PURBANCHAL UNIVERSITY

A NOTE BOOK ON IRRIGATION ENGINEERING

PREPARED BY UMESH RAUT

2072/04/11

Chapter-1: Introduction (4-8 marks)

Defination of irrigation

A/c to Bharat Singh - "the application of water to soil for purpose of supplying the moisture essential or beneficial to plant growth."

A/c to B.C. Punmia - " Artificially supplying water to soil for raising crops."

↳ Application means required water at reqd time
↳ Artificially supplyng wtr to field (soil) to grow the Crops to get the optimum yields.

Rainfall needed for Rice — (90-120) cm

Evaporation → Hot - 5mm
→ Cold - 3mm

Needs of irrigation

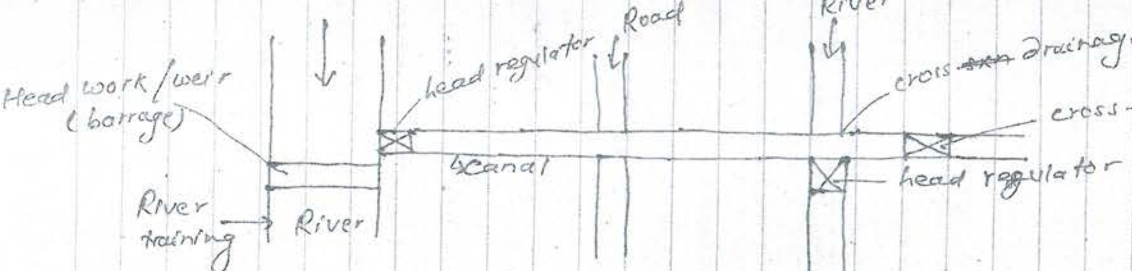
1. To get maximum yields, it is essential to supply the optimum quantity of watr and to maintain direct timing of water.
2. less rainfall
3. Non-Uniform rainfall
4. Commercial crops with additional water

functn of irrigatn

1. Supply moistr to the soil.
2. Producing the nourishing food for the plant
3. Chemical rxn.
4. wash the harmful salt.

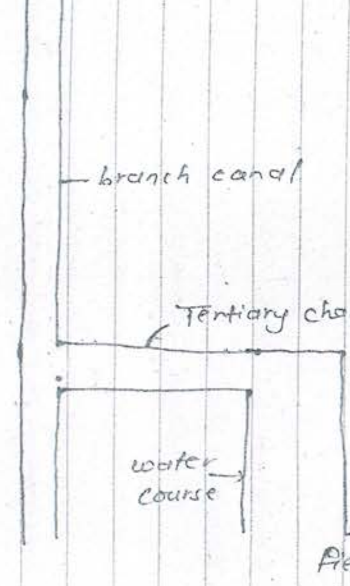
Sources of irrigatn

1. Surface → River, reservoir
2. Sub-Surface → Wells, Springs



Cross-~~sec~~ drainage

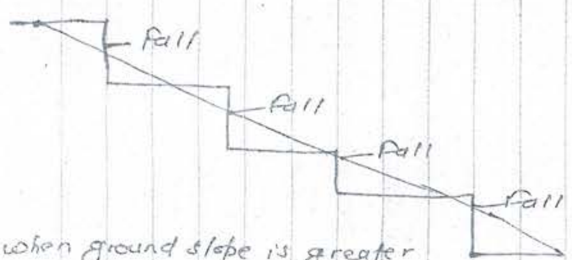
↳ when canal crosses the drain then it is called cross-drainage



→ soil moistR relatd
→ crop wtR requirement
→ water logging

Fig:- Distributd of water

Fall



↳ when ground slope is greater than canal slope the difference is adjusted to construct a structure, a structure is called Fall.

Scope of irrigatn

(1) Engineering Aspects
- storage, diversion or lifting of water
- conveyance of water - Canal
- distributn system
- Regultn

(2) Agricultural aspects
- requirement of watR
- applictn of water

Advantages of irrigatn

(1) increase in prodxn.
(2) increase in GDP (Gross Domestic product)
(3) Hydropower
(4) Recreation
(5) Navigatn
(6) Protn from Soil erosion
(7) improvement of Communicatn (Road)
(8) improves environment (Plantatn)
(9) Fisheries
(10) Control floods

Disadvantages

(1) water pollutn (Pesticides, fertilizer, biodiversity)
(2) Floods → strxtr failure
→ high rainfall
(3) Water logging
(4) Water disputes
(5) Migratn
(6) may result in Colder and damper Climate

→ see www.DOI.GOV.NP
→ irrigatn policy - 2070 BS
→ wtr strategy - 2002 AD
→ wtr plan - 2002 AD

Develpment of irrigatn in Nepal

5000 years ago - during Ramayan.
2500 years ago - Mahabharat

In Nepal 2500 years ago,

Total area - 1,47,181 00 hacter
Agriculture - 26,42,000 hacter
Irrigable Area - 17,66,000 ha
Irrigated Area - 12,79,365 ha

Status of irrigatn Devlopment in Nepal

Nepal is a Country with vast diversity of Climate, topography & vegetatn.
- Nepal is an agricultural Country where 90% of total populatn depend on agriculture.
- In terai large irrigatn area than hilly & himalayan.

Categories of irrigtn system

In Nepal irrigtn system are Catgorised into 3 types

① Extensive irrigtn schemes - which incorporates the main and secondary canals.

② Intensive irrigtn schemes - which incorporates the main, secondary & tertiary Canals.

③ Command area devlopment schemes

History of irrigtn Devlopment in Nepal

- The 1st irrigtn project started from government side in Nepal was Chandra Ca[nal] Constructed in 1985 BS at Saptari District.
- Juddha Canal in Sarlahi & Jagdishpur reservoir in Kapilvastu was Constructed under the Supervsn of Retired Indian engineers.

- After Democracy of 2007, Devlopment of irrigtn was started on institutional basis.
- In 2009B.S, "Department of Canal" was established & finally named as "Department of Irrigtn" in 2044 BS.
- In 2049 New irrigtn policy devlped

Features of New Irrigtn policy 2049

A/c to 2049 policy, there are 4-Categories of irrigtn schemes for devlopment.

1. irrigtn schemes to be handed over to the users.
2. Irrigtn schemes to be jointly managed.
3. Irrigtn schemes Constructed and managed by farmers
4. irrigtn schemes owned individually

2072/04/14

Chapter-2: Soil water relationship

General classification of soil for Agricultural purpose

Soil is divided into 4-groups for Agricultural purpose

(A) A/c to particle size

1. Gravel — >>> 2mm
2. Sand — 0.06 to 2mm
3. Silt — 0.002 to 0.06 mm
4. Clay — particle size smaller than 0.002mm

Sand and silt further dived as

(1) Sand → Coarse sand, Medium sand, Fine sand

(2) Silt → Coarse silt, Medium silt, fine silt

(B) A/c to soil fertility - 2 types

(1) Fertile soil - The soil in which agricultural productivity is high.
eg. loamy soil

(2) Non-fertile soil - The soil in which we cannot produce the agricultural products as per our desire. ie agricultural productivity is very ↓.
- eg. gravel.

→ Generally, the best agricultural soil contains 10-20% clay
5-10% organic matters
and rest divided equally betn sand & silt.

Soil Moisture

- Water held on the soil above the water table is called soil water OR soil Moisture.
- Water below the water table is Grnd wtr.

Types of Soil water or soil Moisture

1. Gravitational or free water
2. Capillary water
3. Hygroscopic water

(1) Gravitational / Free water : water above soil, which can drain wtr easily under the acn of gravity is free water.

(2) Capillary water - It is the rise of water under anti-gravity. Also calld **available** wtr

(3) Hygroscopic water - wtr found below permanent wilting point is Hygroscopic water.
- It is **unavailable** wtr.
- water which is attached to soil moleculues by loose kmical bonds & cannot be extract by capillarity and not available to plant.

VVI पानि soil में [illegible] allows [illegible]

(4) Field Capacity (Fc) :- After Rainfall / irrigtn when all the gravity wtr has drained down to wtr table, a certain amnt of wtr is retained on the surface of soil grain by molecular attrxn and by loose Chemical Bond. This water cannot be drained under the acn of gravity & is called Field Capacity.

ie,

$$\boxed{\text{Field Capacity (Fc)} = \frac{\text{wt. of water retained in certain vol. of soil}}{\text{wt. of same volume of dry soil}} \times 100}$$

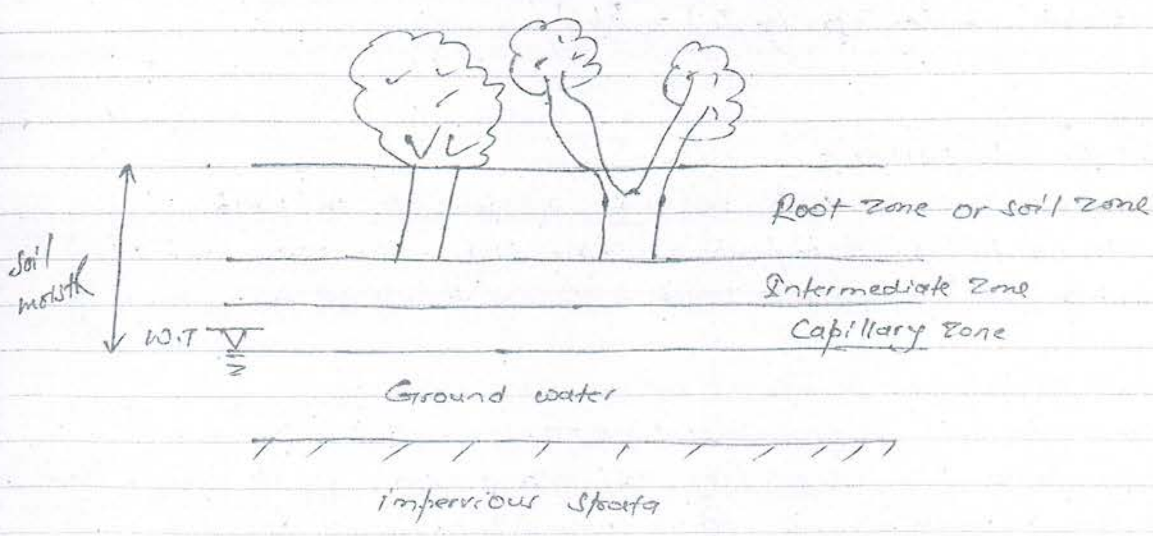
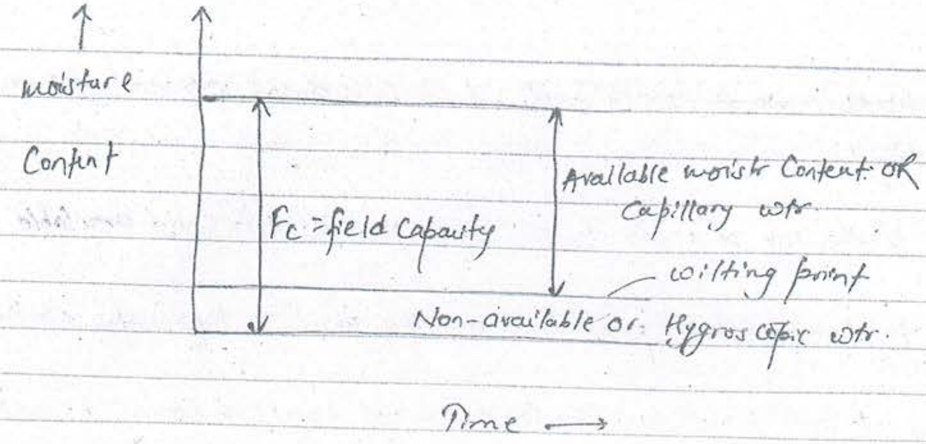


Fig:- Soil moistr - irrigtn relatnship



Soil Zone / Root Zone :- This zone is penetrated part of roots of vegetatn. is imprtant part from irrigtn point of view.

- when water falls over the grnd, a part of it gets absorbed in this Root zone a[nd] rest flow downward undr the axn of Gravity which is called gravity wtr.

Permanent wilting point (PWP) :- It is that water Content at which plant ca[n] No longer extract sufficient wtr for its growth and wilts up.

ie,

$$\boxed{\text{Available moistr} = \text{Field Capacity (Fc)} - \text{PWP}}$$

imp

Crop water Requirement

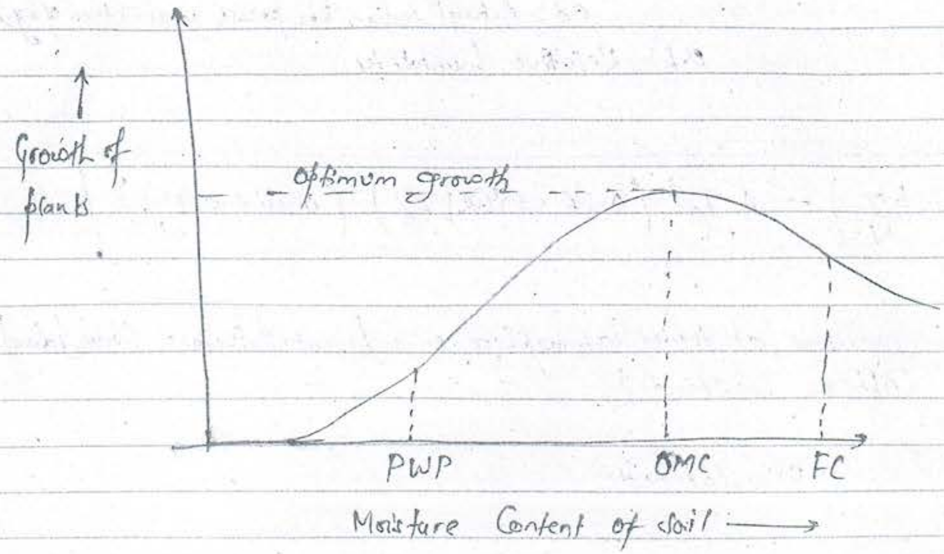
Wtr below wtr table is Grnd table and above wtr table is soil moistr.

Water required for the full growth of plants is called Crop-water requirement (CRW)

It is essential to maintain readily available moistr in the soil for the satisfactory growth of the crops.

- The plant growth may be retarted if the soil-moistr is either less or high.
- Crop water requirement varies crops to crops, place to place and time to time.
- if soil has more wilting Coefficient, the plant must expend extra energy to obtain & plan[t] will not grow healthy. on other hand, excessive wtr fills up the soil pores with water, thus driving air out.
- Air is essential to satisfactory Plant growth & excessive water supply retards plant growth.

- So optimum Moisture Content (OMC) is necessary for full growth of plants



Crop water Requirement by Penman Method

- According to Penman method, the potential evapotranspiration (PET) is given by,

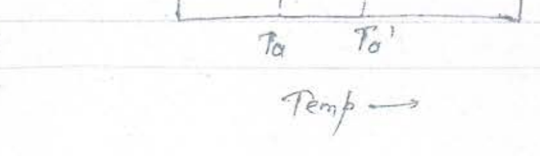
$$\boxed{E_T = \frac{A\,H_n + E_a\,\gamma}{A+\gamma}}$$

E_T = Daily PET (mm/day)

H_n = Net incomming solar radiatn (mm/day) / Radiatn term

E_a = Parameter including wind velocity & saturatn vpr pressR

A = slope of saturatn vapour pressR vs temperatR curve at mean air temp.



$$\boxed{A = \frac{e_s' - e_s}{T_a' - T_a}}$$

γ = Psychometric Constant = 0.49 mm of Hg / °C

$$\boxed{E_a = 0.35\,(e_s - e_a)\left(1 + \frac{V_2}{160}\right)}$$

e_s = Saturtn vpr pressR at mean air t (mm of Hg)

e_a = Actual mean vpr pressR in air (mm of Hg)

R.h = Relative humidity

$$\boxed{e_a = \frac{R.h * e_s}{100}}$$

$$\boxed{H_n = H_c(1-\alpha)\left(a + \frac{bn}{N}\right) - \sigma T_a^4\left(0.56 - 0.092\sqrt{e_a}\right)\left(0.10 + 0.9\frac{n}{N}\right)}$$

H_c = Mean incident solar radiatn at top of atmosphere on a horizntal surface (mm/day)

α = Reflxn Coefficient (Albedo Constant)

a and b = Constant

$$\boxed{a = 0.29\cos\phi}$$

ϕ = latitude

$$\boxed{b = 0.52}$$

n = Actual durtn of bright sunshine in hrs.

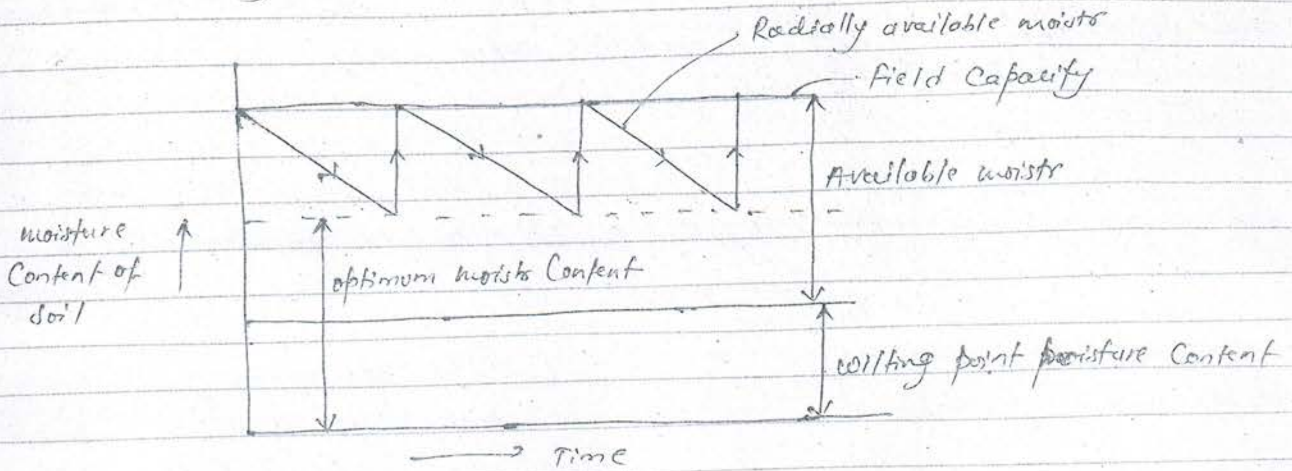
N = Maximum possible hrs of bright sunshine (mean value).

σ = Stefan - Boltzman Constant $= 2.01 \times 10^{-9}$ mm/day

T_a = Mean air temp in Kelvin (°K)

$= 273° + t°C$

Estimating Depth and Frequency of irrigation



Water or moisture is Consumed by plant through their roots. It is therefore become necessary that Sufficient moisture remains available in the soil from the Surface to root zone depth. The soil moistr in root zone Can vary between field Capacity and wilting point.

The irrigation water should be supplied as soon as moistr falls upto this optimum level (Fixing irrigation frequency) and quantity should be just sufficient to bring the moisture Content upto its field Capacity making allowable for application losses.

$$\text{Field Capacity (F.C)} = \frac{\text{wt. of water retained in certain volume of soil}}{\text{wt. of same vol. of dry soil}} \times 100$$

Considering 1 m² area and depth of soil d m (root zone depth)

$\therefore$ Volume of soil $= d \times 1 = d\,m^3$

dry unit wt. of soil $= \gamma_d\ (kN/m^3)$

wt. of $d\,m^3$ of soil $= \gamma_d \times d$ kN

$$\text{Field Capacity (FC)} = \frac{\text{wt. of water retained in unit area of soil}}{\gamma_d \cdot d} \qquad \{F = \text{field Capaci}$$

or, wt. of water retained in unit area of soil $= \gamma_d \cdot d \cdot F$ $\quad kN/m^2$

$\therefore$ Vol. of water stored in unit area of soil $= \frac{\gamma_d \cdot d \cdot F}{\gamma_w}$ $\frac{kN/m^2}{kN/m^3}$

$\therefore$ Total storage capacity of soil in (meter depth of water)

$$= \frac{\gamma_d \cdot d \cdot F}{\gamma_w}$$

$\therefore$ Depth of water stored in Root zone filling the soil upto field capacity

$$= \frac{\gamma_d \cdot d \cdot F}{\gamma_w} \ (m)$$

i.e. $\boxed{\text{Depth of water} = \frac{\gamma_d \cdot d \cdot F}{\gamma_w}}$

F = field capacity
d = depth of root zone
γ_w = unit wt. of water
γ_d = dry unit wt. of water.

