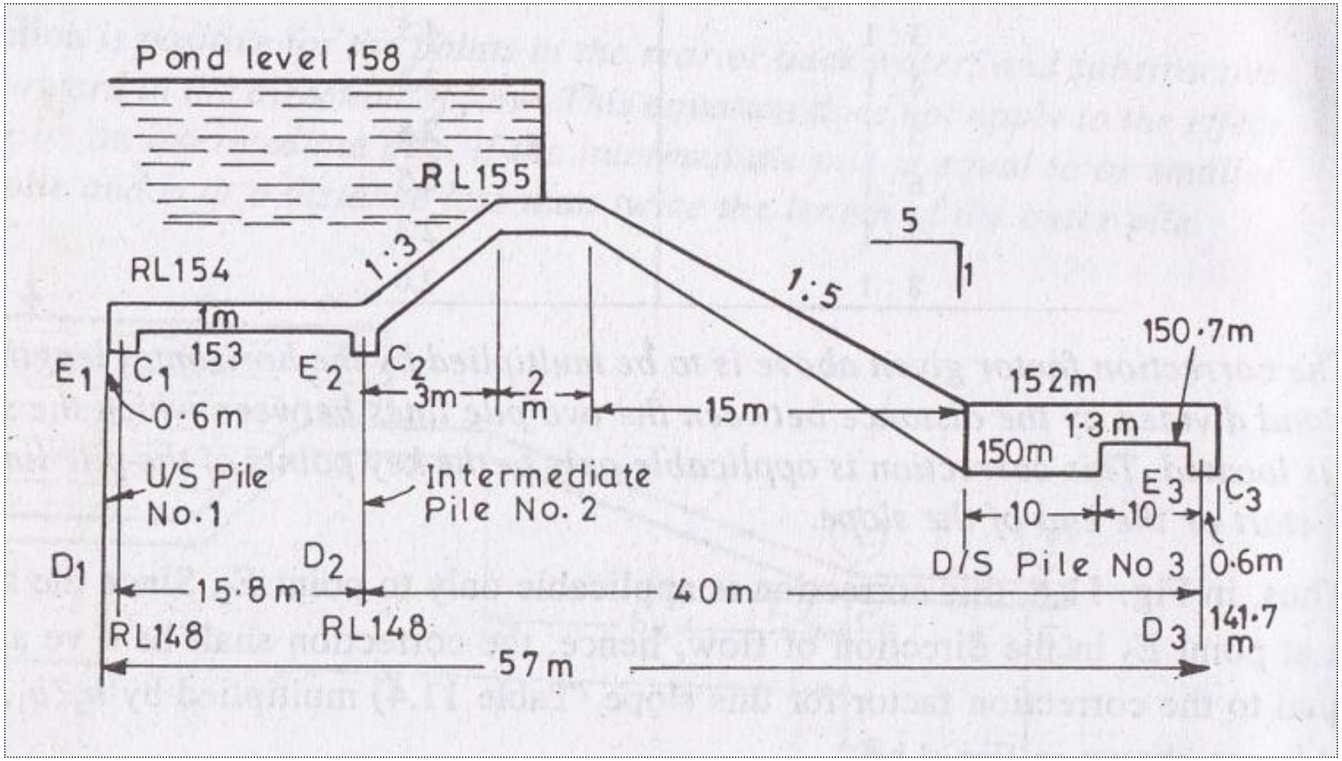
After Corrections (For Pile No.2)

$\therefore\ \varphi_{E2} = 66.95\ \%$

$\therefore\ \varphi_{D2} = 56\ \%$

$\therefore\ \varphi_{C2} = 59.72\ \%$



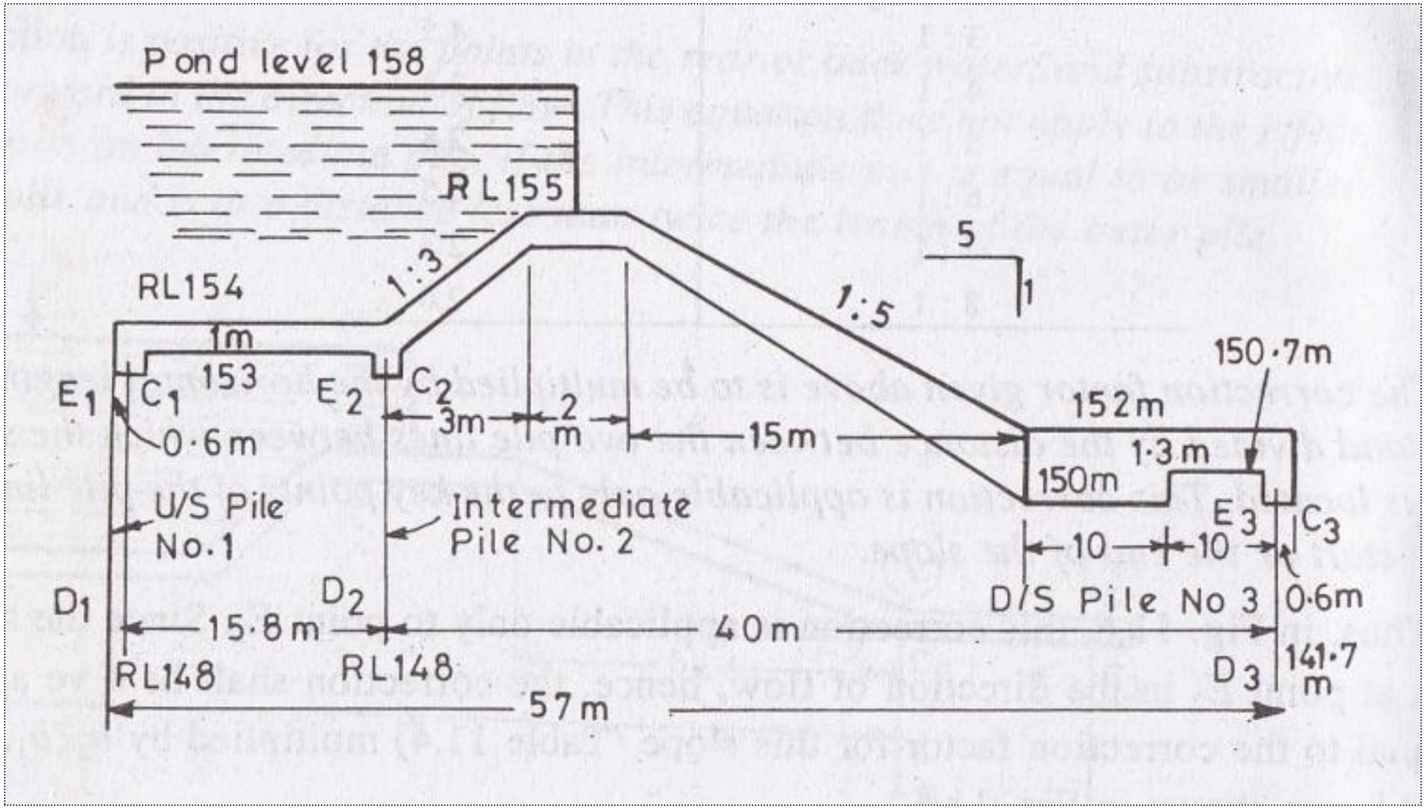


(3) For upstream Pile Line No. 3

$b = 57$ m

$d = 152 - 141.7 = 10.3$ m

$1/\alpha = d/b = 10.3/57 = 0.181$



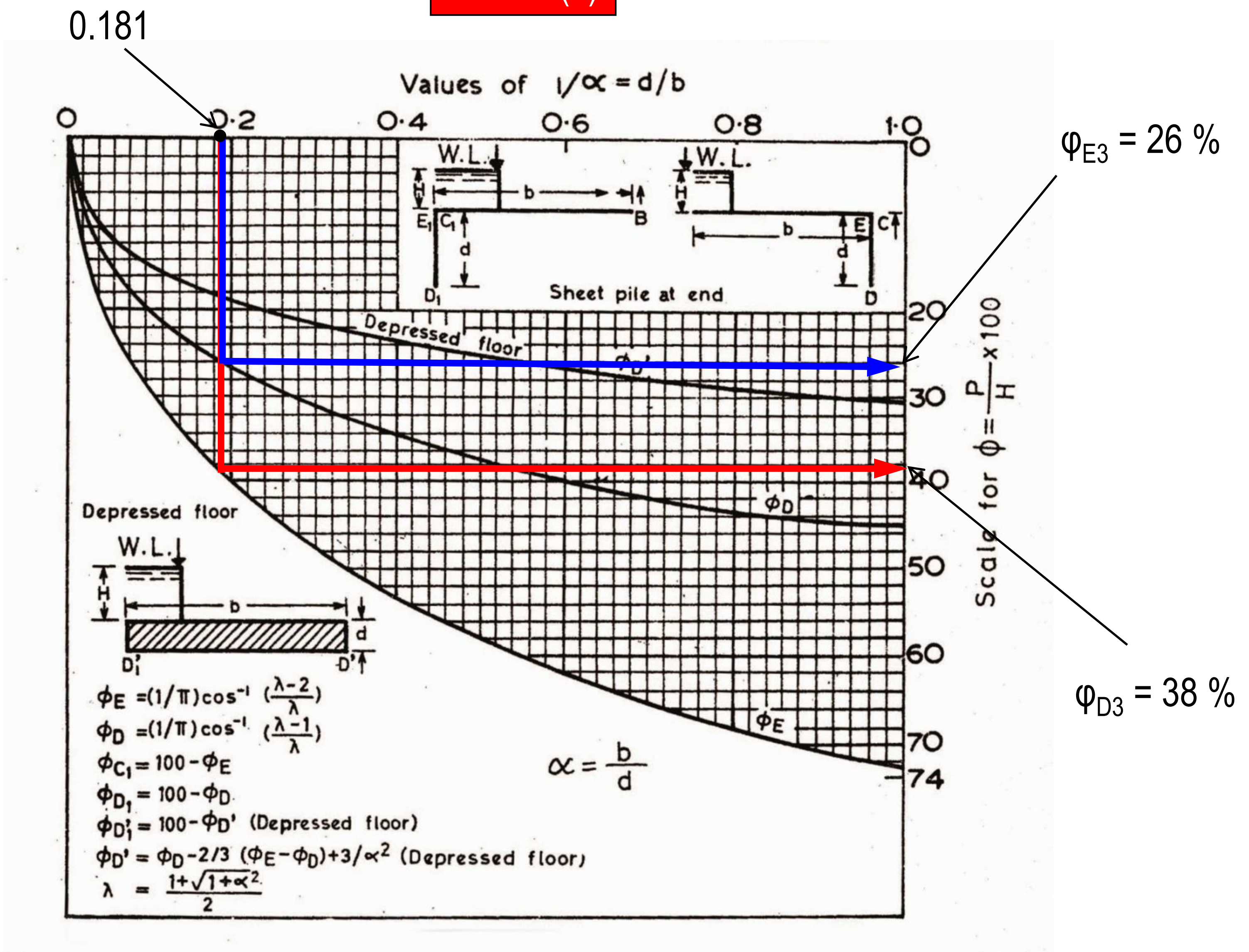
Formula for determining key points pressure at Pile 3:

φ_{E3} = Direct value from chart **(1/ α value)**

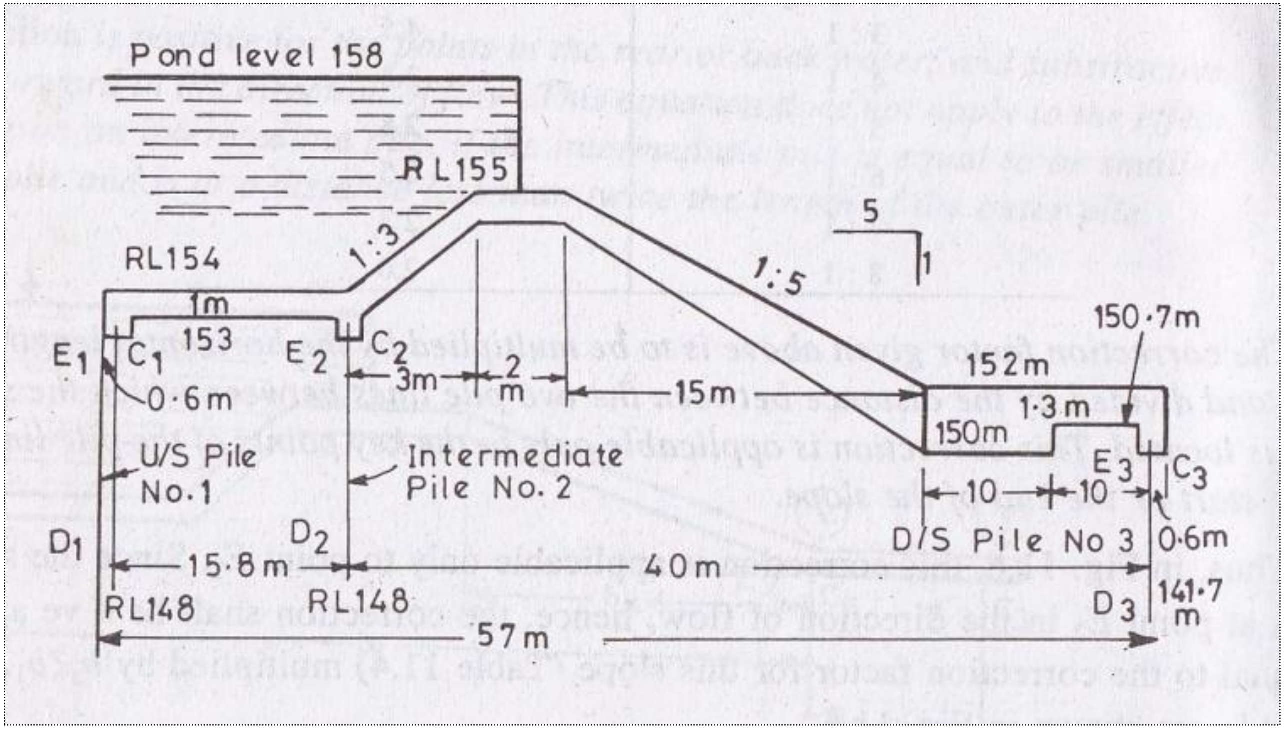
$\varphi_{C3} = 0$

φ_{D3} = Direct value from chart **(1/α value)**

Plate 1 (a)



0.181
Values of 1/α = d/b
0
0·2
0·4
0·6
0·8
1·0
W.L.
H
b
B
E1
C1
d
D1
W.L.
H
b
E
C
d
D
Sheet pile at end
Depressed floor
φD'
φD
φE
Depressed floor
W.L.
H
b
d
D1'
D'
φE =(1/π)cos⁻¹ ((λ−2)/λ)
φD =(1/π)cos⁻¹ ((λ−1)/λ)
φC1 = 100 − φE
φD1 = 100 − φD
φD1' = 100 − φD' (Depressed floor)
φD' = φD − 2/3 (φE − φD) + 3/α² (Depressed floor)
λ = (1+√(1+α²))/2
α = b/d
0
20
30
40
50
60
70
74
Scale for φ = P/H × 100
φE3 = 26 %
φD3 = 38 %



From plate **1 (a)**

$\varphi_{E3} = 38\ \%$

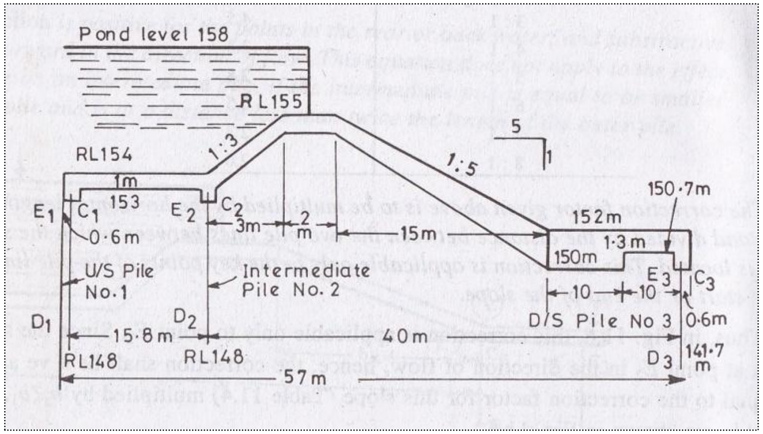
$\varphi_{D3} = 26\%$

Now

Corrections

At point **E3** **only**

Corrections for φ_{E3}



(a) Mutual Interference of Piles

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

$$= 19\sqrt{\frac{2.7}{40}} \times \left(\frac{9+2.7}{57}\right)$$

= 1.02 % (-) ve

Where,

D = Depth of pile No.2, the effect of which is considered = 150.7 – 148 = 2.7 m

d = Depth of pile No. 3, the effect on which is considered = 150 – 141.7 = 9 m

b' = Distance between two piles = 40 m

b = Total floor length = 57 m

(b) Correction due to floor thickness:

$$= \left[\frac{38\% - 32\%}{152 - 141.7}\right] 1.3$$

$$= \left[\frac{16}{10.3}\right] 1.3$$

= 0.76 % (- ve)

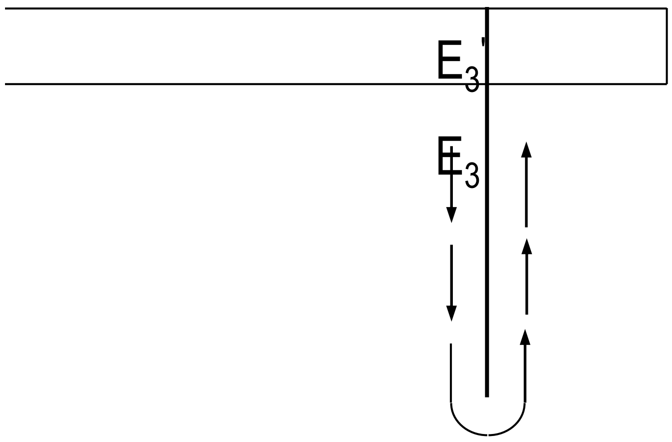


Fig: 7.3

(c) Correction due to slope at E_3 is nil,

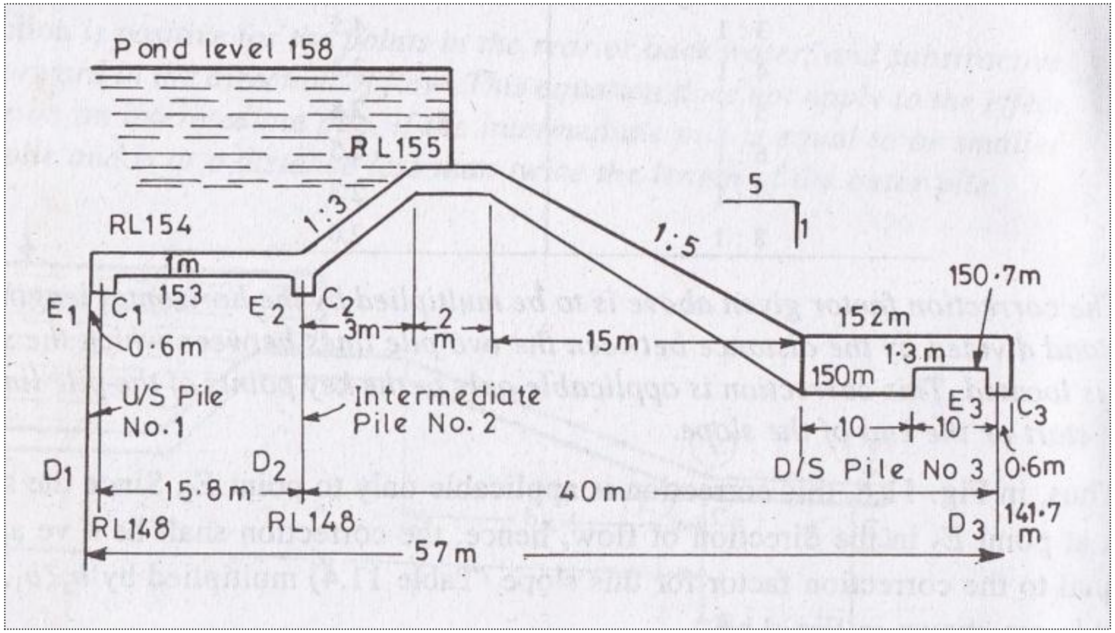
Hence, corrected φ_{E3} = (38 – 1.02 – 0.76) % = 36.22 %

After Corrections (For Pile No.3)

$$\therefore\ \varphi_{E3} = 36.22\ \%$$

$$\therefore\ \varphi_{D3} = 26\ \%$$

$$\therefore\ \varphi_{C3} = 0\ \%$$



The corrected pressures at various key points are tabulated below in Table below

Upstream Pile No. 1	Intermediate Pile No.2	Downstream Pile No. 3
φ_{E1} = 100 %	φ_{E2} = 66.95 %	φ_{E3} = 36.22 %
φ_{D1} = 80 %	φ_{D2} = 63 %	φ_{D3} = 26 %
φ_{C1} = 74.38 %	φ_{C2} = 59.72 %	φ_{C3} = 0 %

Exit gradient

The maximum seepage head, H = 158 – 152 = 6 m

The depth of downstream cur-off, d = 152 – 141.7 = 10.3 m

Total floor length, b = 57 m

$\alpha = b/d = 57/10.3 = 5.53$

For a value of $\alpha = 5.53$, $\dfrac{1}{\pi\sqrt{\lambda}}$ from curves of Plate 2 is equal to 0.18

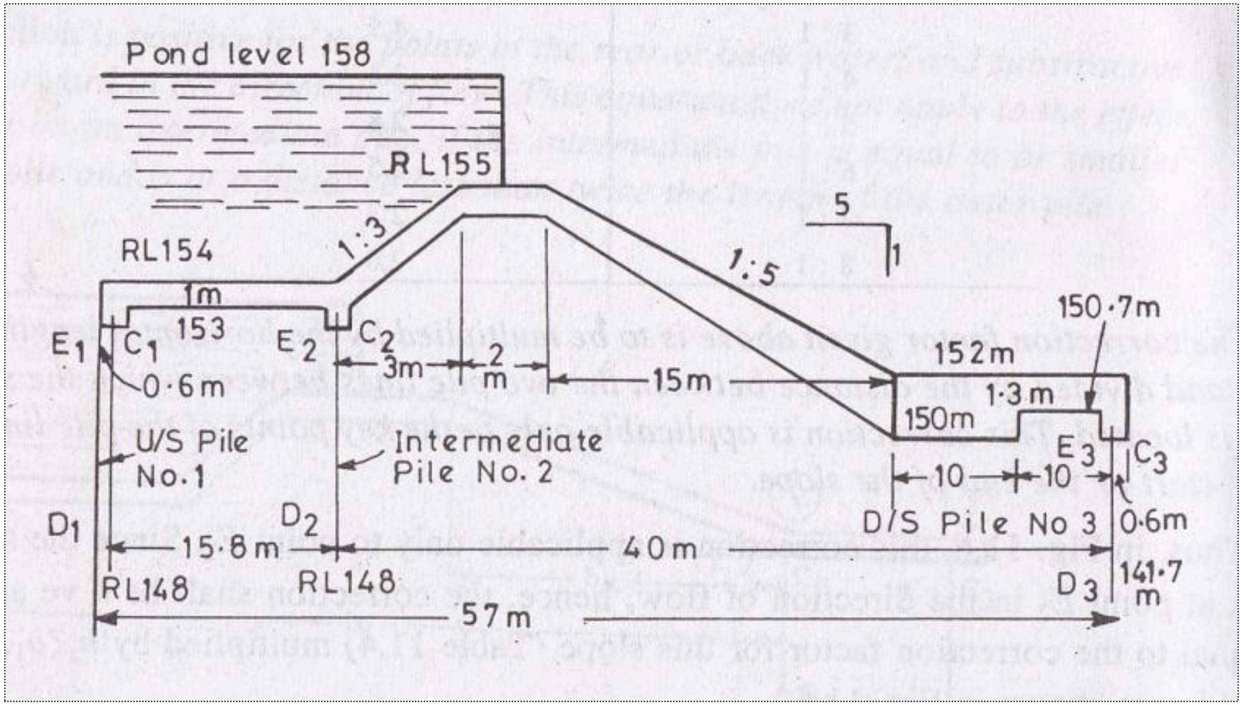
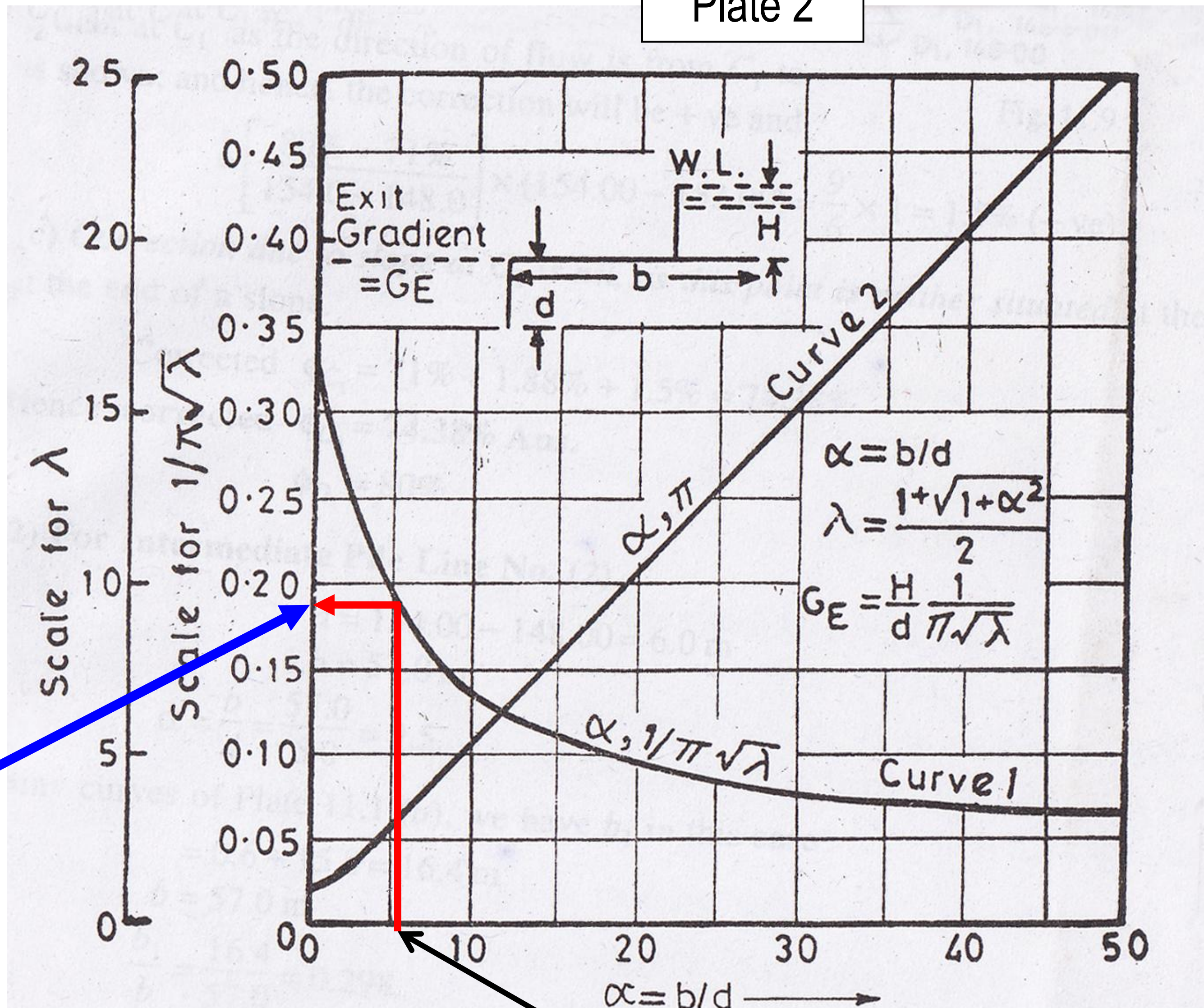


Plate 2



$\frac{1}{\pi\sqrt{\lambda}}$ = 0.18

α = 5.53

$$G_E = \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}} = \frac{6}{10.3} \quad 0.18 = 0.105$$

Hence, the exit gradient shall be equal to 0.105, *i.e.* 1 in 9.53, which is very much safe.

Type of Soil	**Safe exit gradient**
Shingle	1/4 to 1/5 (0.25 to 0.20)
Coarse Sand	1/5 to 1/6 (0.20 to 0.17)
Fine Sand	1/6 to 1/7 (0.17 to 0.14)

End of Chapter – 5

CHAPTER – 6

System of Irrigation Canal

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LECTURE 15

Alluvial and non-alluvial Canal

The soil which is formed by transportation and deposition of silt through the agency of water, over a course of time, is called the **alluvial soil.**

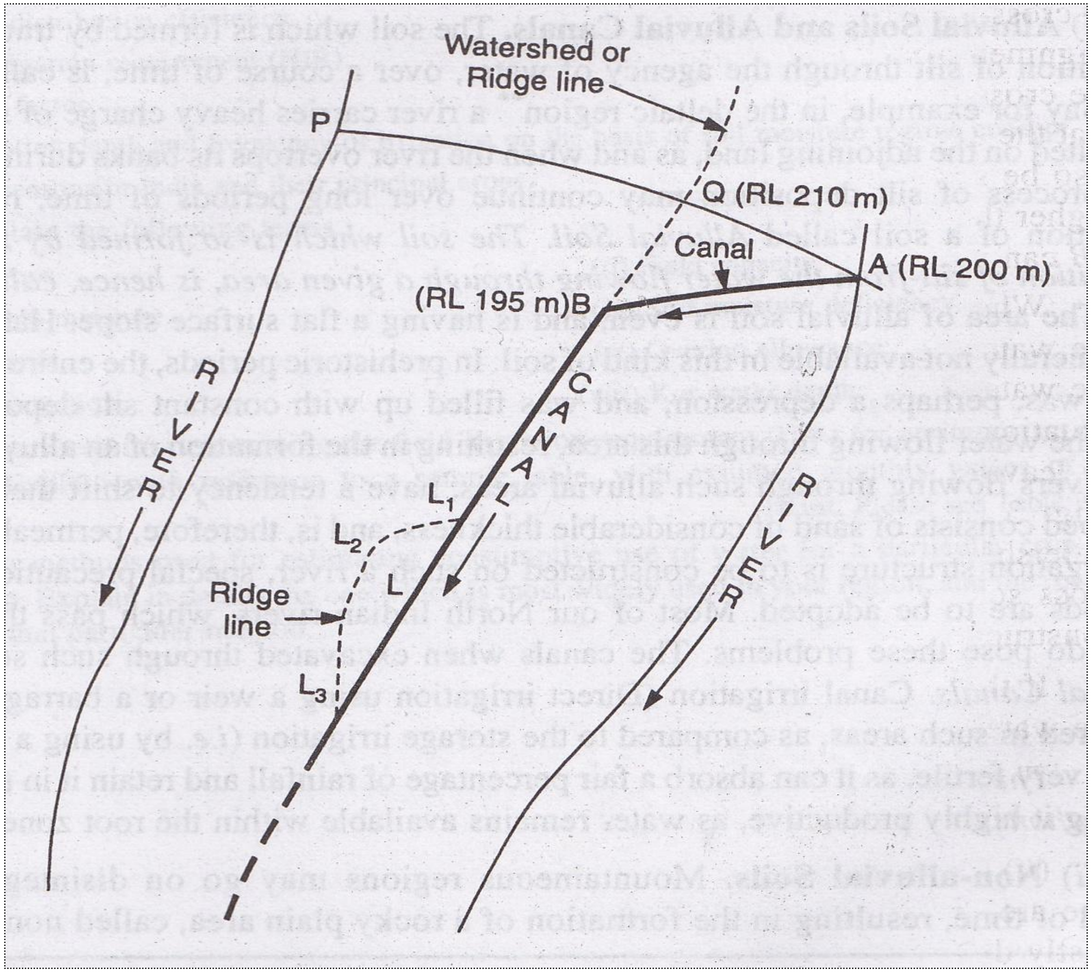
The canals when excavated through such soils are called **alluvial canals**. Canal irrigation (direct irrigation using a weir or a barrage) is generally preferred in such areas, as compared to the storage irrigation (i.e. by using a dam).