$\boxed{\text{Depth of irrigation water } (d_w) = \text{Depth of readily available water} = \frac{\gamma_d \cdot d \cdot (F - OMC)}{\gamma_w}}$

OMC = optimum moisture content

Readily available moisture :- Portion of available moisture which is most easily extracted by plants. it is approximately 75-80% of available moisture.

Mathematically,

$\boxed{\text{Readily available moisture (RAM)} = \text{Field Capacity (FC)} - OMC}$

Optimum moisture Content (OMC) :- It is the water content at which max^m production is obtained

→ The number of times watering is done to maintain moisture content from OMC to FC as per consumption of water is called frequency of irrigation (Fw)

Frequency of irrigation :- If Cu is daily consumptive use rate then frequency of irrigation is given by,

$\boxed{F_w = \frac{d_w}{C_u}}$

d_w = depth of irrigation water $= \frac{\gamma_d \cdot d}{\gamma_w}(FC - OMC)$
C_u = daily consumption use rate

- Frequency of irrigation depends on → Amount of available moisture content in root zone of soil and
→ The consumptive use rate

Available moisture = FC − PWP

Numerical

① After how many days will you supply water to soil in order to ensure sufficient irrigation of given crop if,

① Field Capacity = 28%

② Permanent wilting point = 13%

③ dry density of soil (γ_d) = 1.3 gm/cc

④ effective depth of root zone = 70cm = 0.7m

⑤ Daily Consumptive use of water from given crop = 12mm

Soln.

Available moisture = F.C − Permanent wilting point
= 28 − 13
= 15%

Let,

Readily available moisture = 80% of available moisture (ie 75-80%) take any

RAM = 0.8 × 15
= ~~12.0%~~
= 12%

∴ optimum moisture = 28 − 12 OMC = FC − RAM
= 16%

It means that the moisture will be filled by irrigation betn 16% & 28%

Now,

$$\text{Depth of irrigation } (d_w) = \frac{\gamma_d \cdot d \cdot (F - OMC)}{\gamma_w}$$

$$= \frac{1.3 \times 0.7 \; (0.28 - 0.16)}{1}$$

$\gamma_d = 1.3\,gm/cc$
$d = 0.7m$
$F = 28\% = 0.28$
$OMC = 16\% = 0.16$
$\gamma_w = 1$ in gm/cc

$$\boxed{d_w = 0.1092\,m}$$

or

$$\boxed{d_w = 10.92\,cm}$$

Now,

water available for evapotranspiration = 10.92 cm

1.2 cm of water is utilised by plant in 1 day

∴ 10.92 cm of water will be utilised by plant in $\frac{10.92}{1.2}$ = 9.1 days.

Hence after 9 days water should be supplied to given crop.

VVI must write!

Factors affecting Crop-wtr Requirement

① Climate - wtr lost in evaprtn and transpirtn varies with climate

② Useful Rainfall :- The rainfall falling over the land is useful for growth of crops. More the useful rainfall less will be requirement of irrigtn.

③ Types of soil :- If the permeability of soil undr the irrigated crop is high the los due to percolatn will be more and hence wtr requirement is high.

④ Cultivatn method :- if cultivtn method including tillage & irrigtn is faulty and less efficient resulting in wastage of wtr. which needs the wtr high.

⑤ Types of crop :- wtr requirement varies from crop to crop

⑥ Base period of crop :- if base period of crop is more amnt of wtr require will be

⑦ Quality of water :- The type of wtr hvng fertilizer matters will cause less consum-tn of wtr while containing salt consume ~~les~~ wtr

⑧ Canal Condtn : In Earthen Canal seepage and percolatn losses will be ↑(high), if however Canal is lined losses will be less.

⑨ Depth of irrigtn of water
⑩ Wind velocity of locality
⑪ Topography
⑫ Temperature of the atmosphere

Consumptive use of water (Evapo-transpirtn)

- Consumptive use of water by a crop or Evapo transpirtn of water by crop is the depth of water consumed by evaportn and transpirtn during crop growth.
- Depth of water consumed by plant for evaporatn, transpirtn and weed groth is called consumptive use of water.
- Consumptive use of wtr also called evapo-transpirtn.
- Calculated in terms of depth & unit is mm/day or cm/day or m/day

Evaporation

Transfer of water from liquid state to vapour state is called evaportn.

- If irrigatn water is supplied by flooding method, evaportn is ↑ from soil surface.

Transpiratn

Loss of water from the surface of leaves, stems, root in the form of vapour is called transpirtn.

- The combined axn of evaportn & transpirtn is called evapo-transpirtn.

Principal crops, their season and water requirement

The chief & principal crops of Nepal are

(1) Rice
(2) Wheat
(3) Maize
(4) Millet (कोदो)
(5) Mustard
(6) Sugarcane etc.

From Agricultural point of view the year can be divided in 2-season

(1) Rabi (हिउँद) — 1st oct to 31st march
(2) Kharif (बर्खा) — 1st april to 30th september

↳ Kharif Crops requires 2-3 times the quantity of water than Rabi crops.

Crops	Depth of wtr required (cm)	Season or period of Growth
(A) Kharif Crops		
① Rice	120	July to Nov
② Maize	45	June to sep
③ Pulse	30	July to Nov
④ Cotton	25	May to Nov
⑤ Millet	30	July to Nov
(B) Rabi Crops		
① Wheat	40	Oct to March
② Gram	30	sep to March
③ Mustard	45	Oct to Febouary
④ Potato	75	sep to feb
⑤ Tobacco	60	Oct to feb
(C) Perenial crop (allseason)		
① Sugarcane	90	All season

Methods of applying water to irrigtd field

It is 3-types

① Surface Method

② Sub-Surface Method

③ Sprinkler Method

① Surface irrigatn Method - In this method, water is directly applied to the surface of field or land.

It is 2-types
- ① Flow Irrigatn
- ⑪ Lift irrigatn

(i) Flow Irrigatn :- when water is available at ↑er level and supplied to ↓er level by the axn of gravity is Flow irrigatn.
- it is 2-types
  - ⓐ Perennial (control) irrigatn
  - ⓑ Flood irrigatn / uncontrol irrigatn

ⓐ Perennial / control irrigatn :- In this system Continuous wtr supply occur throught the crop field.
- for this weir, barrage or dam is required.
- it is controlled system.

ⓑ Flood irrigatn / uncontrol irrigatn :- Also called inudation irrigatn. In this method soil is kept sub-merged and throughly flooded with water so as to cause saturatn of land.

(ii) Lift irrigation :- It is done when field is up than wtr sorrce.
- Pumping is done to lift wtr.

Advantage of Surface irrigatn
1. initial cost is less.
2. No highly skill man power
3. No highly develop technology
4. Easy to supervise

Disadvantage
1. No control over wtr (wastage of wtr)
2. Chances of wtr logging
3. levelled land is required
4. chances of erosion of soil.

Techniques of Surface irrigatn and their Suitability

It is 3-types

(1) Flooding → (1) Controlled → (1) Free flooding / ordinary flooding, (2) Border flooding, (3) check flooding, (4) Basin flooding, (5) zig-zag flooding
→ (2) Uncontrolled

(2) furrow irrigatn

(3) Contour farming

(A) Flooding - It is further divided into 2-types
(1) Uncontrolled flooding / wild flooding
(2) Controlled flooding

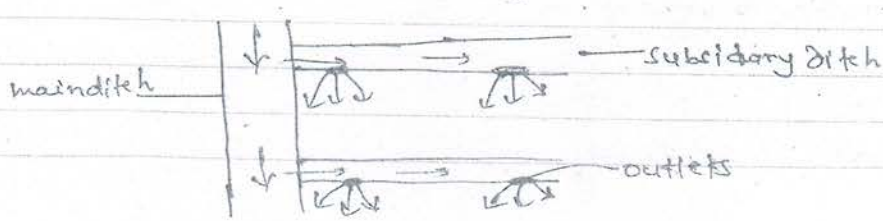
(1) Uncontrolled flooding :- In this method, water is spreaded or flooded on a smooth f land, without much Control to preparation.
- it is used when there is sufficient water.
- Particularly used for inudation irrigatn.

(2) Controlled flooding :- In this method, water is spreaded over the land with proper methods to control the depth of application.

It is 5-types

(a) Free flooding
- also known as irrigatn by plots.
- In this method, ditches are excavated in the field, and they may be either on the Contour or up and down the slope. wtr from this ditches flow across the field. Afte the water leaves the ditches no attempt is made to Control the flow by mea of levees (= आली खेतको)

Suitability
- It is suitable if land is irregular and water is expensive.



(b) Boarder Flooding :- In this method, land is divided into number of strips with the help of low levee called Boarder.

- Here, water fermed into the upper end of each strips and flows slowly 2wards the lower limit.

Suitability

- It is suitable for Regular land.
- Suitable for forage crops.

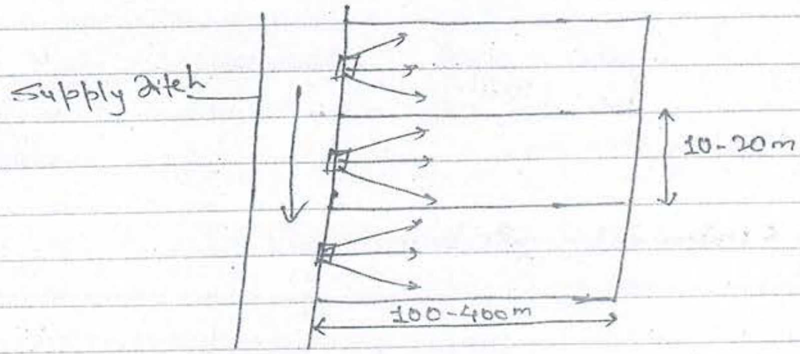


fig:- Boarder Flooding

(c) Check Flooding :- It is similar to free flooding, except that water is controlled by surrounding the area with low and flat levee.

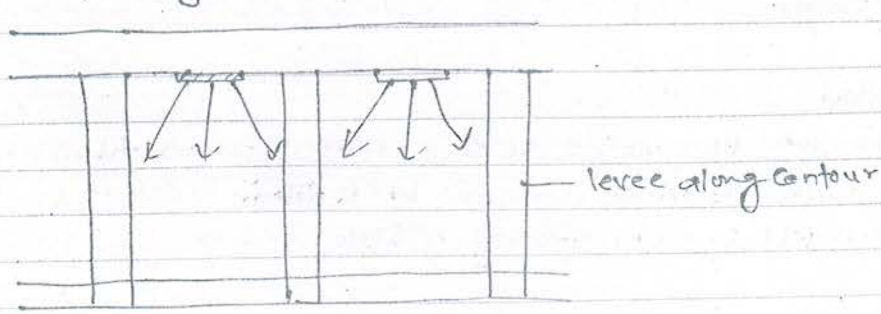


fig:- Cheik Flooding

Suitability

- Suitable for both Permeable and impermeable soil.

(d) Basin flooding
- it is special type of check flooding
- adopted to orchards tree
- one or more trees are generally placed in basin and surface is flooded.

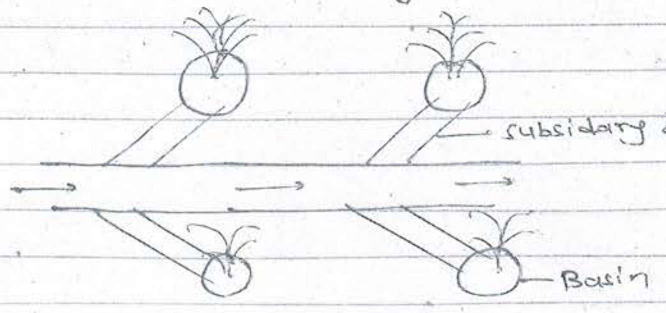


fig:- Basin flooding

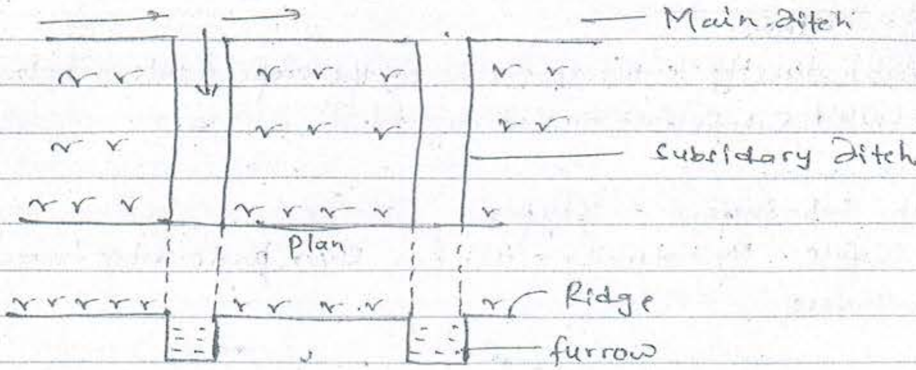
Suitability
- Suitable for permeable + impermeable wtr. soil.

(e) Zig-zag method
- special type of flooding where irrigtn of water is done is zig-zag way reach the dead end of each plot.

Suitability
- for plane grnd
- for impermeable soil.

(2) Furrow method
- suitable for Row crops like maize, potatoes, Cotton, Sugarcane, tobacco etc.
- In this method, furrow is made where water is filled.
- Evaporatn loss is less so water efficiency is high.

Suitability
- for Row crops like cauliflower, maize, potatoes, Sugarcane etc.
- for flatter grnd.



Section

fig:- Furrow Method

Advantage

① Evaporatal loss is Reduced.
② Suitable for crops like maize, Sugarcane, tobacco, potato etc.
③ Labour requirement less.
④ No wastage of land
⑤ furrow can be made befor & after plantatn.

Disadvantage

① If furrow is long percolation is much near upper end & little water at down stream.

③ Contour farming

- Done in hilly areas hvng steep slopes with a quickly falling contour.

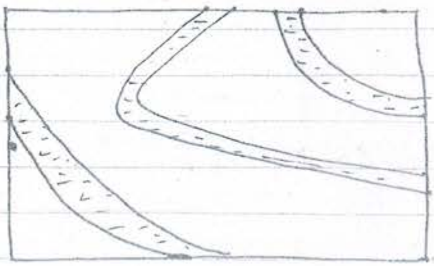


fig:- Contour farming

(2) Sub-Surface Method

- Supplying of water directly to the root zone of the crops inside or below the [surfa]-ce of the earth is called sub-surface method irrigatn.

Types of sub-surface - 3 types

(1) Natural sub surface - By naturally either from River, pond or from other source, wa[ter] reach to soil sub-surface

(2) Artificial sub-surface - By putting perforated pipes irrigatn done.

(3) Drip or trickle sub-surface irrigatn - It is best method

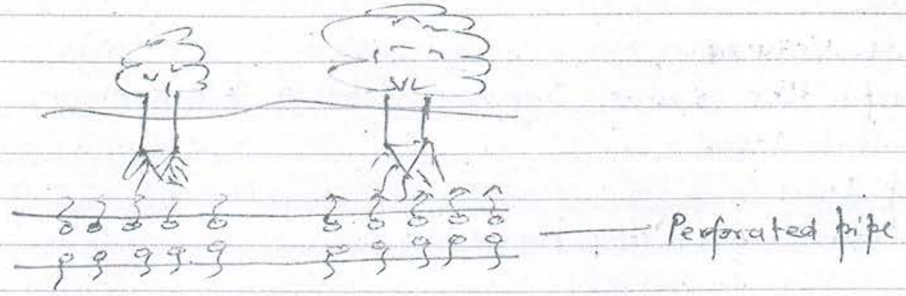
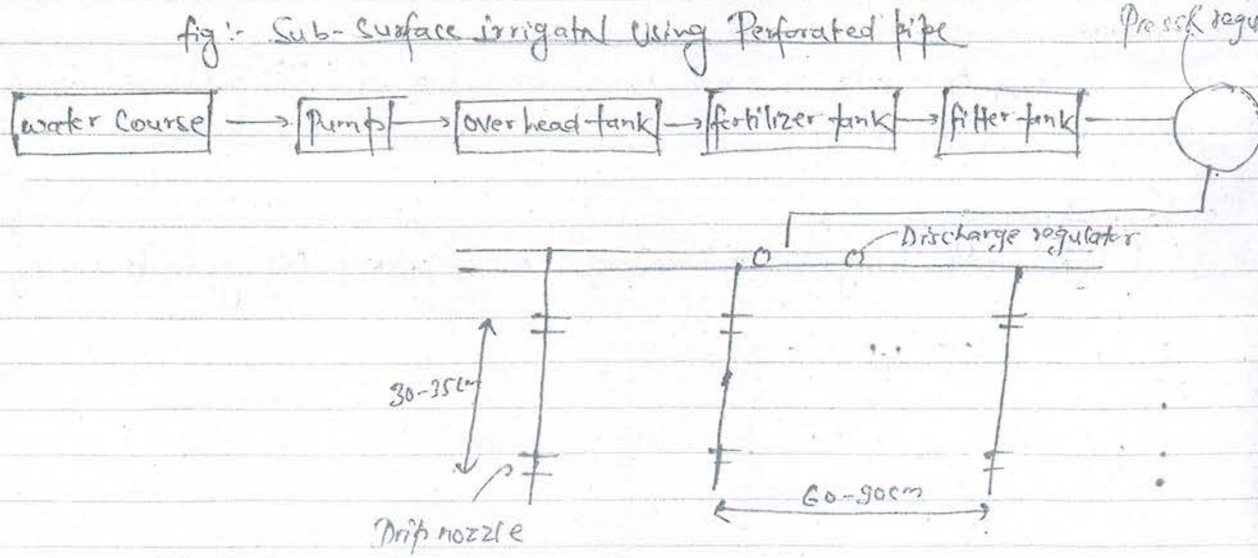


fig:- Sub-Surface irrigatn Using Perforated pipe



Drip trickle irrigtn

Advantages of Sub-Surface

1. less requirement of irrigatn water
2. water supply at optimum level.
3. Water logging avoided
4. Increase in Net irrigable area
5. Nutrients preservatn.

Disadvantages

1. High initial Cost
2. High skilled manpower
3. Difficult to supervise
4. High Devlopment technology.

Suitability

- for permeable soil.
- Uniform topography
- Moderate slops
- Good quality waters.

(3) Sprinkler Method of irrigatn

In this method, water is spread over the land in the form of spray like a ligh[t] rainfall in the form of drizzle.

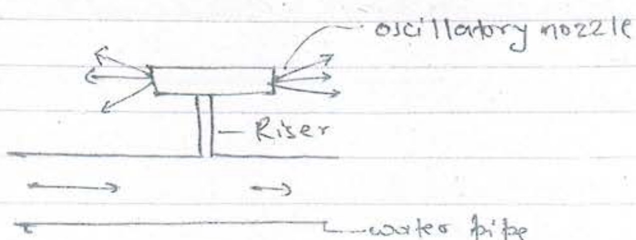


fig:- Sprinkler system

Types of sprinkler irrigatn - 3 types
① Permanent System
② Semi-permanent System
③ Portable System

Advantage of Sprinkler irrigtn
① Used for steep slopes
② shallow depth of soil
③ Erosion can control
④ fertilizer can uniformly distributed
⑤ Controlled irrigtn system
⑥ Seepage losses are eliminated or avoided
⑦ less labour reqd.

Disadvantage
- wind effect may cause non-uniform spreading of water
- At high tempertr loss will be more
- High initial cost
- skilled manpower reqd
- Not suitable for larger depth of irrigatn
- only sand & silt free wtr is used.
- Require electric power

Suitability
- when land can not be prepared by surface method
- Excessive slopes
- irregular topography
- Erosive soil
- Permeable or impermeable soil
- Depth of soil is shallow
- Not suitable for wtr logging area
- for low maintenance cost
-

VVI

factor Affecting irrigatn method

1. Type of Soil (Nature of Soil
2. Topography of soil
3. Availability of water
4. Crop water requirement
5. Net effective rainfall in that locality or area.
6. Availability of funds & economic analysis
7. Experience of farmers or Users.

Design of sprinkler irrigatn

1. Determine the optimum appliktn rate (I)
2. Compute depth of water

$$\boxed{Q = 2.78\, AD}$$

Q = Discharge of pump (lps)
D = Net depth
A = Area in ha.

3. find required Capacity of System

$$\boxed{Q = \frac{2.78\, A \cdot D}{F \cdot H \cdot E}}$$

F = irrigatn interval in day
H = wrkng hr/day
E = field appliktn efficiency

4. Determine the Sprinkler Capacity & Nozzle size

$$\boxed{Q_r = \frac{S_l * S_m \times I}{3600}}$$

Q_r = Required Discharge
S_l = Sprinkler spacing (lateral)
S_m = Sprinkler " along main pipe
I = optimum water appliktn rate

5. Determine lateral Spacing - It depends on wind velocity

6. find Sprinkle Spacing

$$\boxed{R = 1.35 \sqrt{dH}}$$

R = radius of area Covrd by sprinkler
d = diamtr of Nozzle
H = PressR. head

(7) Determine total no. of Sprinkler
(8) Locate the main and lateral line.
(9) Determine size of lateral & main pipe

$$\boxed{H_f = \frac{1.1101 \times 10^{10} \left(\frac{Q}{C}\right)^{1.852}}{D^{4.87}}}$$

(10) Select the pump & power Unit.

$$\boxed{H_p = \frac{Q \cdot TDM}{75\,\eta}}$$

TDM = total dynamic head
η = efficiency

Numericals

(1) Calculate the potential evapotranspiration for an area over Kathmandu in the mo of Janaury by Penmann method. The following data is available

Solⁿ.

Mean monthly temperature $(t) = 19°C$
Relative humidity $(R.H) = 75\%$
Mean observed sunshine hour $(n) = 9hr.$
Potential Sunshine hours $(N) = 11.60 hr$
Wind Velocity at 2m ht. $(V_2) = 100 km/day$
Albedo Constant $(\alpha) = 0.25$
Upper terrestial or solar Radiation $(H_c) = 9.506$ of water/day
Other Values
Latitude $(\phi) = 28°4'$
Saturated vpr pressR at 19°c $(e_s) = 16.5mm$ of Hg
Slope of Saturated press R $(A) = 1.00 mm/°c$
Psychometric Constant $(\gamma) = 0.49 mm/°c$
Boltzman Co-efficient $(\sigma) = 2.01 \times 10^{-9} mm/day$

$E_T = ?$ (E_T = potential evapotranspiration)

Sol^n.

$$\boxed{E_T = \frac{AH_n + E_a \gamma}{A + \gamma}} \quad \text{—— (1)}$$

Here,

$$\boxed{E_a = 0.35\,(e_s - e_a)\left(1 - \frac{V_2}{160}\right)} \quad \text{—— (2)}$$

for, e_a

$$e_a = \frac{R.H \times e_s}{100} = \frac{75 \times 16.5}{100}$$

$$\boxed{e_a = 12.375\ \text{mmhg}}$$

Now from eqn (2)

$$E_a = 0.35\,(16.5 - 12.375)\left(1 + \frac{100}{160}\right)$$

$$\boxed{E_a = 2.34\ \text{mm/day}}$$

Again

$$\boxed{H_n = H_c(1-\alpha)\left(a + \frac{bn}{N}\right) - \sigma T_a^4\left(0.56 - 0.092\sqrt{e_a}\right)\left(0.10 + 0.9\frac{n}{N}\right)} \text{—(3)}$$

for a & T_a, b

$$a = 0.29 \cos\phi$$
$$= 0.29 \cos(28°4')$$

& $T_a = 273 + t°C$
$= 273 + 19°C$

$$\boxed{a = 0.255}$$

$\therefore \boxed{T_a = 292\,K}$

& $\boxed{b = 0.52}$

Now from eqn (3) we get,

$$H_n = 9.506\left(1-0.25\right)\left(0.255+\frac{0.52\times9}{11.60}\right)-\left(2.01\times10^{-9}\times(292)^4\right)\left(0.56-0.092\sqrt{12.3}\right)\left(0.10+\frac{0.9\times9}{11.6}\right)$$

$$= 4.694 - 2.757$$

$$\boxed{H_n = 1.93 \text{ mm of wtr/day}}$$

Now eqn (1) becomes,

$$E_T = \frac{1\times1.93+2.34\times0.49}{1+0.49}$$

$A = 1$ mm/°c
$\gamma = 0.49$ mm/°c

$$\therefore \boxed{E_T = 2.06 \text{ mm/day}}$$ // Ans //.

(2) After how many days will you supply water to soil in order to ensure sufficie[nt] irrigation of the given crop if

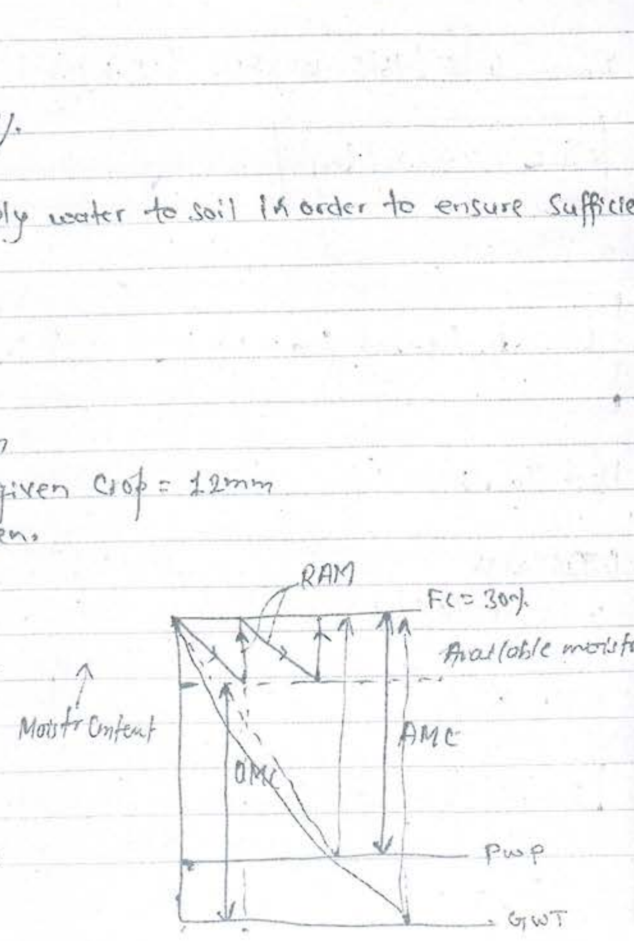
i) Field Capacity of Soil = 28%.
ii) Permanent wilting point = 13%.
iii) Dry density of soil = 1.3 gm/cc
iv) Effective depth of root zone = 70 cm
v) Daily Consumptive use of water for given crop = 12mm

Assume any other data, if not given.

Soln.

Given,
F.C = 28%.
PWP = 13%.
γ_d = 1.3 gm/cc
d = 70cm

Daily Consumptive use of wtr = 12 mm = 1.2 cm
Days = ?



we have,

Available Moisture Content (AMC) = FC - PWP

$= 28\% - 13\%$

$$\boxed{AMC = 15\%}$$

Radially Available Moisture (RAM) = 75-80% of Available moisture Content.
Assume 80%.

$\therefore$ RAM = 80% of AMC

$= \frac{80}{100} \times 15$

$$\boxed{RAM = 12\%}$$

Again,

RAM = FC - OMC

$\therefore$ Optimum moistr Content (OMC) = FC - RAM

$= 28 - 12$

$$\boxed{OMC = 16\%}$$

It means that the moistr will be filled by irrigatn between 16% (OMC) and 28% (FC)

Now,

Depth betn these 2-limits $= \frac{\gamma_d \cdot d (FC - OMC)}{\gamma_w}$ $\gamma_w = 1 gm/cc$ - inc

$= \frac{1.3 \times 70}{1}\left(\frac{28}{100} - \frac{16}{100}\right)$

$= 10.92$ cm

Hence,

water available for evapo-transpiratn = 10.92 cm

$\therefore$ 1.2 cm of water is utilised by plant in 1 day

10.92 cm of water will be Utilised by plant in $= \frac{1}{1.2} \times 10.92$ days

$= 9.1$ days

≈ 9 days

$\therefore$ After 9 days water should be applied to given Crop //Ans//.

Extra Question

(a) if Deth of water stored between FC & PWP $= \frac{\gamma_d \cdot d}{\gamma_w} (FC - PWP)$

$= \frac{1.3 \times 70}{1} \left(\frac{28}{100} - \frac{13}{100} \right)$

$= 13.65$ cm

(b) if Depth of water stored betn FC & GWT $= \frac{\gamma_d \cdot d}{\gamma_w} (FC - \cancel{GWT}^{\,0})$

$= \frac{1.3 \times 70}{1} \times \frac{28}{100}$

$= 25.48$ cm

(3) Find the Field Capacity (FC) with following data :

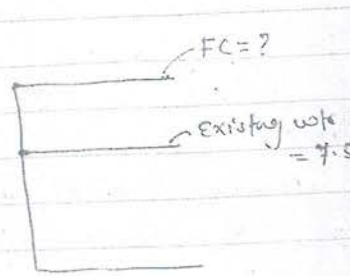
Root zone depth (d) = 2.5 m

Existing water Content = 7.5%

Dry density of Soil = 1.5 gm/cc = 1500 kg/m³

water applied to soil = 600 m³

Area of Plot = 1000 m²



Soln. Give,

Volume of total water applied to soil (V) = 600 m³

Area of plot (A) = 1000 m²

Now,

water used in raising Moisture Content upto Field Capacity from existing 7.5%

$$= \frac{\text{Volume (V)}}{\text{Area (A)}}$$

$$= \frac{600}{1000}$$

$$= 0.6\,m$$

Depth of irrigation water $(d_w) = \frac{\gamma_d \cdot d}{\gamma_w}$ (FC − existing water content)

upper limit — always FC; lower limit

$$0.6 = \frac{1.5 \times 2.5}{1}\left(FC - \frac{7.5}{100}\right)$$

$$FC = 0.235$$

$$\boxed{FC = 23.5\%}$$ // Ans //

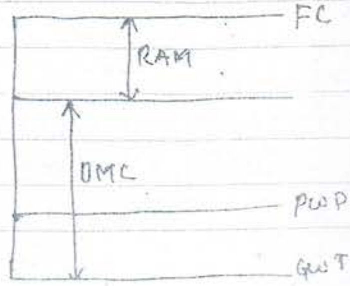
(4) Find out the frequency of irrigation for a crop having
Field Capacity (F.C) = 30%
Permanent Wilting point (PWP) = 12%
effective depth of root zone (d) = 80cm
dry density of soil (γ_d) = 1.4 gm/cc = 1400 kg/m³
Daily Consumptive = 10mm
if application efficiency (η_a) = 75%. Calculate water requirement at field outlet.
Assume RAM (Readily Available Moisture) is 80% of available moisture

Soln.

we know from question,

RAM = 80% of Available moisture
= 80% of (FC − PWP)
$= \frac{80}{100} \times (30-12)$

$$\boxed{RAM = 14.4\%}$$



Now,

$OMC = FC - RAM$

$= 30 - 14.4$

$\boxed{OMC = 15.6\%}$

Here Moist is allowed to vary between 15.6% & 30%.

$\therefore$ Depth of water for irrigatn $(d_w) = \frac{\gamma_d \cdot d}{\gamma_w}(FC - OMC)$

$= \frac{1.4 \times 80}{1}\left(\frac{30}{100} - \frac{15.6}{100}\right)$

$\boxed{d_w = 16.128\,cm}$

we have,

Quantity of water supplied to field outlet $= \frac{NIR}{\eta_a}$ → Net irrigatn rqk

$= \frac{16.128}{\frac{75}{100}}$

$= 21.504\,cm$ // Ans //.

Extra Questn

if Conveyance efficiency is 80%. Find the wtr Requirement at Canal outlet.

Then,

wtr Requirement at Canal outlet $= \frac{\text{Quantity of wtr Supplied to field}}{\eta}$

$= \frac{21.504}{\frac{80}{100}}$

$= 26.88\,cm$ Ans //.

2072/05/11

CHAPTER :- 3 - Canals

Canals :- It is defined as an artificial channel constructed on the grnd to carry water from a source (Reservoir, dam, river etc) to the field.

Classificatn of Canal

(A) On the basis of function :- 4 types

(1) Irrigatn Canal
- Used for irrigation purpose
- it may be → lined or → Unlined
- seepage loss Controlled by lined irrigatn.

(2) Power Canal
- For hydropower generatn.

(3) Drainage Canal
- Used to drain the excess wtr from irrigatn field.

(4) Navigation Canal :- Used for Navigatn or transportatn

(5) feeder Canal :- Used to feed one or more Canal. It is also a link Canal.

(6) Multipurpose Canal - Used for all types

(7) Carrier Canal :- Carry water from another Canal.

(B) Based on Use - 2 types

(1) Permanent Canal
- Canal which is Constructed & operated permanently is Permanent Canal.
- Regular maintenance is necessary.

② Inudation Canal

- Canal which are only used for temporary purposes during floods are inudated c

- Also called **Flood Canal**.

Ⓒ Based on type of Soil - 2 types

① Alluvial canal :- A soil which is formed by Continuous deposit of silt is Called alluvial soil, & the Canals Constructed on alluvial soil is Called Alluvial Canal.

- Alluvial soil is very fertile soil can absorb high rainfall water so ↑ly produc

② Non-Alluvial Canal :- Due to disintegration of mountainous region, formatn of rocky plain area occur which is called Non-alluvial area.

- And Canal passing through such area (hard rocks) is called Non-alluvial Canal.

- Soil infertile & non-permeable generally.

Ⓓ Based on Discharge - 6 types

Cumecs = m^3/s

Q = discharge

① Main canals ($Q \gg 30$ cumecs)

② Brunched Canals ($Q = 2.5$ to 30 Cumecs)

③ Distributary Canals ($Q < 2.5$ cumecs)

④ Minor canals ($Q < 2.5$ Cumecs)

⑤ Field Canals (water Course) - discharge is less than minor Canal.

⑥ Flushing Canal

Components of Canal System

① Head work

② Main Canal / Major Canal

③ Branch Canal

④ Distributary Canal (Major)

⑤ Minor distributaries

⑥ water Course / field canals

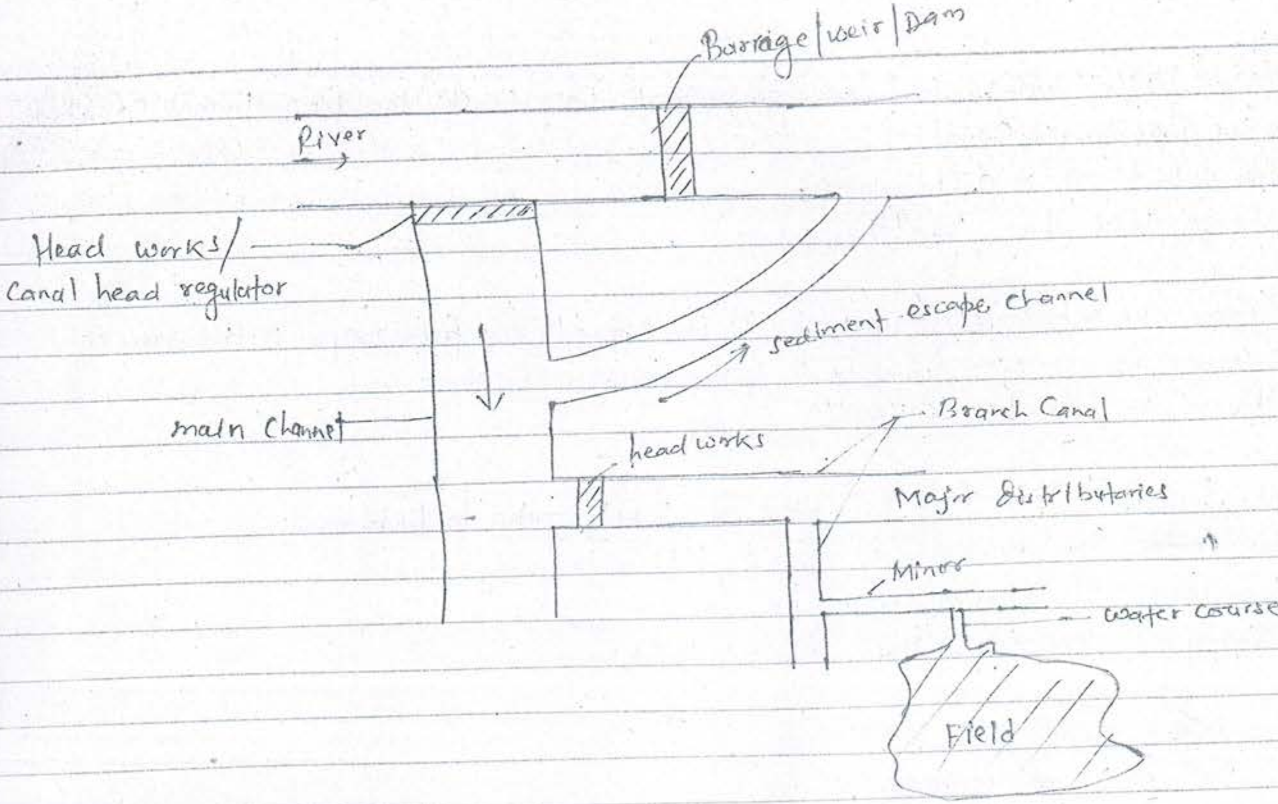


fig: Component of Canal System

- Barrage, weir, dam is used to divert water.

(1) Head works - Used to store, divert and Control water and regulate supply to irrigatn Canal.

(2) Main Canal :- Used to Carry water directly from River.
- Not used for irrigatn.
- Discharge depend on size of Canal. (Generally $Q \gg 30 m^3/s$)

(3) Branch Canal :- when main Canal reaches the area to be irrigated it gets divided into branches, covering the different parts of Area which is called Branch Canal.
- Very little irrigatn is done from branch Canal.
- Discharge is grtr than $30 m^3/s$ ($Q \gg 30 m^3/s$)

(4) Major Distributaries :- These are small canals divided from branch canal sometimes from main canal.
- Supply wtr to minor distributaries
- Discharge less than $30 m^3/s$

(5) Minor Distributaries / Minor :- It is sub-divided from major Distributaries.
- take water from major & supply to water course.
- Discharge is less than $2.5 m^3/s$

(6) Water Courses / Field Canals :- Used to supply water to field.
- water comes from minor distributor & supply to field.
- water course is formed itself by farmers.
- Discharge is less than minor distributors.

VVI

Canal Alignment

Path

The path followed by the canal from the source to the irrigatn field in case of irrigatn canal is called **Canal Alignment**.

Approach

Canal Alignment should

(1) have less number of cross-drainage structrs
(2) have most economical way of distributing water to land
(3) have high command area as possible
(4) Not made in salty rocks & crack strata
(5) Avoid village, roads, religious place & other valuable properties.
(6) have minimum length of main canal.

VVI

Classificatn of Canal Based on Alignment - 3 types

(1) Watershed / Ridge Canal
(2) Contour Canal
(3) Side-slope Canal

① Watershed or Ridge Canal :- The dividing line between the Catchment area of two drains is called watershed
- The Canal which is aligned along only any natural watershed is called wtR shed Canal.

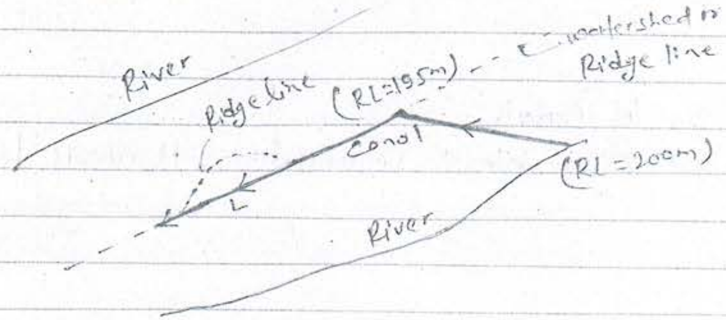


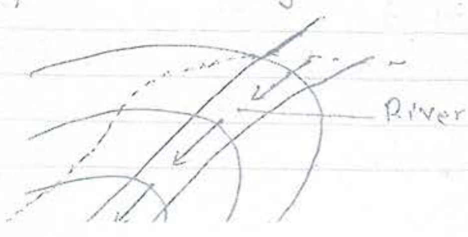
fig:- watershed Canal

Suitability
- When sloped of land are relatively flat & uniform.

Advantage
- It irrigates on both side or both bank of Canal.
- Not necessary of Cross-drainage strxtr.

Disadvantage
- Due to depresson in Ridge line Constrn of Canal bridge or Syphon is necessary
- if wtshed is passing through village or town this Canal may have to leave watershed for some distance.

② Contour Canal
- In hilly area, there is no possibility of aligning the canal along the watershed. So the Canals are aligned along Contour lines.



Advantage
- economical than Ridge Canal.
- Suitable for contour farming.
- Suitable and generally constructed in hilly area.

Disadvantage
- Irrigates only one side of Canal.
- As the drainage flow is right ⊥le to contour line, it needs large number cross-drainage strxtrs.

(3) Side Slope Canal
- A Canal which is aligned at right ⊥led to the Contour lines is called side - slope Canal.
- Such canal is parallel 2 the ntrl drainage flow. So No need of cross-drainage.