It has an uneven topography, and hard foundations are generally available. The rivers, passing through such areas, have no tendency to shift their courses, and they do not pose much problems for designing irrigation structures on them. Canals, passing through such areas are called **non-alluvial Canals**.

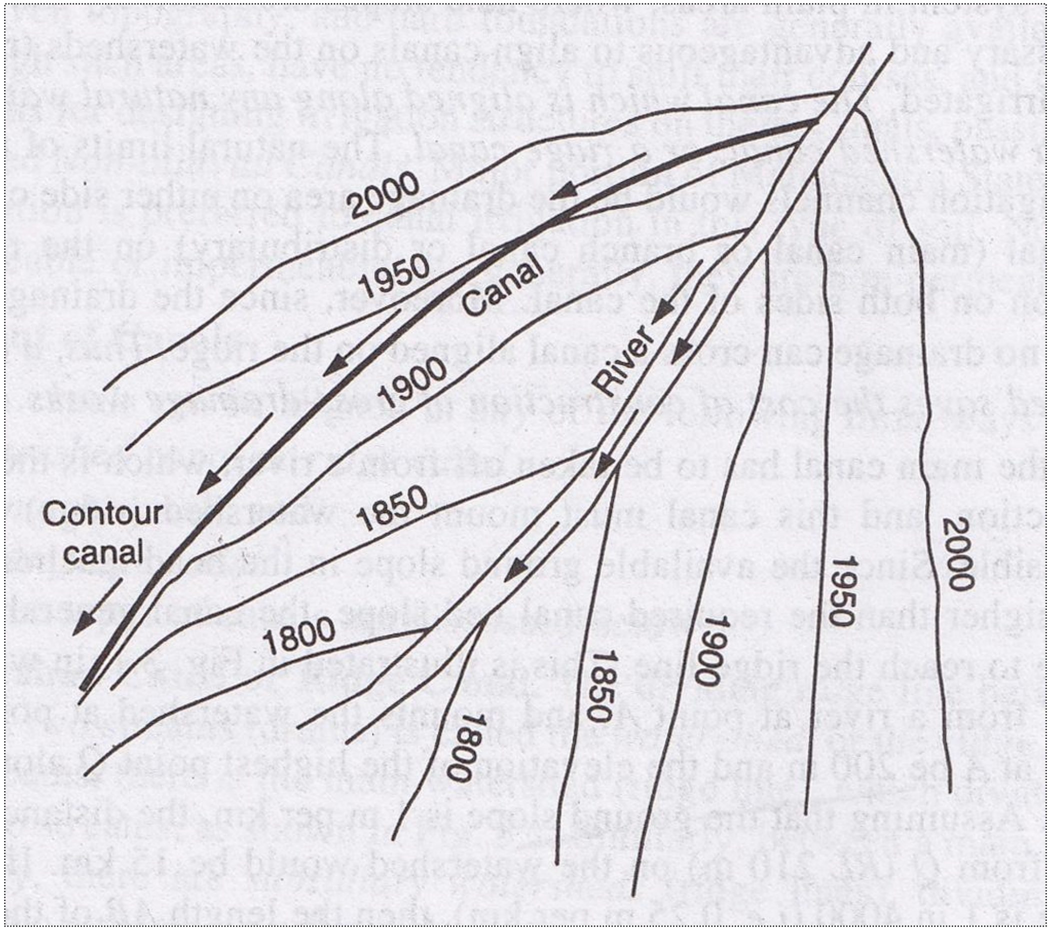
Alignment of Canals

- Water-shed Canal
- Contour Canal
- Side-slope Canal

Water-shed canal



Contour canal



Lecture 15

Side-slope canal

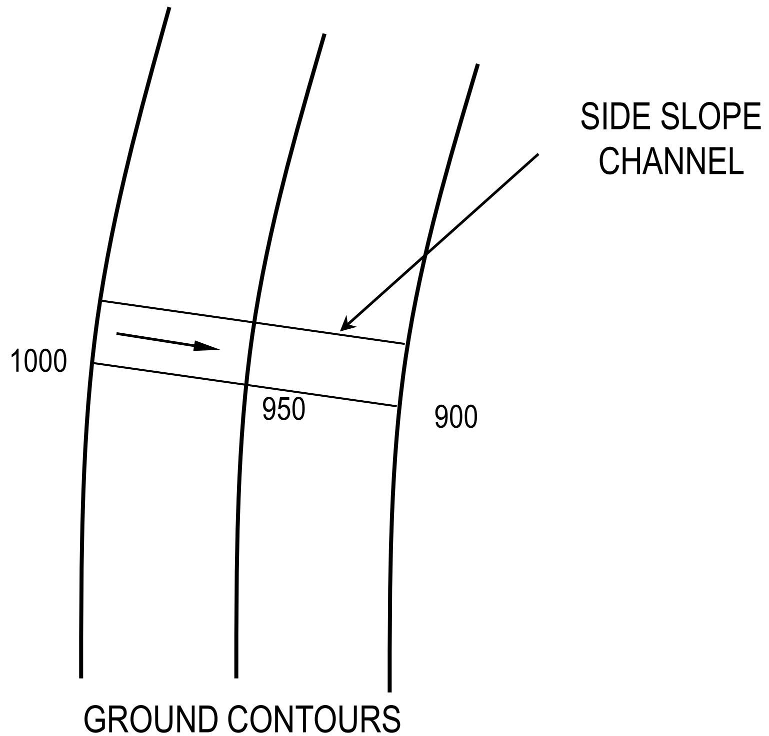
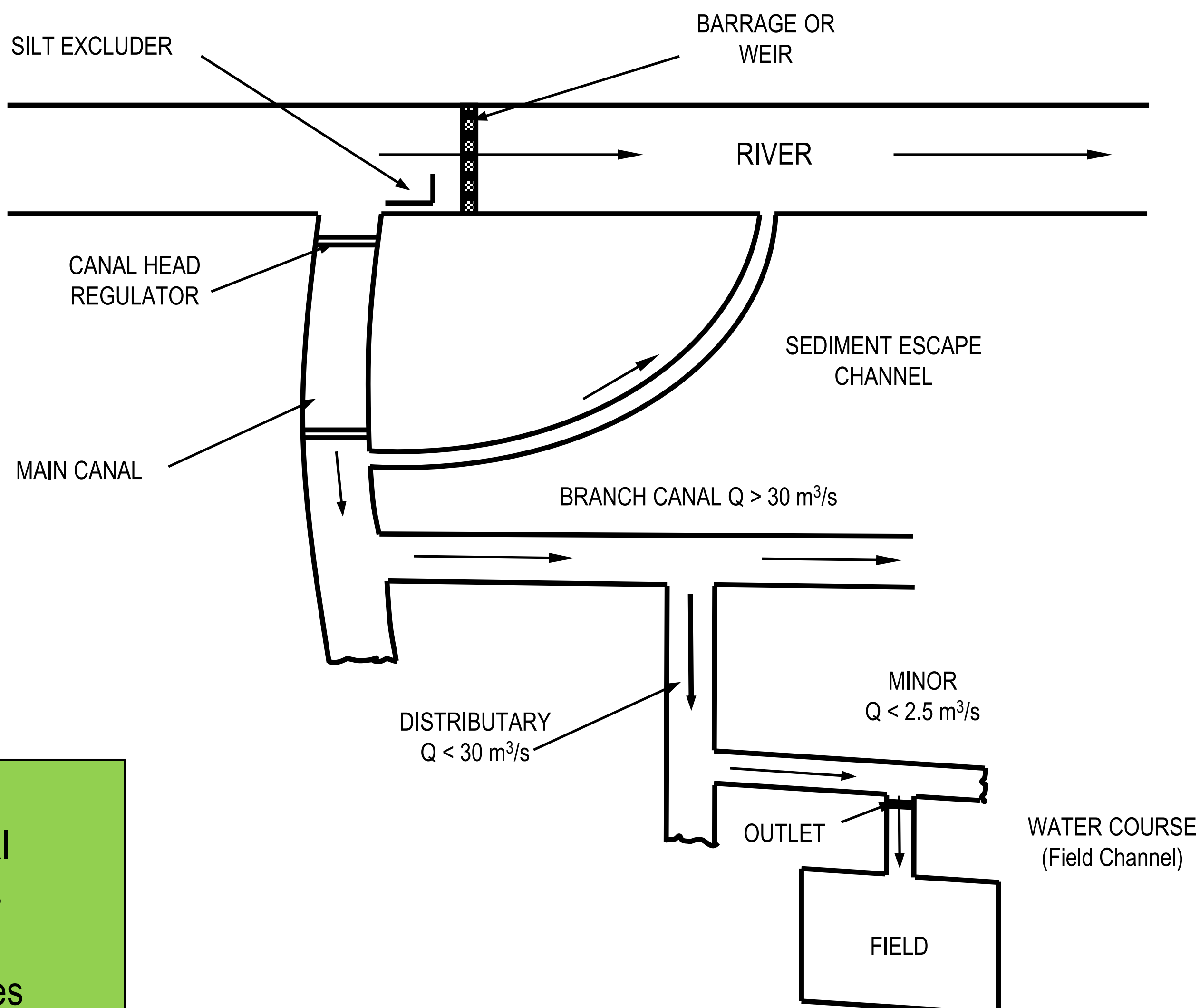


Fig: Alignment of a side slope canal

Distribution system for Canal Irrigation



Main Canal
Branch Canal
Distributaries
Minors
Water-courses

Definition of Important Terms

- **Gross Command Area (GCA)**

The whole area enclosed between an imaginary boundary line which can be included in an irrigation project for supplying water to agricultural land by the net work of canals is known as GCA. It includes both the culturable and unculturable areas.

- **Uncultivable Area**

The area where the agriculture can not be done and crops cannot be grown – marshy lands, barren lands, ponds, forest, villages etc. are considered as uncultivable area.

- **Cultivable Area**

The area where agriculture can be done satisfactorily

❑ Cultivable Command Area (CCA)

The total area within an irrigation project where the cultivation can be done and crops can be grown

❑ Intensity of Irrigation

Ratio of cultivated land for a particular crop to the total culturable command area

$$\therefore \text{Intensity of irrigation, } I_I = \frac{\text{Cultivated Land}}{\text{CCA}}$$

Time factor and capacity factor

Time Factor

The ratio of the number of days the canal has actually been kept open to the number of days the canal was designed to remain open during the base period is known as **time factor**.

For example, a canal was designed to be kept open for 12 days, but it was practically kept open for 10 days for supplying water to the culturable area. Then the time factor is 10/12.

$$\therefore \text{Time factor} = \frac{\text{No. of days the canal practically kept open}}{\text{No. of days the canal was designed to keep open}}$$

$$= \frac{\text{Actual discharge}}{\text{Designed discharge}}$$

Capacity Factor

Capacity Factor

Generally, a canal is designed for a maximum discharge capacity. But, actually it is not required that the canal runs to that maximum capacity all the time of the base period. So, the ratio of the average discharge to the maximum discharge (designed discharge) is known as **capacity factor**.

For example, a canal was designed for the maximum discharge of 50 cumec, but the average discharge is 40 cumec.

$\therefore$ Capacity factor = 40/50 = 0.8

Problem

The culturable commanded area of a watercourse is 1200 hectares. Intensities of sugarcane and wheat crops are 20% and 40% respectively. The duties for the crops at the head of the watercourse are 730 hectares/cumec and 1800 hectares/cumec respectively.

Find (a) The discharge required at the head of the watercourse

(b) Determine the design discharge at the outlet, assuming a time factor equal to 0.8.

Solution:

C.C.A = 1200 hectares

Intensity of irrigation for sugarcane = 20 %

$\therefore$ Area to be irrigated under sugarcane = 1200×(20/100) = 240 ha

Intensity of irrigation for wheat = 40 %

$\therefore$ Area to be irrigated under wheat = 1200×(40/100) = 480 ha

Again,

Duty for sugarcane and wheat = 730 ha/cumec and 1800 ha/cumec

$\therefore$ Discharge required for sugarcane = Area/Duty

$\therefore$ = (240/730) cumec

= 0.329 cumec

$\therefore$ Discharge required for wheat = (480/1800) cumec

$\therefore$ = 0.271 cumec

Now, sugarcane requires water for all the 12 months and wheat requires water for only Rabi season. Hence, the water requirement at the head of the watercourse at any time of the year will be the summation of the two, i.e. equal to 0.329 + 0.271 = 0.6 cumec

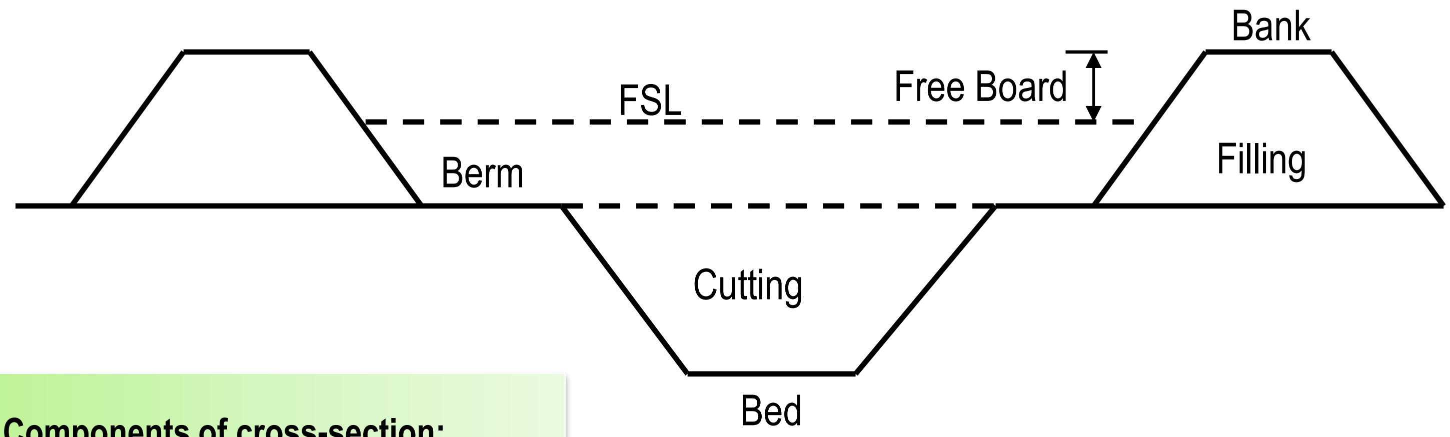
Hence, the discharge required at the head of the watercourse is 0.6 cumec (**ans**)

Note: The discharge during Rabi season will be 0.6 cumec and for the rest of the year, it will be 0.329 cumec

Time factor = 0.8; since the channel runs for fewer days than the crop days, therefore, the actual design discharge at the outlet = (0.6/0.8) = 0.75 cumec (**ans**)

LECTURE 16

Cross-section of an irrigation canal

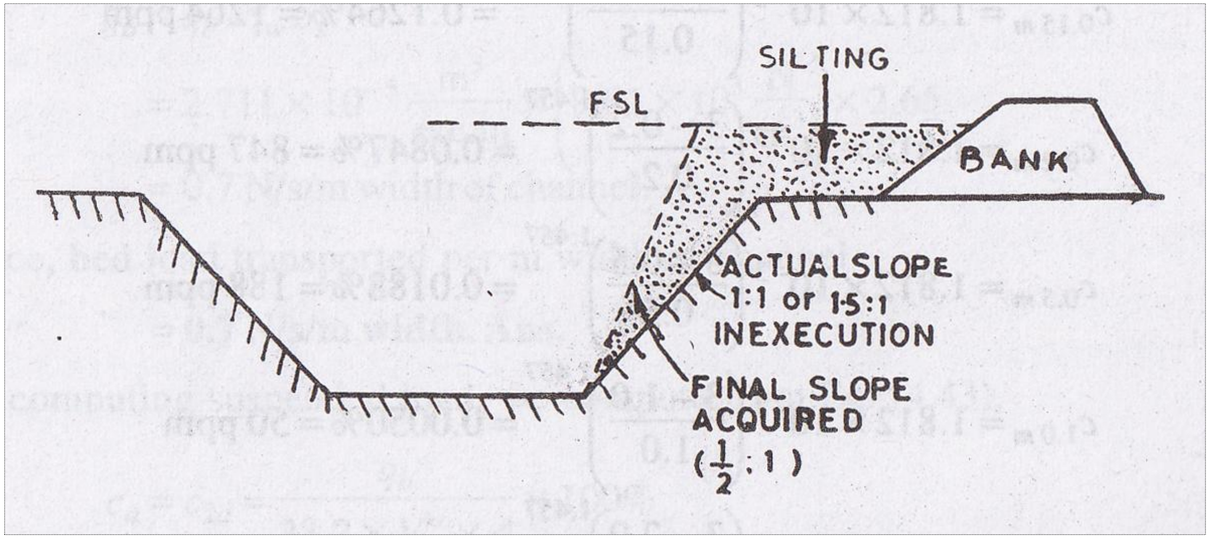


Components of cross-section:

- Side slopes
- Berms
- Freeboard
- Banks
- Service roads
- Back Berm or Counter Berms
- Spoil Banks
- Borrow Pits

Side Slopes

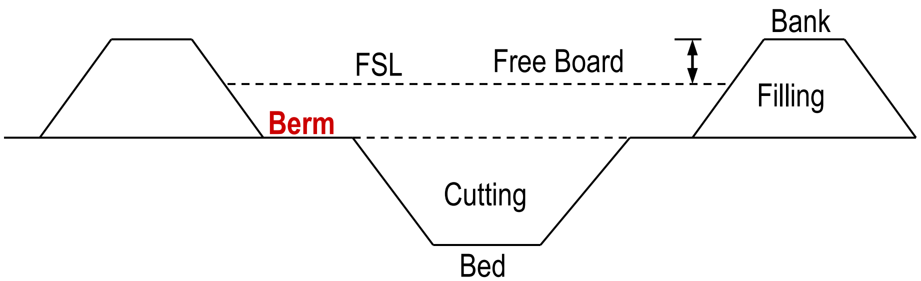
The side slopes should be such that they are stable, depending upon the type of the soil. A comparatively steeper slope can be provided in cutting rather than in filling, as the soil in the former case shall be more stable.

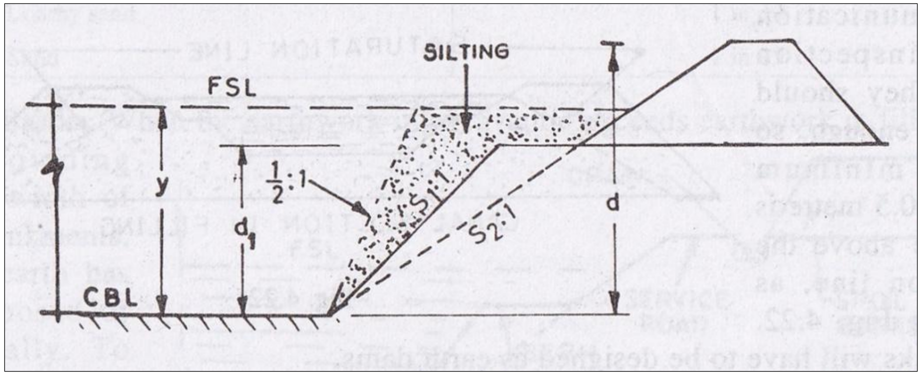


In cutting ------- 1H: 1V to 1.5 H: 1V
In filling ------ 1.5 H: 1V to 2H: 1V

BERMS

Berm is the horizontal distance left at ground level between the toe of the bank and the top edge of cutting.





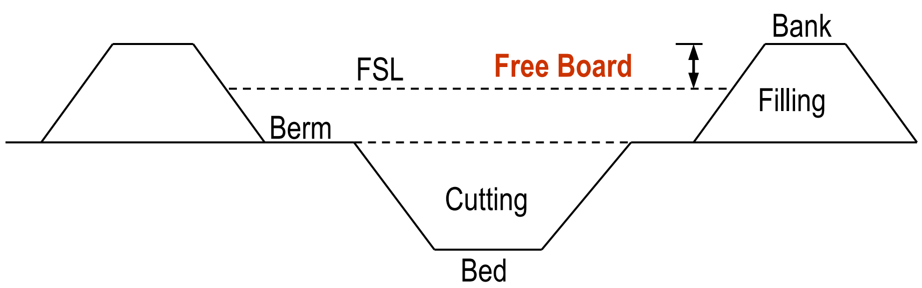
The berm is provided in such a way that the bed line and the bank line remain parallel. If s_1: 1 is the slope in cutting and s_2:1 in filling, then the initial berm width = $(s_2 - s_1)\ d_1$.

Purposes of Berms

- They help the channel to attain regime conditions.
- They give additional strength to the banks and provide protection against erosion and breaches.
- They protect the banks from erosion due to wave action.
- They provide a scope for future widening of the canal.

Free Board

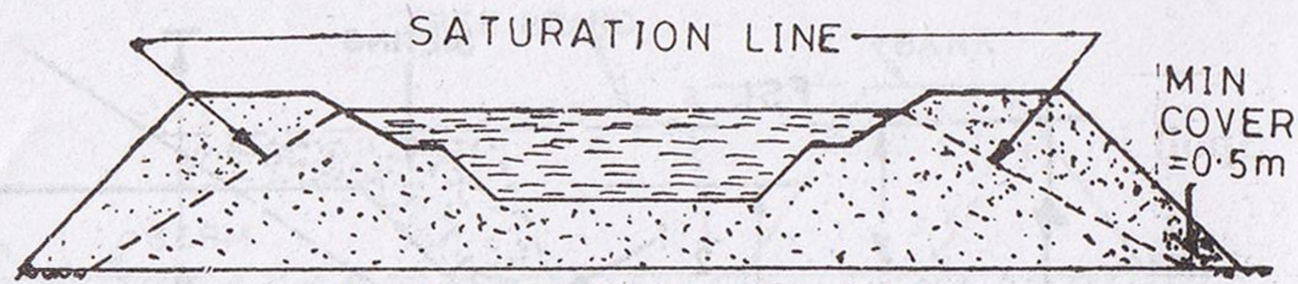
The margin between FSL and bank level is known as **freeboard**. The amount of freeboard depends upon the size of the channel. The generally provided values of freeboard are given in the table below:



Discharge (m^3/s)	Extent of freeboard (m)
1 to 5	0.50
5 to 10	0.60
10 to 30	0.75
30 to 150	0.90

Banks

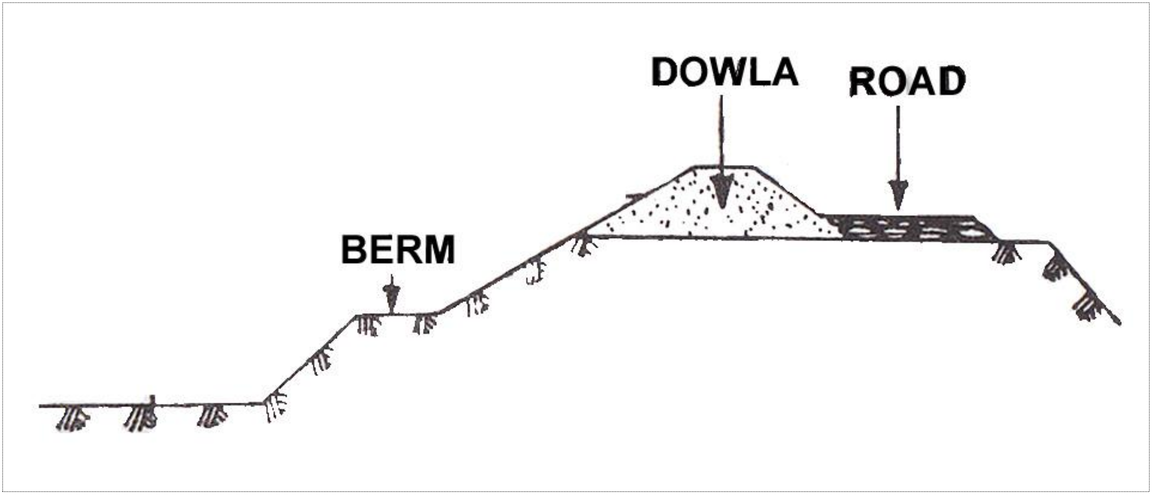
The primary purpose of banks in to remain water. This can be used as means of communication and as inspection paths. They should be wide enough, so that <u>a minimum cover of 0.50 m</u> is available above the saturation line.



CANAL SECTION IN FILLING

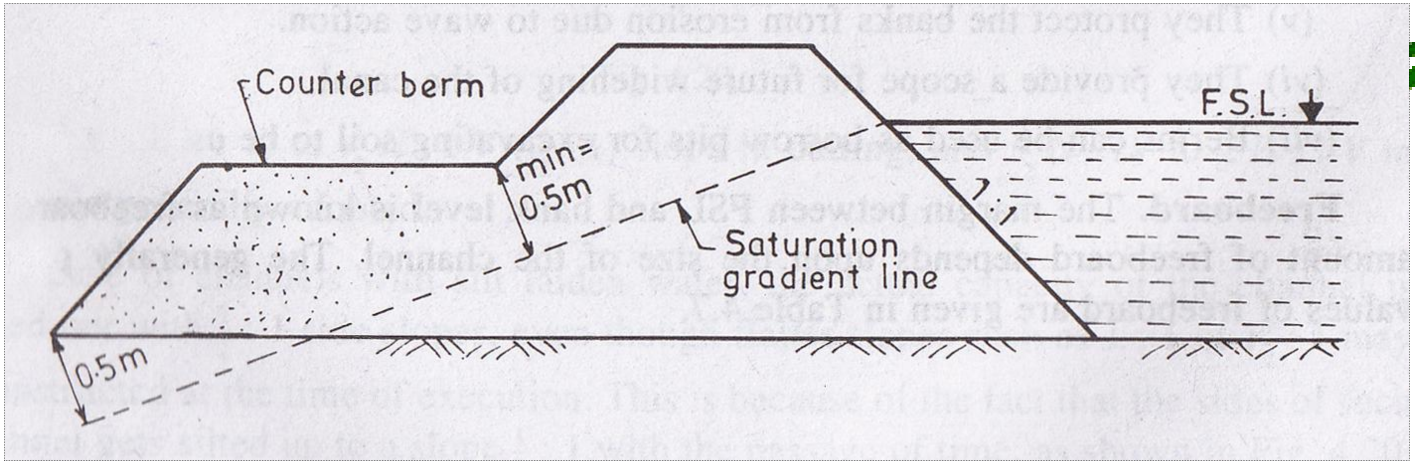
Service Roads

Service roads are provided on canals for inspection purposes, and may simultaneously serve as the means of communication in remote areas. They are provided 0.4 m to 1.0 m above FSL, depending upon the size of the channel.



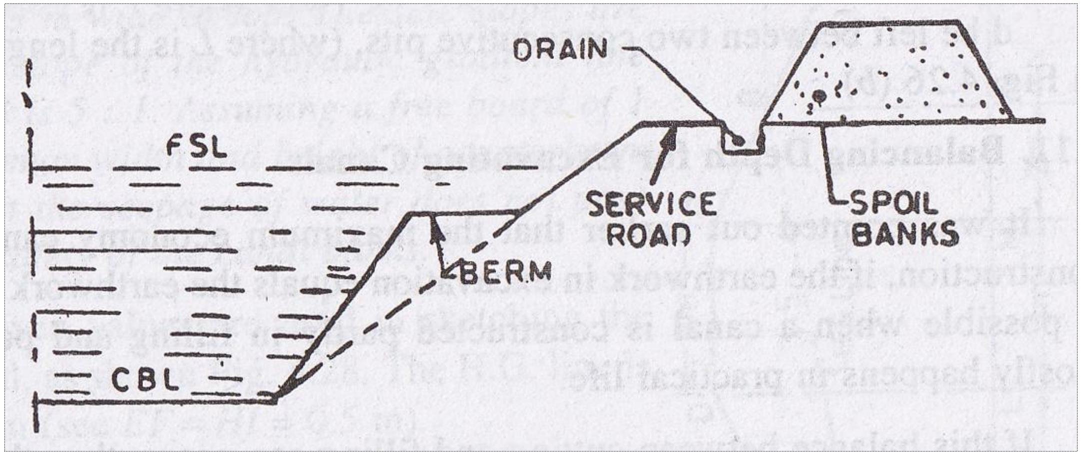
Back Berms or Counter Berms

Even after providing sufficient section for bank embankment, the saturation gradient line may cut the downstream end of the bank. In such a case, <u>the saturation line can be kept covered at least by 0.5 m</u> with the help of counter berms as shown in figure below.