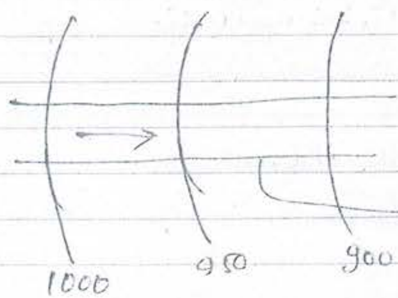


Fig:- Side - slope Channel

Suitability
- for very flat Area

Advantage
- no need of Cross-Drainage strxtrs.

NYE

Canal losses Due to Seepage & Evaporation

water loss is due to
(i) Evaporation - 1-2% of total loss
(2) Seepage - 98% of total loss - which is called as Channel loss or Canal loss or Conveyance loss.

(1) Evaporation loss
- loss of water in the form of vapour is evaporation loss.
- in Canal, evaporation loss is small (ie generally 1-2%) of total loss.
- Evaporation loss depends on
  1. Temperature
  2. Humidity
  3. Exposed Surface
  4. Wind velocity

(2) Seepage loss
- Main cause of loss in the Canal is due to seepage.
- abt. 98% loss due to Seepage or percolation.
- seepage loss depends on
  1. Type of soil (ie Permeability of soil)
  2. Position of water-table
  3. cross-sxn of channel
  4. Condition of canal — lined or unlined / New or old
  5. Physical properties of Canal
  6. Velocity of water
  7. Sediment Concentration (↑er Concentration ↓es Seepage)

Position of water Table
Depending upon position of water table 2-types of Seepage
(1) Absorption
(2) Percolation

① Absorption: when water table is considerably below grnd level, the water se[eps] through the pores is unable to join Grnd wtr table (GWT) & wets the subsoil locally forming a saturated zone. The zone beth saturated zone remains unsaturated.

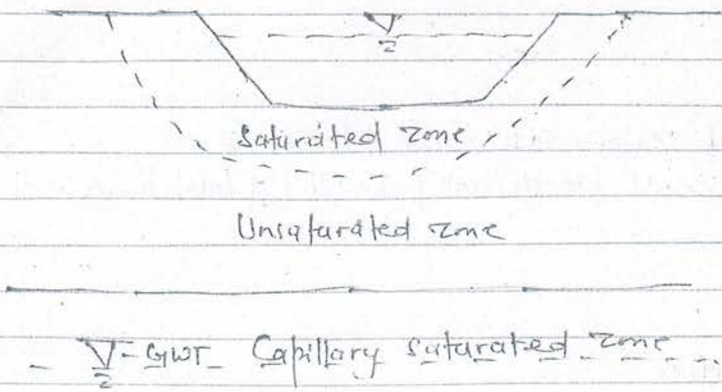


fig:- Absorption

② Percolation: when water table is closer to the grnd level the seepage water may establish a direct & continuous flow in beth canal sxn & wtr table.

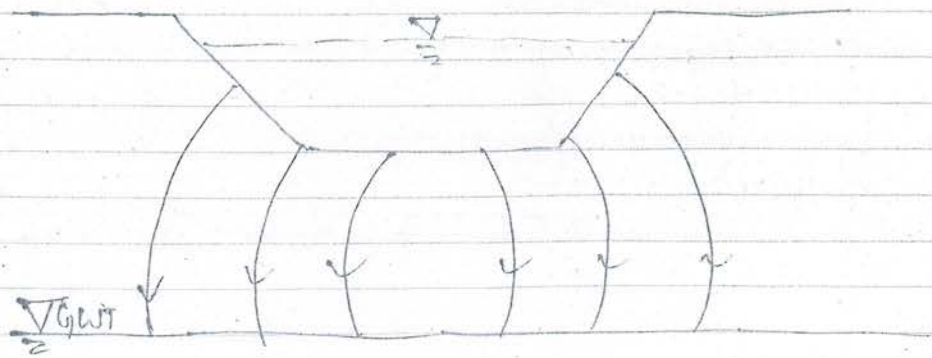


fig:- Percolation

Calculatn of loss

(1) Direct Measurement of seepage loss
- inflow & outflow method
- Ponding method

(2) Theoritical method
- Darcy's law
- $Q = KIA$
- Greater the k greater the loss

Control of Canal loss

(1) Lining of Canal
(2) Slxn of efficient sxn
(3) Avoiding Canal alignment in filling
(4) Good hydraulics
(5) Use of Closed Conduits (Concrete & polythene pipes)

Assesment of water Requirement in Canals of Command Area

- water requirement in Canals depend upon the Command area.
- Greater the Command area, ↑er will be requirement of water.
- water requirement in canals also depend upon loss of water through Various portal.

Assesment of Command Area

(1) Gross Command Area (GCA)
(2) Cultivable Command Area (CCA)
(3) Net Command Area (NCA)

(1) GCA (Gross Command Area) :- It is the total area that Can be irrigated economically without Considering the limitatn of Quantities of available water.
- it includes both Cultivable & Non-Cultivable area.

$$\boxed{\begin{aligned} GCA &= \text{Cultivable area} + \text{Non Cultivable area} \\ &= \text{Culturable} + \text{Non-Culturable} \end{aligned}}$$

(2) Cultivable Command Area (CCA) :- It is total area of irrigation on wh
Cultivation is possible.
- It is Cultivable part of GCA & includes all land of GCA on which Cultivation is possible like, Pastures & Fallow lands.
- Does not include uncultivable part of GCA like ponds, reserved forests, roads etc.
- CCA = 80% of GCA

(3) Net Command Area (NCA) :- It is the portion of Culturable Command area (CCA) under the Command of any Irrigation channel Considering the limitation of available water is called NCA.

Here,
$$\boxed{FIR = \frac{NIR}{\eta_a}}$$

Cu = Consumptive use
Re = Effective Rainfall
FIR = Field Irrigation Requirement
NIR = Net Irrigation Requirement
η_a = application efficiency
CIR = Consumptive irrigation Requirement

NIR = Cu − Re + loss
CIR = Cu − Re

$$\boxed{\text{Frequency of irrigation} = \frac{\text{Allowable moisture depletion}}{\text{Rate of Consumptive Use}}}$$

Intensity of irrigation

The percentage of CCA proposed to be irrigated in a given season is called irri-tion intensity of that season.
- Total annual intensity of irrigation is sum of each intensity for different season
for eg.
The intensity of irrigation under Bagmati Irrigation project is only 27.6% Fo Kharif and 34.4% for Rabi season.

AII = Annual intensity of irrigatn.

∴ Annual intensity = Sum of Irrigatn intensity of each season
= 27.6 + 34.4
= 62%.

- CCA * AII = Actual area to be irrigated in a year

Assesment of water Requirement

① First obtain CCA
② Fix intensity of irrigatn so that Area to be actually irrigated may be obtai

| Actual Area to be irrigated = CCA * AII |

③ The water depth required by each crop during a given interval multiplied by the area would give volume of water required by crop during that interval
④ The sum of water requirement for all crops could thus be obtain for all interval and they channel capacity designed for that interval which has maximum demand during the year.

Water requirement of crop :- Total quantity and the way in which a crop requi water from the time it is harvested.
- It depends upon type of crop & place

V.I.V. imp unit.

Duty, delta and their Relatnship

Duty (D) :- It is defined as the number of hectors of land that can be irrigated by a unit quantity of water flow ($1 m^3/s$)
- it is relatnship both volume of water & the area of crop it matures.
- Unit : ha/cumecs

$$\boxed{\text{Duty (D)} = \frac{\text{Area of land irrigated}}{\text{Flow of water}}}$$

Delta (Δ) :- It is defined as the total quantity of water reqd for the full growth of a plant is called its Delta.
- Total depth of water (in cm) reqd by crop to come to maturity is called Delta
- expressed in term of depth (cm) of water.

eg.

Rice — 120 cm
Sugarcane — 90 cm
wheat — 30 cm
Maize — 5 cm

Relatn between Delta (Δ) & Duty (D), Base period (B)

let,

B = Base period of crop
Δ = Delta of crop
D = Duty of water
Q = Flow of wtr in m^3/sec.

Then,

Volume of water applied to the field during 'B' period $= Q * B * 24 * 60 * 60$
$= 86400\, Q B$
$= 86400 B\ m^3$ (for $Q = 1 m^3/sec$) — ①

Also,

Volume of water in field $= D * \Delta * Q$ ha-m
$= D \Delta Q \times 10^4\ m^3$ — ② ($1 ha = 10^4 m^2 \cdot m = 10^4 m^3$)

equating ① & ②

$$86400\, B\, \cancel{m^3} = D \Delta Q \times 10^4\, \cancel{m^3} \qquad Q = 1 m^3/sec$$

$$86400\, B = D \Delta \times 10^4$$

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

in metre (m)

$$\boxed{D = \frac{864\,B}{\Delta}}$$ in cm.

B = in days
D = ha / cumecs
Δ = in metre (m) or cm

Next method

Total depth of water applied $(\Delta) = \frac{\text{Volume}}{\text{Area}}$

$V = Q \times B \times 24 \times 60 \times 60$
$A = 10^4 D\ m^2$

$= \frac{Q\,B \times 86400}{10^4\,D}$

$Q = 1\,m^3/sec.$

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

$$\boxed{D = \frac{8.64\,B}{\Delta}}$$ in (m).

Duty (D) at Various places

- Duty of water for a crop is the no. of hectors of a land which the water ca irrigate.
- So, if the water requirement of crop is high, Duty (D) will be less & viceversa.

VVI Factor affecting Duty

① Type of crop
- different crop require diff. quantities of water.
- Duty of wtr will be ↑er for the crop which require less amnt of wtr.

② Climate and season
- Duty includes water losses due to evaporatn, percolatn, seepage, transpiratn etc & losses of water varies from season to season.

③ Type of Soil
- If Permeability of soil is high, wtr requirement is ↑ so Duty will be ↓

④ Effective Rainfall

- Greater the effective Rainfall, ↓er the irrigatn wtr requirement & hence ↑er the Duty of irrigation water

⑤ Efficiency of Cultivatn

- For advanced Cultivatn, wtr required will be more & less duty of irrigtn w

Importance of Duty

- To design Canal sxn.
- To find discharge,

$$\boxed{Q = \frac{Area}{Duty}}$$

Base period

The time between first watering of crop at the time of sowing to last watering before harvesting.

Crop period :- Time period that elapsed from instant of its sowing to the instant of its harvesting.

- it is grtr than Base period.
- Practically,

$$\boxed{\text{crop period} = \text{Base period}}$$

KOR Period & KOR Depth

- irrigatn wtr reqd is maximum when crop grow a few cm and its deman decreases as crop gets maturity & almost Nill when it gets matured.

Kor watering :- The 1st watering which is given 2 a crop when a crop is a few cm high is known as Kor watering.

KOr period :- It is a critical growth period of a crop during which the wtr demand is maximum.

- KOR depends on-
  1. Climate
  2. Type of crops

		KOR period	
eg. for Rice	——	2-4 weeks	→ Kharif
wheat	——	3-8 weeks	→ Rabi

KOR depth :- The depth of water required during KOR period (KOR watering) is called KOR - depth.

eg.

		KOR depth	
for Rice	——	19 cm	→ Kharif
wheat	——	13.5 cm	→ Rabi

Numericals

① Find the delta of a crop if duty is 1800 ha/cumecs and Base period is 130 days. What would be the duty if Delta is increased by 20% & base period is reduced by 10 days.

soln: Given,

Delta (Δ) = ?

Duty (D) = 1800 ha/cumecs

Base period (B) = 130 days

we know,

① 1st Case : Delta(Δ) = ?

$$\Delta = \frac{8.64\ B}{D} = \frac{8.64 \times 130}{1800}$$

$$= 0.624\ m$$

$$\therefore \boxed{\Delta = 62.4\,cm}$$ //

② 2nd Case Duty (D) = ?

According to questn,

$\Delta = (100+20)\%$ of ~~62.4~~ 0.624 (Δ inireased by 20%.

$= \frac{120}{100} \times 0.624$

$\boxed{\Delta = 0.75}$

And,

$B = 130 - 10 = 120$ days (B decrease by 10 day)

Now,

$\text{Duty}(D) = \frac{8.64 \times 120}{0.75}$

$\boxed{D = 1382.4 \text{ ha/cumecs}}$ //Ans//.

② The CCA for a distributory is 1700 ha. The Intensity of irrigatn is 50% for Rabi & 30% for kharif crops. If the KOR period is 4-weeks & 2.5 weeks for Rabi & kharif crops respectively. Calculate the outlet discharge. Take KOR depth for Rabi crops = 12 cm &

kharif crops = 25 cm

Soln.

Given,

CCA = 1700 ha

intensity for Rabi = 50%.

∴ Area for Rabi = 50% of CCA

$= \frac{50}{100} \times 1700$

$\boxed{A_R = 850 \text{ ha}}$

Again,

intensity for kharif = 30%.

∴ Area of kharif (A_K) = 30% of 1700

Cumecs $\rightarrow m^3/s$

$$A_K = \frac{30}{100} \times 1700$$

$$\boxed{A_K = 510\ ha}$$

Given, for Rabi

Kor period or Base period for Rabi (B_R) = 4-weeks = 28 days

Kor depth or Delta for Rabi (Δ_R) = 12cm = 0.12m

$\therefore$ Duty for Rabi $(D_R) = \frac{8.64 \times B_R}{\Delta_R} = \frac{8.64 \times 28}{0.12}$

$$\boxed{D_R = 2016\ ha/cumecs}$$

Given, for kharif

B_K = 2.5 weeks = 17.5 day

Δ_K = 25cm = 0.25m

$\therefore$ Duty for Kharif $(D_K) = \frac{8.64 \times B_K}{\Delta_K} = \frac{8.64 \times 17.5}{0.25}$

$$\boxed{D_K = 604.8\ ha/cumecs}$$

Now,

Discharge $(Q) = \frac{A}{D}$

for Rabi

$$Q_R = \frac{A_R}{D_R} = \frac{850}{2016}$$

$$\boxed{Q_R = 0.42\ cumecs}$$

for kharif

$$Q_K = \frac{A_K}{D_K} = \frac{510}{604.8} = 0.84$$

$\boxed{Q_k = 0.84 \text{ cumecs}}$ //.

∴ The outlet discharge = Q_k = 0.84 cumecs (ie maxm value take)

// Ans //.

(3)

The GCA for a distributary is 6000 hectares, 80% of which is culturable irrigable. The intensity of irrigation for Rabi season is 50% & that for kharif season is 25%. If the average duty at the head of the distributary is 2000 ha/cumec for Rabi season and 900 ha/cumec for kharif season, find out the discharge reqd at the of distributary from average demand considerations.

Soln.

Given,

$GCA = 6000 \text{ ha}$

$CCA = 80\%$ of GCA

$= \frac{80}{100} \times 6000$

$\boxed{CCA = 4800 \text{ ha}}$

Area to be irrigated in Rabi season,

intensity of irrigation for Rabi = 50%

$A_r = CCA \times$ intensity of irrigation

$= 4800 \times \frac{50}{100}$

$\boxed{A_r = 2400 \text{ ha}}$

Area to be irrigated in kharif season

intensity for kharif = 25%

$A_k = CCA \times$ intensity

$= 4800 \times \frac{25}{100}$

$\boxed{A_k = 1200 \text{ ha}}$

Given,

Duty for Rabi season $(D_r) = 2000$ ha/cumecs

& " " Kharif " $(D_K) = 900$ ha/cumecs

Now,

for Rabi

$$Q_R = \frac{A_R}{D_R} = \frac{2400}{2000}$$

$$\boxed{Q_R = 1.20 \text{ cumecs}}$$ //

for Kharif

$$Q_K = \frac{A_K}{D_K} = \frac{1200}{900}$$

$$\boxed{Q_K = 1.33 \text{ cumecs}}$$ //

$\therefore$ Required Discharge $= Q_K = 1.33$ cumecs //

(4) Determine the discharge required at the head of the distributary in above questn for fulfilling maximum crop requirement. Assume suitable values for kor depth and kor-period.

Soln,

Assume B { Kor period for Rabi = 4 weeks = 28 day (7×4) day
Kor period for Kharif = 2.5 weeks = 7×2.5 = 17.5 day

Δ { Kor depth for Rabi = 13.5cm (wheat)
" " for kharif = 19cm (Rice)

Now,

$$D_R = \frac{864\times 28}{13.5} = 1792 \text{ ha/cumecs}$$

And,

$$D_K = \frac{864\times 17.5}{19} = 796 \text{ ha/cumecs}$$

from above questn,

$A_R = 2400$ ha

$A_K = 1200$ ha

Now,

$$Q_R = \frac{2400}{1792} = 1.34 \text{ cumecs}$$

$$\boxed{Q_R = 1.34 \text{ cumecs}}$$

And,

$$Q_K = \frac{A_K}{D_K} = \frac{1200}{796} = 1.50$$

$$\boxed{Q_K = 1.50 \text{ Cumecs}}$$

$\therefore$ Required discharge $= Q_K = 1.50$ cumecs // Ans.

Here,

Required Kor demand has increased from 1.33 to 1.50 cumecs ie increase of 12.78%.

$$\Rightarrow \frac{1.50 - 1.33}{1.33} \times 100$$

$$= 12.78\% \text{ // Ans.}$$

2072/05/12

Design Capacity

Design Capacity should be fixed by Considering following parts.

① Different types of crops that would be Sown in any season should be known. (रोपने)
② Peak rate of water requirements of all crops in each season of years should be known.
③ Capacity of Canal should be sufficient to fulfill the maximum of peak demand of all crops.
④ Canal should be designed for non-scouring Velocity (खोतल्ने)

Manning's Uniform flow equation

A/c to Manning's, Velocity of Flow is given by,

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

V = Velocity of flow
n = manning's Rugosity Constant

R = hydraulic radius $\left(\frac{A}{P}\right) \rightarrow \frac{\text{Area}}{\text{wetted perimeter}}$

S = longitudnal slope of canal

$$\therefore Q = AV$$

$$\boxed{Q = \frac{1}{n} A R^{2/3} S^{1/2}}$$

For Value of 'n'

① Manning's n is given by strickler's formula

$$\boxed{n = \frac{d^{1/6}}{24}}$$

d = size of particle in meters.

- Value of n depends upon Roughness of Canal Surface.

- Greater the roughness, greater will be Value of n & hence lesser will be Velocity through such surfaces.

② Chezy's formula

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

C = Chezy's coefficient obtained by Kutter

To find C

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

n = Rugosity constant = Kutter Co

S = slope

Semi theoretical Approach of Canal Design

Whenever wtr flows from canal (or channel), it tries to scour its surface. S... or gravel are detached from bed of canal and are remvd downstream of canal. This phenomena is called **Sediment transport** in canal.

Types of sediment - 2 types

① Bed load sediment

② Suspended load sediment

① Bed load sediment :- The deposited sediment on the bed material in laye... where suspension is impossible is called bed load.

② Suspended load sediment :- Suspended loads are one in which the mtrls is maintained in suspension due to the turbulence of flowing water.

Procedure of sediment transport

① When average shear stress (τ_o) on the bed of an alluvial channel exceeds the critica shear stress (τ_c), the sediment particle starts mvng (ie if $\tau_o >> \tau_c$)

② A relatively low shear stress (τ_o) ie average shear stress just exceeds the critical shear (τ_c), the particle on the bed start to roll, slide along the bed remai-ning in contact with bed. This is called **Contact load**.

③ On increasing shear stress some sediment loose contact with the bed for sometimes with bounce. This type of sediment are called **Saltation load**.

④ It is difficult to distinguish clearly contact load and saltatn load. Hence both combine 2gether (contact load + saltatn load = Bed load) & form **Bed load**.

⑤ The addtnl load from the erosion in catchment is called **Wash load**.

VVI only in 2077

Mechanics of Sediment transport

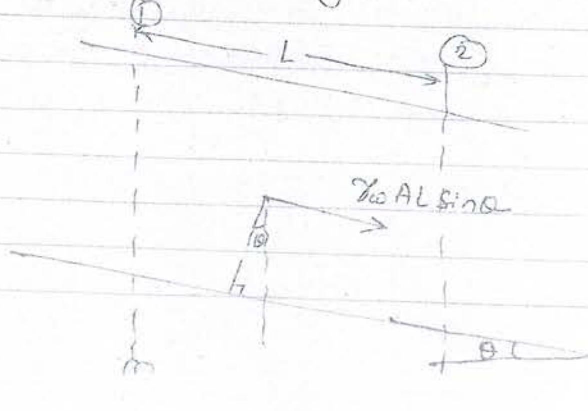
for mechanics of sediment transprt, Assume soil as an incoherent soil
ie, $C=0$ (c = cohesion)

eg. gravels, sand etc.

- By assuming soil as an incoherent each soil grain can be studied individually.
- The basic mechanism of sediment transprt is drag force in the dirxn of flow.

Drag force / Tractive force / shearforce

Drag force is nothing but pull of sediment particles by the flow of water in the dirxn of flow is known as Drag force.



let us Consider a channel sxn of length 'L' & Cross-sxnl Area A

Volume of water stored = $A \cdot L$

Wt. of water stored = $\gamma_w \cdot AL$

Horizontal Component of this wt. = $\gamma_w A L \sin\theta$

$= \gamma_w A L S$ (s = slope of bed)

i.e.,

for small θ

$\sin\theta \simeq \tan\theta \simeq S$

The horizontal force exerted by water is Tractive force

$\therefore$ Average tractive force per Unit area = $\tau_o = \dfrac{\text{Tractive force}}{\text{wetted Area}}$

$$\tau_o = \frac{\gamma_w A \cancel{L} S}{P \times \cancel{L}}$$

$$= \gamma_w \frac{A}{P} \cdot S \qquad (R = A/P)$$

$$\boxed{\tau_o = \gamma_w R S}$$ //

Incipient Motion (Threshold Motion)

when the Velocity of flow through a channel is very small, the channel bed matrl do not move at all and the channel behaves as a **Rigid boundary channel**. As the Velocity increases steadily a stage is reached when the shear force (τ) exerted by flowing water the bed particle will just exceed the force opposing their movement.

At this stage few particles on the bed will just start mvng. This Condtn is known as Incipient motion.

Design Approach

The movement of sediment at bed is caused by a frce exerted on grains by the following water. This force is known as tractive force and is equal to component of weight of water in the dirxn of flow.

for Uniform flow, median tractive force per unit Area.

$$\boxed{\tau_o = \gamma_w R S}$$

Critical tractive Stress (τ_c) :- It is defined as maximum tractive stress that will not cause movement of material forming the channel bed.

↳ when tractive stress (τ_o) exceeds critical tractive stress (τ_c) the particle starts mv

Shield equation

According to shield, the critical tractive stress (τ_c) is proportional to grain diamter (d) and the sub-merged unit wt. of sediment and is given by,

$$\boxed{\tau_c = 0.056 \cdot \gamma_w \cdot d \, (G_s - 1)}$$

γ_w = Unit wt of water

d = Diamtr of grain (m)

G_s = specific gravity of sediment particle

↳ For no movement of bed particles:

Average tractive force stress (τ_o) ≤ critical tractive force stress (τ_c)

$$\tau_o \le \tau_c$$

$$\cancel{\gamma_w} R S \le 0.056 \cancel{\gamma_w} \cdot d \, (G_s - 1)$$

$$R S \le 0.056 d \, (2.65 - 1) \qquad (G = 2.65)$$

$$R S \le \frac{d}{11}$$

$$\boxed{d \ge 11 R S}$$

// This gives the minimum size of bed mtrl that remains in Rest.

- The critical tractive stress on a sloping face is less than than on levelled Su

$$\boxed{\frac{\tau_s}{\tau_L} = \sqrt{\frac{1-\sin^2\theta}{1-\sin^2\phi}} = k} \text{ ——— (1)}$$

τ_s = critical shear stress on slope

τ_L = " " " on levelled grnd

k = tractive force Ratio $\qquad k = 0$ to 1

ϕ = ∠le of internal frxn of bed mtrl

θ = ∠le of sloping surface

The above eqn shows that,

$$\boxed{\tau_s \le \tau_L}$$

which shows that shear stress Reqd to move a grain on side slope is less than shear stress reqd to move grain on levelled surface.

Note

$$\boxed{\begin{array}{l} \text{On bed: } \tau_0 = \gamma_w R S \\ \text{On slope: } \tau_0 = 0.75\,\gamma_w R S \end{array}}$$

$\rightarrow$ V:H

$\rightarrow$ side slop $\rightarrow$ H:V

Numericals

(V:H)

(1) Design a Canal to carry discharge of 50 cumec. The slope of canal is 1 in 5000. The soil in coarse alluvium with lung grain size of 6cm. Tractive shear stress not to exceed 2.35 N/m². Side slope = $\frac{1}{2}$: 1 $\rightarrow$ 1 : 2

(H : V)

soln.

Given,

Discharge (Q) = 50 m³/s = 50 cumec

Slope (S) = $\frac{1}{5000}$

Diamtr (d) = 6cm = 0.06m

we know. By manning's formula,

$$\boxed{V = \frac{1}{n} R^{2/3} s^{1/2}} \quad \text{——— (1)}$$

from strickler's formula

$$n = \frac{d^{1/6}}{24} = \frac{(0.06)^{1/6}}{24}$$

$$\boxed{n = 0.0260}$$

Tractive stress on bed level $(\tau_c) = 2.35 \, N/m^2$

$$\tau = \tau_c = \gamma_w R s \qquad (\because \gamma_w = 9810 \, N/m^3)$$

$$2.35 = 9810 \times R \cdot \frac{1}{5000}$$

$$\boxed{R = 1.198 m}$$

Then, from eqn (1)

$$V = \frac{1}{0.0260} \times (1.198)^{2/3} \cdot \left(\frac{1}{5000}\right)^{1/2}$$

$$\boxed{V = 0.61 \, m/s}$$

Now,

$$Area = \frac{Q}{V} = \frac{50}{0.61}$$

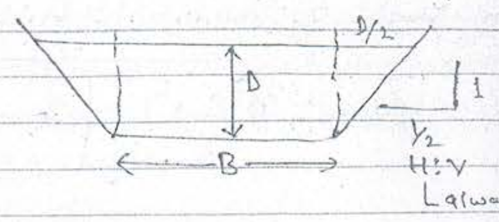
$$\boxed{A = 81.96 m^2}$$

Then,

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

$$P = \frac{A}{R} = \frac{81.96}{1.198} = 68.41 m$$

$\boxed{P = 68.41\,m}$ //Ans.



Trapezoidal

Now,

$$Area\,(A) = BD + 0.5D^2$$

$$81.96 = BD + 0.5D^2 \quad \text{——— (1)}$$

Again,

$$Perimeter\,(P) = B + D\sqrt{5}$$

$$68.41 = B + D\sqrt{5} \quad \text{——— (2)}$$

Solving (1) & (2)

from eqn (2)

$$B = 68.41 - D\sqrt{5}$$

Then eqn (1) becomes

$$(68.41 - D\sqrt{5})D + 0.5D^2 = 81.96$$

$$68.41D - 2.23D^2 + 0.5D^2 = 81.96$$

$$-1.73D^2 + 68.41D - 81.96 = 0$$

$$\therefore D = 38.30 \text{ \& } 1.237$$

$\therefore \boxed{D = 1.237}$ //Ans. (ie take always small value)

Putting value in eqn (2) we get,

$$B = 68.41 - 1.23 \times \sqrt{5}$$

$\boxed{B = 65.65\,m}$ //Ans.

Design of stable channel in alluvium

If the average shear stress (τ_o) is actng on the boundary of alluvial channel is less than critical shear stress (τ_c) then the channel can be assumed as rigid boundar and resistance equatn by manning's, chezy's will be applicable on such channel.

However as soon as the sediment movemnt starts undulatn (wavement) devlop on the bed which increases boundary resistance of channel besides this some energy is required to move the grain. Suspended loads carry due to turbulence in the flow further affects the resistance of the alluvial stream all these factors render the evaluatn of resistance of alluvial streams to be very much complex problem and the complexity further Ies if one includes the effect of channel shape, non-uniformity of sediment size, discharge variation and other such factors known of the resistance equatn developed so far takes all these factor into account. The direct accurate mathematical solutn to the design of canal in alluvial soil is there4R not an easy Job of hence in india Alluvial channel are design on the basis of hypothetical theory given by **Kennedy** & **Lacy**. This theories are based on Experiment & experience gained on the existing channel over passed many years.

Design procedures

Given, - Q,

- maximum permissible velocity (V)
- manning's roughness coefficient 'n'
- Bed slope : 's'
- Side slope : Z : 1 (H : V)

$$\underset{H}{Z} : \underset{V}{1}$$

Steps

① Find Catchment area (A)

$$Q = A.V$$

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

② Determine R (hydraulic Radius) from Manning's eqn.

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

$$\boxed{R = \left(\frac{V * n}{s^{1/2}}\right)^{3/2}}$$

Bed slope → V : H
Side slope → H : V

(3) Determine wetted perimeter (P)

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

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

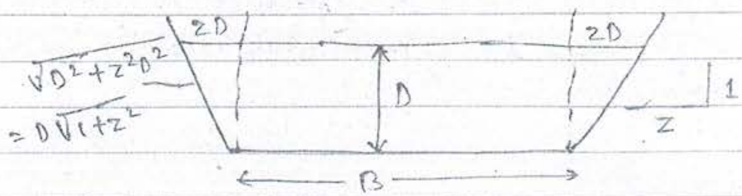
(4) Determine Depth (D) & width (B) from value of A & P by solving.

$$\boxed{A = BD + ZD^2}$$

$$\boxed{P = B + 2D\sqrt{1+Z^2}}$$

Z = horizontal slope
1 = V = Vertical slope
ie (Z : 1) ⇒ H = V ⇒ H : V

(5) figure

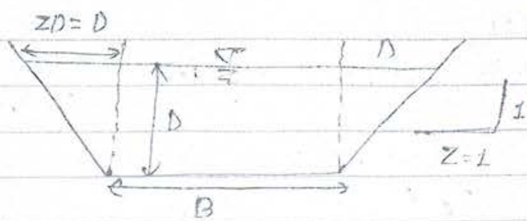


Numericals

(2) Design an irrigatal Canal in a non-erodible materials to carry a discharge $15 m^3/sec$ when maximum permissible velocity is 0.8 m/s. bed slope 1 : 4000 & side slope 1 : 1 & n = 0.025

Soln.

Given,
$Q = 15 m^3/sec$
$V = 0.8 m/s$
Bed slope (V : H) = 1 : 4000
Side slope (H : V) = 1 : 1
n = 0.025



Trapezoidal channel

For design find remaining parameters
A = ?
R = ?
P = ?
B = ?

we know,

$$Q = A \cdot V$$
$$15 = A \times 0.8$$
$$\boxed{A = 18.75 m^2}$$

Now by Manning's formula,

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$
$$0.8 = \frac{1}{0.025} \times R^{2/3} \times \left(\frac{1}{4000}\right)^{1/2}$$
$$\boxed{R = 1.42 m}$$

Now,

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

R = hydraulic Radius

P = wetted perimeter

$$P = \frac{18.75}{1.42}$$
$$\therefore \boxed{P = 13.20 m}$$

Again,

$$A = BD + ZD^2$$

$Z = 1$

$$\therefore \boxed{18.75 = BD + D^2} \quad \text{———— (a)}$$

and,

$$P = B + 2D\sqrt{1+Z^2}$$
$$= B + 2D\sqrt{2}$$
$$= B + D\,2\sqrt{2}$$
$$\boxed{13.20 = B + 2.828 D} \quad \text{———— (b)}$$

from eqn (b)

$$B = 13.20 - 2.828 D \quad \text{———— (c)}$$

Now,

$$(13.20 - 2.82D)\,D + D^2 = 18.75$$

Solving we get,

$$D = 5.277 \text{ or } 1.943$$

$$\therefore \boxed{D = 1.943\,m} \qquad \text{(take small value always)}$$

Putting value we get,

$$\boxed{B = 7.69\,m}$$

Adopt free Board : 0.6m

$$\therefore \text{Total depth } (d) = D + \text{free board}$$
$$= 1.943 + 0.6$$
$$= 2.55\,m \text{ Ans}$$

Silt theory of Kennedy & Lacy & their Comparison

If the velocity of flow in the Canal is more, the bed & their bank are likely to be **eroded**. Similarly if velocity is less **Silting** may occur. Therefore alluvial Cana should be design such that neither scouring nor silting occurs. The velocity at which neither scouring nor silting occurs is called **Critical Velocity**. Such Canal are known as **Stable Canal OR Regime Canal**.

Silt theory of Kennedy

Silt theory is proposed by **Kennedy**, an executive engineer of Irrigatn department from the experiment & observatn of 30yrs, he Concluded following:-

- The silt Supporting power in a Canal Cross-sxn is mainly dependent upon generatn of eddies rising to the surface.
- This eddies are generated due to friction of flowing water with Canal Surface.

Critical velocity Concept

The vertical Component of this eddies try to move this Sediment up but while the weight of sediment try to bring it down. Thus keeping the sediment in Suspension. Based on Concept, he defines **Critical velocity** (V_0) as the mean Velocity which will jus keep the Canal free from Silting & scouring.

Based on observatn, Critical Velocity is given by,

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

y = depth (ie water depth in Canal)
V_0 = Critical velocity.

C_1 & C_2 = Constant & value is

$C_1 = 0.55$

$C_2 = 0.64$

$$\therefore \boxed{V_0 = 0.55 y^{0.64}}$$

This eqn is not valid for all type of soil & he introduced a factor "m" which depen upon silt Grade & named as critical velocity ratio ie, $\boxed{V_o = m\, 0.55\, y^{0.64}}$ // is eq^d eq^n.

m = critical Velocity Ratio (CVR) $= \frac{V}{V_o}$

Design procedure for kennedy

Case-I :

Q, N, m & bed slope (S) given

N = manning's Constant
m = critical velocity ratio

Steps:

(1) Assume trial value of y (depth) (y = depth)

(2) Calculate velocity

$$\boxed{V_o = 0.55\, m\, y^{0.64}}$$

(3) Calculate cross-sectional Area (A)

$$\boxed{A = Q/V_o}$$

(4) Assume side slope : 0.5 : 1 ($\frac{1}{2}$: 1) $\Rightarrow$ 1 : 2
H : V H : V

(5) Find 'B'

$$A = BD + 0.5D^2$$

$$\boxed{B = \frac{A - 0.5D^2}{D}}$$

(D = y = assumed value in step-1)

(6) Find wetted perimeter (P) for side slope 2 : 1
V : H

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

(7) Find hydraulic Radius (R)

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

⑧ Calculate Actual mean Velocity & Compare with previous Velocity (at step-2)

(a) if N = kutter's Constant

$$V = \frac{\left\{\frac{1}{N} + \left(23 + \frac{0.00155}{S}\right)\right\}\sqrt{RS}}{1 + \left(23 + \frac{0.00155}{S}\right)\frac{N}{\sqrt{R}}}$$

(b) if N = Manning's Constant

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

(c) if N = chezy's Constant

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

if V in step-⑧ is equal to V_0 in step-②. Assumed depth (y) is ok//.

ie,

if $\boxed{V = V_0}$ then y = ok//.

And

if not procedure is Repeted Such that both velocity become nearly equal.

ie,

if $\boxed{V_0 < V}$ then greater depth (y) is assumed

Numericals

① Given,

$Q = 45 m^3/s$

N = 0.0225 (Kutters Constant

m = 1

$S = 0.16$ per km $= \frac{0.16}{1000} = \frac{Y}{H}$

Generally y is taken from 1 to 2

If for y value V_0 is small then value of y is ↑ed.

(1) Take trial depth (y)

$$\boxed{y = 1.8m}$$

(2) $V_0 = 0.55 m y^{0.64}$

$= 0.55 \times 1 \times 1.8^{0.64}$

$$\boxed{V_0 = 0.801 m/s}$$

(3) $A = \frac{Q}{V_0} = \frac{45}{0.801}$

$$\boxed{A = 56.17 m^2}$$

(4) let, Side slope = 0.5 : 1
H : V

(5)

$A = BD + 0.5D^2$

$B = \frac{56.17 - 0.5 \times 1.8^2}{1.8}$ $\{ D = y = $ depth $= 1.8m$ (ie assum

$$\boxed{B = 30.31 m}$$

(6) wetted perimeter (P)

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

$= 30.31 + 1.8\sqrt{5}$

$$\boxed{P = 34.32 m}$$

(7) Hydraulic Radius (R)

$R = \frac{A}{P} = \frac{56.17}{34.32}$

$$\boxed{R = 1.64 m}$$

(8) Actual mean velocity (V)

$$V = C\sqrt{RS} \qquad \left(\because S = \frac{0.16}{1000}\right)$$

where, $N = 0.0225$

$$C = \frac{\frac{1}{N} + \left(23 + \frac{0.00155}{S}\right)}{1 + \left(23 + \frac{0.00155}{S}\right)\frac{N}{\sqrt{R}}} = \frac{\frac{1}{0.0225} + \left(23 + \frac{0.00155}{0.16/1000}\right)}{1 + \left(23 + \frac{0.00155}{\frac{0.16}{1000}}\right)\cdot\frac{0.0225}{\sqrt{1.64}}}$$

$C = 48.99$

$\boxed{C \simeq 49}$

then,

$$V = C\sqrt{R \cdot S}$$
$$= C\sqrt{RS} = 49\sqrt{1.64 \times \frac{0.16}{1000}}$$

$\boxed{V = 0.792\,m/s}$

i.e,

$\boxed{V \simeq V_0 \simeq 0.80\,m/s}$

i.e OK//

So, Assumed depth $y = 1.8m$ is OK//.

or, $\boxed{D = 1.8m}$ //

Case-II : Q, N (kutters), m and $\frac{B}{D}$ ratio Given,

steps

(1) Calculate Area (A) in terms of D

$$A = BD + 0.5D^2 = D^2(B/D + 0.5)$$
$$A = D^2(x + 0.5) \qquad \text{where, } x = \frac{B}{D}$$

(2) write Continuity equation and substitute Kennedy equation for Velocity

$$Q = AV \qquad\qquad A = D^2(x+0.5)$$
$$= D^2(x+0.5) \times 0.55\,m\,D^{0.64} \qquad\qquad V = 0.55\,m\,D^{0.64}$$

$$Q = 0.55m(x+0.5)D^{2.64}$$

$$D = \left(\frac{Q}{0.55m(x+0.5)}\right)^{1/2.64}$$

(3) Calculate B

$$\boxed{B = xD}$$

(4) Compute hydraulic Radius (R)

$$R = \frac{A}{P} = \frac{BD + 0.5D^2}{B + D\sqrt{5}}$$

(5) Compute velocity from (V)

$$V = 0.55mD^{0.64}$$

(6) Compute slope from Kutter's OR Manning's eqn

for Kutter's,

$$V = \frac{\frac{1}{N} + \left(23 + \frac{0.00155}{S}\right)\sqrt{RS}}{1 + \left(23 + \frac{0.00155}{S}\right) \cdot \frac{N}{\sqrt{R}}}$$

('S' will come from quadratic)

Find 'S' from trial & Error process

Numerical

(1) Given,

$N = 0.0225$ (Kutter)

$m = 1$

$\frac{B}{D} = 5.7$

$Q = 14 m^3/s$ or 14 cumecs

Find $S = ?$

S/n.

① Area $(A) = BD + 0.5D^2 = D^2\left(\frac{B}{D} + 0.5\right)$ $\therefore \frac{B}{D} = 5.7$

$= D^2(5.7 + 0.5)$

$$\boxed{A = 6.2D^2}$$

② $Q = A \cdot V$ $Q = 14 m^3/s$

$14 = 6.2D^2 \times 0.55 m D^{0.64}$ $m = 1$

$14 = 6.2D^2 \times 0.55 \times 1 \times D^{0.64}$

$\frac{14}{6.2 \times 0.55} = D^{2+0.64}$

$D = \left(\frac{14}{6.2 \times 0.55}\right)^{1/2.64}$

$$\boxed{D = 1.70 m}$$

③ $\frac{B}{D} = 5.7$ $x = \frac{B}{D}$

$B = 5.7 \times D$

$= 5.7 \times 1.70$

$$\boxed{B = 9.7 m}$$

④ $R = \frac{A}{P} = \frac{BD + 0.5D^2}{B + D\sqrt{5}} = \frac{9.7 \times 1.7 + 0.5 \times 1.7^2}{9.7 + 1.7\sqrt{5}}$

$$\boxed{R = 1.32 m}$$

⑤ $V = 0.55 m D^{0.64}$

$= 0.55 \times 1 \times 1.7^{0.64}$

$$\boxed{V = 0.772 m/s}$$

⑥ for Kutters,

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

$$0.772 = \left\{ \frac{\frac{1}{0.0225} + \left(23 + \frac{0.00155}{s}\right)}{1 + \left(23 + \frac{0.00155}{s}\right) \cdot \frac{0.0225}{\sqrt{1.32}}} \right\} \cdot \sqrt{1.32 \times s}$$

$$s = 2.05 \times 10^{-4} \simeq 2 \times 10^{-4}$$

$$= \frac{\cancel{2}.00}{\cancel{1}0000}$$

$$\boxed{s = \frac{1}{5000}}$$

Drawbacks in Kennedy's Theorem

1. Kennedy didnot notice the importance of $\frac{B}{D}$ ratio
2. No account for silt Concentratal & bed load.
3. Silt grade and silt charge were not defined
4. Kennedy did not give any slope equatal
5. Kennedy uses Kutters formula so Kutters drawbacks are also taken.