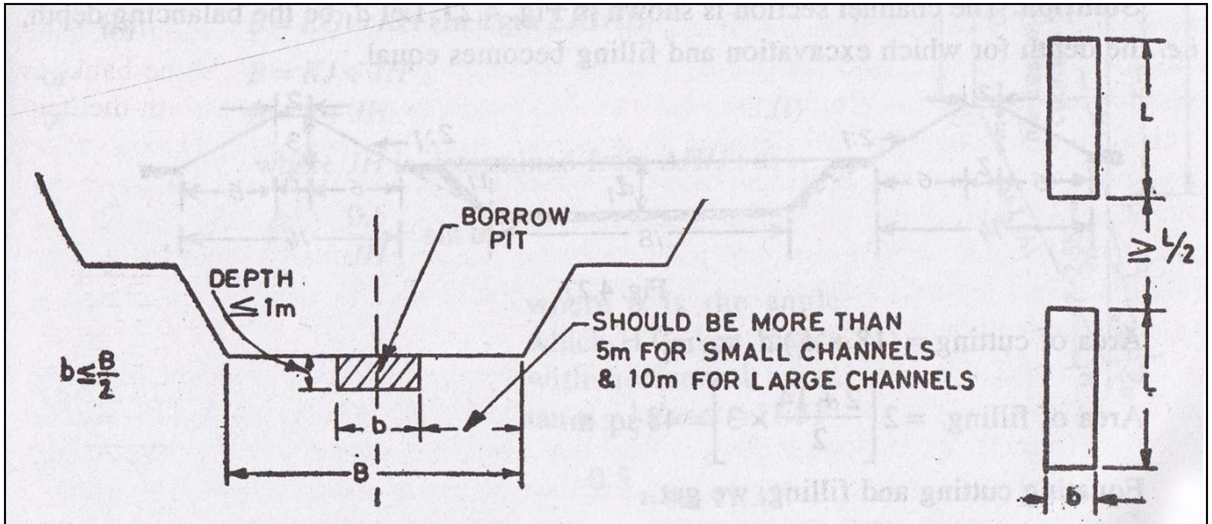
Spoil Banks

When the earthwork in excavation exceeds earthworks in filling, even after providing maximum width of bank embankments, the extra earth has to be disposed of economically. To dispose of this earth by mechanical transport, etc. may become very costly, and an economical mode of its disposal may be found in the form of collecting this soil on the edge of the bank embankment itself.



Borrow Pits

When earthwork in filling exceeds the earthwork in excavation, the earth has to be brought from somewhere. The pits, which are dug for bringing earth, are known as **Borrow Pits**.



(a). Elevation. (b). Plan.

Design Requirements:

✓ The borrow pits should start from a point at a distance more than 5 m from the toe for small channels, and 10 m for large channels.

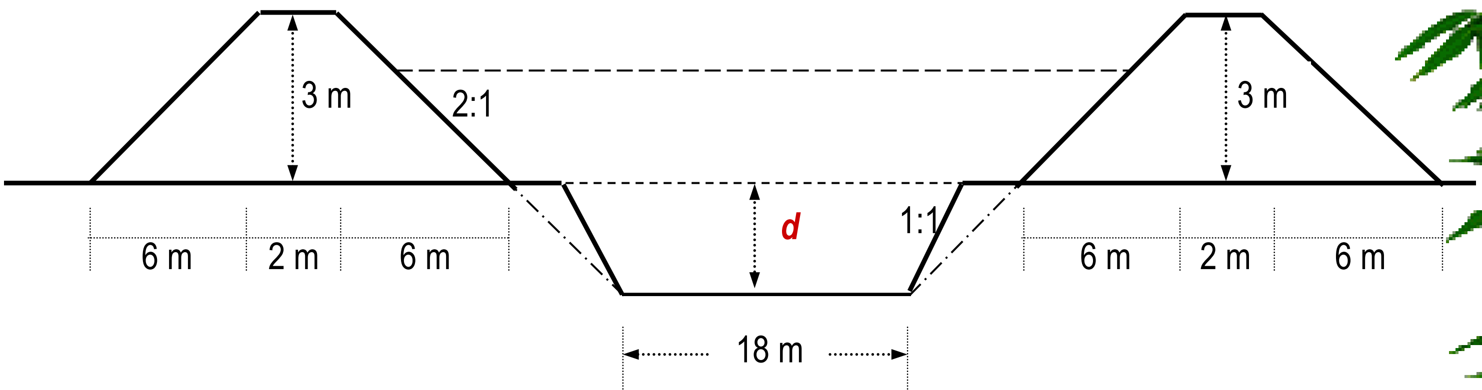
✓ The width of these pits *b*, should be less than half the width of the canal *B*, and should be dug in the entire.

✓ The depth of these pits should be equal to or less than 1 m.

✓ Longitudinally, these pits should not run continuous, but a minimum space of *L*/2 should be left between two consecutive pits, (where *L* is the length of one pits).

Problem

Calculate the **balancing depth** for a channel section having a bed width equal to 18 m and side slopes of 1:1 in cutting and 2:1 in filling. The bank embankments are kept 3.0 m higher than the ground level (berm level) and crest width of banks is kept as 2.0 m



Solution:

The channel section is shown below. Let d be the balancing depth, i.e. the depth for which excavation and filling becomes equal.

Area of cutting = $(18 + d)\, d$ m^2

Area of filling = $2(2+14)/2 \times 3 = 48$ m^2

Equating cutting and filling, we get

$$(18 + d)\, d = 48$$

$\Rightarrow d^2 + 18d - 48 = 0$

$\Rightarrow d = 2.35$ m (neglecting –ve sign)

$\therefore$ Balancing depth = 2.35 m

End of Chapter – 6

CHAPTER 07

CANAL DESIGN

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LECTURE 17

Canal Design Types

Canal Design

- Drainage Channel Design
- Irrigation Canal Design

Topic will be discussed……. →

Design Parameters

- The design considerations naturally vary according to the type of soil.
- Velocity of flow in the canal should be ***critical***.
- Design of canals which are known as '*Kennedy's theory*' and '*Lacey's theory*' are based on the characteristics of sediment load (i.e. silt) in canal water.

Important Terms Related to Canal Design

- Alluvial soil
- Non-alluvial soil
- Silt factor
- Co-efficient of rugosity
- Mean velocity
- Critical velocity
- Critical velocity ratio (c.v.r), m
- Regime channel
- Hydraulic mean depth
- Full supply discharge
- Economical section

Alluvial Soil

The soil which is formed by the continuous deposition of silt is known as **alluvial soil**. The river carries heavy charge of silt in rainy season. When the river overflows its banks during the flood, the silt particles get deposited on the adjoining areas. This deposition of silt continues year after year. This type of soil is found in deltaic region of a river. This soil is permeable and soft and very fertile. The river passing through this type of soil has a tendency to change its course.

Non-alluvial Soil

The soil which is formed by the disintegration of rock formations is known as **non-alluvial soil**. It is found in the mountainous region of a river. The soil is hard and impermeable in nature. This is not fertile. The river passing through this type of soil has no tendency to change its course.

Silt Factor

During the investigations works in various canals in alluvial soil, *Gerald Lacey* established the effect of silt on the determination of discharge and the canal section. So, Lacey introduced a factor which is known as '**silt factor**'.

It depends on the mean particle size of silt. It is denoted by 'f'. The silt factor is determined by the expression,

$$f = 1.76\sqrt{d_{mm}}$$

where d_{mm} = mean particle size of silt in mm

Particle	Particle size (mm)	Silt factor (f)
Very fine silt	0.05	0.40
Fine silt	0.12	0.60
Medium silt	0.23	0.85
Coarse silt	0.32	1.00

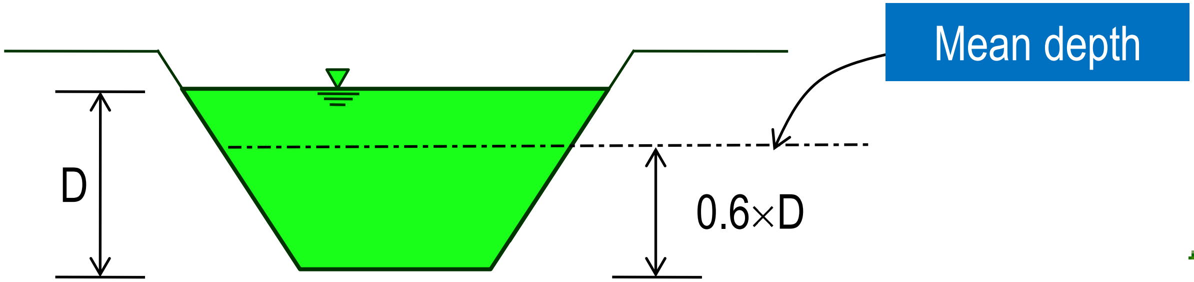
Coefficient of Rugosity (n)

The roughness of the canal bed affects the velocity of flow. The roughness is caused due to the ripples formed on the bed of the canal. So, a coefficient was introduced by *R.G Kennedy* for calculating the mean velocity of flow. This coefficient is known as coefficient of rugosity and it is denoted by ‘n’. The value of ‘n’ depends on the type of bed materials of the canal.

Materials	Value of n
Earth	0.0225
Masonry	0.02
Concrete	0.013 to 0.018

Mean Velocity

It is found by observations that the velocity at a depth 0.6D represents the mean velocity (V), where 'D' is the depth of water in the canal or river.



(a) Mean Velocity by Chezy's expression:

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

(a) Mean Velocity by Manning's expression:

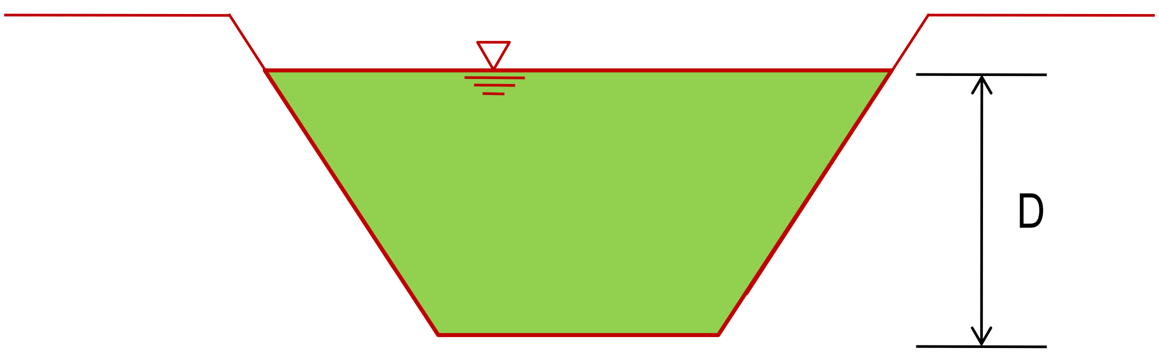
$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

Critical velocity

When the velocity of flow is such that there is no silting or scouring action in the canal bed, then that velocity is known as **critical velocity.** It is denoted by 'V_o'. The value of V_o was given by Kennedy according to the following expression,

$$V_o = 0.546 \times D^{0.64}$$

; where, D = Depth of water



Critical velocity ratio (C.V.R)

The ratio of mean velocity 'V' to the critical velocity 'V_o' is known as critical velocity ratio (CVR). It is denoted by *m* i.e.

$$\text{CVR } (m) = V/V_o$$

When m = 1, there will be no silting or scouring.
When m > 1, scouring will occur
When m < 1, silting will occur

So, by finding the value of *m*, the condition of the canal can be predicted whether it will have silting or scouring

Regime Channel

When the character of the bed and bank materials of the channel are same as that of the transported materials and when the silt charge and silt grade are constant, then the channel is said to be in its regime and the channel is called regime channel. This ideal condition is not practically possible.

Hydraulic Mean Depth/Ratio

The ratio of the cross-sectional area of flow to the wetted perimeter of the channel is known as hydraulic mean depth or radius. It is generally denoted by R.

$$R = A/P$$

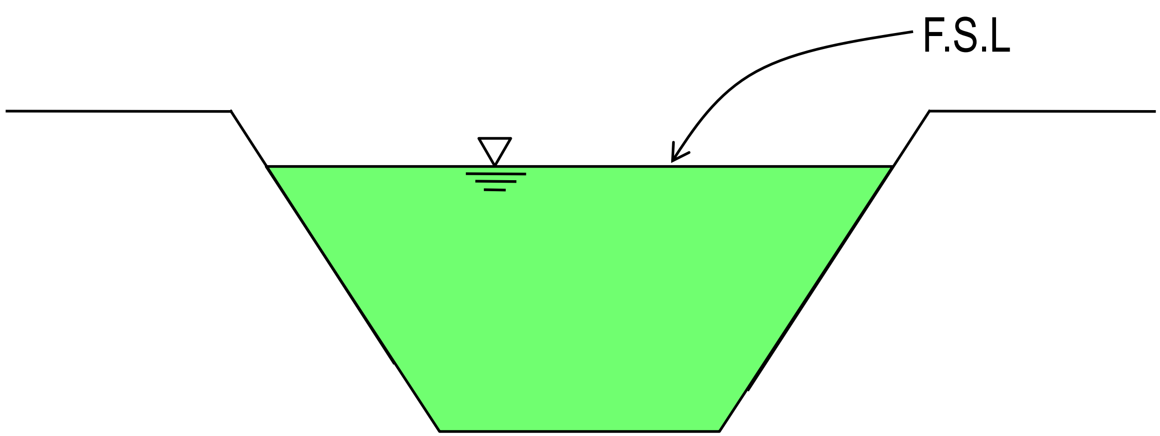
Where,

A = Cross-sectional area

P = Wetted perimeter

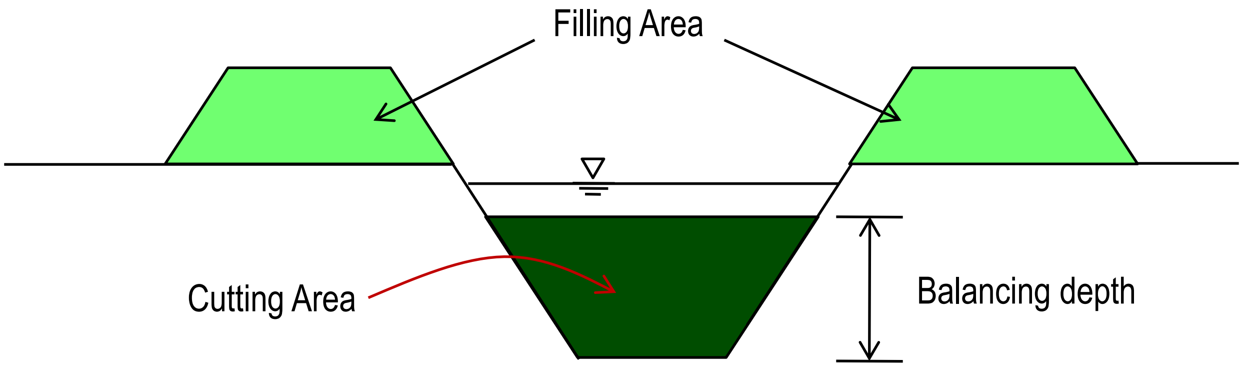
Full Supply Discharge

The maximum capacity of the canal for which it is designed, is known as full supply discharge. The water level of the canal corresponding to the full supply discharge is known as **full supply level (F.S.L).**



Economical Section

If a canal section is such that the earth obtained from cutting (i.e. excavation) can be fully utilized in forming the banks, then that section is known as **economical section**. Again, the discharge will be maximum with minimum cross-section area. Here, no extra earth is required from borrow pit and no earth is in excess to form the spoil bank. This condition can only arise in case of partial cutting and partial banking. Sometimes, this condition is designated as balancing of cutting and banking. Here, the depth of cutting is called **balancing depth**.



LECTURE 18

Unlined Canal Design on Non-alluvial soil

The non-alluvial soils are stable and nearly impervious. For the design of canal in this type of soil, the coefficient of rugosity plays an important role, but the other factor like silt factor has no role. Here, the velocity of flow is considered very close to critical velocity. So, the mean velocity given by Chezy's expression or Manning's expression is considered for the design of canal in this soil. The following formulae are adopted for the design.

(1) Mean velocity by Chezy's formula

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

Where,

V = mean velocity in m/sec,
C = Chezy's constant,
R = hydraulic mean depth in m
S = bed slope of canal as 1 in *n*.

Again, the Chezy's constant C can be calculated by:

(a) Bazin's Formula:

$$C = \frac{87}{1+\frac{K}{\sqrt{R}}}$$

Where,

K = Bazin's constant,
R = hydraulic mean depth

(b) Kutter's Formula:

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

Where,

n = Co-efficient of rugosity,
S = bed slope,
R = hydraulic mean depth

(2) Mean velocity by Manning's formula

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

(3) Discharge by the following equations:

$$Q = A \times V$$

Where,

Q = discharge in cumec

A = cross-sectional area of water section in m^2

V = mean velocity in m/sec

Note:

- If value of K is not given, then it may be assumed as follows,
  - For unlined channel, K = 1.30 to 1.75.
  - For line channel, K = 0.45 to 0.85
- If the value of N is not given, then it may be assumed as follows,
  - For unlined channel, N = 0.0225
  - For lined channel, N = 0.333

Problem

Design an irrigation channel with the following data:
Discharge of the canal = 24 cumec
Permissible mean velocity = 0.80 m/sec.
Bed slope = 1 in 5000
Side slope = 1:1
Chezy's constant, C = 44

Solution:
We know, A = 24/0.80 = 30 m^2
30 = (B + D)D
And P = B + 2.828 D
But, R = 30/(B + 2.828 D)
From Chezy's formula, V = $C\sqrt{RS}$

$$\Rightarrow 0.80 = 44 \times \sqrt{R \times 0.0002}$$

$\therefore$ R = 1.65 m

Putting the value of R and solving, D = 2.09 m and B = 12.27 m

Exercise Problems

Problem – 1

Design a most economical trapezoidal section of a canal having the following data:

Discharge of the canal = 20 cumec

Permissible mean velocity = 0.85 m/sec.

Bazin's constant, K = 1.30

Side slope = 1.5:1

Find also the allowable bed slope of the canal

Problem – 2

Find the bed width and bed slope of a canal having the following data:

Discharge of the canal = 40 cumec

Permissible mean velocity = 0.95 m/sec.

Coefficient of rugosity, n = 0.0225

Side slope = 1:1

B/D ratio = 6.5

Problem – 3

Find the efficient cross-section of a canal having the discharge 10 cumec. Assume, bed slope 1 in 5000, value of n = 0.0025, C.V.R (m) = 1, full supply depth not to exceed 1.60 m and side slope = 1:1

Unlined Canal Design on Alluvial soil by Kennedy's Theory

After long investigations, R.G Kennedy arrived at a theory which states that, the silt carried by flowing water in a channel is kept in suspension by the vertical component of eddy current which is formed over the entire bed width of the channel and the suspended silt rises up gently towards the surface.

The following assumptions are made in support of his theory:

- The eddy current is developed due to the roughness of the bed.
- The quality of the suspended silt is proportional to bed width.
- It is applicable to those channels which are flowing through the bed consisting of sandy silt or same grade of silt.
- It is applicable to those channels which are flowing through the bed consisting of sandy silt or same grade of silt.

Continue…. Kennedy's Theory

He established the idea of critical velocity 'V_o' which will make a channel free from silting or scouring. From, long observations, he established a relation between the critical velocity and the full supply depth as follows,

$$V_o = C \times D^n$$

The values of C and n where found out as 0.546 and 0.64 respectively, thus

$$V_o = 0.546 \times D^{0.64}$$

Again, the realized that the critical velocity was affected by the grade of silt. So, he introduced another factor (m) which is known as critical velocity ratio (C.V.R).

$$V_o = 0.546 \times m \times D^{0.64}$$

Drawbacks of Kennedy's Theory

- The theory is limited to average regime channel only.
- The design of channel is based on the trial and error method.
- The value of m was fixed arbitrarily.
- Silt charge and silt grade are not considered.
- There is no equation for determining the bed slope and it depends on Kutter's equation only.
- The ratio of 'B' to 'D' has no significance in his theory.

Design Procedure

- Critical velocity, $V_o = 0.546 \times m \times D^{0.64}$.
- Mean velocity, $V = C \times (RS)^{1/2}$

Where, m = critical velocity ratio,
D = full supply depth in m,
R = hydraulic mean depth of radius in m,
S = bed slope as 1 in 'n'.

The value of 'C' is calculated by Kutter's formula,

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

Where, n = rugosity coefficient which is taken as unlined earthen channel.

- B/D ratio is assumed between 3.5 to 12.
- Discharge, $Q = A \times V$.

Where, A = Cross-section area in m^2,
V = mean velocity in m/sec

- The full supply depth is fixed by trial to satisfy the value of 'm'. Generally, the trial depth is assumed between 1 m to 2 m. If the condition is not satisfied within this limit, then it may be assumed accordingly.

Problem

Design an irrigation channel with the following data:
Full supply discharge = 6 cumec
Rugosity coefficient (n) = 0.0225
C.V.R (m) = 1
Bed slope = 1 in 5000
Assume other reasonable data for the design

Solution:

First Trial:

Assume, Full supply depth, D = 1.5 m

Critical velocity, $V_o = 0.546 \times 1 \times 1.5^{0.64} = 0.707$ [Assume, m = 1]

As m = 1, $V = V_o$
$\therefore A = 6/0.707 = 8.49$ m

$A = (2B + 3)/2 \times 1.5 = 1.5B + 2.25 \Rightarrow B = 4.16$ m
$P = B + 2\sqrt{2} \times 1.5 = B + 4.24 = 8.40$ m
$R = 4.16/8.40 = 1.0$ m

By Kutter's formula, C = $\left[\dfrac{\dfrac{1}{0.0225}+\left(23+\dfrac{0.00155}{0.0002}\right)}{1+\left(23+\dfrac{0.00155}{0.0002}\right)\dfrac{0.0225}{\sqrt{1.0}}}\right]$ = 44.49

By Chezy's formula, V = 44.49×√(1×0.0002) = 0.629 m/sec

C.V.R = 0.629/0.707 = 0.889 < 1

As the C.V.R is much less than 1, the channel will be in silting. So, the design is not satisfactory. Here, the full supply depth is to be assumed by trials to get the satisfactory result.

Second Trial:

Assume, Full supply depth, D = 1.25 m

Critical velocity, $V_o = 0.546 \times 1 \times 1.25^{0.64} = 0.629$ [Assume, m = 1]

As m = 1, $V = V_o$

$\therefore A = 6/0.629 = 9.53$ m

$A = 1.25B + 1.56 \Rightarrow B = 6.38$ m

$P = B + 2\sqrt{2} \times 1.25 = 9.92$ m

R = 0.96 m

By Kutter's formula, $C = \left[\dfrac{\dfrac{1}{0.0225} + \left(23 + \dfrac{0.00155}{0.0002}\right)}{1 + \left(23 + \dfrac{0.00155}{0.0002}\right)\dfrac{0.0225}{\sqrt{0.96}}} \right] = 44.23$

By Chezy's formula, $V = 44.23 \times \sqrt{(1 \times 0.0002)} = 0.613$ m/sec

C.V.R = 0.613/0.629 = 0.97 < 1

In this case, the CVR is very close to 1. So, the design may be accepted. So, finally,

D = 1.25 m and B = 6.28 m

Exercise Problems

Problem – 1

Find the maximum discharge through an irrigation channel having the bed width 4 m and fully supply depth is 1.50 m. Given that n = 0.02, S = 0.0002, side slope = 1:1

Assume reasonable data, if necessary. Comment whether the channel will be in scouring or silting.

Problem – 2

Design an irrigation channel with the following data:

Full supply discharge = 10 cumec

Bazin's constant, K = 1.3

C.V.R (m) = 1

B/D ratio = 4

Side slope = 1:1

Assume other reasonable data for the design

LECTURE 19

Unlined Canal Design on Alluvial soil by Lacey's Theory

Lacey's theory is based on the concept of regime condition of the channel. The regime condition will be satisfied if,

- ❑ The channel flows uniformly in unlimited incoherent alluvium of the same character which is transported by the channel.
- ❑ The silt grade and silt charge remains constant.
- ❑ The discharge remains constant.

In his theory, he states that the silt carried by the flowing water is kept in suspension by the vertical component of eddies. The eddies are generated at all the points on the wetted perimeter of the channel section. Again, he assumed the hydraulic mean radius *R*, as the variable factor and he recognized the importance of silt grade for which in introduced a factor which is known as silt factor '*f*'.

Thus, he deduced the velocity as; $V = \sqrt{(2/5 f R)}$

Where, V = mean velocity in m/sec, f = silt factor,

R = hydraulic mean radius in meter

Then he deduced the relationship between A, V, Q, P, S and f are as follows:

- $f = 1.76 \times \sqrt{d_{mm}}$
- $Af^2 = 140 \times V^5$
- $V = \left(\frac{Q \times f^2}{140}\right)^{1/6}$
- $P = 4.75 \times \sqrt{Q}$
- Regime flow equation, $V = 10.8 \times R^{2/3} S^{1/3}$
- Regime slope equation,

(a) $S = \frac{f^{3/2}}{4980 \times R^{1/3}}$

(b) $S = \frac{f^{5/2}}{3340 \times Q^{1/6}} \Rightarrow Q = \left[\frac{f^{5/3}}{3340 \times S}\right]^6$

- Regime scour depth, $R = 0.47 \times \left(\frac{Q}{f}\right)^{1/3}$

Problem

Design an irrigation channel with the following data:

Full supply discharge = 10 cumec

Mean diameter of silt particles = 0.33 mm

Side slope = ½:1

Find also the bed slope of the channel

Solution:

f = 1.76 $\times\sqrt{0.33}$ = 1.0 and V = $\left(\dfrac{Q\times 1^2}{140}\right)^{1/6}$ = 0.64 m/sec

A = 10/0.64 = 15.62 m^2

P = 4.75 $\times\sqrt{10}$ = 15.02 m

R = 0.47$\times$(10/1)$^{1/3}$ = 1.02 m

S = $\dfrac{1^{5/2}}{3340\times 10^{1/6}}$ = 1/4902

But, A = BD + 0.5 D^2

$\Rightarrow$ 15.62 = BD + 0.5 D^2 ------------------- (i)

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

15.02 = B + 2.24 D ------------------------- (ii)

Solving equation (i) & (ii) D = 1.21 m and B = 12.30 m

Exercise Problem

Problem – 1

Find the section and maximum discharge of a channel with the following data:

Bed slope = 1 in 5000

Lacey's silt factor = 0.95

Side slope = 1:1

Drawbacks of Lacey's Theory

- The concept of true regime is theoretical and con not be achieved practically.
- The various equations are derived by considering the silt factor f which is not at all constant.
- The concentration of silt is not taken into account.
- Silt grade and silt charge is not taken into account.
- The equations are empirical and based on the available data from a particular type of channel. So, it may not be true for a different type of channel.
- The characteristics of regime channel may not be same for all cases.

Comparison between Kennedy's and Lacey's theory

Kennedy's theory	Lacey's theory
It states that the silt carried by the flowing water is kept in suspension by the vertical component of eddies which are generated from the bed of the channel.	It states that the silt carried by the flowing water is kept in suspension by the vertical component of eddies which are generated from the entire wetted perimeter of the channel.
It gives relation between 'V' and 'D'.	It gives relation between 'V' and 'R'.
In this theory, a factor known as critical velocity ratio 'm' is introduced to make the equation applicable to different channels with different silt grades.	In this theory, a factor known as silt factor 'f' is introduced to make the equation applicable to different channels with different silt grades.
In this theory, Kutter's equation is used for finding the mean velocity.	This theory gives an equation for finding the mean velocity.
This theory gives no equation for bed slope.	This theory gives an equation for bed slope.
In this theory, the design is based on trial and error method.	This theory does not involve trial and error method.

Design of Lined Canal

The lined canals are not designed by the use of Lacey's and Kennedy's theory, because the section of the canal is rigid. Manning's equation is used for designing. The design considerations are,

- The section should be economical (i.e. cross-sectional area should be maximum with minimum wetted perimeter).
- The velocity should be maximum so that the cross-sectional area becomes minimum.
- The capacity of lined section is not reduced by silting.

Section of Lined Canal:

The *following two* lined sections are generally adopted:

❑ **Circular section:**

The bed is circular with its center at the full supply level and radius equal to full supply depth 'D'. The sides are tangential to the curve. However, the side slope is generally taken as 1:1.

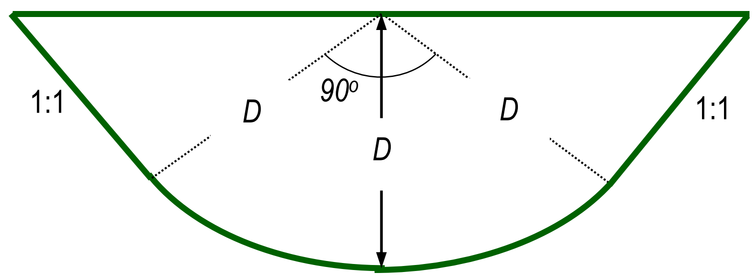


Table – 1:Design parameters for circular section

Design parameter	Side slope		
	1:1	**1.5:1**	**1.25:1**
Sectional area (A)	$1.785\times D^2$	$2.088\times D^2$	$1.925\times D^2$
Wetted perimeter (P)	$3.57\times D$	$4.176\times D$	$3.85\times D$
Hydraulic mean depth or radius (R)	$0.5\times D$	$0.5\times D$	$0.5\times D$
Velocity (V)	$V = (1/n)\times R^{2/3}\times S^{1/2}$	–	–
Discharge (Q)	$A\times V$	–	–

❑ Trapezoidal section:

The horizontal bed is joined to the side slope by a curve of radius equal to full supply depth D. The side slope is generally kept as 1:1

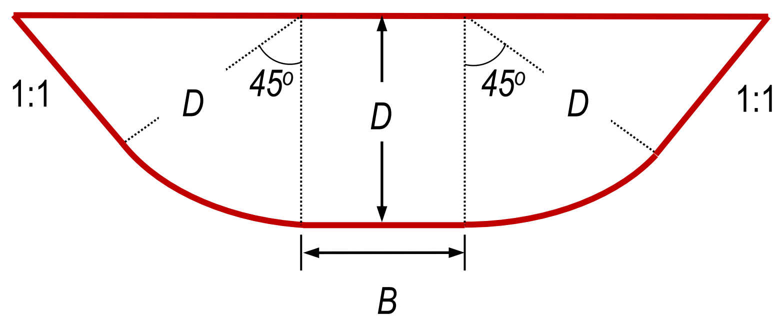


Table – 2:Design parameters for circular section

Design parameter	Side slope		
	1:1	**1.5:1**	**1.25:1**
Sectional area (A)	BD + 1.785×D^2	BD + 2.088 ×D^2	BD + 1.925×D^2
Wetted perimeter (P)	B + 3.57×D	B + 4.176×D	B + 3.85×D
Hydraulic mean depth or radius (R)	A/P	A/P	A/P
Velocity (V)	$V = (1/n) \times R^{2/3} \times S^{1/2}$	–	–
Discharge (Q)	A×V	–	–

Note: For the discharge up to 50 cumec, the circular section is suitable and for the discharge above 50 cumec trapezoidal section is suitable.