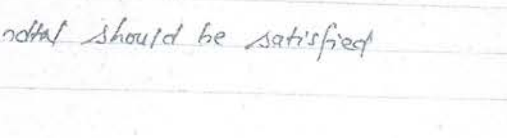
LACEY'S Silt Theory / Lacey Regime Theory

Rigid (नियमित)

Kennedy say's that Canal will be under the regime Canal if there is neither silting non-scouring.

But Lacey's states that even a channel showing no silting and No-scouring may actually not be in Regime. He therefore differentiated betn 3-Regime Condtals

1. True Regime
2. Initial Regime
3. Final Regime



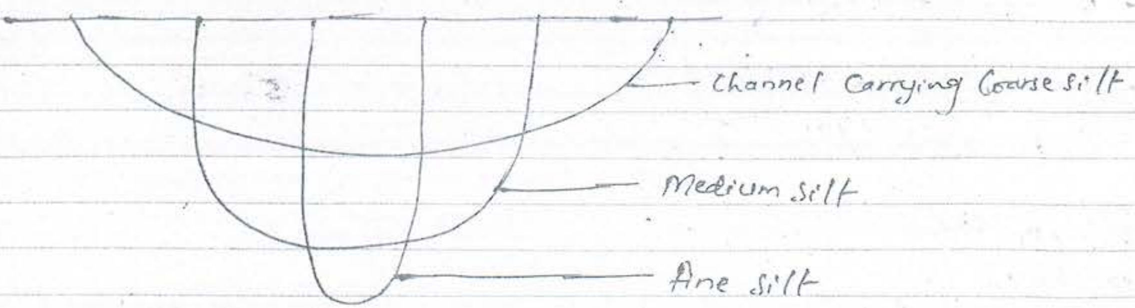
- Lacey's theory is applicable only in true regime & final regime.

1. True Regime : For true regime following Condtal should be satisfied
- Discharge is Constant
- Flow should be Uniform

- Silt charge is Constant
- Silt grade is Constant
- Channel should not be flowing through a matrial which Can be scoured as easily as it Can be deposit.
- But in practise, all these Conditn Can **Never** be satisfied and there4R artificial channels Can never be in True regime, they Can either be in initial or in final regime.
ie **True Regime is Not possible in Real**

VVE
(2) Initial Regime :- If the cross-sxn remains Unchanged and only the bed slope of Canal Varies due to silting, then the Canal is said to be **initial Regime**.

VVE
(3) final Regime :- It is an ultimate state of Regime attempt by an alluvial channel when in additn to bed slope and depth, width of Canal has also be adjusted.
- More or less a permanent stability is achieved which is known as **Final Regime**
- final Regime Canal has a 10dency to assume a **Semi-elliptical** Sxn.



four Basic equatn Based on Observatn
(1) Silt factor(f)

$$\boxed{f = 1.76\sqrt{d}} \quad \text{———} \quad (1) \quad d \text{ in mm}$$

(2) Relatn between Mean Velocity(V) & Hydraulic Radius(R) (V & R)

$$\boxed{V = \sqrt{\frac{2}{5} fR}} \quad \text{———} \quad (2)$$

(3) Relation between cross-sxn Area(A) and mean velocity(V) (A & V)

$$\boxed{Af^2 = 140V^5} \quad \text{——— (3)}$$

(4) By plotting a large nos of data from different source, Lacey gave,

$$\boxed{V = 10.8 R^{2/3} S^{1/3}} \quad \text{——— (4)}$$

S = slope.

Lacey's Derived equations (Relation beth V, Q, F)

(1) Velocity equation (V)

we have,

$$AF^2 = 140V^5 \qquad \text{(from eqn (3))}$$

multiplying both side by V

$$AVf^2 = 140V^6 \qquad Q = AV$$

$$Qf^2 = 140V^6$$

$$\boxed{V = \left(\frac{Qf^2}{140}\right)^{1/6}} \quad \text{——— (5)}$$

(2) Perimeter (P)

we have,

$$V^2 = \frac{2}{5} fR \qquad \text{(from eqn (2))}$$

Squaring we get

$$V^4 = \frac{4}{25} f^2 R^2$$

Also,

$$F^2 = \frac{140V^5}{A} \qquad \text{from eqn (3)}$$

Putting value of f we get,

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

$$\frac{25}{4}\left(\frac{AV}{R^2}\right) \cdot \cancel{V^3} = \cancel{V^5}^{\,2} \cdot 140$$

$$\frac{25}{4}\left(\frac{AV}{R^2}\right) = 140 V^2 \qquad R = \frac{A}{P}$$

$$\frac{25}{4}\left(\frac{AV}{A^2/P^2}\right) = 140 V^2$$

$$\frac{25}{4}\left(\frac{Q}{A^2/P^2}\right) = 140 V^2$$

$$\frac{P^2 Q}{A^2} = \frac{140 \times 4}{25} V^2$$

$$P^2 Q = 22.4 V^2 A^2$$

$$P^2 \cancel{Q} = 22.4\, Q^{\cancel{2}}$$

$$\boxed{P = 4.75 \sqrt{Q}} \quad \text{———————} \quad (6)$$

(3) Hydraulic Radius eqn ('R')

we have,

$$V^2 = \frac{2}{5} f R \qquad \text{from eqn (2)}$$

$$R = \frac{5}{2} \frac{V^2}{f}$$

$$R = \frac{5}{2f}\left(\frac{Qf^2}{140}\right)^{1/3} \qquad \because V^2 = \left(\frac{Qf^2}{140}\right)^{1/3} \text{ from eqn (5)}$$

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

Generally taken as, 0.47

$$\boxed{R = 0.47 \left(\frac{Q}{f}\right)^{1/3}} \quad \text{———————} \quad (7)$$

④ Slope equation (S)
we have

$$V = 10.8\, R^{2/3}\, S^{1/3}$$ from eqn ④

Cubing both side,

$$V^3 = 1260 \cdot R^2 \cdot S \quad \text{——— (a)}$$

and,

from eqn ① & Cubing,

$$V^3 = \left(\frac{2}{5}\right)^{3/2} f^{3/2}\, R^{3/2} \quad \text{——— (b)}$$

from (a) & (b)

$$1260\, R^2 S = \left(\frac{2}{5}\right)^{3/2} \cdot f^{3/2} \cdot R^{3/2}$$

$$\boxed{S = \frac{f^{3/2}}{4980\, R^{1/2}}} \quad \text{——— (8)}$$

"OR" Alternative equation in terms of Q.
we have,

$$V = \sqrt{\frac{2}{5} fR} \quad \text{——— (c)}$$ from ②

and,

$$V = \left(\frac{Qf^2}{140}\right)^{1/6} \quad \text{——— (d)}$$ from ③

equating eqn (c) & (d)

$$\left(\frac{2}{5} fR\right)^{1/2} = \left(\frac{Qf^2}{140}\right)^{1/6}$$

$$R^{1/2} = \left(\frac{5}{2f}\right)^{1/2} \cdot \left(\frac{Qf^2}{140}\right)^{1/6}$$

$$\boxed{R^{1/2} = 0.69\left(\frac{Q}{f}\right)^{1/6}} \quad \text{——————— (e)}$$

Also,

$$R^{1/2} = \frac{f^{3/2}}{4980 \times S} \quad \text{——————— (f)} \qquad \text{from eqn (8)}$$

from eqn (e) & (f)

$$0.69\left(\frac{Q}{f}\right)^{1/6} = \frac{f^{3/2}}{4980 \times S} \qquad \therefore S = \frac{f^{5/3}}{3455\, Q^{1/6}}$$

But in denominater 3340 is taken, instead of 3455

$$\boxed{S = \frac{f^{5/3}}{3340\, Q^{1/6}}} \quad \text{——————— (9)}$$

Design Procedure by Lacey Regime approach

Given: Q & f or d

Steps

(1) Determine Velocity (V)

$$V = \left(\frac{Qf^2}{140}\right)^{1/6}$$

(2) Calculate (A)

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

(3) Compute wetted perimeter (P)

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

(4) Known A & P; Determine B and D for (0.5 : 1)

$$A = BD + 0.5D^2$$

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

⑤ Determine bed slope (s)

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

⑥ Check : Compute R from B & D

$$R = \frac{A}{P} = \frac{BD + 0.5D^2}{B + D\sqrt{5}}$$

Compare with,

$$R = \frac{5}{2}\frac{V^2}{f}$$

They must be approximately equal.

Numericals

① Design a Regime channel for a discharge of 30 cumecs and silt factor 1 using Lacey's theory.

Soln. Given,

$Q = 30$ cumecs

Silt factor $(f) = 1$

Note: if f is not given than d is given then find 1st

$d' = f = 0.76\sqrt{d}$ mm

① $V = \left(\frac{Qf^2}{140}\right)^{1/6} = \left(\frac{30 \times 1^2}{140}\right)^{1/6}$

$\boxed{V = 0.778 \text{ m/s}}$

② $A = \frac{Q}{V} = \frac{30}{0.773}$

$\boxed{A = 38.78 \text{ m}^2}$

③ $P = 4.75\sqrt{Q}$

$= 4.75\sqrt{30}$

$\boxed{P = 26.01 \text{ m}}$

$$\boxed{R^{1/2} = 0.69\left(\frac{Q}{f}\right)^{1/6}} \quad \text{———— (e)}$$

Also,

$$R^{1/2} = \frac{f^{3/2}}{4980 \times S} \quad \text{———— (f)} \qquad \text{from eqn (8)}$$

from eqn (e) & (f)

$$0.69\left(\frac{Q}{f}\right)^{1/6} = \frac{f^{3/2}}{4980 \times S} \qquad \therefore S = \frac{f^{5/3}}{3455\, Q^{1/6}}$$

But in denominater 3340 is taken instead of 3455

$$\boxed{S = \frac{f^{5/3}}{3340\, Q^{1/6}}} \quad \text{———— (9)}$$

Design Procedure by Lacey Regime approach

Given: Q & f or d

Steps

(1) Determine Velocity (V)

$$V = \left(\frac{Qf^2}{140}\right)^{1/6}$$

(2) Calculate (A)

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

(3) Compute wetted perimeter (P)

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

(4) Known A & P: Determine B and D for (0.5 : 1)

$$A = BD + 0.5D^2$$

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

⑤ Determine bed slope (S)

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

⑥ Check : Compute R from B & D

$$R = \frac{A}{P} = \frac{BD + 0.5D^2}{B + D\sqrt{5}}$$

Compare with,

$$R = \frac{5}{2}\frac{V^2}{f}$$

They must be approximately equal.

Numericals

① Design a Regime channel for a discharge of 30 cumecs and silt factor 1 using Lacey's theory.

Soln. Given,

$Q = 30$ cumecs

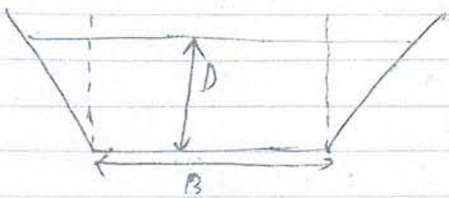
Silt factor $(f) = 1$

Note: if f is not given than d is given then find 1st

$d' = f = 0.76\sqrt{d_{mm}}$

① $V = \left(\frac{Qf^2}{140}\right)^{1/6} = \left(\frac{30 \times 1^2}{140}\right)^{1/6}$

$\boxed{V = 0.778 m/s}$



② $A = \frac{Q}{V} = \frac{30}{0.773}$

$\boxed{A = 38.78 m^2}$

③ $P = 4.75\sqrt{Q}$

$= 4.75\sqrt{30}$

$\boxed{P = 26.01 m}$

④ $A = BD + 0.5D^2$

$38.78 = BD + 0.5D^2$ ———— ⓐ

and,

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

$26.01 = B + 2.23D$

$B = 26.01 - 2.23D$ ———— ⓑ

Putting value of ⓑ in eqn ⓐ

$38.78 = (26.01 - 2.23D)D + 0.5D^2$

$38.78 = 26.01D - 2.23D^2 + 0.5D^2$

$1.73D^2 - 26.01D + 38.78 = 0$

$D = 13.35,\ 1.67$

Adopt, $\boxed{D = 1.67m}$ (ie take small value)

Then,

$B = 26.01 - 2.23 \times 1.67$

$\boxed{B = 22.28m}$

⑤ $S = \dfrac{f^{5/3}}{3340\ Q^{1/6}} = \dfrac{1^{5/3}}{3340 \times 30^{1/6}} = \dfrac{1}{5887.5} \simeq \dfrac{1}{5900}$

$\boxed{S = 1 : 5887.5}$ OR $\boxed{S = 1 : 5900}$ //Ans//.

⑥ Check,

$R = \dfrac{A}{P} = \dfrac{38.78}{26.01} = 1.49m$

Again, $R = \dfrac{5}{2}\dfrac{V^2}{f} = \dfrac{5 \times 0.773^2}{2 \times 1} = 1.49m$

Hence,

The value of R is same ie $\boxed{R = 1.49m}$ So ok//.

Difference between Kennedy and Lacey

Kennedy Theory	Lacey's Theory
① He introduced a term 'm' but didnot gave idea to measure. $V = 0.55\, m y^{0.64}$	① He introduced Concept of silt factor(f) and determined as, $V = \sqrt{\frac{2}{5} fR}$ where, $f = 0.76\sqrt{d_{mm}}$
② Kutter's formula is used to calculate mean velocity (v) $V = C\sqrt{RS}$ where, $C = \dfrac{\frac{1}{N} + \left(23 + \frac{0.00155}{S}\right)}{1 + \left(23 + \frac{0.00155}{S}\right)\frac{N}{\sqrt{R}}}$	② Lacey gave his own formula. $V = \left(\frac{Qf^2}{140}\right)^{1/6}$
③ No formula for longitudanal slope.	③ Gave formula as, $S = \dfrac{f^{5/3}}{3340\, Q^{1/6}}$
④ Suspension silt rises from bed.	④ Suspension silt rises both from bed & side.
⑤ only gave idea about Regime Channel.	⑤ Shape of Regime should be semi-elliptical.

Defects of Lacey's Theory

① Lacey didnot properly define the silt & silt charge.
② introduced semi-ellipse as ideal shape of regime channel which is not true.
③ In Real artificial channel is not a Regime channel.

Lined Canals, Various types of lining, Advantages & Economics of lining

VVI

Canal lining :- Canal lining is a treatment given to the canal bed and banks so as to make the canal sxn impervious.

- It is the process of applying stable surfaces like Concrete, tiles, asphalt etc. on the earthen surface of canal which is in direct contact with water.

Needs of Canal lining

(1) To minimize seepage losses
(2) To increase discharge by increasing velocity.
(3) To prevent the erosion of bed and side
(4) To Reduce maintenance of Canal
(5) To prevent weeds & grass growth
(6) To improve canal operation.

Various Types of Lining - 3types

(1) Hard Surface lining
- (a) Cement Concrete lining
- (b) Plaster OR Cement mortar lining
- (c) Pre-cast Concrete lining
- (d) Brick or tile lining
- (e) Asphalt lining
- (f) Boulder (Stone Pitching) lining

(2) Earth type lining
- (a) Soil Cement lining
- (b) Compacted earth lining
- (c) Sodium carbonate lining

(3) Buried and protected membrane lining
- (a) Fabricated light membrane lining
- (b) Bene tonite Soil & Clay lining
- (c) Road oil lining.

(1) Hard Surface lining - It is of 6-types

(a) Cement Concrete lining
- Concrete used as lining material
- has excellent hydraulic properties
- durable, highly impermeable

(b) Plaster OR Cement mortar lining
- Cement mortar used for lining
- Usually used as sandwich material beth Brick layers.
- Large cement is consumed and Costly.

(c) Precast Cement lining
- Consists of well prepared precast slabs & laid on well Compacted and prepared sub-grade.

(d) Brick or Tile lining
- Consists of a single or double layer of bricks or tiles or Combination of both.
- lining is efficient as Concrete lining

(e) Asphalt lining
- mxtr of asphalt & graded aggregate are in Asphalt lining.

(f) Boulder lining
- also called stone pitching or dry stone lining.

(2) Earth type Lining - It is of 3 types

(a) Cement type lining
- Mxtr of Cement & Soil
- Placed is subgrade & properly Compacted.

(b) Compacted soil lining
- Fine soil is placed on subgrade & Compacted properly.

(c) Sodium Carbonate lining
- Consists of Clayey soil and sodium Carbonate mxtr.
- used in small Canal.

(3) Buried and Protected Membrane type lining - 3 types
(a) Pre-fabricated light membrane lining
- Fibres of abestos or jute Coated with Asphalt
- laid on smooth & prepared subgrade.

(b) Bentonite soil & Clay membrane
- Consists Bentonite Soil or Clay membranes for lining.

(c) Road oil lining
- Road oil is sprinkled on a Sub-grade and then subgrade is rolled so that oil enters the soil pores.

Advantages of Lining
(1) To Reduce seepage
(2) Prevention of water logging
(3) Increases the Capacity of Canal by increasing velocity
(4) Redxn in maintenance Cost
(5) Reduces weed growth (ii) Decrease in evapotn & transpiratn
(6) Prevents erosion of bed & side materials (i) Higher velocity so decrease siltatn.
(7) increases in Command Area
(8) Eliminatn of flood danger.
(9) ↑es the life of Canal.

Disadvantage
(1) Initial investment is high (5) Difficult to Repair (6) Increase head
(2) Lining being permanent, difficult to shift Canal outlet
(3) Difficult to maintain damaged lining (7) joints always create problem.
(4) Needs high skilled

Economics of lining
Lining is a treatment given to the canal bed and banks so as to make the canal sxn impervious.
- By lining life of Canal is ↑ed.

In Considering the economics of Canal lining it is necessary to evaluate the tangible and addtnl benefits and then to Compare these with Cost of lining.
- $\frac{B}{C}$ (Benefit Cost) Ratio Can be worked out to satisfy necessity of lining.

ie, $\boxed{\frac{B}{C} \gg 1}$

Justificatn of lining

(a) Annual benefits : Irrigatn water is sold to Cultivators at certain rate.

let.

irrigatn water Rate = R_1 per Cumec.

if m cumec of water is saved by lining.

$\therefore$ Annual money saved by lining = mR_1

let,

Annual maintenance Cost for unlined Canal (from pervious records) = R_2

if P is % fraxn Saving in maintenance by lining.

$\therefore$ Total saved = PR_2

$\therefore$ Total amnt profit = $mR_1 + PR_2 = B$

(b) Annual Cost

Total Capital 'C' lining has N life years : I% of interest

$$\therefore \text{Annual Cost} = \frac{C\{I(1+I)^N\}}{(1+I)^N - 1} = c.$$

$$\text{Now, } \frac{B}{c} = \frac{mR_1 + PR_2}{\frac{C\{I(1+I)^N\}}{(1+I)^N-1}}$$

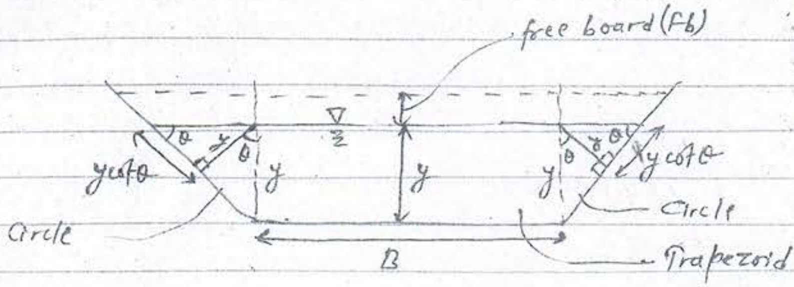
$$\boxed{\frac{B}{c} = \frac{(mR_1 + PR_2)\{(1+I)^N - 1\}}{C\{I(1+I)^N\}}}$$

For feasibility of lining $\frac{B}{c} \gg 1$ //.