Problem - 1

Design a lined canal to carry a discharge of 40 cumec. Assume bed slope as 1 in 5000, N = 0.0225 and side slope – 1:1

Solution:

Since the discharge is less than 50 cumec, the circular section is suitable slope as 1 in 5000, n = 0.0225 and side slope = 1:1

From table 1, $A = 1.785\ D^2$

$P = 3.57\ D$

$R = 1.785\ D^2/3.57D = 0.5\ D$

Again, $V = (1/0.0225) \times (0.5D)^{2/3} \times (0.0002)^{1/2} = 0.38 \times D^{2/3}$

But, $50 = 1.785\ D^2 \times 0.38 \times D^{2/3}$

$\therefore\ D = 4.61$ m

$V = 1.05$ m/s

$A = 37.93\ m^2$

$P = 16.45$ m

Problem - 2

Design a lined canal having the following data:

Full supply discharge = 200 cumec

Side slope = 1.25:1

Bed slope = 1 in 5000

Rugosity coefficient = 0.018

Permissible velocity = 1.75 cumec

Solution:

Since the discharge is more than 50 cumec, the trapezoidal section will be acceptable.

From table – 2,

Sectional area, $A = BD + 1.925\ D^2$ -------------- (i)

Wetted perimeter, $P = B + 3.85\ D$ ------------- (ii)

Now, $A = 200/1.75 = 114.28\ m^2$

Again, $V = (1/n)\times R^{2/3}\times S^{1/2}$

$\Rightarrow 1.75 = (1/0.018)\times R^{2/3}\times(0.0002)^{1/2}$

$\therefore\ R = 3.32$ m

$P = 114.28/3.32 = 34.42$ m

From equation (i) $\Rightarrow 114.28 = BD + 1.925\times D^2$

From equation (ii) $\Rightarrow 34.42 = B + 3.85\times D$

Solving equation (i) & (ii) $\Rightarrow D = 4.4$ m (Full supply depth)

$\therefore\ B = 17.5$ m (Bed width)

END OF CHAPTER – 7

CHAPTER – 8

Irrigation Structures – 1

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LECTURE 20

Diversion Head Works

Definition:

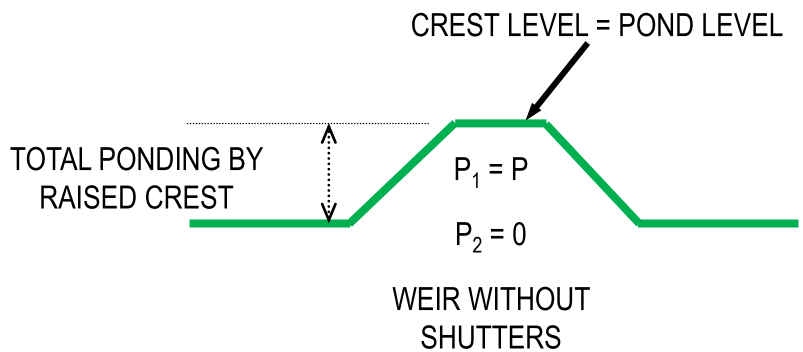
The works, which are constructed at the head of the canal, in order to divert the river water towards the canal, so as to ensure a regulated continuous supply of silt-free water with a certain minimum head into the canal, are known as ***diversion heads works***.

Objectives:

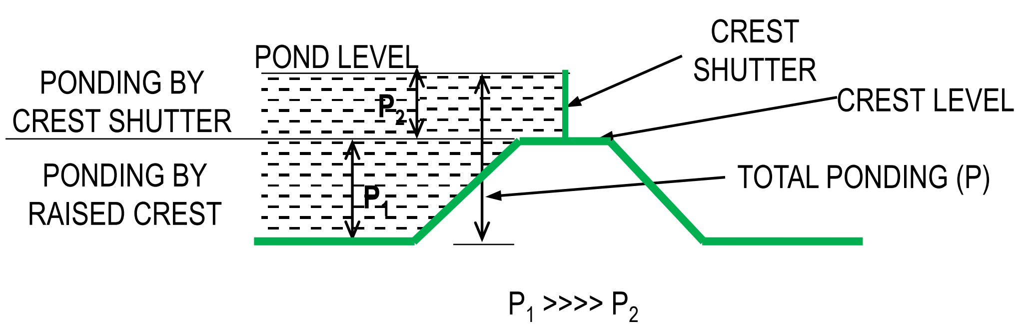
- To rise the water level at the head of the canal.
- To form a storage by constructing dykes (embankments) on both the banks of the river so that water is available throughout the year
- To control the entry of silt into the canal and to control the deposition of silt at the head of the canal
- To control the fluctuation of water level in the river during different seasons

Weir & Barrage

If the major part or the entire ponding of water is achieved by a raised crest and a smaller part or nil part of it is achieved by the shutters, then this barrier is known as a ***weir***.

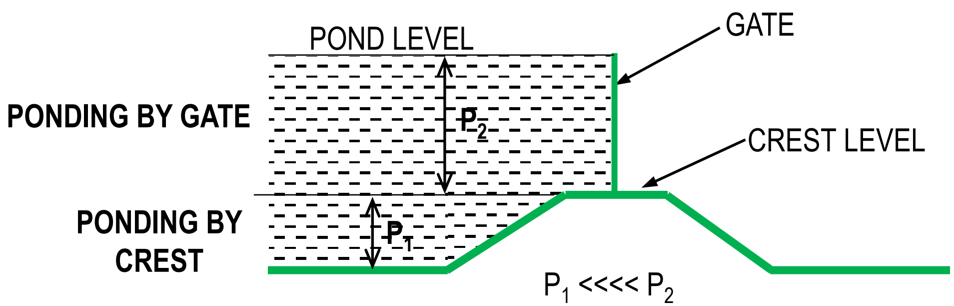


(a) Fig: Weir without shutters

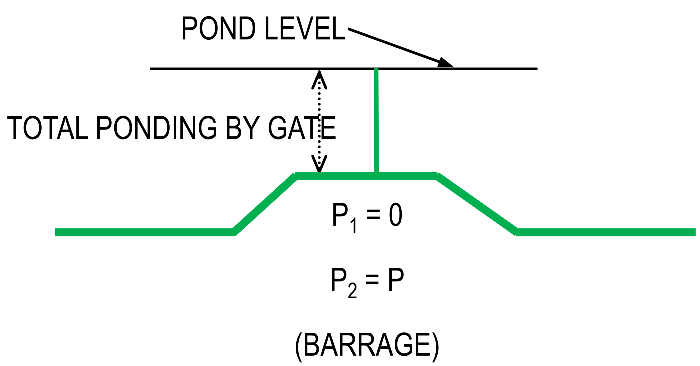


(b) Fig: Weir with shutters

If most of the ponding is done by gates and a smaller or nil part of it is done by the raised crest, then the barrier is known as a ***barrage*** *or a river regulator*.



(c) Fig: Barrage with a small raised crest



(d) Fig: Barrage without any raised crest

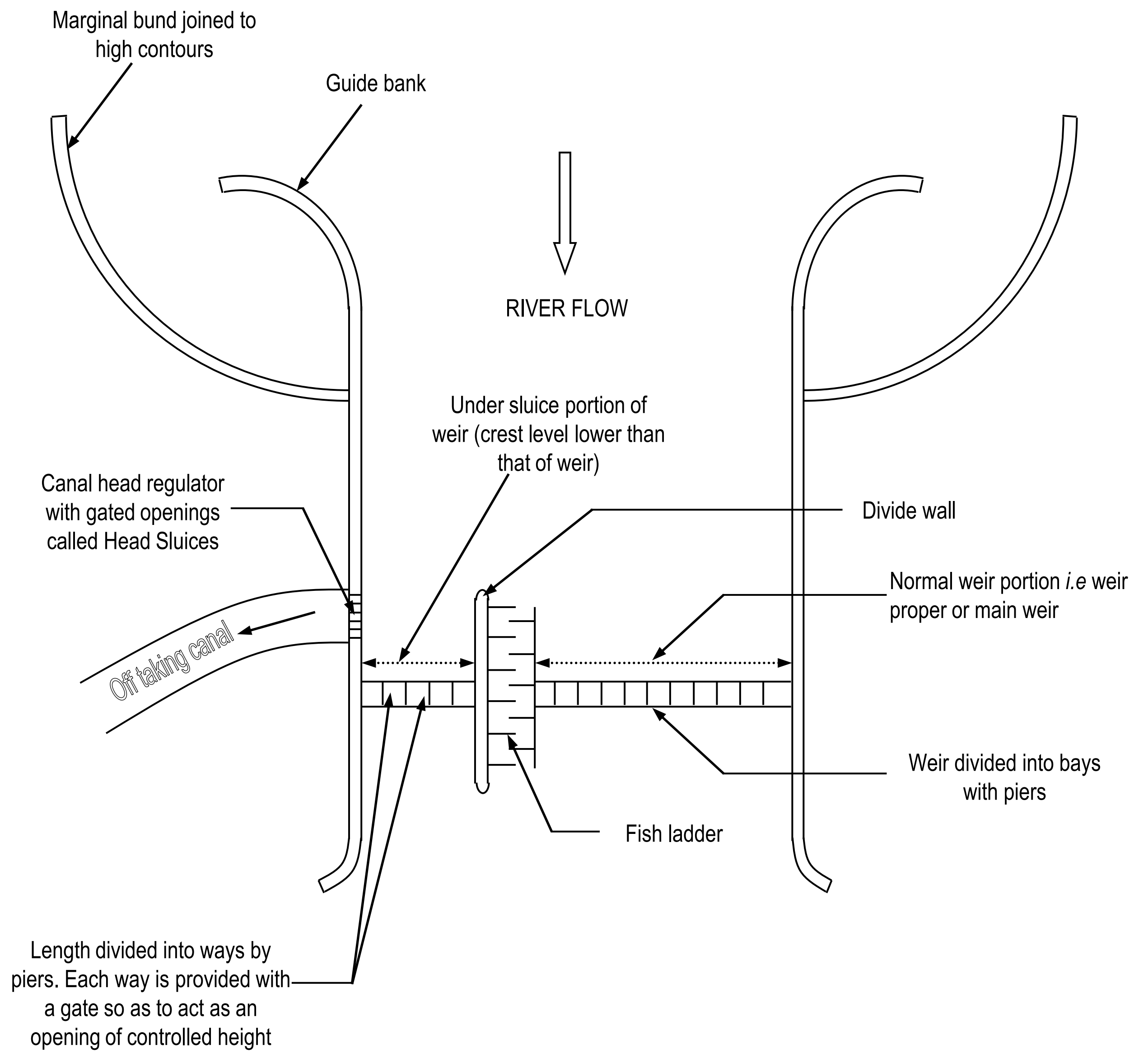
Layout of a Diversion Head Works and its components

A typical layout of a canal head-works is shown in the next slide.

***Such a head-works consists of*:**

(a) Weir portion
(b) Under-sluices
(c) Divide wall
(d) River Training works
(e) Fish Ladder
(f) Canal Head Regulator
(g) Weir's ancillary works, such as shutters, gates, etc.
(h) Silt Regulation Works

Typical Layout of Diversion Head Works



LECTURE 21

Types of Weirs

(a) Masonry weirs with vertical drop
(b) Rock-fill weirs with sloping aprons
(c) Concrete weirs with sloping glacis

Masonry weirs with Vertical Drop

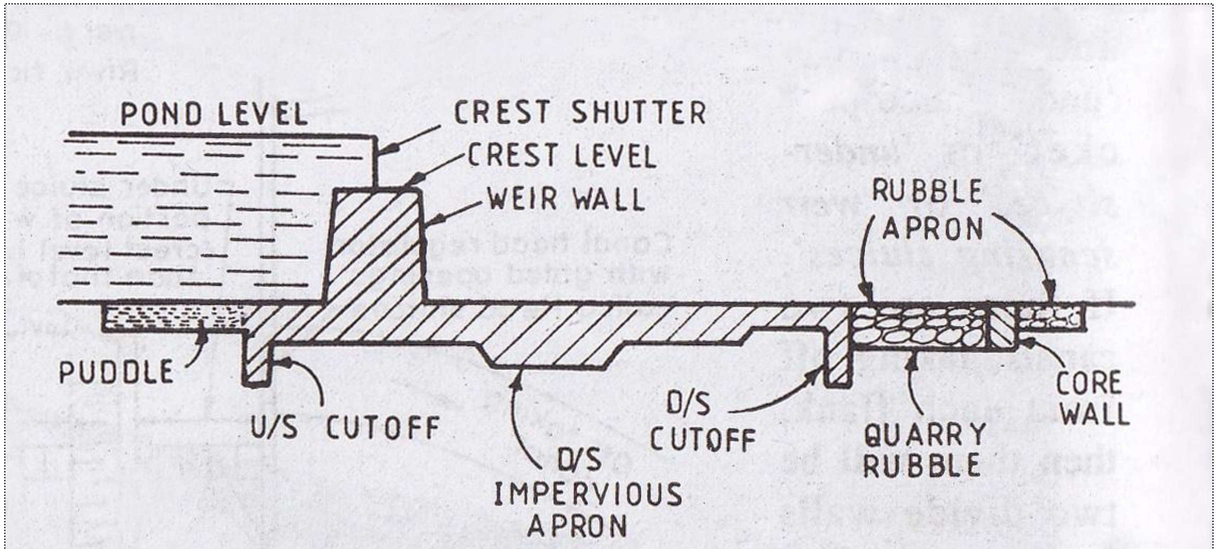


Fig. Masonry weir

Rock-fill weirs with Sloping aprons

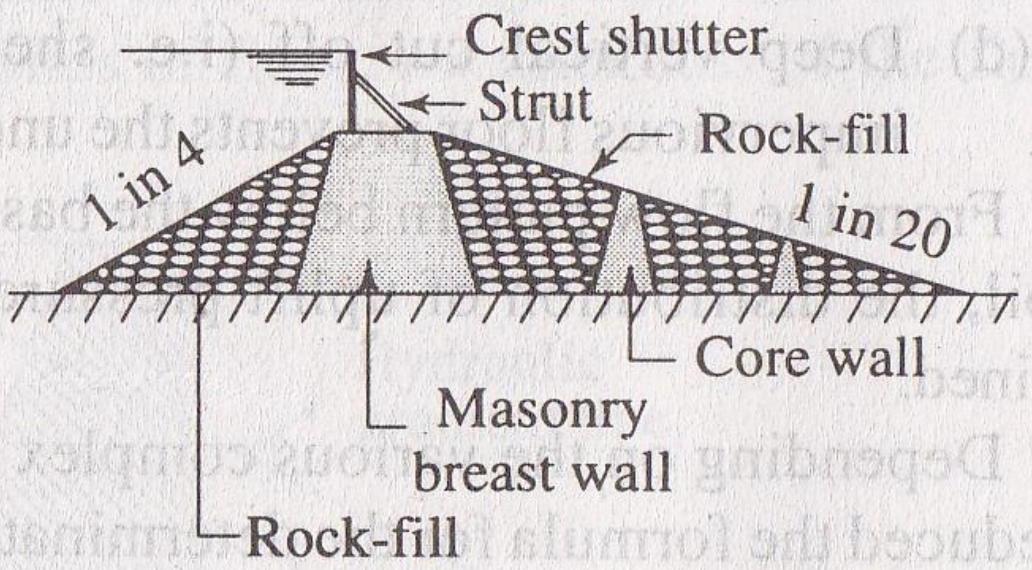


Fig: Rock-fill Weir

Concrete Weir

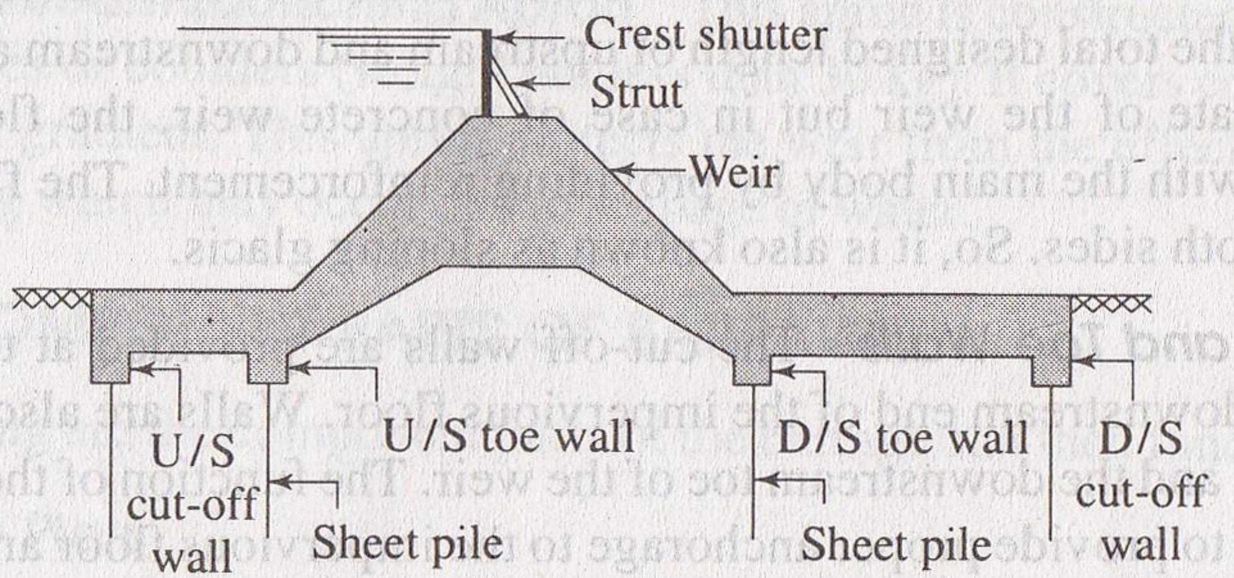


Fig: Concrete Weir

River Training Works

River training works are required near the weir site in order to ensure a smooth and an axial flow of water, and thus, to prevent the river from outflanking the works due to a change in its course.

The river training works required on a canal headwork are:

(a) Guide banks

(b) Marginal bunds

(c) Spurs or groynes

Guide Bank

When a barrage is constructed across a river which flows through the alluvial soil, the guide banks must be constructed on both the approaches to protect the structure from erosion.

Guide bank serves the following purposes:

- ✓ It protects the barrage from the effect of scouring and erosion.
- ✓ It provides a straight approach towards the barrage.
- ✓ It controls the tendency of changing the course of the river.
- ✓ It controls the velocity of flow near the structure.

Marginal Bunds

The *marginal bunds* are earthen embankments which are constructed parallel to the river bank on one or both the banks according to the condition. The top width is generally 3 m to 4 m. The side slope on the river side is generally 1.5: 1 and that on the country side is 2:1.

The marginal bunds serve the following purposes:

- It prevents the flood water or storage water from entering the surrounding area which may be submerged or may be water logged.
- It retains the flood water or storage water within a specified section.
- It protects the towns and villages from devastation during the heavy flood.
- It protects valuable agricultural lands.

Spurs

These are temporary structures permeable in nature provided on the curve of a river to protect the river bank from erosion. These are projected from the river bank towards the bed making angles 60^o to 75^o with the bank of the river. The length of the spurs depends on the width of the river and the sharpness of the curve.

Types of Spur

- Bamboo spur
- Timber spur
- Boulder spur

Bamboo and Timber Spur

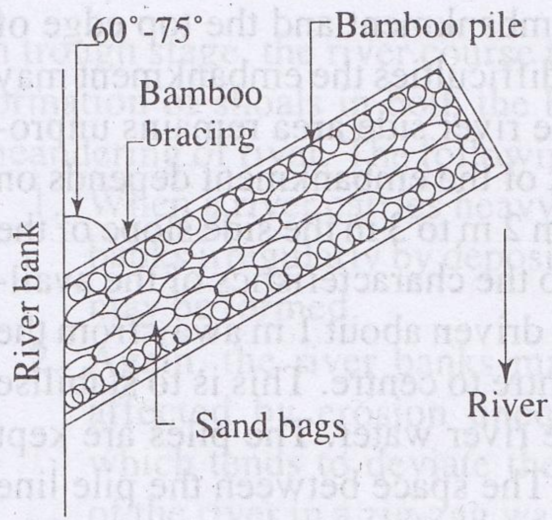


Fig: Bamboo Spur

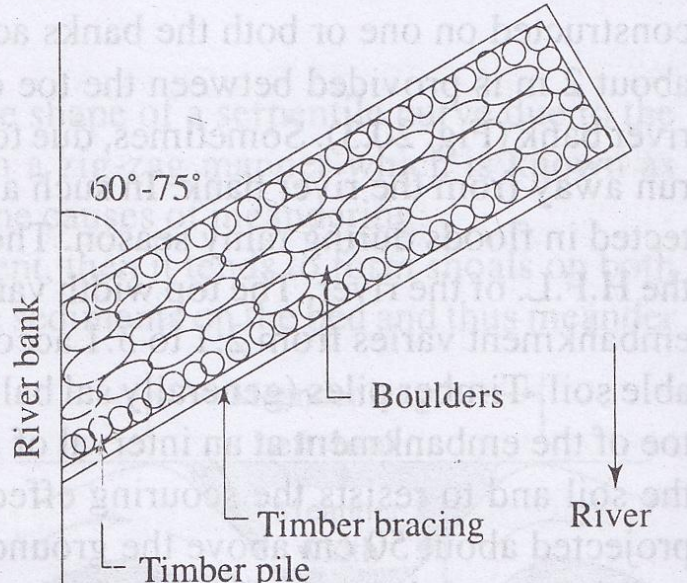


Fig: Timber Spur

Boulder Spur

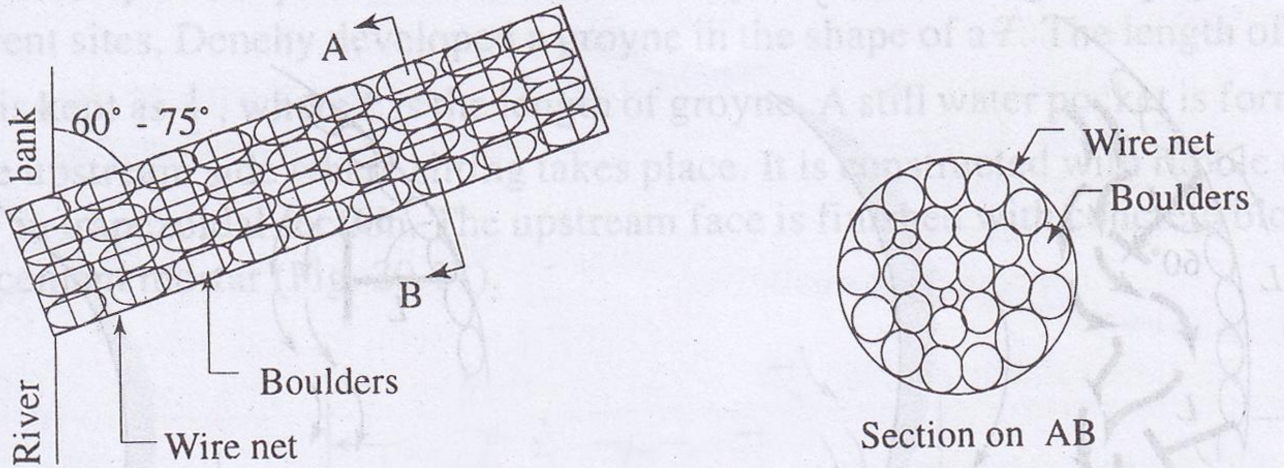


Fig: Boulder Spur

Groynes

The function of groynes is similar to that of spur. But these are impervious permanent structures constructed on the curve of a river to protect the river bank from erosion. They extend from the bank towards the bed by making an angle of 60^o to 75^o with the bank. The angle may be towards the upstream or downstream. Sometimes, it is made perpendicular to the river bank.

Types of Groyne

(a) Attracting Groyne
(b) Repelling Groyne
(c) Deflecting Groyne

Attracting Groyne

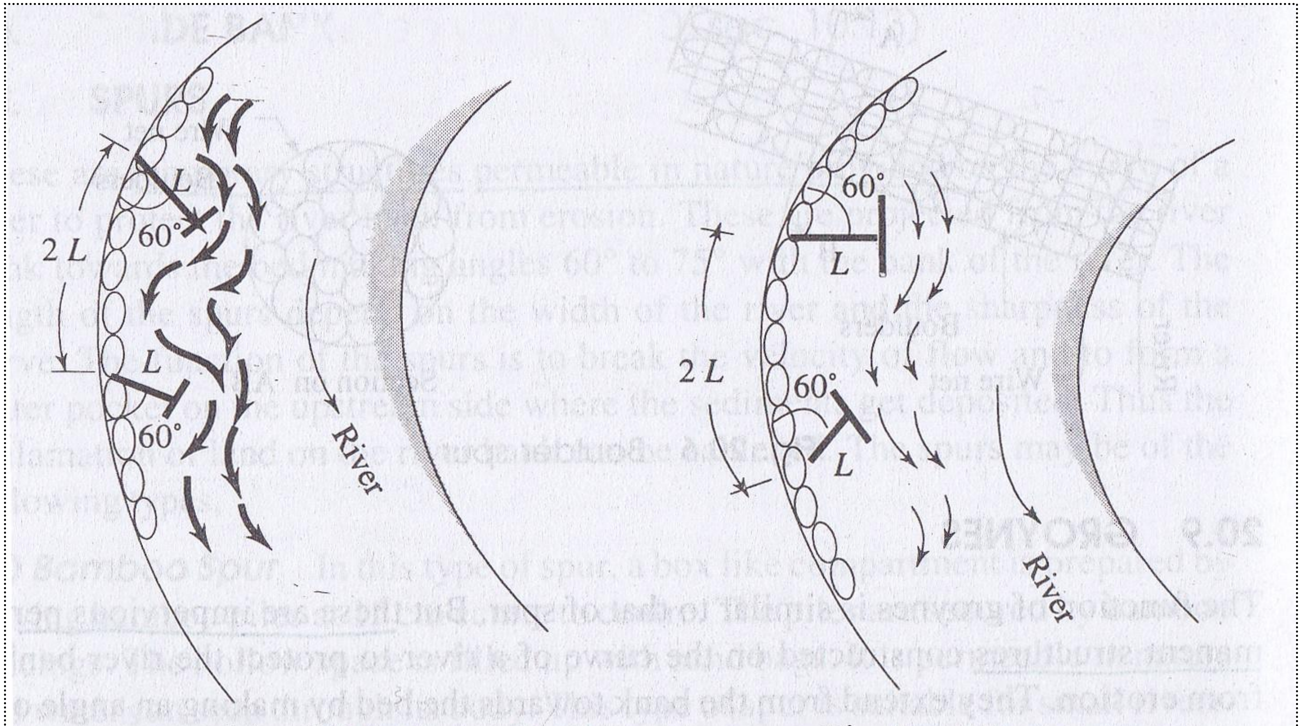


Fig: Attracting groyne

Fig: Repelling groyne

Deflecting Groyne

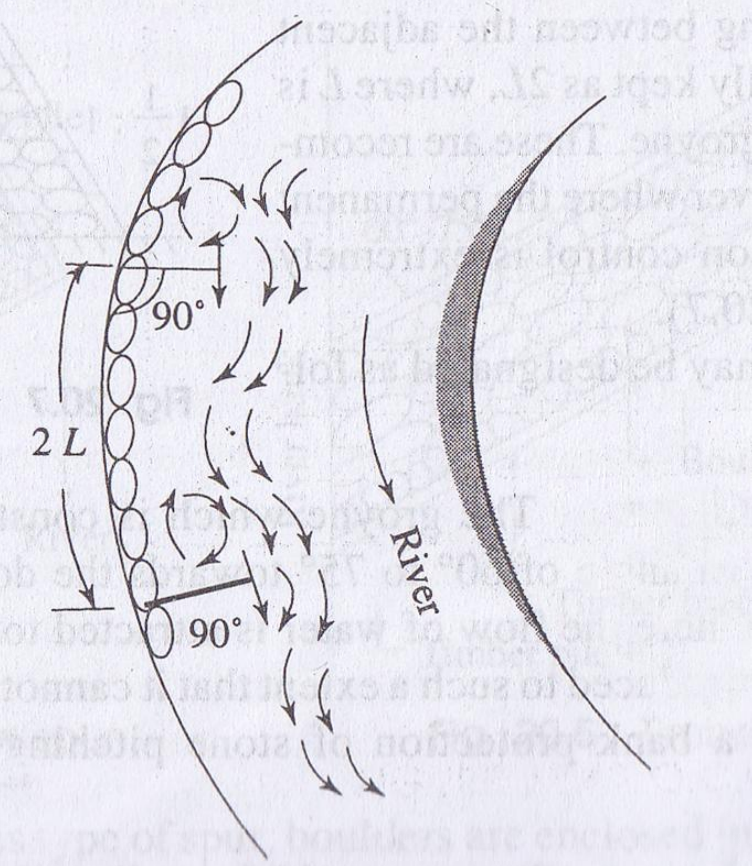


Fig: Deflecting Groyne

Lecture 21

Fish Ladder

The fish ladder is provided just by the side of the divide wall for the free movement of fishes.

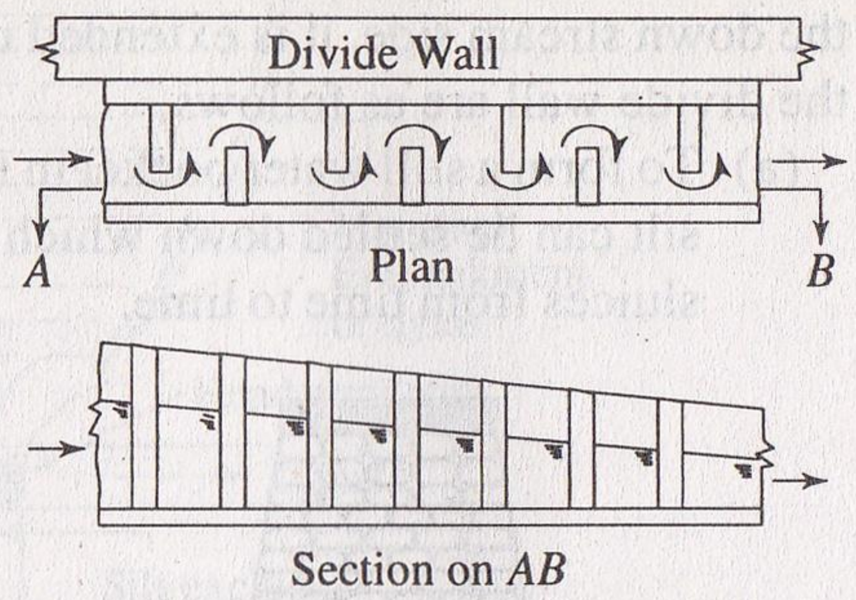


Fig: Fish Ladder

Cont……. Fish Ladder….

Fishpass at Shariakandi Bogra

Fish Friendly Structure at Tangail

Lecture 21

Fish Pass

Silt Regulation Works

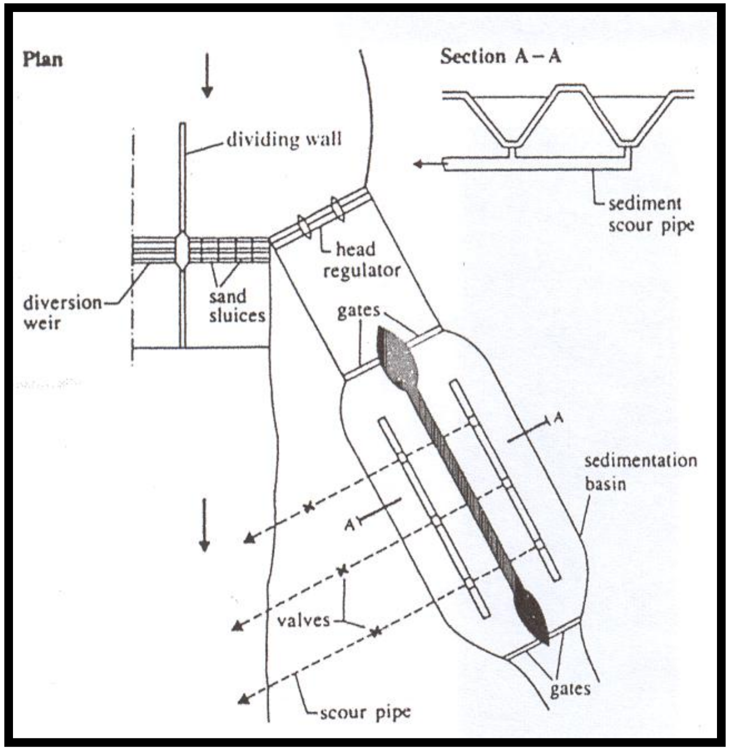
The entry of silt into a canal, which takes off from a head works, can be reduced by constructed certain special works, called silt control works.

Two types of Silt regulation works:

(a) Silt Excluders
(b) Silt Ejectors

(a) Silt Excluders

Silt excluders are those works which are constructed on the bed of the river, upstream of the head regulator. The clearer water enters the head regulator and silted water enters the silt excluder. In this type of works, the silt is, therefore, removed from the water before in enters the canal.



(b) Silt Ejectors

Silt ejectors, also called silt extractors, are those devices which extract the silt from the canal water after the silted water has traveled a certain distance in the off-take canal. These works are, therefore, constructed on the bed of the canal, and little distance downstream from the head regulator.

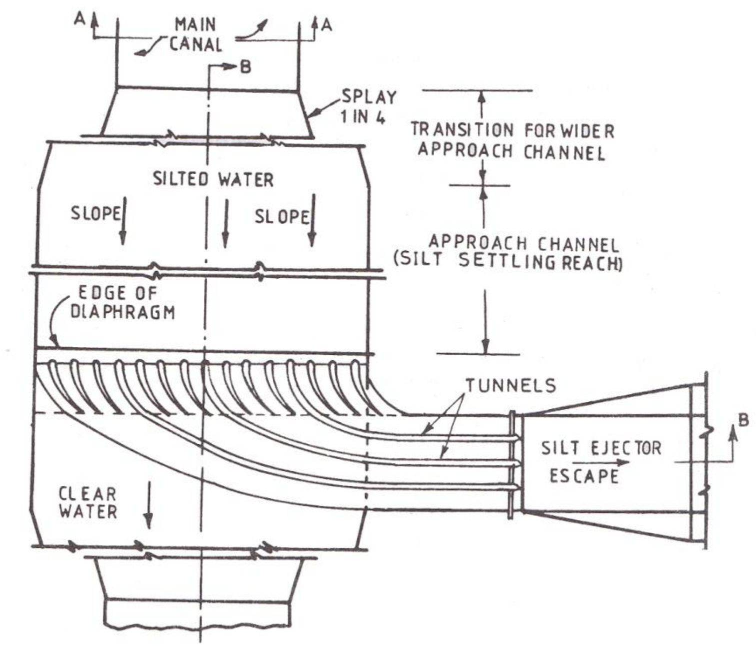


Fig: Plan of Silt Ejector

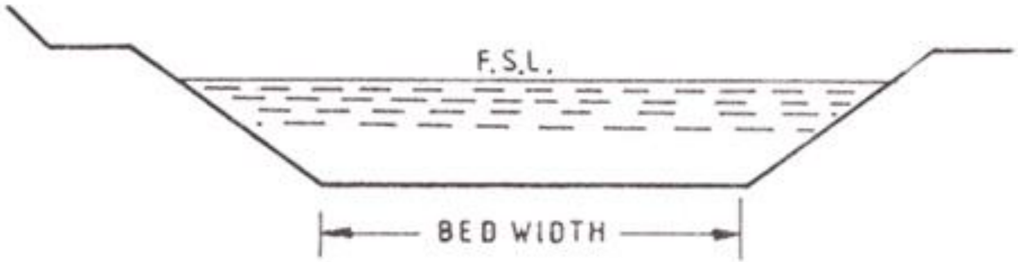


Fig: Section A-A
(Main Canal)

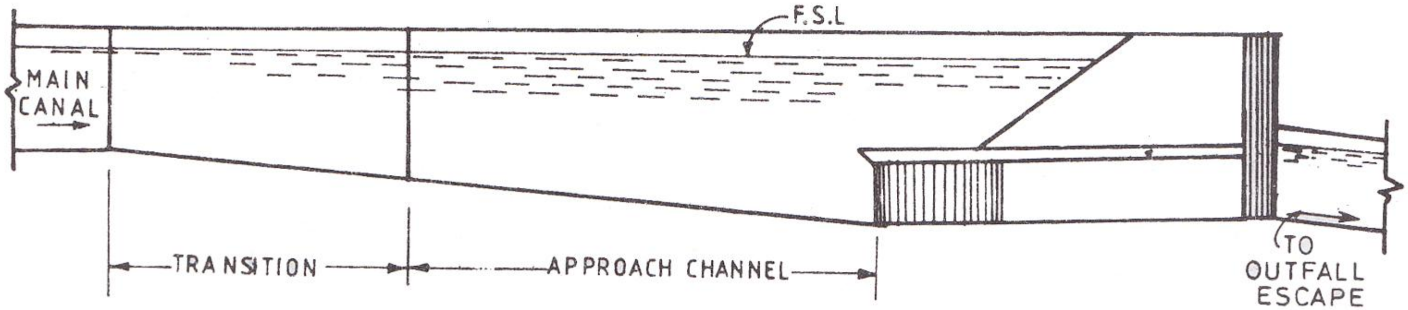


Fig: L-section along tunnel
(Section B-B)

Teesta Barrage

Lecture 21

Head Regulator

Head Regulator at teesta

Canal Head Regulator or Head Sluices

A canal head regulator (C.H.R) is provided at the head of the off-taking canal, and serves the **following functions**:

- It regulates the supply of water entering the canal
- It controls the entry of silt in the canal
- It prevents the river-floods from entering the canal

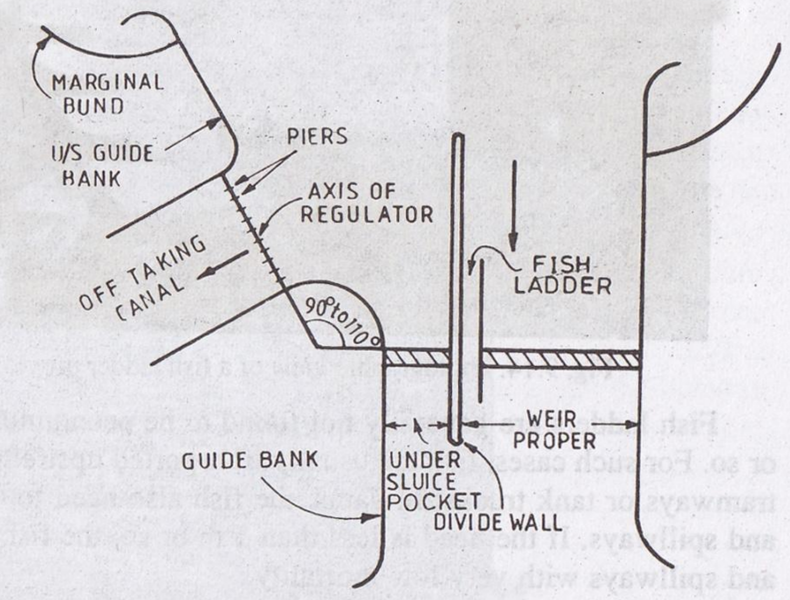


Fig: Alignment of a canal head regulator

End of Chapter – 8

CHAPTER – 9

Irrigation Structures – 2

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LECTURE 22

What is Cross Drainage Works?

In an irrigation project, when the network of main canals, branch canals, distributaries, etc. are provided, then these canals may have to cross the natural drainages like rivers, streams, nallahs, etc at different points within the command area of the project. The crossing of the canals with such obstacle cannot be avoided. So, suitable structures must be constructed at the crossing point for the easy flow of water of the canal and drainage in the respective directions. These structures are known as ***cross-drainage works***.

Navigation Lock

Cont…..Navigation Lock

Lecture 22

Necessity of Cross Drainage Works

- The water-shed canals do not cross natural drainages. But in actual orientation of the canal network, this ideal condition may not be available and the obstacles like natural drainages may be present across the canal. So, the cross drainage works must be provided for running the irrigation system.

- At the crossing point, the water of the canal and the drainage get intermixed. So, far the smooth running of the canal with its design discharge the cross drainage works are required.

- The site condition of the crossing point may be such that without any suitable structure, the water of the canal and drainage can not be diverted to their natural directions. So, the cross drainage works must be provided to maintain their natural direction of flow.

Types of Cross Drainage Works

- **Type I (Irrigation canal passes over the drainage)**
  - (a) Aqueduct
  - (b) Siphon Aqueduct

- **Type II (Drainage passes over the irrigation canal)**
  - (a) Super passage
  - (b) Siphon super passage

- **Type III (Drainage and canal intersection each other of the same level)**
  - (a) Level crossing
  - (b) Inlet and outlet

Selection of Type of Cross Drainage Works

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

Aqueduct

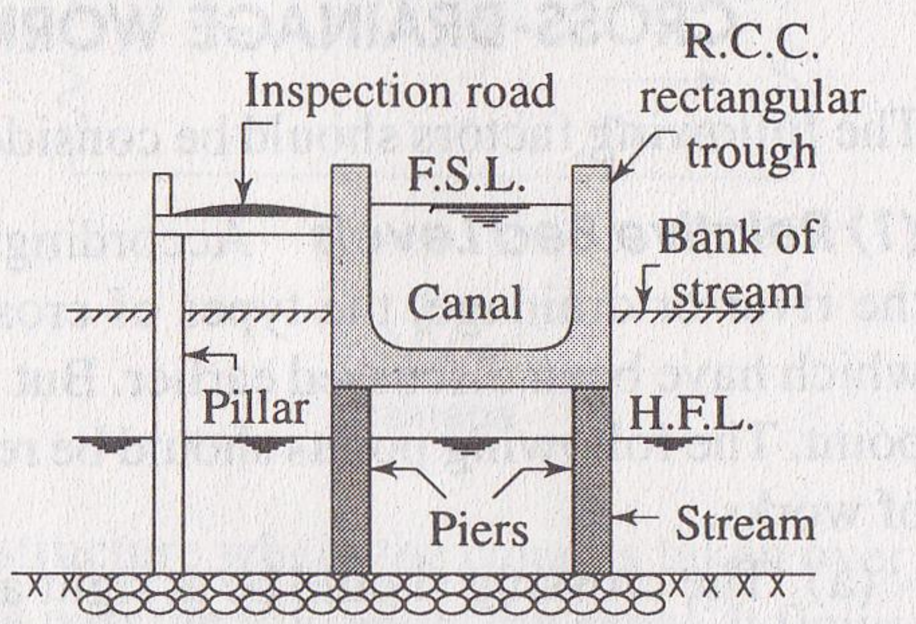


Fig: Aqueduct

Cont……. Aqueduct

Cont……. Aqueduct

Siphon Aqueduct

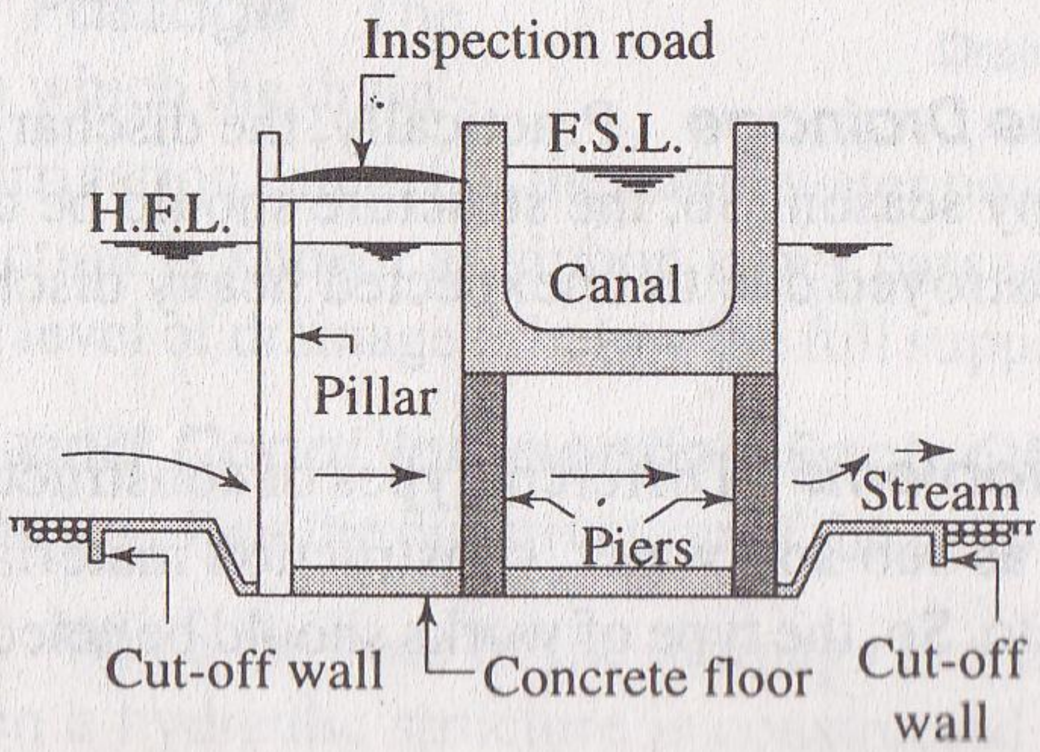


Fig: Siphon Aqueduct

Super Passage

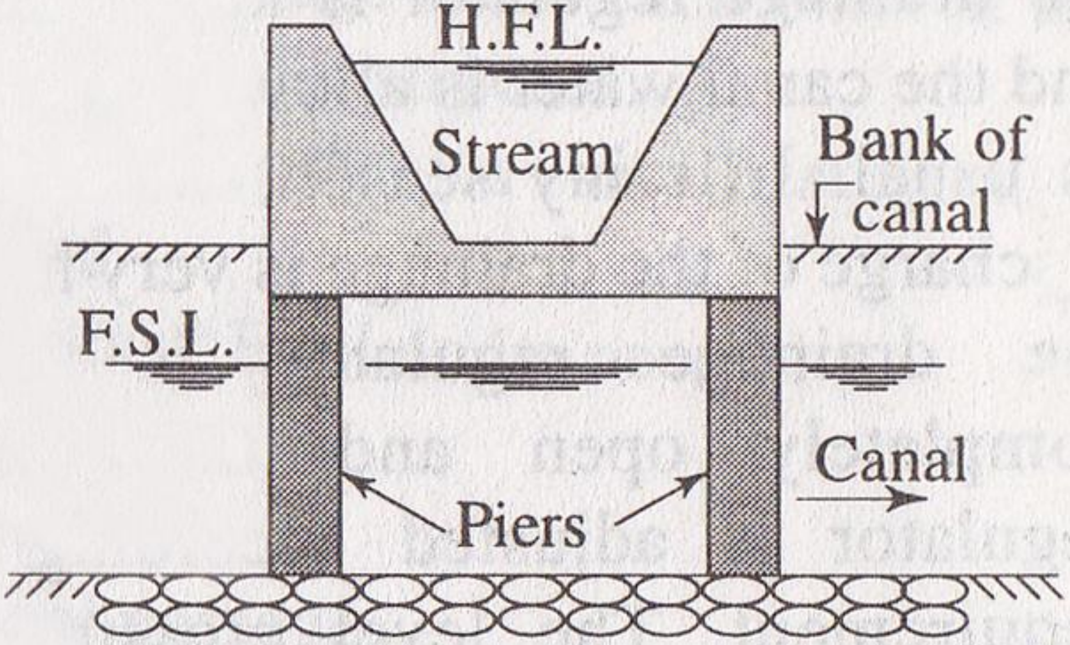


Fig: Super Passage

Siphon Super Passage

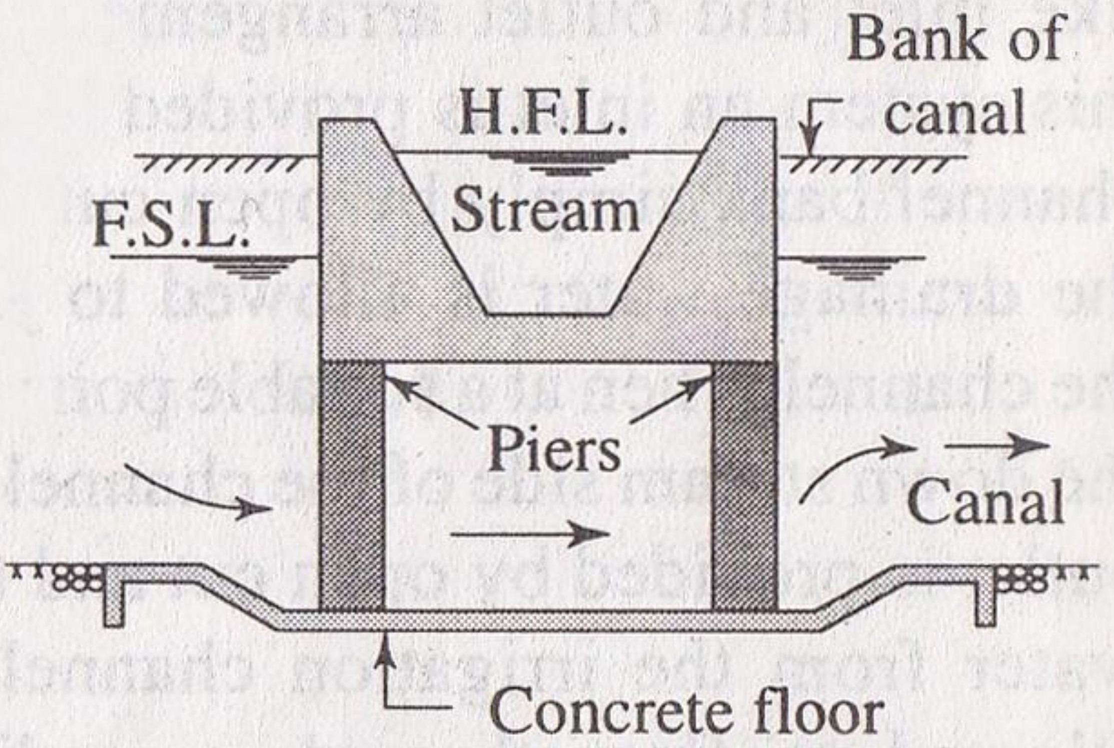


Fig: Siphon Super Passage

Level Crossing

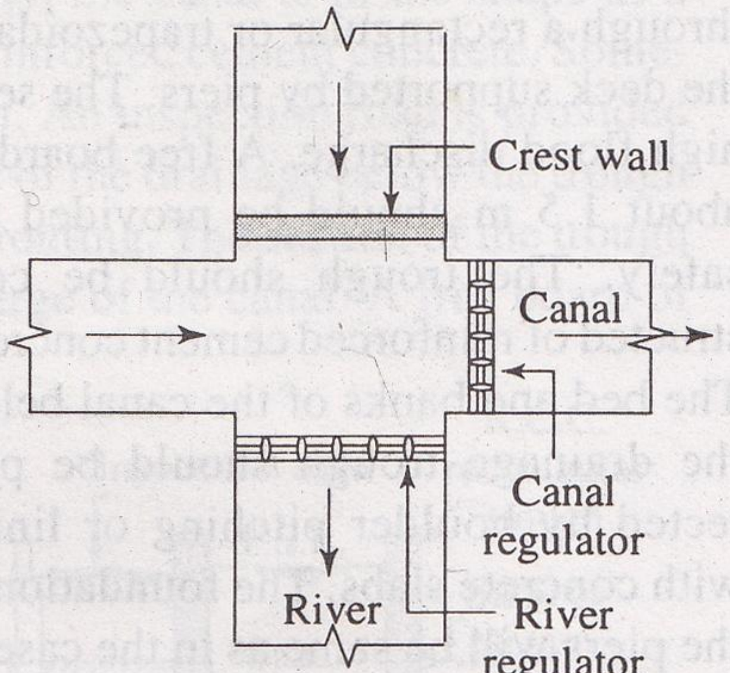


Fig: Level Crossing

Inlet and Outlet

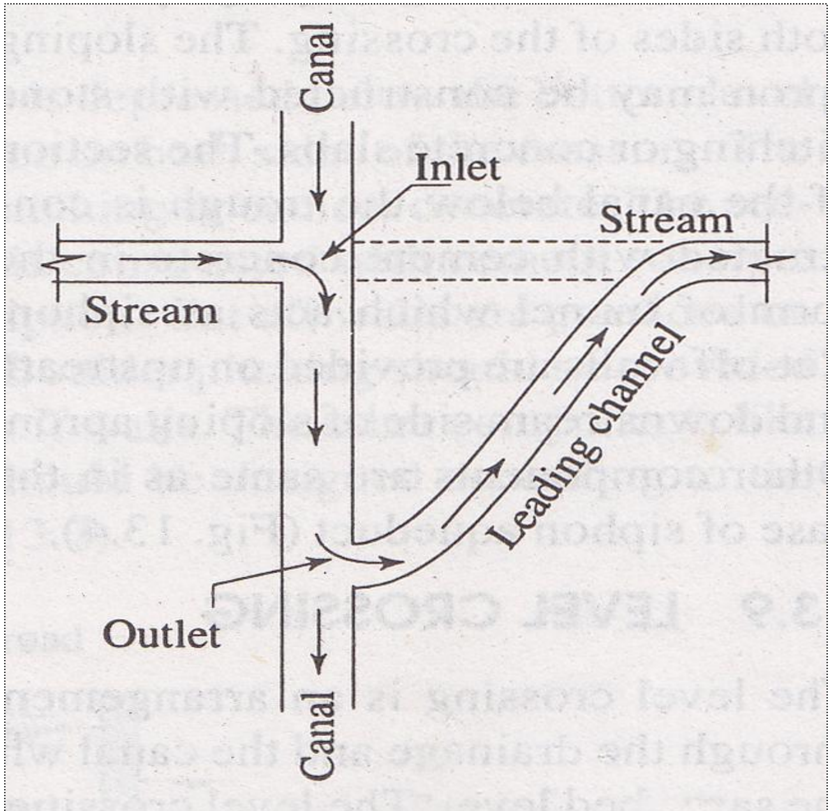


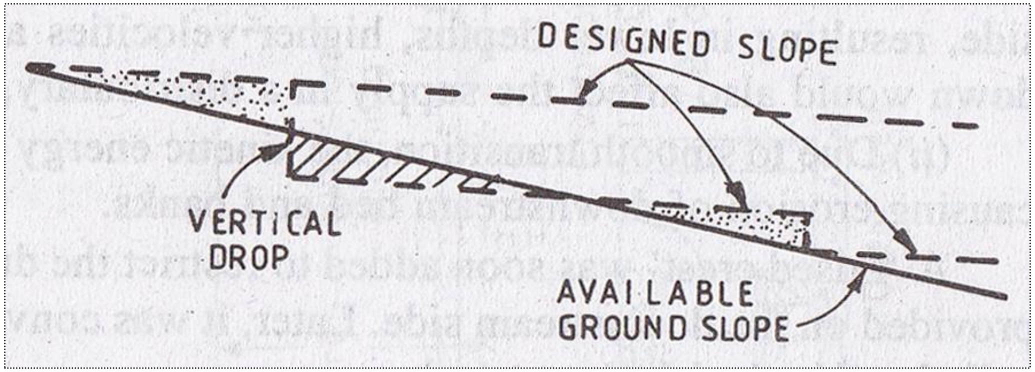
Fig: Inlet and outlet

LECTURE 23

What is Canal Fall?

Whenever the available natural ground slope is steep than the designed bed slope of he channel, the difference is adjusted by constructing vertical 'falls' or 'drops' in the canal bed at suitable intervals, as shown in figure below.

Such a drop in a natural canal bed will not be stable and, therefore, in order to retain this drop, a masonry structure is constructed. Such a pucca structure is called a Canal Fall or a Canal drop.



Necessity of Canal Falls

- When the slope of the ground suddenly changes to steeper slope, the permissible bed slope can not be maintained. It requires excessive earthwork in filling to maintain the slope. In such a case falls are provided to avoid excessive earth work in filling (**Fig. 1**)

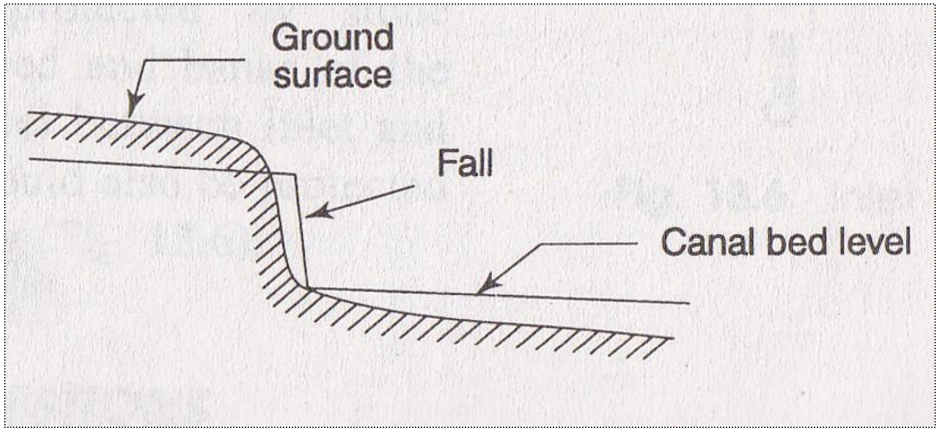


Fig. 1

- When the slope of the ground is more or less uniform and the slope is greater than the permissible bed slope of canal. (**Fig.2**)

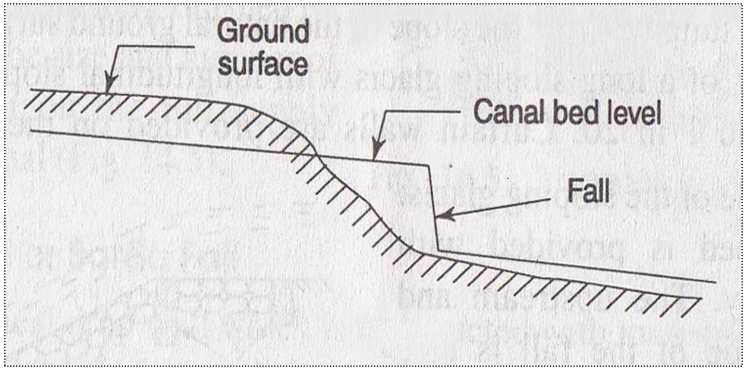


Fig. 2

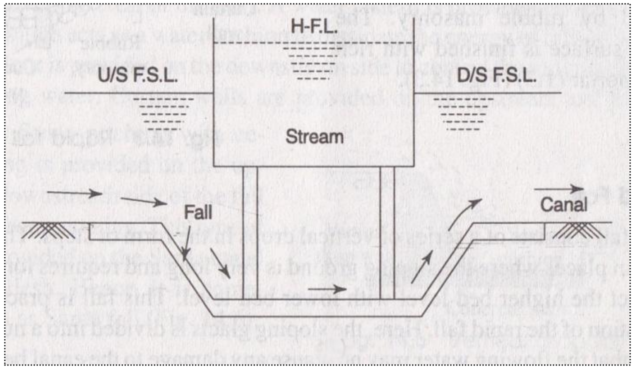


Fig. 3

- In cross-drainage works, when the difference between bed level of canal and that of drainage is small or when the F.S.L of the canal is above the bed level of drainage then the canal fall is necessary to carry the canal water below the stream or drainage. (**Fig. 3**)

Types of Canal Falls

- Ogee Fall
- Rapid Fall
- Stepped Fall
- Trapezoidal Notch Fall
- Vertical Drop Fall
- Glacis Fall
    - (a) Montague Type Fall
    - (b) Inglis Type Fall