Designed of lined Canals
- ① Trapezoidal sxn → $Q >> 55 m^3/sec$
- ② Triangular sxn → $Q \le 55 m^3/sec.$

① Trapezoidal canal sxn ($Q >> 55$ cumecs)

- Adopted for higher discharges



$$Area = B\cdot y + 2\left(\pi y^2 \times \frac{\theta}{2\pi}\right) + 2 \times \left(\frac{1}{2} \times y \times y\cot\theta\right)$$

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

$$\boxed{A = y(B + y\theta + y\cot\theta)} \quad \text{——— ①}$$

$\theta \to$ in Radian

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

$$\boxed{P = B + 2y\theta + 2y\cot\theta} \quad \text{——— ②}$$

Now,

$$\boxed{R = \frac{A}{P} = \frac{y(B + y\theta + y\cot\theta)}{B + 2y\theta + 2y\cot\theta}}$$

If: Q, V, S, n is Given

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

② $A = \frac{Q}{V}$

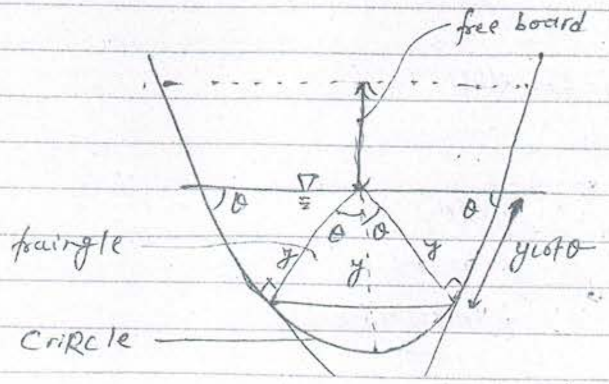
③ $P = A/R$

④ from eqn ① & ② find B & y

⑤ Take free board 0.6 – 0.8m

(2) Traingular Canal sxn ($Q \le 55$ cumecs)

- Used for small discharging & it is best discharging sxn.



$$\text{Area }(A) = \frac{1}{\cancel{2}} \times \cancel{2} \times y\cot\theta \times y + \pi y^2 \times \frac{2\theta}{360^\circ} \qquad (360 = 2\pi)$$

$$= y^2\cot\theta + \cancel{\pi} y^2 \cdot \frac{\cancel{2}\theta}{\cancel{2\pi}}$$

$$\boxed{A = y^2\cot\theta + y^2\theta} \text{ ——— (1)}$$

$$\text{Perimeter }(P) = \cancel{2\pi} y \times \frac{2\theta}{\cancel{2\pi}} + 2y\cot\theta$$

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

$$\boxed{P = 2y(\theta + \cot\theta)} \text{ ——— (2)}$$

Now,

$$R = \frac{A}{P} = \frac{y^2\cot\theta + y^2\theta}{2y(\theta + \cot\theta)} = \frac{y^{\cancel{2}}\cancel{(\theta + \cot\theta)}}{2\cancel{y}\cancel{(\theta + \cot\theta)}}$$

$$\boxed{R = \frac{y}{2}}$$

To find other parameter : use same Method as Trapezoidal sxn method.

Numericals

(1) Design a lined Canals for following data

$Q = 350\ m^3/sec$

Channel slope (s) = 1 in 5000

Side slope $= \underset{H}{1.5} : \underset{V}{1} = \frac{3}{2} : 1$

Roughness Coefficient (n) = 0.014

Limiting velocity of flow (V) = 2 m/sec.

Soln.

we have,

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

$2 = \frac{1}{0.014} \times R^{2/3} \times \left(\frac{1}{5000}\right)^{1/2}$

$\boxed{R = 2.78\ m}$

(2) $Q = AV$

$A = \frac{Q}{V} = \frac{350}{2} = 175 m^2$

Now,

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

$P = \frac{A}{R} = \frac{175}{2.78}$

$\boxed{P = 62.9 m}$

Here

$Q >> 55 m^3/s$ so, Assume Trapezoidal channel sxn.

Then,

$A = y\,(B + y\theta + y\cot\theta)$

and $P = B + 2y\theta + 2y\cot\theta$

$\pi^c = 180°$

$180° = \pi^c$

$1 = \frac{\pi^c}{180}$

For θ, Take side slop

$\therefore \tan\theta = \frac{1}{1.5}$

$\therefore \theta = \tan^{-1}\left(\frac{1}{1.5}\right)$

$\theta = 33.69^\circ$

$= 33.69 \times \frac{\pi}{180}^c$

$\boxed{\theta = 0.59^c}$

we have,

$\tan\theta = \frac{1}{1.5}$

$1.5 = \frac{1}{\tan\theta}$

$\boxed{1.5 = \cot\theta}$

Now,

$A = y(B + y\theta + y\cot\theta)$

$175 = y(B + 0.59y + y\cot\theta)$ $\quad (\cot\theta = 1.5)$

$= y(B + 0.59y + y \times 1.5)$

$175 = y(B + 2.09y)$

$\boxed{y(B + 2.09y) = 175}$ ———— (1)

Again,

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

$62.9 = B + 2y \times 0.59 + 2y \times 1.5$

$62.9 = B + 4.18y$

$\boxed{B = 62.9 - 4.18y}$ ———— (2)

Putting Value of B in eqn (1)

$y(62.9 - 4.18y + 2.09y) = 175$

$62.9y - 2.09y^2 = 175$

$2.09y^2 - 62.9y + 175 = 0$

$\therefore y = 26.99, 3.10$

for economic design

Take, $\boxed{y = 3.10m}$ (ie small value)

Putting value of y in eqn (2)

$B = 62.9 - 4.18 \times 3.10$
$= 49.942$
$\boxed{B \simeq 50m}$

Take free board (FB) = 0.6m
∴ Total depth (y) = 3.1 + 0.6 = 3.7m
width (B) = 50m } Ans//.

(2) Design a Concrete lined Channel to carry a discharge of $350 m^3/s$ at a slope of 1 in 6400. The side slopes of the Channel may be taken as $\frac{3}{2}:1$. The value of n for lining material may be taken as 0.013. Assume limiting water depth of channel as 4m.

Soln.

Given,
Discharge (Q) = $350 m^3/s$
Bed slope(S) = 1 : 6400 (V:H)
Side slope = $3/2 : 1$ = 1.5 : 1 (H:V)
n = 0.013
Depth (y) = 4m
B = ?
A = ?
P = ?
R = ?

Soln. we know,
Take Trapezoidal sxn (ie $Q >> 55 m^3/s$)

∴

$A = y(B + y\theta + y\cot\theta)$
$= 4(B + 4 \times 0.59 + 4 \times 1.5)$
$\boxed{A = 4B + 33.44}$

$\tan\theta = \frac{1}{1.5}$ ∴ $\theta = 33.69$
$\boxed{\theta^c = 0.59}$
$\cot\theta = 1.5$

and,

$P = B + 2 \times y\theta + 2y\cot\theta$

$= B + 2 \times 4 \times 0.59 + 2 \times 4 \times 1.5$

$\boxed{P = B + 16.72}$

we have,

$Q = A \times V$

$= A \times \frac{1}{n} R^{2/3} . s^{1/2}$

$= A \times \frac{1}{n} \left(\frac{A}{P}\right)^{2/3} . s^{1/2}$

$350 = (4B + 33.44) \times \frac{1}{0.013} \times \left(\frac{4B + 33.44}{B + 16.72}\right)^{2/3} . \left(\frac{1}{6400}\right)^{1/2}$

$364 = \frac{(4B + 33.44)^{5/2}}{(B + 16.72)^{2/3}}$

$\boxed{B = 32.5 m}$

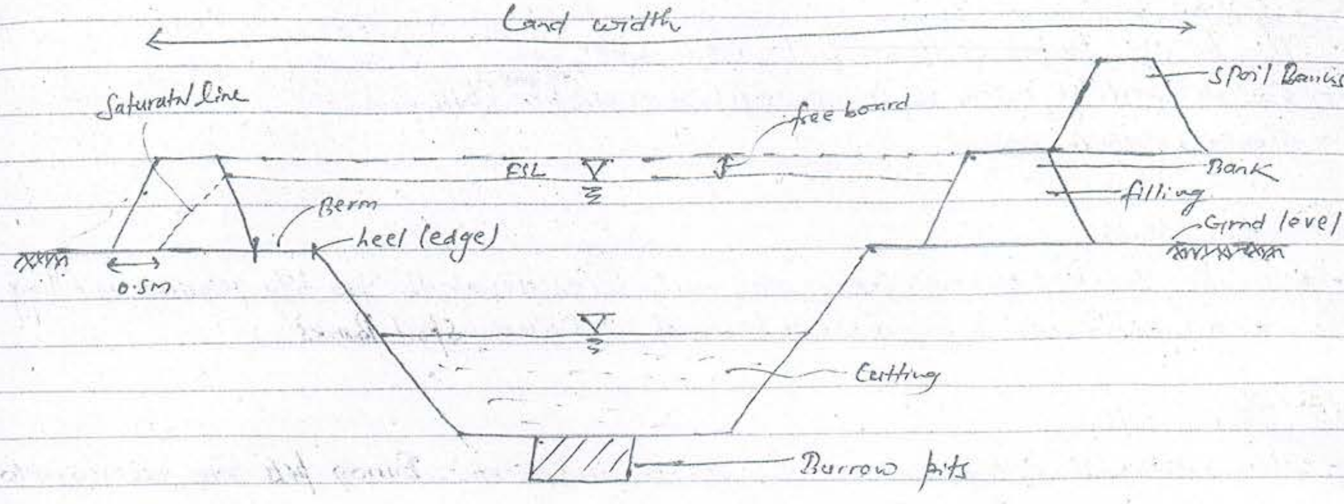
and

$\boxed{y = 4m}$

Assume free board (FB) = 0.6m

$\therefore$ Total depth (y) = 0.6 + 4 = 4.6m
and width (B) = 32.5m } Ans//.

Cross-Section of Canal, Berms, Banks, Roadways and Spoil banks, Balance depth



(1) Side Slopes
- Side slope should be such that they are stable depending upon the type of soil.
- Comparatively steep slope can be provided in Cutting Rather than in filling ie,

Cutting – 1H : 1V to 1.5H to 1V
filling – 0.5H : 1V to 2H : 1V

(2) **Berm**: It is horizontal distance left at the grnd level between the toe of bank & heel or edge of Cutting.
- width of berm is Variable but kept such that bed line & bank line remains parallel.

Purpose of Berm
(1) The silt deposited on the side is very fine & impervious which serves as good lining for reducing absorption & losses.
(2) To increase Capacity of Canal
(3) To protect the bank from erosion
(4) To provide additional Strength
(5) To provide Scope for future widening.

③ Banks
- The primary purpose of bank is to retain water.
- Can be used as means of Communicatn & as inspxn (निरिक्षण) path.
- Should be wide enough

④ Spoil Banks
- When the quantity of earth cutting is much in excess of the quantity required in filling then extra earth has 2 be deposited which is called **Spoil Banks**

⑤ Burrow pits
- When erthwork in filling exceeds the erthwork in excavtn, **Burrow pits** are necessary to make good requirement of filling.

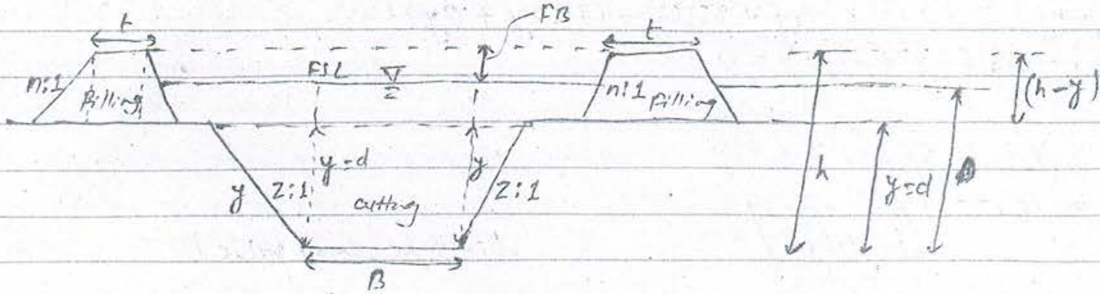
⑥ Land width
- It is width of land required for constrxn of canal ~~cross~~-sxn.

⑦ Road ways
- It is very necessary to have access in all parts of canal so that inspxn may become ea
- Road ways increase the efficiency of maintenance
- Main canal and branch canal have Roadways on both side.

⑧ Free board (FB)
- Gap or margin of FSL and top of banks
- Depends on → size of canal
  → Locatn
  → water surface fluctuatn

⑨ Balancing Depth
- A canal sxn will be economical if the earth work involved at a particular sxn has an equal amnt of cut and fill.
- For a given sxn there is always only one depth for which cutting and filling will be equ
This depth is known as **Economic depth OR Balancing depth.**

Illustratn of Balancing depth or example



let,

B = Bed width

h = ht. from top of bank

D = fall depth

t = top width of Canal bank

n : 1 = side slope of bank in filling

Z : 1 = Side slope of canal in Cutting

y = depth

for Balance depth

$$\boxed{\text{Area of Cutting} = \text{Area of filling}} \quad \text{——— (1)}$$

$\therefore$ Area of Cutting $(A_c) = By + \cancel{2} \times \frac{1}{\cancel{2}} \times Z \times y \times y$

$$\boxed{A_c = By + zy^2} \quad \text{——— (a)}$$

Again,

$$\text{Area of filling } (A_f) = 2\left[t(h-y) + n(h-y)^2\right] \quad \text{——— (b)}$$

equating (a) & (b)

$$A_c = A_f$$

$$By + zy^2 = 2\left[t(h-y) + n(h-y)^2\right] \qquad \therefore y = d$$

Put

$B = 5m$, $t = 2m$, $h = 2.92$, $n = 1.5$, $Z = 1$

$$5xy + 1y^2 = 2\left[2(2.92-y) + 1.5(2.92-y)^2\right]$$
$$5y + y^2 = 11.68 - 4y + 3(8.52 - 5.84y + y^2)$$
$$5y + y^2 = 11.68 - 4y + 25.56 - 17.52y + 3y^2$$
$$2y^2 - 26.52y + 37.24 = 0$$

$$y = 11.66,\ 1.6$$

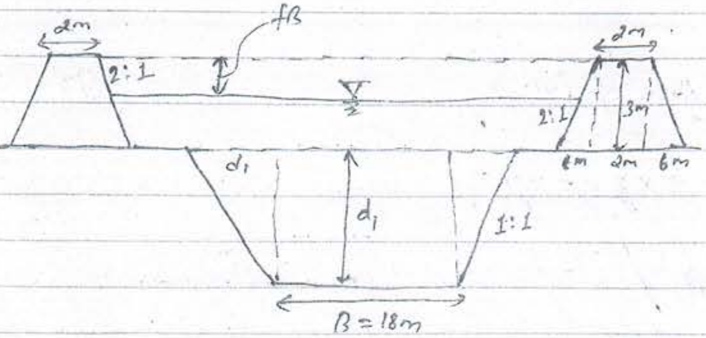
for economic, take $\boxed{y = 1.6\,m}$ (ie take small value)

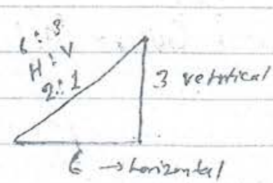
$\therefore$ Required **Economic Depth or Balancing depth** $\boxed{y = 1.6\,m}$ //Ans//

Numerical

① Calculate the Balancing depth for a Canal sxn hvng bed width 18m, side slope 1:1 in cutt and 2:1 in filling. The bank embankment are kept 3m higher than grnd level (Berm level). Th crest width of bank is kept as 2m.

Soln. Given,





Here,

Area of Cutting $(A_c) = 18 \times d_1 + \left(\frac{1}{2} \times d_1 \times d_1\right) \cdot 2$

$$\boxed{A_c = (18 + d_1)\,d_1} \quad \text{———— (1)}$$

Area of filling $(A_f) = 2\left(\frac{2+14}{2} \times 3\right)$ OR $= 2\left[\left(2 \times \frac{1}{2} \times 6 \times 3\right) + 2 \times 3\right]$

$$\boxed{A_f = 48\,m^2} \quad \text{———— (2)}$$

equating (1) & (2)

$(18 + d_1) d_1 = 48$

$d_1 = y$

$\boxed{d_1 = 2.35 m}$

$\therefore$ $\boxed{\text{Balancing depth } (d_1 = y) = 2.35 m}$ // Ans //

2072/05/30

Chapter: 5 - Headworks and Distributul System

Headworks :- Any hydraulic structures which supplies water to the off-taking canal is called **Headworks**
- It is Constructed across River

Types of Headworks - 2 types

① Storage Headworks
② Diversion Headworks

① Storage headworks :- The headworks that Comprises the Constrxn of dam across the river in order to create a Reservoir in the Canal bed is called **Storage headworks**.
- storage headworks stores water during the period of excess supplies in the River and relese it when demand overtakes the available supply.

eg. Dam.

② Diversion headworks :- These are located across the river so as to Raise the Norm water level of the River to divert the required River wtr Supply to the Canal.

eg. Barrage, weir

Types of Diversion headworks - 2 types

(a). Temporary spurs or bunds
(b) Permanent weirs & Barrages

(a) Temporary spurs or bunds - These are temporary strxtrs made during flood.

(b) Permanent weirs & Barrage - These are permanent strxtrs.

eg. koshi barrage

Purpose / functn of : Headworks

① To Raise the wtr level in the River.
② To supply water 2 the Canal
③ To Control entry of silt into the Canal
④ To reduce the River fluctuatn.
⑤ To store water for small periods.
⑥ To divert the River water.

WVL wet wated.

Components of Headworks

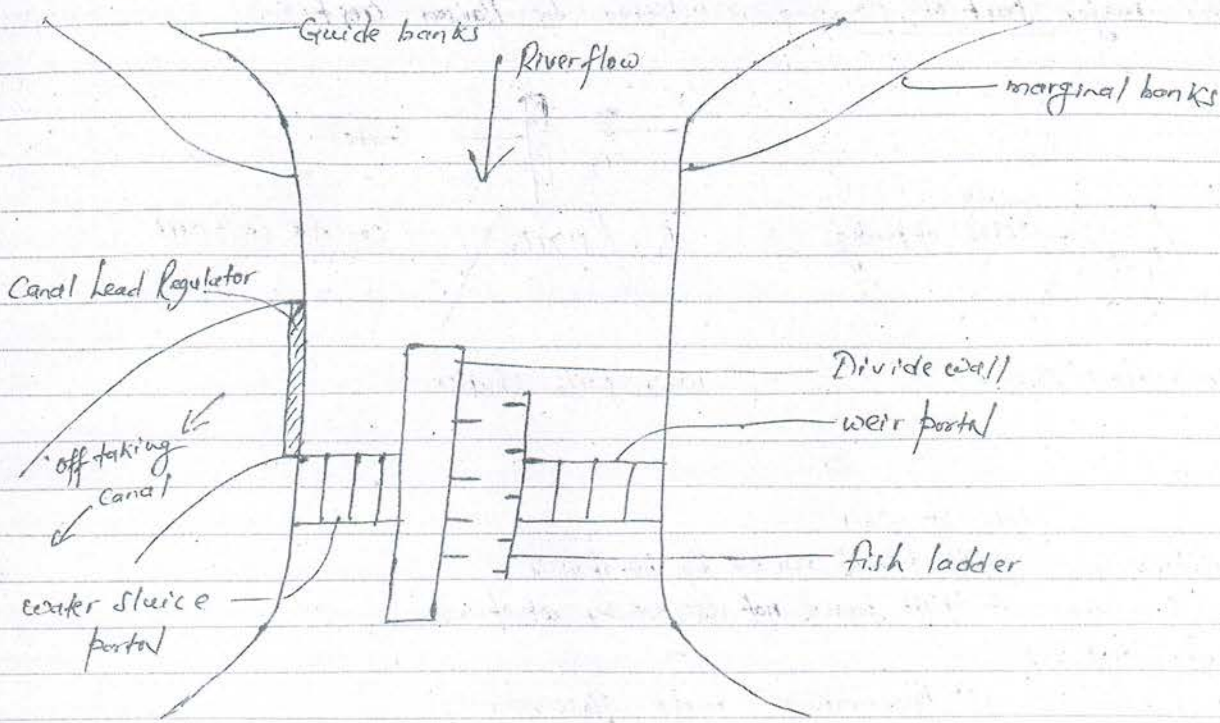


Fig:- Typical layout of Diversion headwrks

1. weir / Barrage
2. Under sluice
3. Fish ladder
4. Canal head Regulator
5. Divide wall
6. Sediment Regultn works
7. River training wrks
   - Guida banks @ GMSR
   - Marginal banks
   - Spurs
   - Revetment (stone pitching) & launching apron

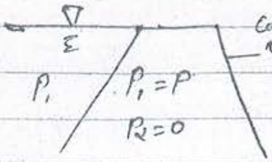
WVL

(1) weir or Barrage

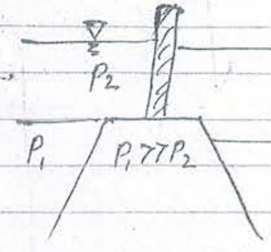
weir :- A solid obstrxn put across the River to Raise the water level in the River and to divert water to Canal is Called **weir**.