Ogee Fall

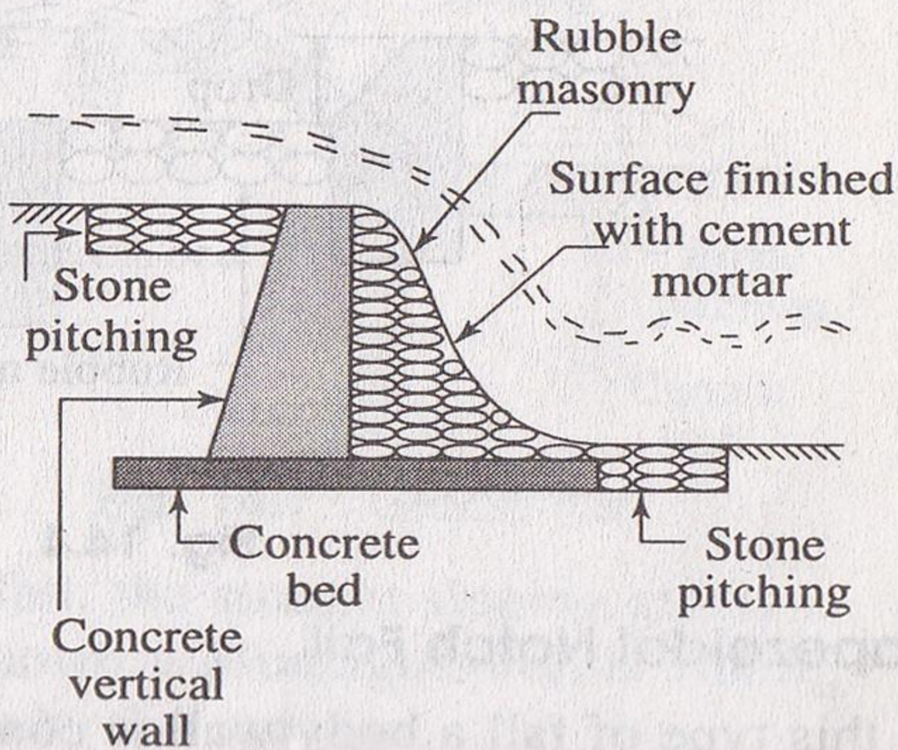


Fig: Ogee Fall

Rapid Fall

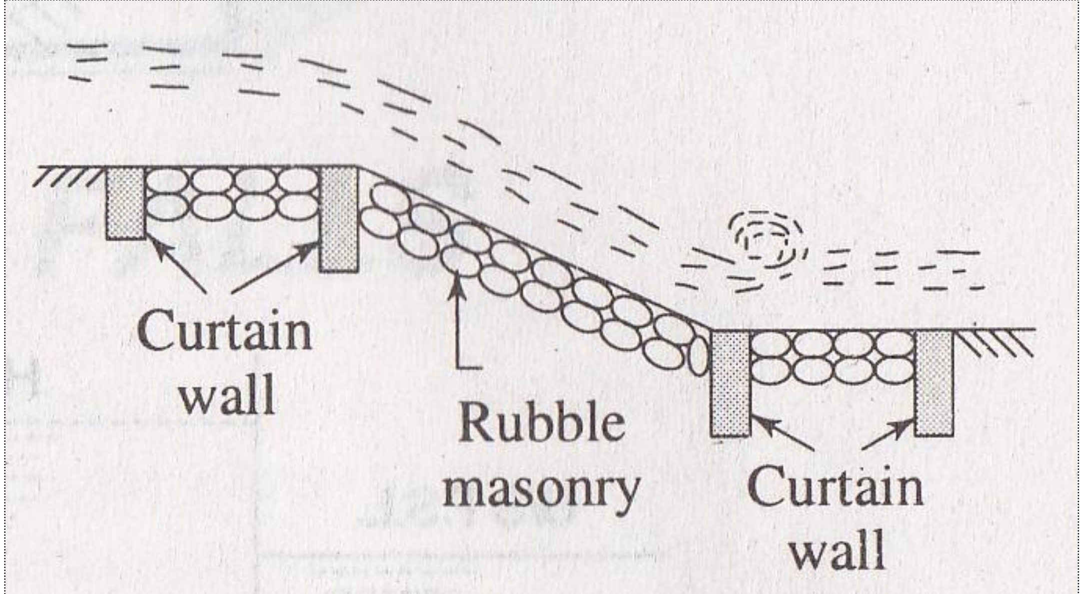


Fig: Rapid Fall

Stepped Fall

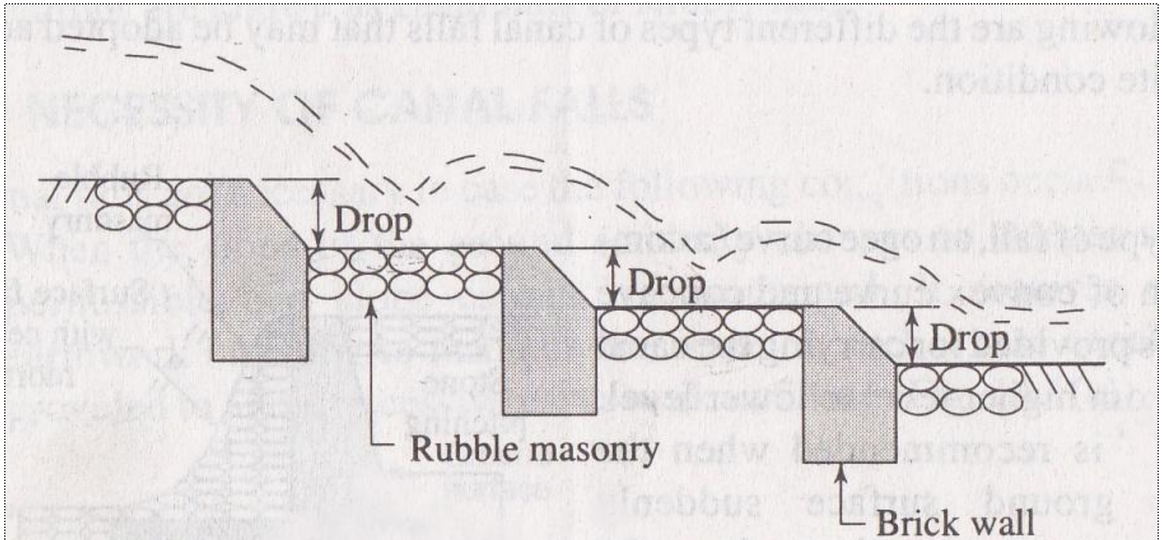


Fig: Stepped Fall

Trapezoidal Notch Fall

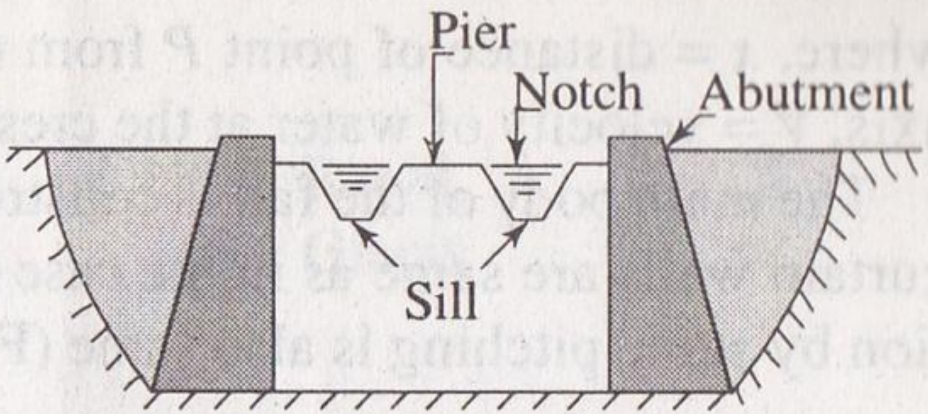


Fig: Trapezoidal Notch Fall

Vertical Drop Fall

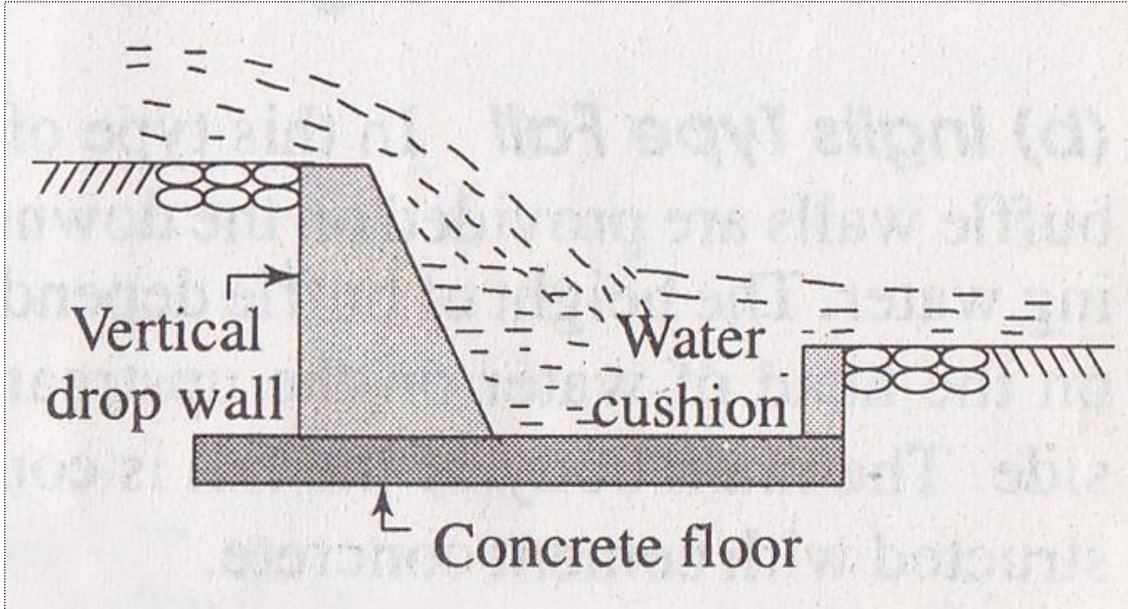


Fig: Vertical Drop Fall

Glacis Fall

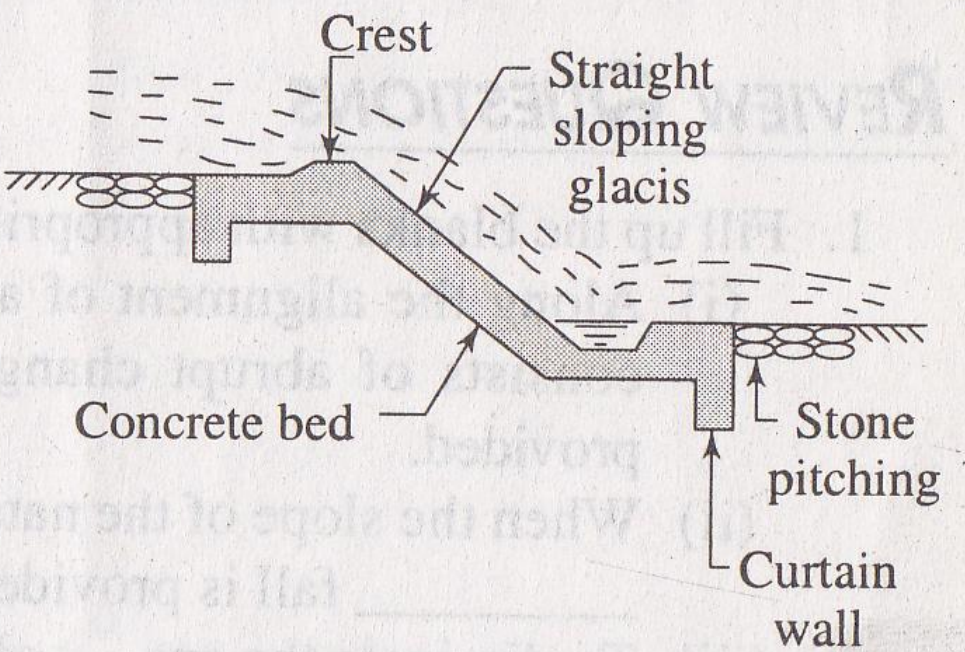


Fig: Glacis Fall

(a) Montague Type Fall

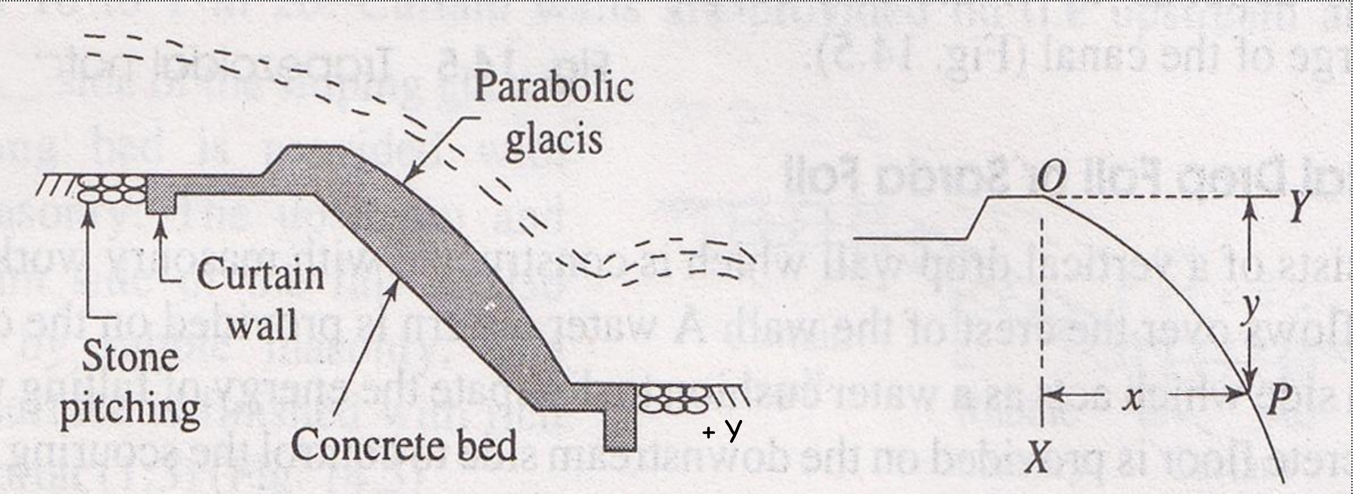


Fig: Montague Type Fall

$$X = \upsilon \sqrt{\frac{4y}{g}}$$

Where, x = distance of point P from OX axis,
Y = distance of point P from OY axis,
υ = velocity of water at the crest,
g = acceleration due to gravity

(b) Inglis Type Fall

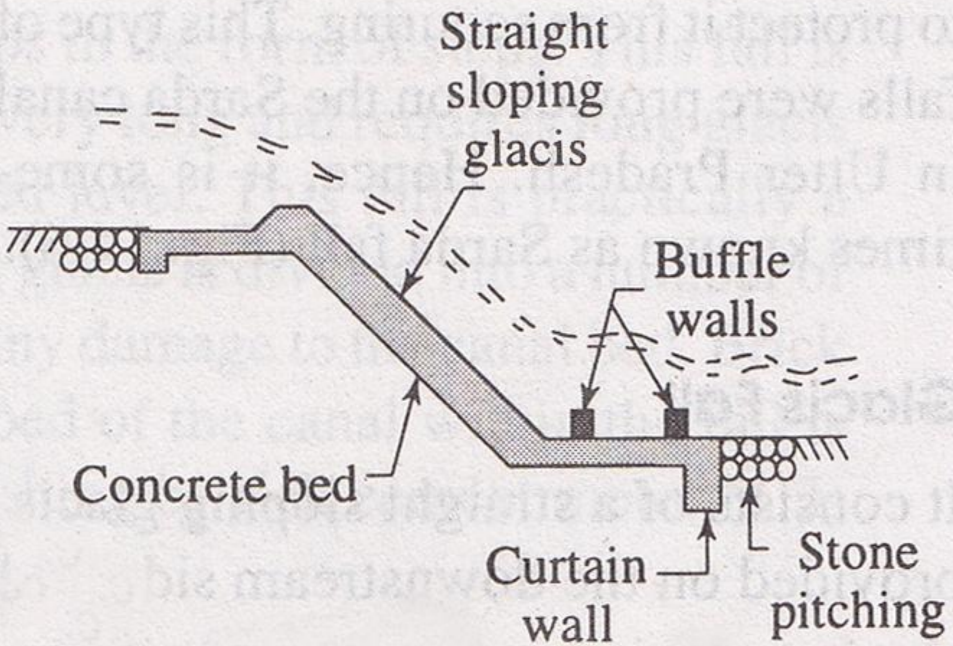
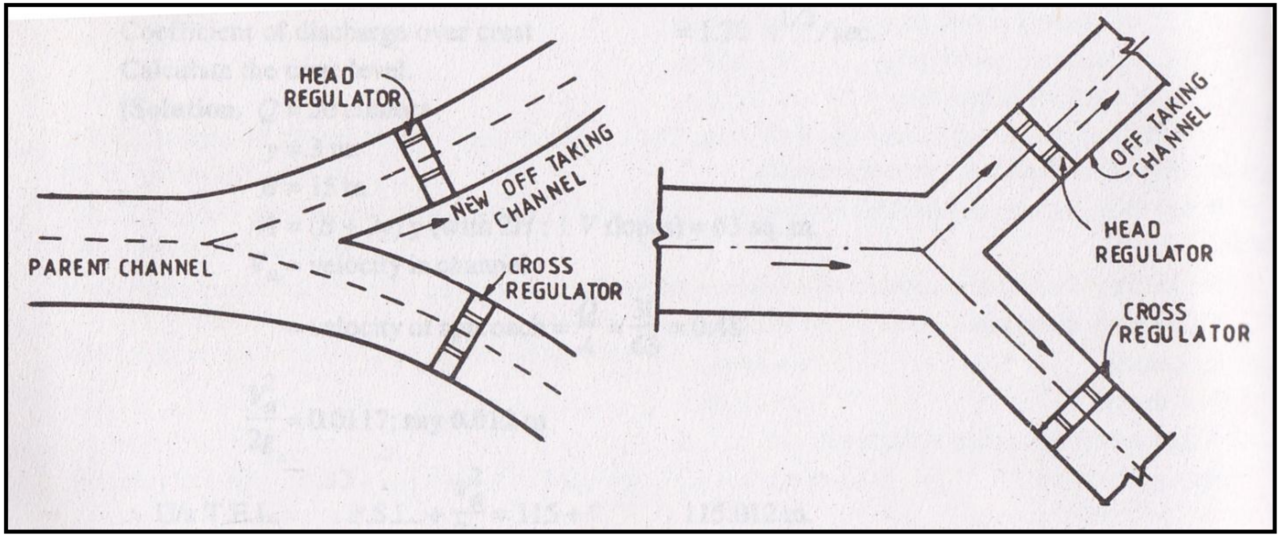


Fig: Inglis Type Fall

LECTURE 24

Canal Regulators



A head regulator provided at the head of the off-taking channel, controls the flow of water entering the new channel.

While a cross regulator may be required in the main channel downstream of the off-taking channel, and is operated when necessary so as to head up water on its upstream side, thus to ensure the required supply in the off-taking channel even during the periods of low flow in the main channel.

Main functions of a head regulator:

- To regulate or control the supplies entering the off-taking canal
- To control the entry of silt into the off-taking canal
- To serve as a meter for measuring discharge.

Main functions of a cross regulator:

- To control the entire Canal Irrigation System.
- To help in heading up water on the upstream side and to fed the off-taking canals to their full demand.
- To help in absorbing fluctuations in various sections of the canal system, and in preventing the possibilities of breaches in the tail reaches.
- Cross regulator is often combined with bridges and falls, if required.

Typical layout and cross-section of a regulator

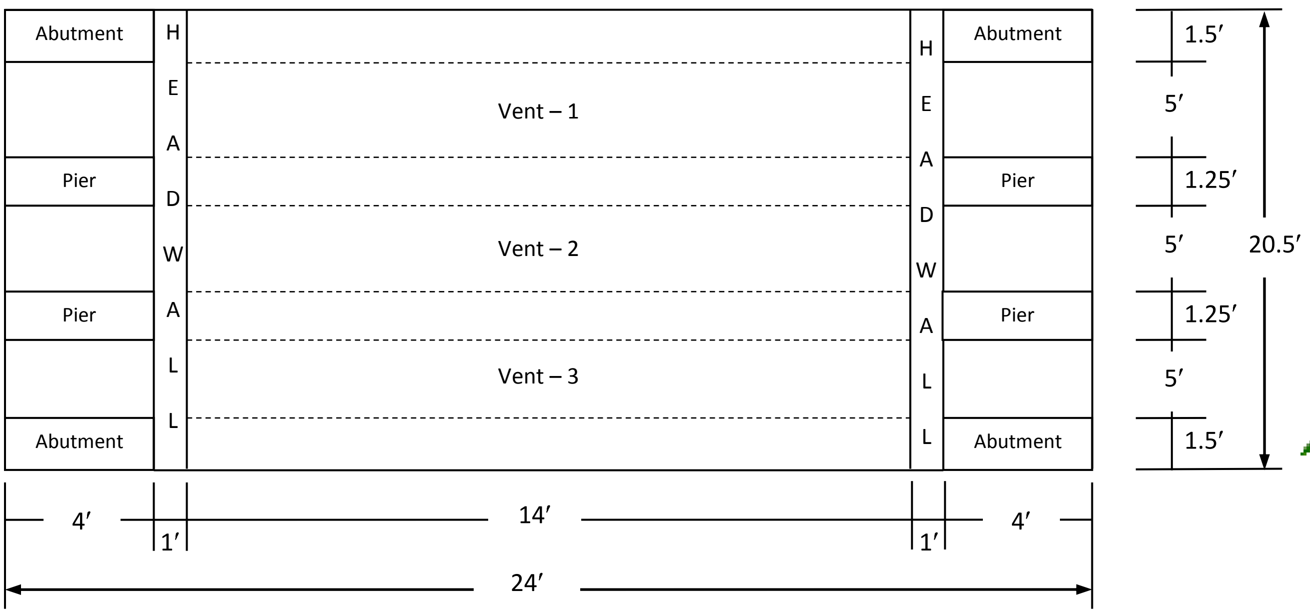


Fig: Plan view of a 3-vent regulator

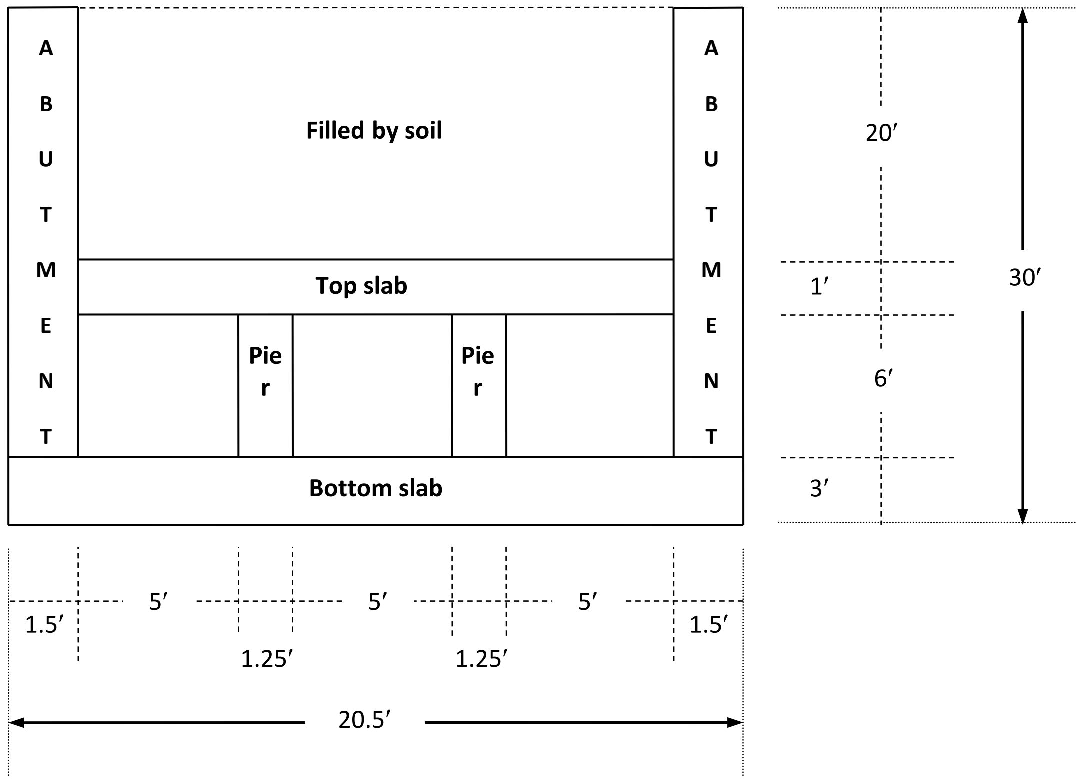


Fig: Front Elevation of a 3-vent regulator

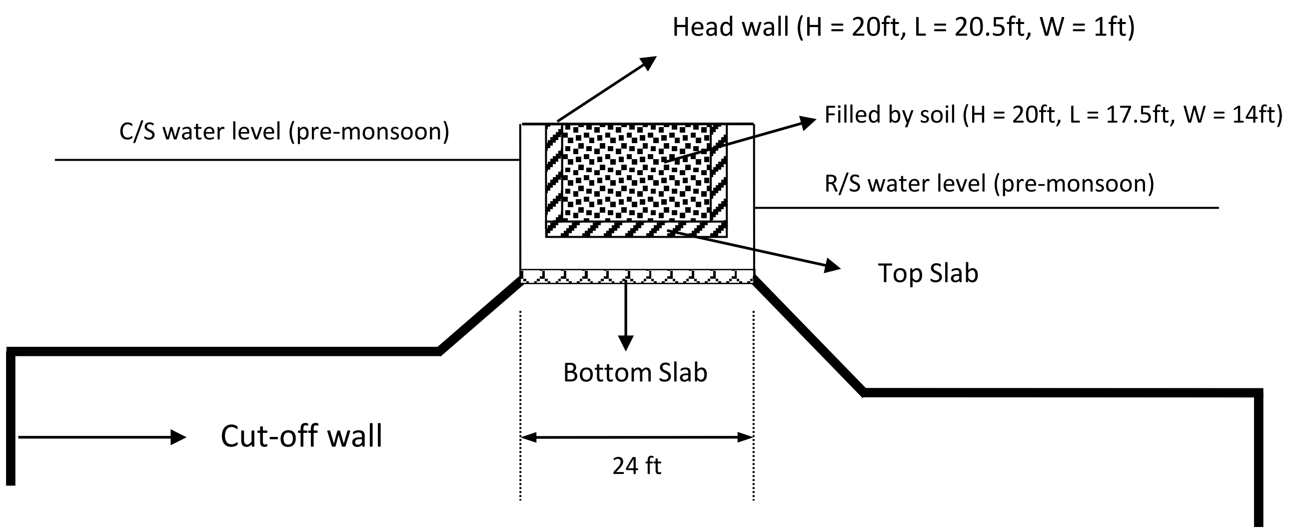


Fig: Longitudinal Section of a 3-vent regulator

Canal Escape

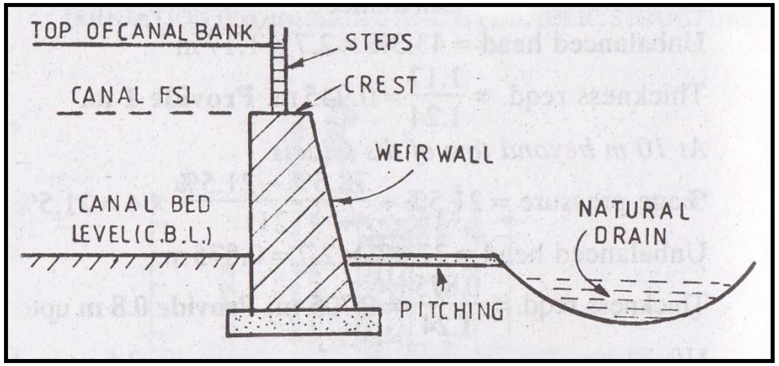
It is a side channel constructed to remove surplus water from an irrigation channel (main canal, branch canal, or distributary etc.) into a natural drain.

The water in the irrigation channel may become surplus due to -

- Mistake
- Difficulty in regulation at the head
- Excessive rainfall in the upper reaches
- Outlets being closed by cultivators as they find the demand of water is over

Types of Canal Escapes:

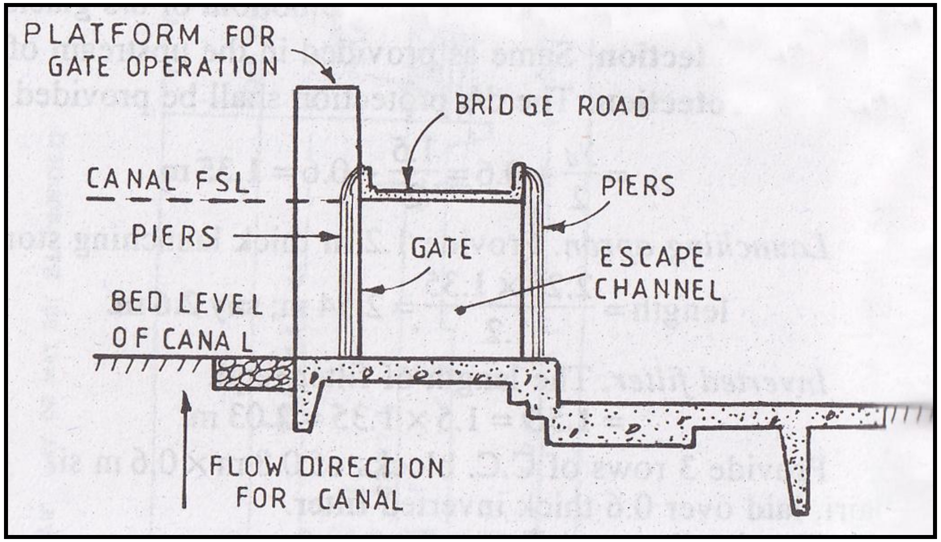
(a) Weir type escape:



Crest level = FSL of the canal
Water escapes if w_L > FSL

The crest of the weir wall is kept at R.L equal to canal FSL. When the water level rises above FSL, it gets escaped.

(a) Regulator/ sluice type escape:



The silt of the escape is kept at canal bed level and the flow can be used for completely emptying the canal.
They may be constructed for the purpose of scouring off excess bed silt deposited in the head reaches from time to time.

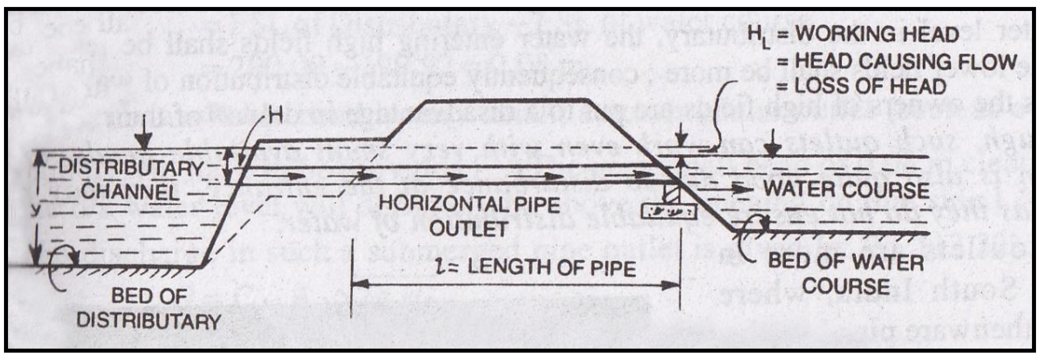
Canal Outlet/modules

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 distributary channel.

It acts as a connecting link between the system manager and the farmers.

Types of Outlet/modules:

(a) Non-modular modules:



Non-modular modules are those through which the discharge depends upon the head difference between the distributary and the water course.
Common examples are:
(i) Open sluice
(ii) Drowned pipe outlet

(b) Semi-modules or Flexible modules:

- Due to construction, a super-critical velocity is ensured in the throat and thereby allowing the formation of a jump in the expanding flume.
- The formation of hydraulic jump makes the outlet discharge independent of the water level in water course, thus making it a semi module.Semi-modules or flexible modules are those through which the discharge is independent of the water level of the water course but depends only upon the water level of the distributary so long as a minimum working head is available.
- Examples are pipe outlet, open flume type etc.

(c) Rigid modules or Modular Outlets:

Rigid modules or modular outlets are those 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 both.
An example is Gibb's module:

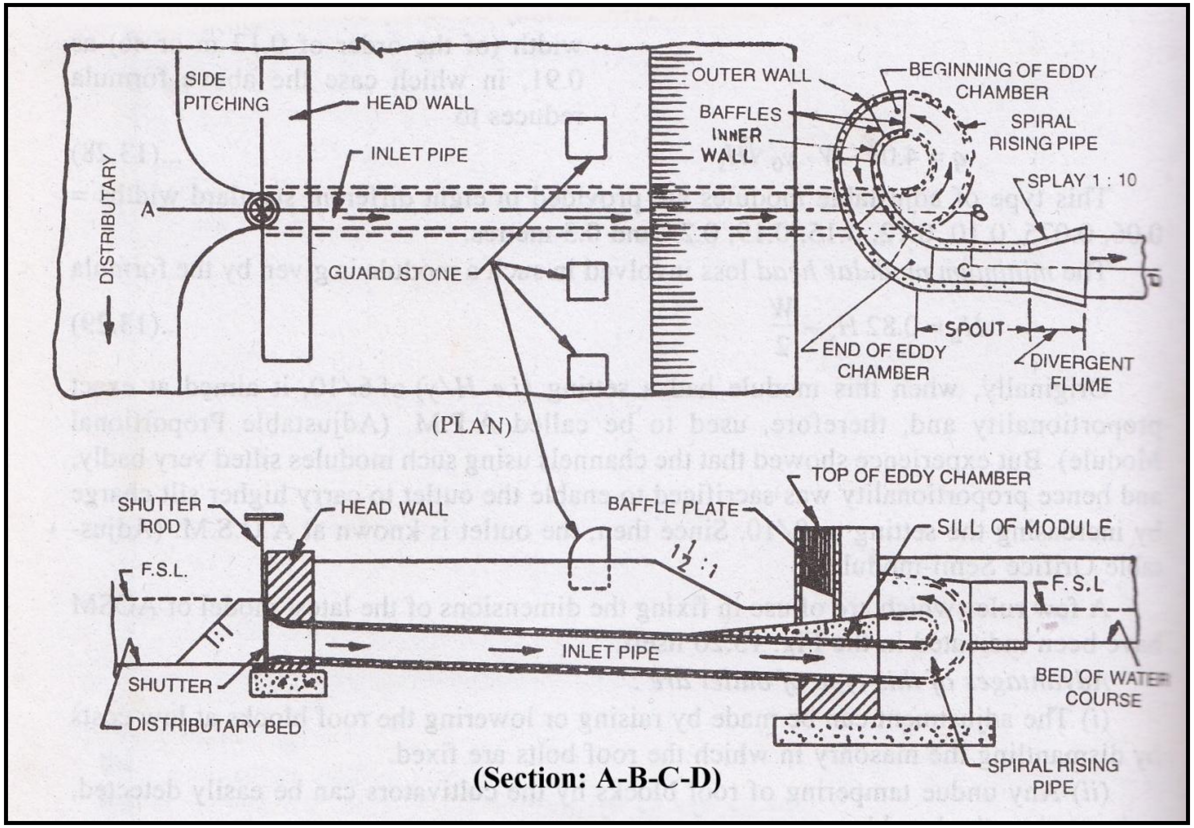


Fig: Gibb's Module

LECTURE 25

Performance Criteria

(a) Flexibility, F:

It is defined as the ratio of the 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, F = Flexibility of the outlet
q = Discharge passing through the outlet
Q = Discharge in the distributary channel

If H = the head acting on the outlet,

$$q = CH^m$$

Where, C and m are constants depending upon the type of outlet

If y = the depth of water in the distributary,

$$Q = Ky^n$$

Where, k and n are constants

$$\frac{dq}{dH} = CmH^{m-1} = (CH^m)\times(m/H) = q\times \frac{m}{H}$$

$$\Rightarrow \frac{dq}{q} = \frac{m}{H} \times dH$$

Again, $\frac{dQ}{dy} = Kny^{n-1} = (Ky^n)\times(n/y) = Q\times \frac{n}{y} \Rightarrow \frac{dQ}{Q} = \frac{n}{y}$ y

Thus, $F = \frac{\frac{m}{H}\times dH}{\frac{n}{y}\times dy} = \frac{m}{n} \quad \frac{y}{H} \quad \frac{dH}{dy}$

A change in water depth of the distributary (dy) would result in an equal change in the head working on the outlet (dH), so that

$$dy = dH$$

So, $F = \frac{m}{n} \quad \frac{y}{H}$

(b) Proportionality:

The outlet is said to be proportional when the rate of change of outlet discharge equals the rate of change of channel discharge

Thus $\frac{dq}{q} = \frac{dQ}{Q}$

So, F = 1, i.e. $\frac{m}{n} \quad \frac{y}{H} = 1$

$$\Rightarrow \frac{H}{y} = \frac{m}{n} = \frac{\text{Outlet index}}{\text{Channel index}}$$

The outlet is said to be sub-proportional, if F < 1,

Or, $\frac{H}{y} > \frac{m}{n}$

The outlet is said to be hyper-proportional, if F > 1,

Or, $\frac{H}{y} < \frac{m}{n}$

(c) Setting:

It is the ratio of the depth of the silt level of the outlet below the FSL of the distributary, to the full supply depth of the distributary.

$$\text{Setting} = \frac{H}{y}$$

For proportional outlet , setting = $\frac{H}{y} = \frac{m}{n}$

For a wide trapezoidal channel, the discharge is proportional to $y^{5/3}$,
so, n = 5/3

Discharge through an orifice type outlet is proportional to $H^{1/2}$, so, m = ½

$$\text{Thus, setting} = \frac{H}{y} = \frac{m}{n} = \frac{1/2}{5/3} = \frac{3}{10} = 0.3$$

For a weir type outlet, the discharge is proportional to $H^{3/2}$

Hence, the setting for a combination of a weir type outlet and a trapezoidal channel,

$$= \frac{m}{n} = \frac{3/2}{5/3} = \frac{9}{10} = 0.9$$

Thus an orifice or a weir type outlet shall be proportional, if the outlet is set at 0.3 and 0.9 times depth below the water surface respectively.

(d) Sensitivity, S:

It is defined as the ratio of the rate of change of discharge through the outlet to the ratio of change of water level of the distributary.

$$S = \frac{\frac{dq}{q}}{\frac{dG}{y}}$$

Relation between Sensitivity and Flexibility

$$F = \frac{\frac{dq}{q}}{\frac{dQ}{Q}}$$

But, $\frac{dq}{q} = \frac{n}{y}$ dy $\qquad \therefore F = \frac{\frac{dq}{q}}{\frac{n}{y} \times dy} = \frac{1}{n} \frac{\frac{dq}{q}}{\frac{dy}{y}}$

Since, dG = dy,

So, $F = \frac{1}{n}$ S $\qquad \boxed{\text{Thus, } S = n\ F}$

For rigid modules, the discharge is fixed, and hence sensitivity is zero.
The greater the variation of discharge through an outlet for a given rise or fall in water level of the distributary, the larger is the sensitivity of the outlet.

Water Measurement Structures

(a) Constant Head Orifice
(b) Weir
(c) Parshall Flume
(d) Cut Throat Flume

Purpose of measurement:

- ❑ Efficient water distribution
- ❑ Efficient water use at farm level
- ❑ Project evaluation
- ❑ Equitable distribution of limited supply
- ❑ Provides basis for water charge

Location of measurement structures:

- ❑ Headworks
- ❑ Intake of the secondary canal
- ❑ Farm outlet/turnout

(a) Constant Head Orifice (CHO)

There are two gates. The upstream gate or the orifice gate controls the size of the opening. The downstream gate or the turnout gate controls the depth below the orifice and is operated to maintain a constant head (0.2 ft)

Discharge is given by,

$$Q = C \ A \ \sqrt{2gh}$$

Where, C = 0.7 and h = 0.2 ft

Advantage:

- It can regulate and measure discharge simultaneously
- There is no problem of sediment deposit in front of the gate
- It can be used for large fluctuations of water level in the parent canal

Disadvantage:

- It collects floating debris
- Flow measurement is not so accurate
- Discharge regulation needs two gate settings

(b) Weir

Weir can be installed in case of a drop in bed level. There are different types of weirs based upon shape of the opening through water flows.

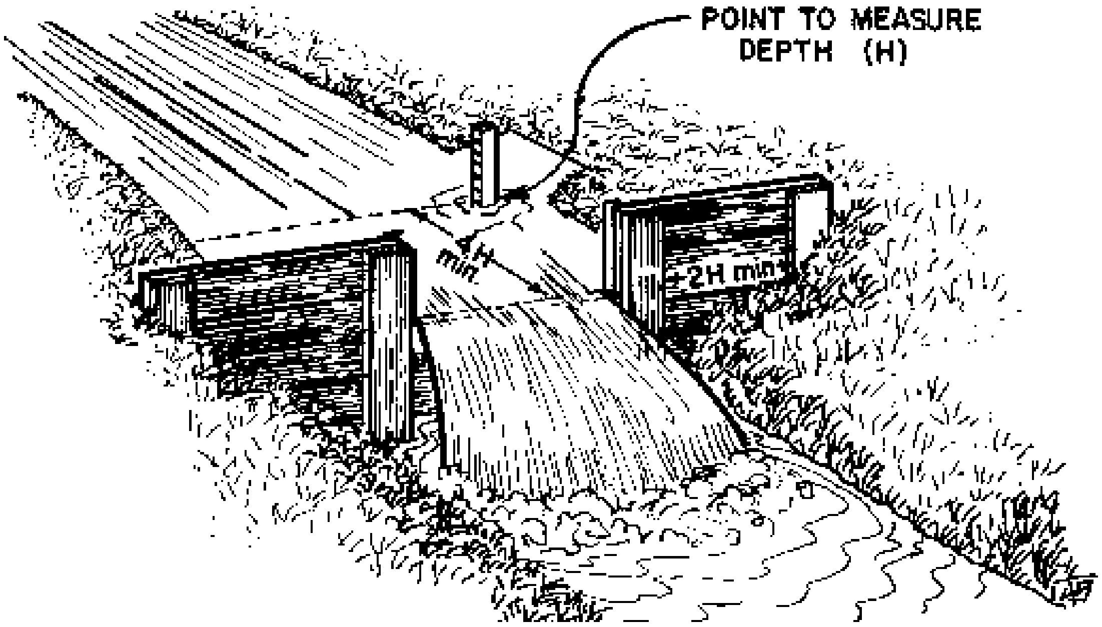


Fig: Weir

Discharge is given by,

(i) Rectangular weir:

$$Q = 1.84 \times (L - 0.2 \times H) \times H^{1.5}$$

Where,

Q = Discharge in cumec (m^3/s)

L = Length of crest (*m*)

H = Head (*m*)

(ii) Trapezoidal (Cipolletti) Weir:

$$Q = 1.86 \times L \times H^{1.5}$$

(iii) 90^o V-notch Weir:

$$Q = 8/15 \times \sqrt{2g} \times \tan(\theta/2) \times H^{2.5}$$

Advantage:

- It is capable of measuring a wide range of discharge.
- It is simple and easy to construct.
- No obstruction by moss or any floating debris.
- It can be combined with turnout.
- It is durable and its accuracy is higher.

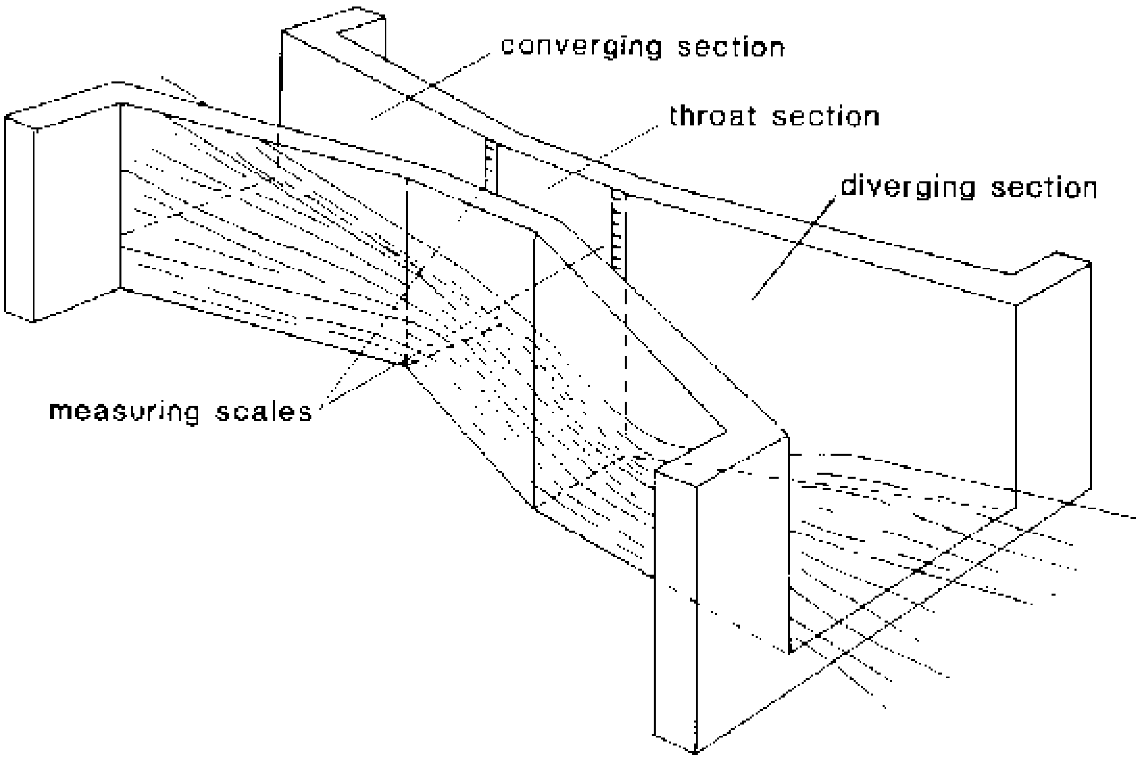
Disadvantage:

- Considerable fall in head is required.
- Silt deposition occurs in the upstream side.

Rules for setting and operating weirs:

- Weir should be placed at the lower end of a long, wide and deep pool such that $V_{approach} \leq 15$ cm/s
- The centre line of the weir should be parallel to the direction of flow.
- The face should be vertical.
- The crest should be level.
- The upstream edge should be sharp.
- The crest height should be $\geq 3 \times H$
- The edge of the weir should be at least $2 \times H$ form the edge of the channel.
- $H \leq 1/3 \times L$
- $H \geq 15$ cm
- Fall should be enough to provide ventilated condition.
- Weir gauge should be 5~6 time H upstream from the weir.

Parshall Flume:



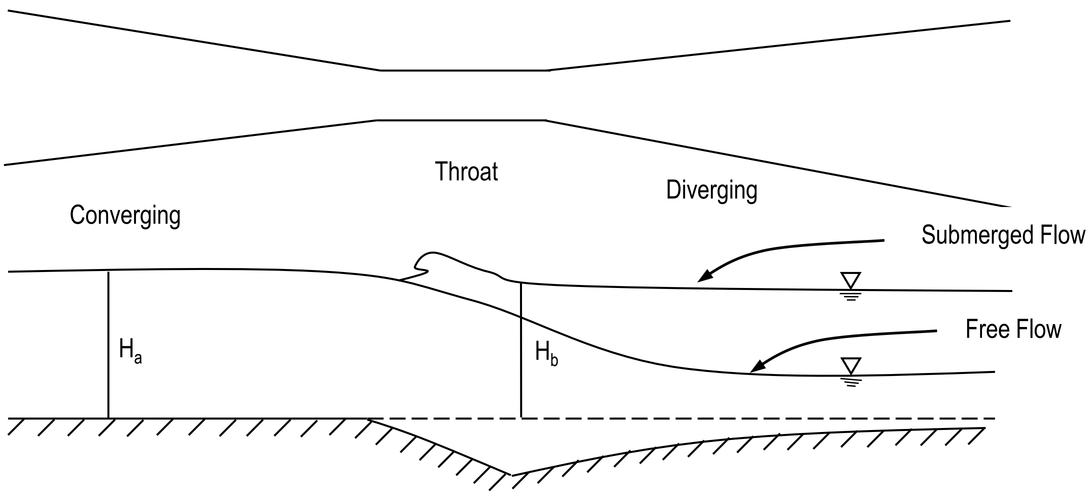


Fig: Free flow and submerged flow condition at Parshall flume

Discharge for free flow condition is given by,

$$Q_{free} = K \; H_a^{\,n}$$

Where,
Q = Discharge (cumec)
K = A constant depending on the system of units
n = Exponent
H_a = Upstream depth (m)

The value of K and n depend on the throat width and for 6″ throat width,

$$Q_{free} = 0.3812 \; H_a^{\,1.58}$$

When H_b/H_a exceeds 0.6, submergence occurs and discharge is reduced:

$$Q = Q_{free} - Q_{correction}$$

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Advantage:

- Discharge measurement is more accurate.
- It can be withstand a relatively high degree of submergence over a wide range of backwater condition downstream of the structure.
- It acts as a self cleaning device.

Disadvantages:

- Complicated and costly to construct.
- Cannot be combined with a turnout.
- May become invalid in case of heavy burden of erosion debris.
- Downstream ditch needs protection under free flow condition.

(d) Cut Throat Flume:

The cut-throat flume is an attempt to improve on the Parshall flume mainly by simplifying the construction details.

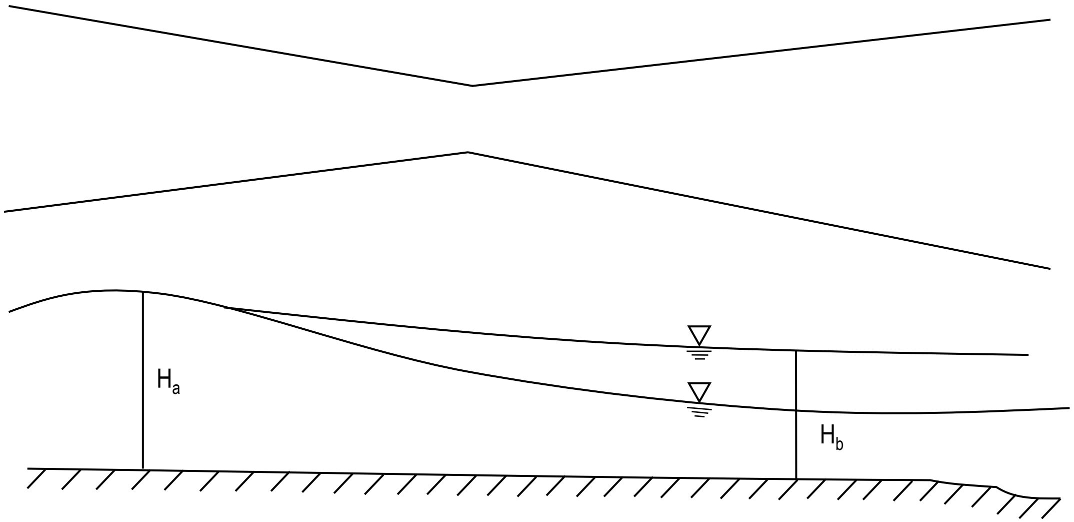

Fig: Discharge measurement by cut throat flume

Free flow condition is said to exist if $H_b/H_a \leq S_t$

Discharge for free flow condition is given by,

$$Q_{free} = C \times H_a^n$$

Where,

Q = Discharge in cumec

C = Free flow coefficient given by,

$$C = K \times W^{1.025}$$

H_a = Upstream depth (m) measured at a distance of 2L/9 from the throat.

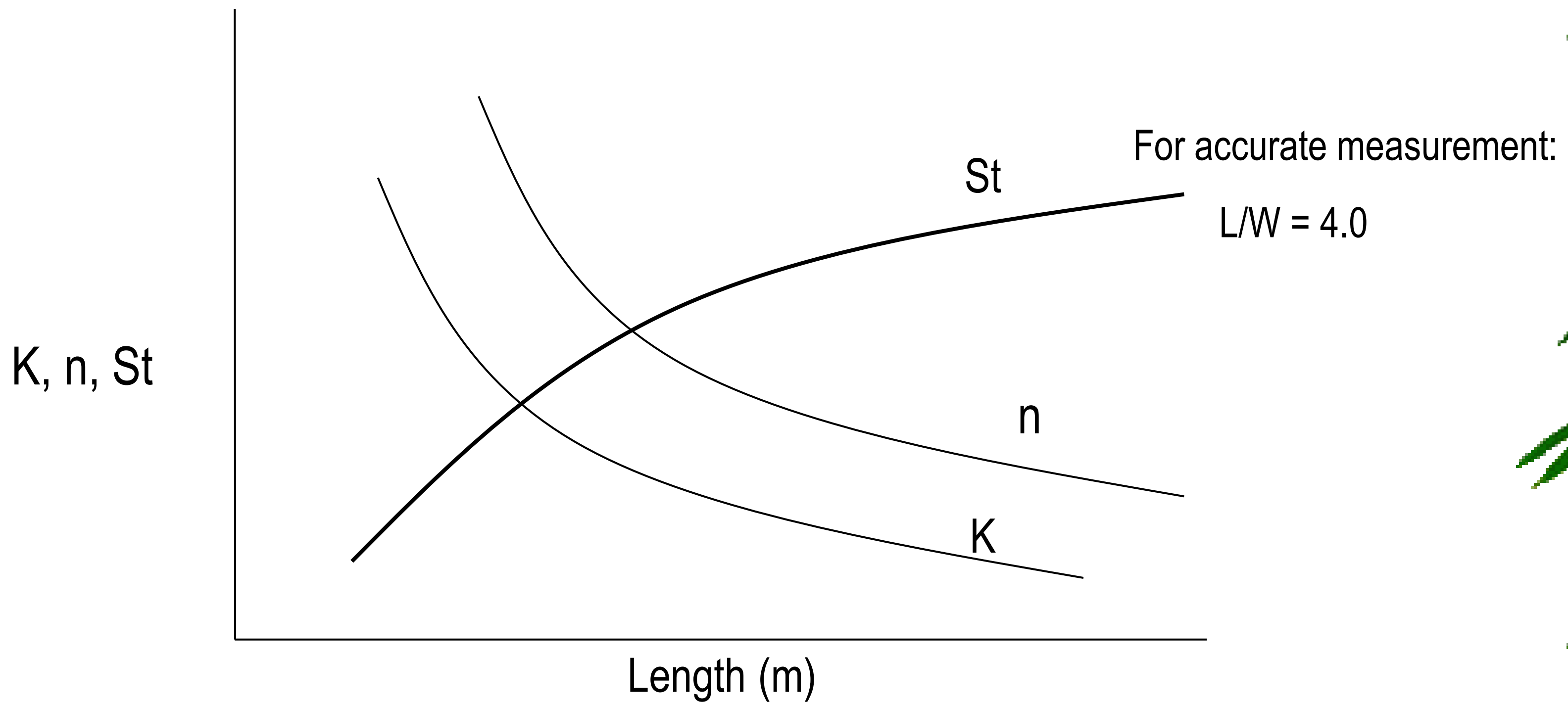

Fig: Cut-throat flume co-efficient

Advantage:

- Construction is facilitated by providing a horizontal floor and removing the throat section.
- The angle of divergence and convergence remain same for all flumes so the size of the flume can be changed by merely moving the vertical walls in or out.
- Calibration parameters remain same for a given length.
- More economic as mass fabrication is possible.

End of Chapter – 9

CHAPTER – 10

SPILLWAY AND PUMPS

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LECTURE 26

Definition of Spillway

The spillways are openings provided at the body of the dam to discharge safely the excess water or flood water when the water level rises above the normal pool level.

Necessity of Spillway

- The height of the dam is always fixed according to the maximum reservoir capacity. The normal pool level indicates the maximum capacity of the reservoir. The water is never stored in the reservoir above this level. The dam may fail by over turning so, for the safety of the dam the spillways are essential.

- The top of the dam is generally utilized by making road. The surplus water in not be allowed to over top the dam, so to stop the over topping by the surplus water, the spillways become extremely essential.

- To protect the downstream base and floor of the dam from the effect of scouring and erosion, the spillways are provided so that the excess water flows smoothly.

Location of Spillway

Generally, the spillways are provided at the following places

- Spillways may be provided within the body of the dam.
- Spillways may sometimes be provided at one side or both sides of the dam.
- Sometimes by-pass spillway is provided which is completely separate from the dam.

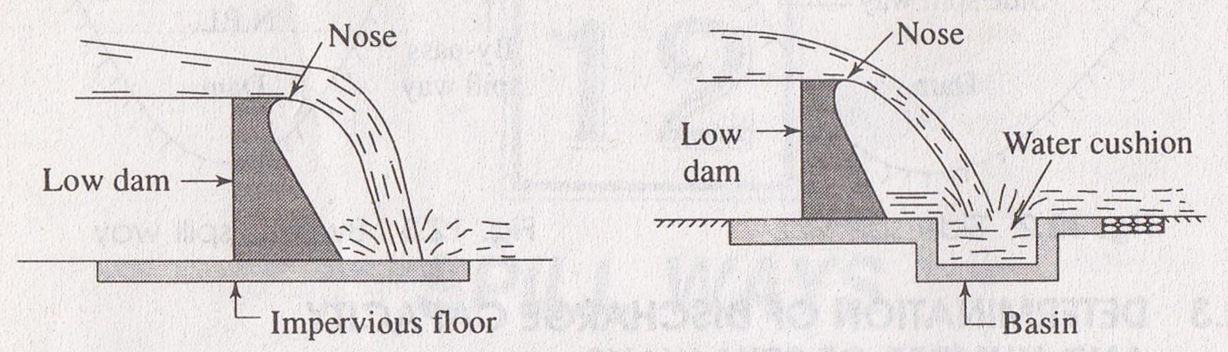
Determination of Discharge Capacity and Number of Spillway

- By studying the flood hydrograph of past ten years, the maximum flood discharge may be computed which is to be disposed off completely through the spillways.
- The water level in the reservoir should never be allowed to rise above the maximum pool level and should remain in normal pool level. So, the volume of water collected between maximum pool level and minimum pool level computed, which indicates the discharge capacity of spillways.
- The maximum flood discharge may also be computed from other investigation like, rainfall records, flood routing, empirical flood discharge formulae, etc.
- From the above factors the highest flood discharge is ascertained to fix the discharge capacity of spillways.
- The natural calamities are beyond the grip of human being. So, an allowance of about 25 % should be given to the computed highest flood discharge which is to be disposed off.
- The size and number of spillways are designed according to the design discharge.

Types of Spillway

- Drop Spillways
- Ogee Spillway
- Siphon Spillway
- Chute or Trough Spillway
- Shaft Spillway
- Side Channel Spillway

Drop Spillway



(a) Drop spillway with impervious apron
(b) Drop spillway with water cushion

- In drop spillway, the over flowing water falls freely and almost vertically on the downstream side of the hydraulic structure.
- The crest of the spillway is provided with nose so that the water jet may not strike the downstream base of the structure.
- To protect the structure from the effect of scouring horizontal impervious apron should be provided on the downstream side.

Ogee Spillway

- In Ogee Spillway, the downstream profile of the spillway is made to coincide with the shape of the lower nappe of the free falling waterjet from a sharp crested weir.
- In this case, the shape of the lower nappe is similar to a projectile and hence downstream surface of the ogee spillway will follow the parabolic path where “0” is the origin of the parabola.
- The downstream face of the spillway forms a concave curve from a point “*T*” and meets with the downstream floor.

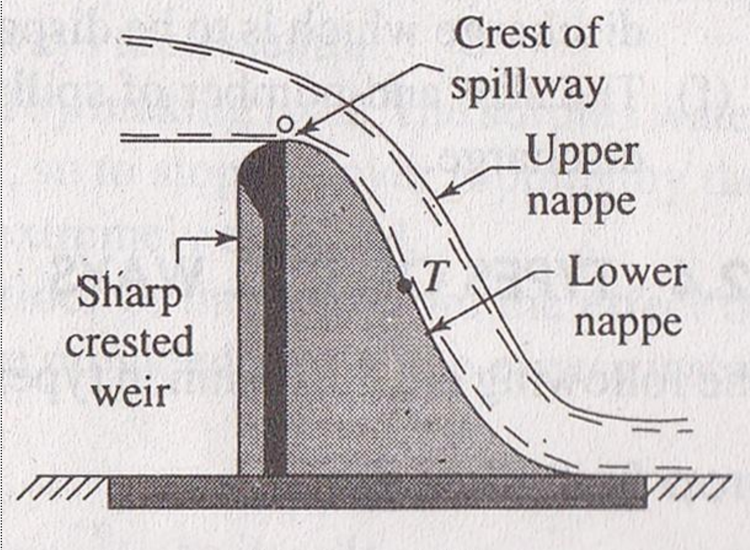


Fig: Ogee Spillway

The shape of the ogee spillway has been developed by U.S Army Corps Engineers which is known as "Water-way experimental station spillway shape". The equation given by them is, $X^n = K \times H^{n-1} \times Y$, where, *x* and *y* are the coordinates of a point *P* on the ogee profile taking *O* as origin. *K* and *n* are the constants according to the slope of the upstream face of the spillway (figure aside).

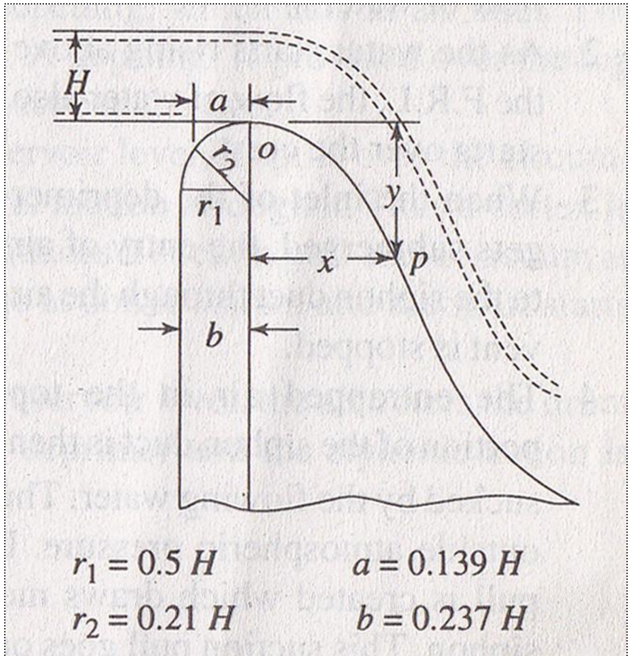


Fig: Shape of Ogee Spillway

The value of *K* and *n* are given as follow:

Shape of upstream face of spillway	K	n
Vertical	2.0	1.85
1:3 (H:V)	1.936	1.836
1:1½ (H:V)	1.939	1.810
1:1 (H:V)	1.873	1.776

LECTURE 27

Irrigation Pump

The mechanism by which the water is lifted from the under ground source to some height or to some place is known as pump

Pumping in ancient time

Pumping in modern time

Traditional irrigation system

Types of Pump

The Pumps may be of the following types:

- Reciprocating Pump
- Centrifugal Pump
- Turbine Pump
- Submersible Turbine Pump
- Rotary Pump
- Air Lift Pump

Reciprocating Pump

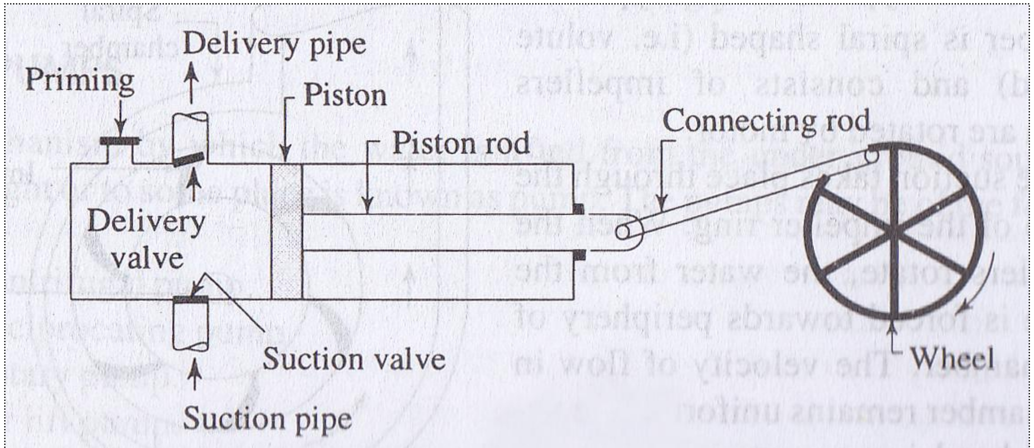


Fig: Reciprocating Pump

Components are:
- ❑Cylinder/piston
- ❑Suction Pipe
- ❑Delivery Pipe
- ❑Suction/inlet valve
- ❑Delivery/discharge valve

- ❑ A piston moves to and fro by a connecting rod.
- ❑ The connecting rod is again hinged with a wheel which is rotated by a motor.
- ❑ During the suction stroke, the suction valve is opened and delivery valve remains closed and water enters the cylinder.
- ❑ During the delivery stroke, the delivery valve is opened and suction valve remains closed and water is forced through the delivery pipe.
- ❑ An inlet is provided for the priming which is necessary for starting the pump.

Advantages of Reciprocating Pump

- It is suitable for large pumping units.
- It gives constant discharge.

Disadvantages of Reciprocating Pump

- It requires large space for installation.
- It is unsuitable for pumping water containing high sediment.

Centrifugal Pump

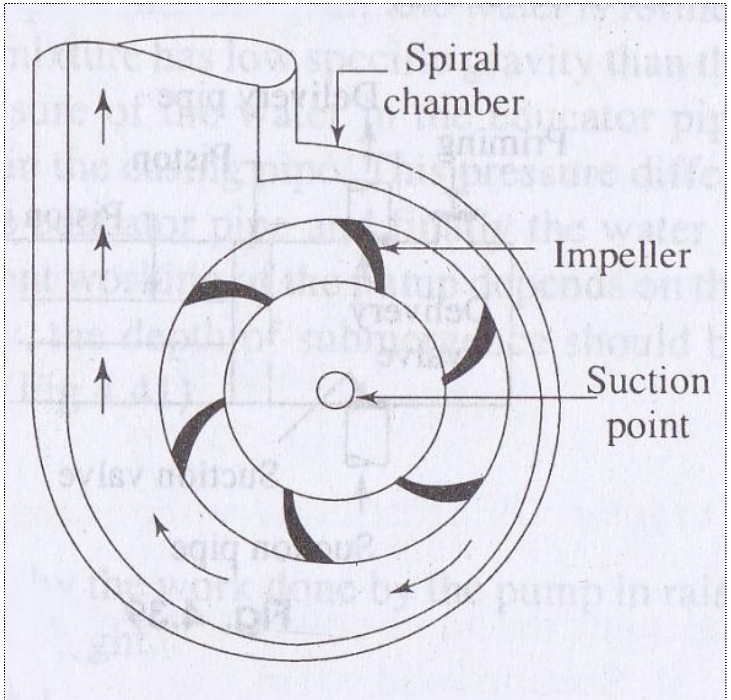


Fig: Centrifugal Pump (Volute Type)

- When the water in the chamber (casing) of a pump is rotated vigorously by the impellers about the central point, the centrifugal force develop which forces the water towards the periphery of the chamber.

- Thus the velocity head is converted to pressure head and this head forces the water through the delivery pipe.

- At the same time the water form the ground water source is lifted up by suction through the suction pipe.

Advantages of Centrifugal Pump

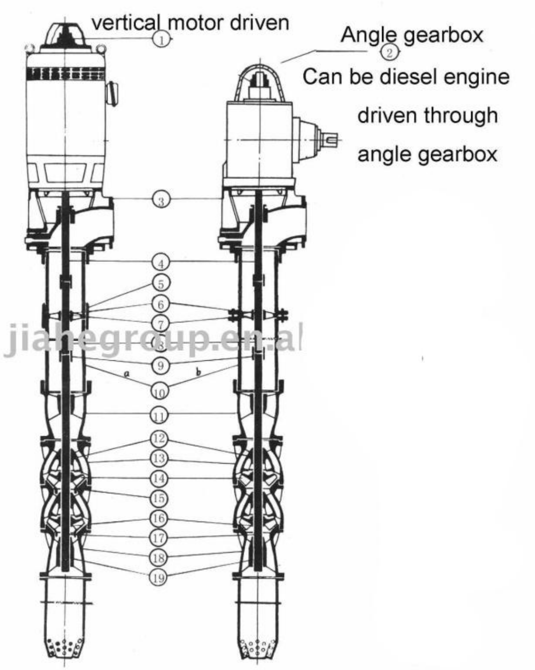
- It requires minimum space for installation as it is compact in design.
- It can be installed for high speed driving mechanism.
- The working is simple and there is no valve in the pump, hence it is reliable and durable

Disadvantages of Centrifugal Pump

- The pump will not work, if the chamber is not full of water. So, the priming should always be done before starting the pump
- The pump will not work if there is any leakage in the suction side.

Turbine Pump

- The impeller is surrounded by stationary guide vanes that reduce the velocity of water and convert velocity head to pressure head. The casting surrounding the guide vanes is usually circular and concentric with the impeller.
- A deep will turbine pump is a multi-stage pump that accommodates several impellers on a vertical shaft and stationary bowls pressing guide vanes.
- The two bowl assemblies are nearly always located beneath the water surface.
- The several bowls are connected in series to obtain the desired total head.



vertical turbine Long shaft deep well pump

Applicability:
Deep-well turbine pumps are used for irrigation when the water surface is below the practical lift of the centrifugal pump.
Q = 56 liter/sec, h = up to 300 m below ground level

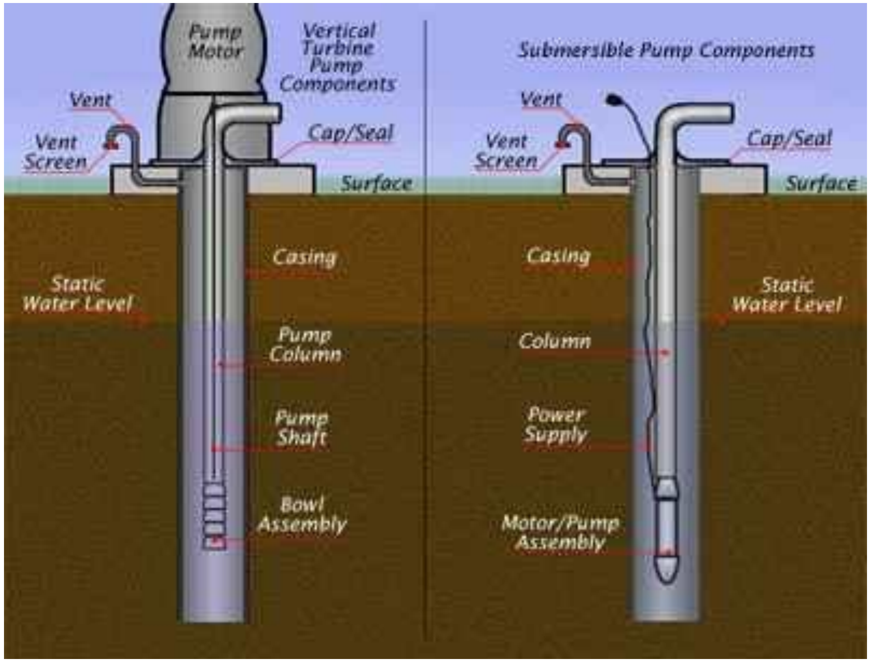
Advantages of Turbine Pump

- ❑ Priming is not required.
- ❑ Adapted to high lifts.
- ❑ Adapted to seasonal fluctuations in water level in the well

Disadvantages of Turbine Pump

- ❑ Operating parts are inaccessible and difficult to inspect
- ❑ Low efficiency is common
- ❑ Frequent shaft rupture
- ❑ Higher initial cost

Submersible Turbine Pump



- ❑ A submersible turbine pump is one in which the pump and the electric motor are placed below the water surface of a well. Delivery of water to the surface is through a riser pipe on which the assembly is suspended.
- ❑ The characteristics of the pump unit are similar to a conventional vertical turbine pump.
- ❑ They have been used in wells over 4000 meter deep. Units with more than 250 stages have been used.

Advantages of Submersible Turbine Pump

- It eliminates the long vertical shaft from the ground surface to the pump which reduces bearing friction and provides an unobstructed pipe for delivery of water to the surface.
- It can be used where the installation is flooded or where an above ground pump house would be inconvenient, unsightly or hazardous.

Disadvantages of Submersible Turbine Pump

- Operating parts are inaccessible and difficult to inspect

Rotary Pump

- It consists of tow cams which are pivoted in a casing.
- These cams rotate in opposite directions and thereby the suction takes place through the suction pipe.
- The rotation of the came pushes the water in upward direction through the delivery pipe.

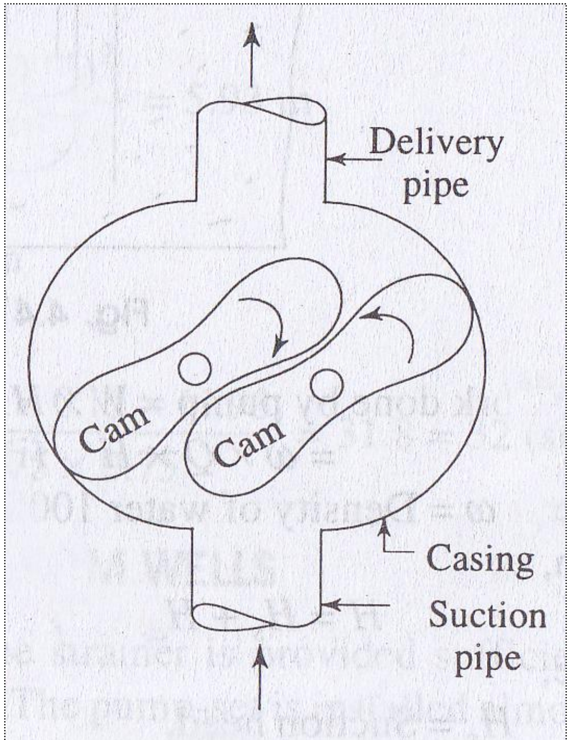


Fig: Rotary Pump

<u>Advantages of Rotary Pump</u>:

- The flow of water is uniform.
- No priming is required.
- It requires no valves and its operation is simple.

<u>Disadvantages of Rotary Pump</u>:

- It requires replacement of cams frequently and hence is involves more maintenance cost.
- It can not be used for pumping water containing high sediment.

Air Lift Pump

- When compressed air is forced through the air pipe, a mixture of air and water is formed and rises up in the form of bubbles.
- Thus the pressure of the water in the educator pipe becomes less than the pressure of water in the casing pipe.
- This pressure difference forces the water to rise through the educator pipe and finally the water is discharged through the outlet.

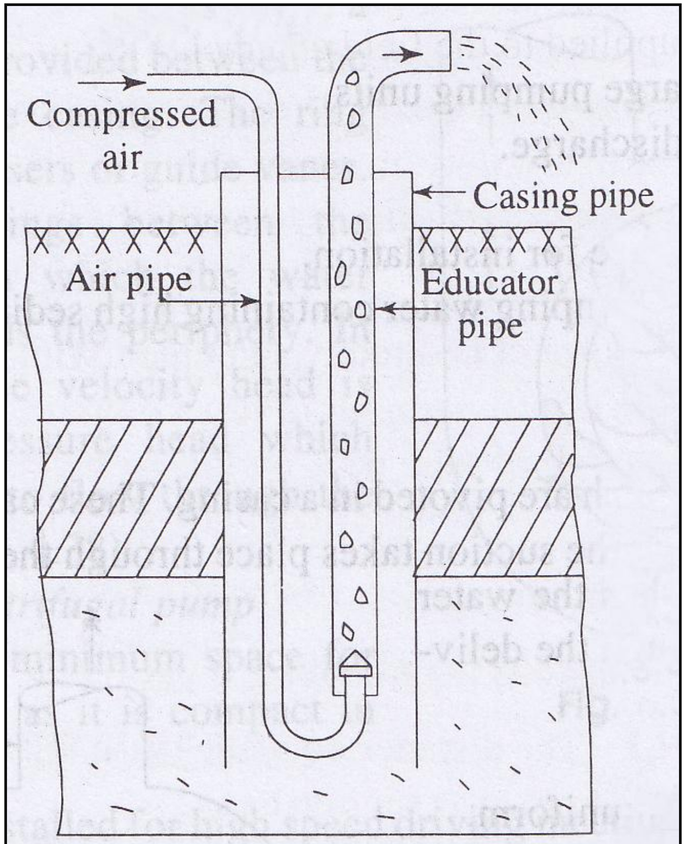
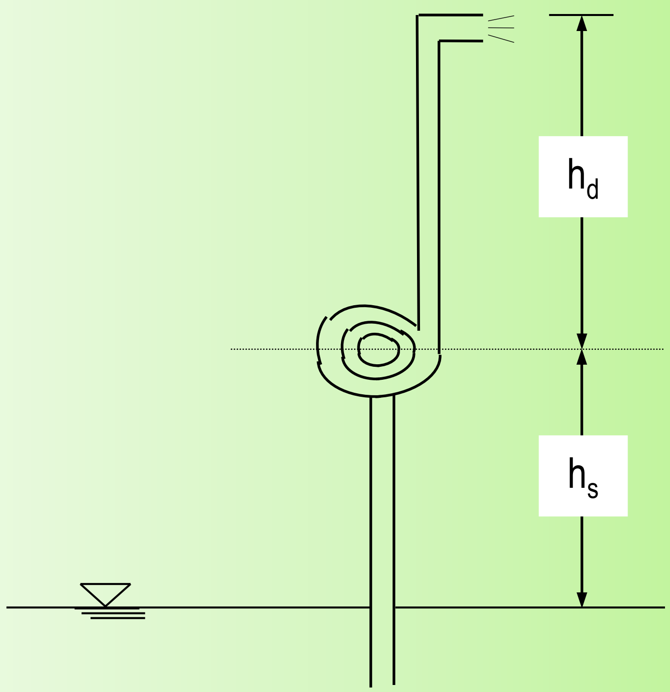


Fig: Air Lift Pump

LECTURE 28

Pumping Head



h_s = Static suction lift = vertical distance from the free suction water level to the centre line of the pump

h_d = Static discharge head = vertical distance from the centre line of the pump to the discharge water level

$h_s + h_d$ = Total static head = vertical distance from the suction water to the discharge water level

$h = h_s + h_d + d_f$ = Total pumping head

Where, h_f = Total frictional head loss in the suction and delivery pipes

Horse power of pump

The horse power of a pump is determined by work done by the pump in raising a particular quantity of water to some height.

Let, W = Quantity of water (kg)

H = Total head (m)

Then, work done by pump = $W \times H$

$= w \times Q \times H$ ------------- (i)

$[W = w \times Q]$

Where, w = Density of water (1000 kg/m^3)

Q = Discharge (m^3/s)

Again, $H = H_s + H_d + H_f$

Where, H_s = Suction head (m)

H_d = Delivery head (m)

H_f = Head loss due to friction (m)

The head lost by friction is given by,

$$H_f = \frac{f \times l \times Q^2}{3d^5}$$

Where, f = Coefficient of friction

l = Total length of pipe (suction and delivery)

d = Diameter of pipe (m)

From equation (i) $\Rightarrow$

$$\text{Water Horse Power (W.H.P)} = \frac{w \times Q \times H}{75}$$

Considering the coefficient of the pump as η, brake horse power

$$\text{Brake Horse Power (B.H.P)} = \frac{w \times Q \times H}{75\eta}$$

Problem-2:

A centrifugal pump is required to lift water at the rate of 100 lit/sec. Calculate the brake horse power of the engine from the following data when the water is directly supplies to the field channel.

(a) Suction head = 6 m

(b) Coefficient of friction = 0.01

(c) Efficiency of pump = 65%

(d) Water is directly supplied to the field channel

(d) Diameter of pipe = 15 cm

Solution:

Q = 100 lit/sec = (100/1000) m^3/s = 0.1 m^3/s

Delivery head, H_d = 0 (As water is directly supplied to field)

H = 5 m

f = 0.01

d = 15 cm = 0.15 m

The length of the pipe where frictional effect may occur is taken equal to the suction head, so *l* = 6 m

$$H_f = \frac{f \times l \times Q^2}{3d^5} = \frac{0.01 \times 6 \times (0.1)^2}{3 \times (0.15)^5} = 2.63 \text{ m}$$

So, total head, $H = H_s + H_d + H_f$

$= 6 + 0 + 2.63$

$= 8.63$ m

Efficiency, $\eta = 65\% = 0.65$

$$\text{Horse Power (H.P)} = \frac{w \times Q \times H}{75\eta} = \frac{1000 \times 0.1 \times 8.63}{75 \times 0.65}$$

$= 17.70 \approx 18$ (Ans)

Power requirement

WHP (Water Horse Power) is the theoretical horse power required for pumping. It is the head and capacity of the pump expressed in terms of horse power.

$$WHP = Q \times h/76$$

Where, Q = Discharge (liter/sec)
h = Total pumping head (meter)

Efficiency is the ratio of the power output to power input

$$\therefore \text{Pump efficiency, } E_p = WHP/SHP$$

Where, SHP = Shaft horse power

Break Horse Power (BHP) is the actual horse power required to be supplied by the engine or electric motor for driving the pump

$$BHP = SHP/E_d = WHP/(E_p \times E_d)$$

Where, E_d = Delivery efficiency

Horse Power input to electric motor = $WHP/(E_p \times E_d \times E_m)$

Kilowatt input to electric motor, $KW = (BHP/E_m) \times 0.746$

Pump characteristics

- The interrelations between speed, head, discharge and horse power of a pump are usually represented by curves which are designated as 'Characteristics curves'
- Knowledge of pump characteristics enables one to select a pump which fits operating conditions and thus attain a relatively high efficiency with low operating cost.
- As the discharge increases, the head decreases. The resulting efficiency is observed to increase from zero where the discharge is zero to a maximum of 82% when the discharge is 86 liter/sec and the head 23 m (and then found to decrease to zero at zero head).

Cont…. Pump characteristics

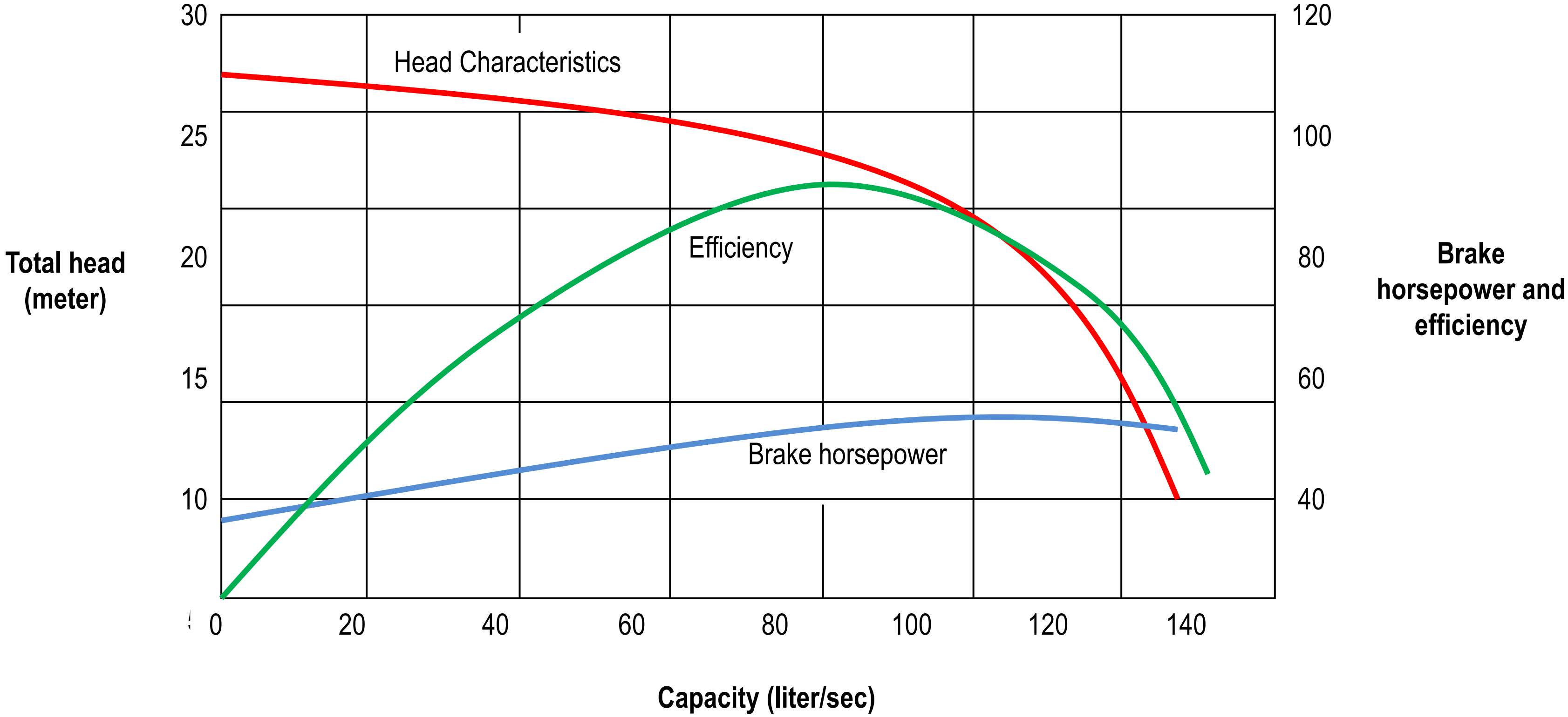


Fig: Pump characteristics curve

The BHP curve for a centrifugal pump usually increases over most of the range as the discharge increases, reaching a somewhat higher rate of discharge then that which produce maximum efficiency.

Selection of power plant

The power plant must be capable of delivering the required power under varying conditions. The factors are:

- Brake Horse Power required
- Initial cost
- Availability and cost of energy or fuel
- Depreciation
- Dependability of unit
- Portability required
- Maintenance and convenience of operation
- Labor availability and quality

Effect of speed and impeller diameter on pump performance

Effect of change of pump speed:

When the speed of a centrifugal pump is changed, the operation of the pump is changed as follows:

- ✓ The capacity varies directly as the speed.
- ✓ The head varies as the square of the speed.
- ✓ The brake horse power varies as the cube of the speed.

Expressed mathematically,

$$Q = Q_1 \times (n/n_1) \text{ ---------------- (i)}$$
$$H = H_1 \times (n/n_1)^2 \text{ --------------- (ii)}$$
$$P = P_1 \times (n/n_1)^3 \text{ --------------- (iii)}$$

From equation (i), (ii) & (iii) $\Rightarrow$

$$n/n_1 = Q/Q_1 = (H/H_1)^{1/2} = (P/P_1)^{1/3}$$

Where,

n = New speed desired, rpm

Q = Capacity at the desired speed n, liter/sec

H = Head at the desired speed n for capacity Q, meter

P = BHP at the desired speed n at H and Q

n_1= Speed at which the characteristics are known, rpm

Q_1 = Capacity at speed n, liter/sec

H_1 = Head at speed n_1 and capacity Q_1, meter

P_1 = BHP at speed n_1 at H_1 and Q_1

Effect of Change of Impeller Diameter

Changing the impeller diameter has the similar effect on the pump performance as changing the speed.

Thus, $D/D_1 = Q/Q_1 = (H/H_1)^{1/2} = (P/P_1)^{1/2}$

Where,

D = Changed diameter of impeller, mm

D_1 = Original diameter of impeller, mm

Pump and system characteristics

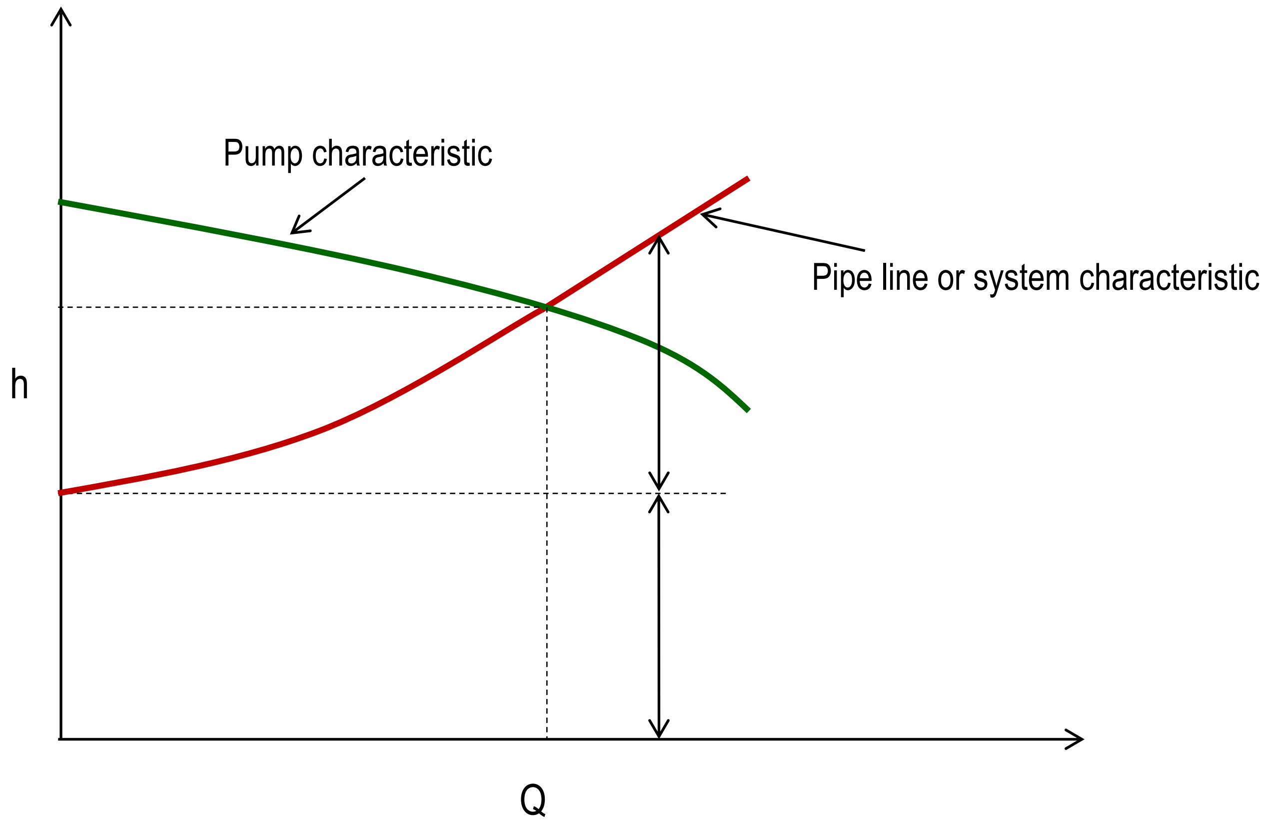


Fig: Graphical method for finding the operating condition of a pump and pipeline

If under this mode of operation the efficiency of the pump is not very high, an improvement in efficiency can be made by changing the speed of the pump or the impeller diameter or by selecting a different type of pump

Pumping Cost

Cost of pumping includes fixed cost and operating cost.

Fixed cost:

Total annual cost = capital cost ‘X’ CRF + annual O & M cost

Where,

CRF = Capital Recovery Factor

$$= \frac{i(1+i)^n}{(1+i)^n - 1}$$

i = Discount rate,

n = Project life, years

Operating cost:

Annual Operating cost = energy consumption rate X hours of operation X energy cost rate

End of Chapter – 10