- it is raised Concrete Crest. wall Constructed across the River.

- in weir major part of ponding is achieved by Raised Crest.



weir without shutter



weir with shutter

Types of weir

① Gravity weir - uplift pressR resisted by wt. of weir

② Non-Gravity weir - uplift pressR not resisted by wt. of weir

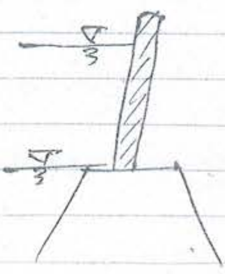
③ Vertical drop weir

④ Sloping weir → ⓐ Masonary or Concrete slope weir
→ ⓑ Dry stone slope weir

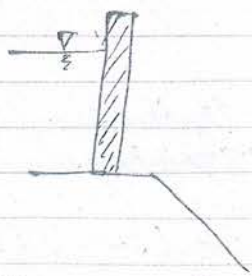
⑤ Parabolic weir

Barrage :- It is similar to weir but it has a low crest wall with high Gates.

- in barrage must of the ponding is done by Gates or shutter.



Barrage with small raised crest



Barrage without crest

② Under Sluice OR Scouring Sluices

These are the opening provided in the weir wall with lower crest level then the crest of

Nrml portal to weir.
- A divide wall separates the main weir portal & under sluice portal.

Function
- To Control silt entry to Canal
- To pass the low floods
- To provide greater wtr way for the Flood discharge.

(3) Fish ladder :- It is the strxtr provided adjacent to divide wall near under sluice to facilate the migratn of fish From upstream to ↓ stream.
- slope of fish ladder is generally 1V: 10H.
- Velocity is maintained 3 to 3.5m/s

(4) Canal head Regulator :- It is provided at the entrance of off-taking Canal. The crest level of head Regulator is kept slightly ↑er than the crest of under sluice.

Functn
- To Control entry of water into Canal.
- To Control the entry of flood
- To Control the entry of Silt into Canal.

(5) Divide walls :- It is masonary or Concrete wall which separates the weir portal from the under sluices.

functn
- To separate under sluice portal from weir
- ↑es the effectiveness of undersluice
- prevents cross-current and flow parallel to weir.
- Serves as one of the side walls of fish ladder.

(6) Sedimentatn Regulatn works
- sediment can be Controlled by 2 method
  - (1) Using sediment preventive measure
  - (2) Sediment exclusion dete
    - silt excluder
    - silt ejector
    - setting basins

(1) Using sediment preventive measures
- Adopting proper alignment
- Proper approach
- proper Regulatn

(2) Sediment Exclusion Device (silt) - 3 types

(a) Silt excluder : It is a device by which silt is excluded from wtr entering to canal.
- Constructed near River bed in front of head regulator.

(b) Silt ejector :- It is a device by which silt is ejected from canal to River.
- Constructed near bed of off-taking canal.

(c) Settling Basins :- To exclude very fine suspended mtrls settled mtrls are continuously flushed or Remvd from settling basins.

(7) River Training works - These are provided for smooth flow of water in River.
- It consists

(a) Guide bank :- These are provided on the either side of diversion headworks for a smo approach to diversn headwork & prevent the river from out-flanking.

(b) Marginal banks :- These R provided at upper sides of headwork to prevent the Area fr sub-mergence.

(c) Spur or groynes :- These R strxtrs constructed transverse to the River flow.
- also known as spur or groynes or dikes

(d) Revetments :- Stitching & pitching of banks by stones, bamboo or timber piles, sand bags, concrete slabs or gabion filled with stone is called Revetment.

Bligh Creep Theory for seepage Flow

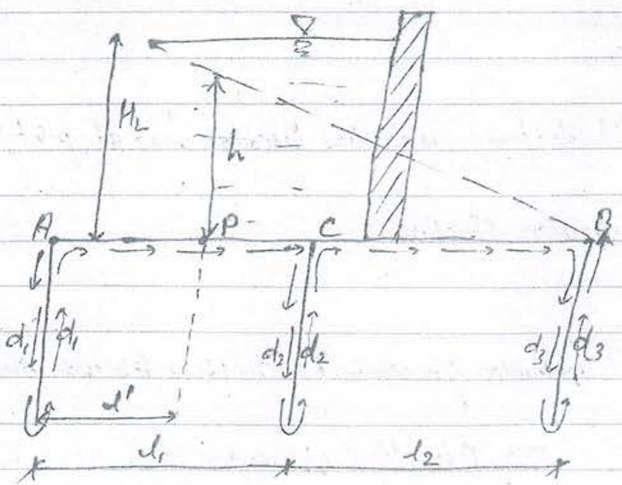
Hydraulic Structures like Dam, weir, barrage, head Regulators, cross regulators etc are found on a pervious foundatn due to which seeping may occur beneath the hydraulic strctrs. This seeping wtr may cause its failure by ① Piping
② Uplift.

① Piping: Piping occurs when soil beneath the foundtn are progressively remvd by residual force and emerging downstream end of the work (or strctr).

② Uplift :- failure by direct uplift occurs when water seeping below the strctr exerts an uplift pressR on the floor of the strctr. If this process is not counter balanced by the weight of concrete, the strctr will Rupture & fails.

Bligh's creep theory

According to Bligh's, "the percolating water, follow the outline of the base of the foundatn of hydraulic strctrs. The length of the seepage path traversed by percolating water is called creep length.



from figure,

$$\text{Creep length } (L) = d_1 + d_1 + l_1 + d_2 + d_2 + l_2 + d_3 + d_3$$
$$= 2(d_1 + d_2 + d_3) + l_1 + l_2$$

As the water creeps from upstream to downstream end head loss occurs which is propor to creep distance.

- If "H_L" is the **Total Head Loss** between upstream and downstream and "L" is the **creep length**, then the Head loss per unit length $\left(\frac{H_L}{L}\right)$ is called **Hydraulic Gradient**

ie,

$$\text{Hydraulic gradient } (i) = \frac{H_L}{L}$$

$$= \frac{H_L}{2(d_1+d_2+d_3)+l_1+l_2}$$

let us consider a point 'P' within the horizontal floor at length l' from upstream edg the uplift PressR at point $P = \gamma h$

where,

$$h = H_L - \text{loss of head between A to P}$$

$$= H_L - i \times L' \qquad (\because L' = d_1 + d_1 + l' \text{ — ie upto } l')$$

$$= H_L - \frac{H_L}{L}(2d_1 + l')$$

$$\boxed{h = \frac{H_L}{L}(L - 2d_1 - l')}$$

is reqd seepage head OR Residual head at point P.

Design criteria for safety of hydraulic structure

(a) Safety against Piping OR Underpinning

To prevent piping failure the sub-soil hydraulic gradient (i) should be less than the permi-ble value.

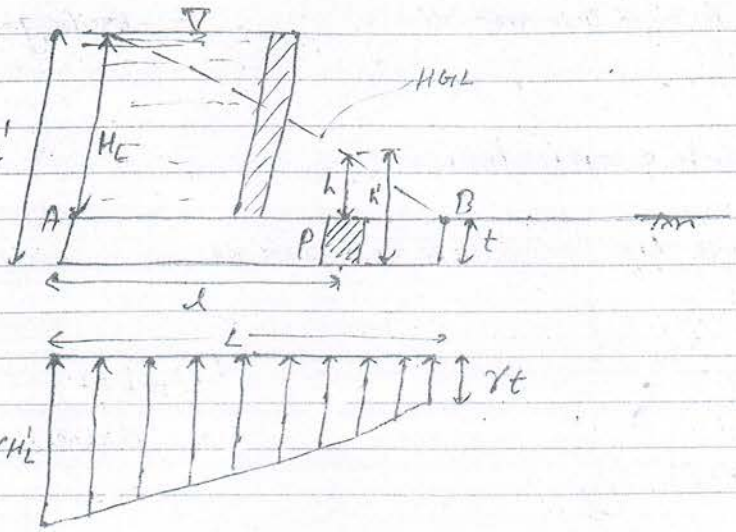
ie, $\boxed{i \le \frac{1}{C}}$, C = Bligh's Coefficient.

$$i = \frac{H_L}{L} \le \frac{1}{C}$$

$$\boxed{L \geq CH_L}$$

Value of C	Value of C
light Sand OR mud	— 8
Fine Sand	— 15
Coarse Sand	— 12
Sand mxd with shoulder	— 5-9

⑥ Safety Against uplift



The ordinate of HGL (Hydraulic gradient line) above the bottom of the floor represent the Residual uplift water head. To prevent Rupture due to uplift pressR the floor should sufficient thick. It means the weight of impervious floor must be sufficient to counter balance the uplift pressR.

Consider an impervious floor of length 'L', seepage head H_L.
let us Consider point 'P' at distance 'l' from point 'A' (ie upstream edge)

$\therefore$ Residual head at P, $h = H_L - i \times l$

$$= H_L - \frac{H_L}{L} \times l$$

$$\boxed{h = \frac{H_L}{L}(L-l)}$$ $\left(i = \frac{H_L}{L}\right)$

uplift pressR at point P ie,

$U =$ pressR intensity * Unit area
$= \gamma_w h'$ * Unit area.

weighing Concrete $= \gamma_{concrete} \times$ Volume
$= \gamma_w G \times$ Unit area $\times t$

G = specific gravity $= \frac{\gamma_s}{\gamma_w}$

At equilibrm,
wt. of Concrete = uplift pressR

$\cancel{\gamma_w} h' \times \cancel{\text{Unit Area}} = \cancel{\gamma_w} G \times \cancel{\text{Unit area}} \times t$

$h' = Gt$
$t + h = Gt$
$h = Gt - t$
$h = t(G-1)$

$$\boxed{t = \frac{h}{G-1}}$$

$(h' = h + t)$
t = thickness

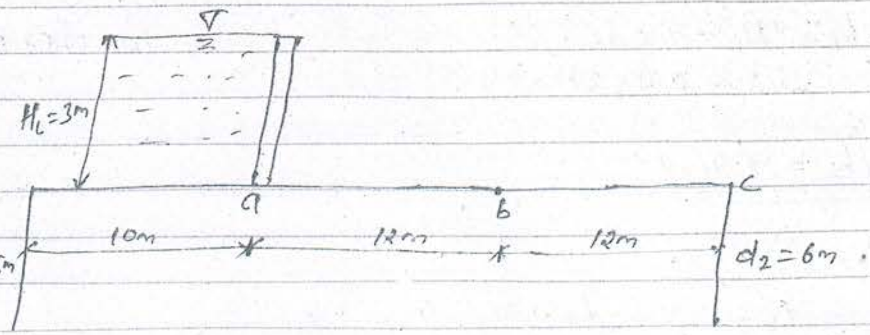
Taking factor of safety $\frac{4}{3}$

$$\boxed{t_{actual} = \frac{4}{3}\frac{h}{G-1}}$$

//.

Numerical

(1) A hydraulic structure founded on Coarse sand. Determine the average hydraulic gradient and uplift pressR at points a, b, c and also determine thickness required at these points.
Take $G = 2.2$
$H_L = 3m$



Soln.
find,
thickness at a = ?
" " b = ?
" " c = ?

$H_L = 3m$

and Total creep length (L) = 5+5+10+12+12+6+6
= 56m

we have,

$$t = \frac{4}{3}\frac{h}{G-1}$$

and $h = H_L - i \times l$

i = hydraulic gradient $= \frac{H_L}{l}$

(1) for point a
$L = 56m$
$H_L = 3m$

PressR head available $(h_a) = H_L - i \times l_a$
$= 3 - 0.53 \times 20$
$= 2m$

$i = \frac{H_L}{L} = \frac{3}{56} = 0.053$

$l_a = 5+5+10 = 20$ — i.e upto a only

Now,
thickness at a $(t_a) = \frac{4}{3} \times \frac{h_a}{G-1}$

$= \frac{4}{3} \times \frac{2}{2.2-1}$

$\boxed{t_a = 2.22m}$

(2) for point b

$L = 56m$

$H_L = 3m$

$i = \frac{H_L}{L} = \frac{3}{56} = 0.05$

$h_b = H_L - i \times l_b$

$l_b = 5+5+10+12 = 32$

$= 3 - 0.05 \times 32$

$\boxed{h_b = 1.4m}$

Now,

$t_b = \frac{4}{3} \times \frac{h_b}{G-1} = \frac{4}{3} \times \frac{1.4}{2.2-1}$

$\boxed{t_b = 1.55m}$ //

(3) for point c

$L = 56m$

$H_L = 3m$

$i = 0.05$

$l_c = 5+5+10+12+12 = 44m$

$h_c = H_L - i \times l_c$

$= 3 - 0.05 \times 44$

$\boxed{h_c = 0.8m}$

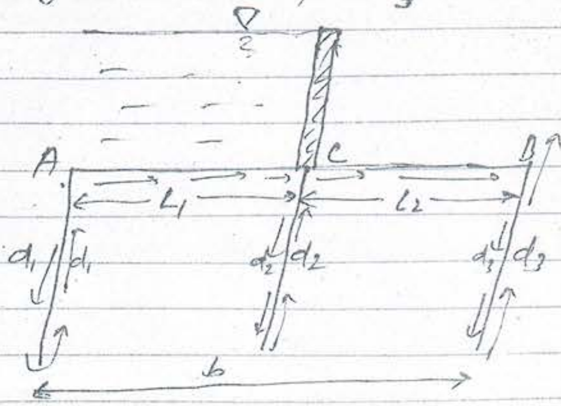
Now,

$t_c = \frac{4}{3} \times \frac{h_c}{G-1} = \frac{4}{3} \times \frac{0.8}{2.2-1}$

$\boxed{t_c = 0.88m}$ //

Lane's Weighted Creep Theory

Bligh's Calculated the length of creep by simply adding the horizontal and vertical creep creep length without any distinction [illegible] between these two creeps. But **According** to **Lane** horizontal creep is less effective in Reducing uplift or Causing head loss than the vertical creep. So he gave weightage factor $\frac{1}{3}$ for **horizontal** creep against **1** For **vertical creep**.



from figure,

$$\boxed{\text{Lane's Creep length } (L_L) = \frac{1}{3}\left(L_1 + L_2\right) + 2\left(d_1 + d_2 + d_3\right)}$$

//This reqd expresN.

(a) Safety against piping

Creep length $(L_L) \nless C_L H_L$

H_L = Head Causing Flow
C_L = Lane's Creep Coefficient.

Soil type	Value of C_L
Very fine sand/silt	8.5
Fine sand	7.0
Coarse sand	5
Gravel and sand	3.5 to 3
Boulders, gravels & sand	2.5 to 3
Clayey soils	3 to 1.6

KHOSLA'S Seepage Theory / khosla's Theory

Principle of Design for Sub-surface Flow of structure, khosla Theory

Er. Umesh Raut

(A) Critical Exist Gradient and Safe Exist Gradient

→ The exist gradient is said to be critical, when the upward disturbing force on the gr
is just equal to the Submerged weight of the grain at the exist. When a factor of safety
equal to 4 or 5 is used, then exist gradient is taken as safe exist gradient.

Here,

for Unit Volume of Soil,

Submerged weight $\boxed{(w_s) = \gamma_w (1-n)(G-1)}$

γ_w = Unit wt. of wtr
G = Specific gravity of soi
n = Porosity of soil

for critical Condtn at exist gradient

$F = w_s$

F = upward force

Force F = PressR gradient at that point

$= \gamma_w \cdot \frac{dh}{dl}$

where, $\frac{dh}{dl}$ = Rate of loss of head OR gradient at exist end.

Now,

$F = w_s$

$\cancel{\gamma_w} \cdot \frac{dh}{dl} = \cancel{\gamma_w} \cdot (1-n)(G-1)$

$\boxed{\frac{dh}{dl} = (1-n)(G-1)}$ // is exprsn for critical exist gradient.

for safe exist gradient $\frac{1}{4}$ to $\frac{1}{5}$ exist gradient provided//.

Khosla Method for Determinatn of PressR and Exist Gradient ~~for~~ Seepage flow Below weir OR Barrage

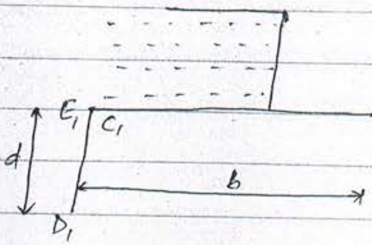


Fig:- Pile on upstream end

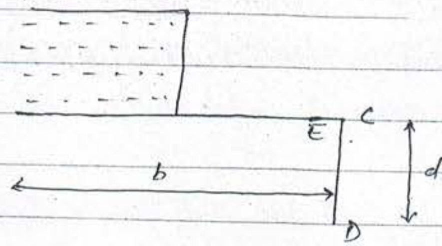


Fig:- Pile at downstream end

$$\phi C_1 = 100 - \phi E$$
$$\phi D_1 = 100 - \phi D$$

$$\phi E = \frac{1}{\pi} \cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$$

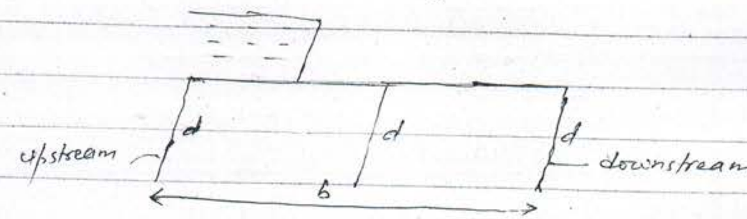
$$\phi D = \frac{1}{\pi} \cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right)$$

where,

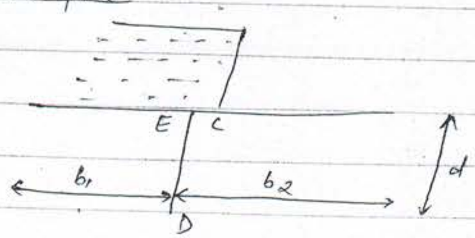
$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2}$$

$$\alpha = \frac{b}{d}$$

b = distance from upstream to downstream dist



for intermediate pile



$$\phi E = \frac{1}{\pi} \cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right)$$

$$\phi D = \frac{1}{\pi} \cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$$

$$\phi C = \frac{1}{\pi} \cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda}\right)$$

where,

$$\lambda = \frac{\sqrt{1+\alpha_1^2} + \sqrt{1+\alpha_2^2}}{2}$$

$$\lambda_1 = \frac{\sqrt{1+\alpha_1^2} - \sqrt{1+\alpha_2^2}}{2}$$

$$\alpha_1 = \frac{b_1}{d}$$

$$\alpha_2 = \frac{b_2}{d}$$

Corrxn (A) Corrxn for mutual interference of piles

downstream ले गर्दा upstream मा affect

(a) Correction due to downstream pile

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

d = आफ्नो value, ie upstream को d

D = affected part, आफ्नो भन्दा अर्को ie downstream

b' = Distance betn two pile

b = Total floor length

$\therefore$ $\boxed{\text{Corrected } \phi C_1 = \phi C + C}$

(b) Corrxn due to upstream pile

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

$\therefore$ $\boxed{\text{Corrected } \phi E_2 = \phi E_2 - C}$

(B) Corrxn for slope

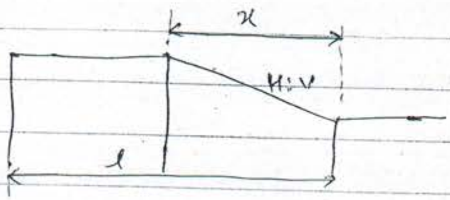
A Corrxn is applied for sloping Floor and is taken as +ve for down and −ve for up.

(Slope) H:V	Corrxn factor (C.F)
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

→ The Corrxn factor given above is multiplied by horizontal length of the slope and divided by the distance between the two sheet pile lines between which the sloping floo. is located.

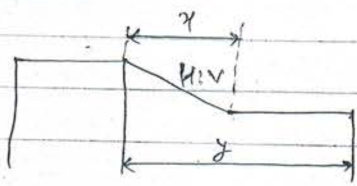
- The Corrxn is applicable only to the key points of pile line fixed at the start or enc of slope.

Case - I



Corrxn = $\frac{x}{l} \times C.F$

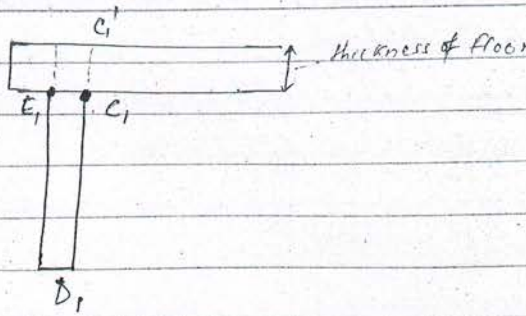
Case - II



Corrxn = $\frac{x}{y} \times C.F$

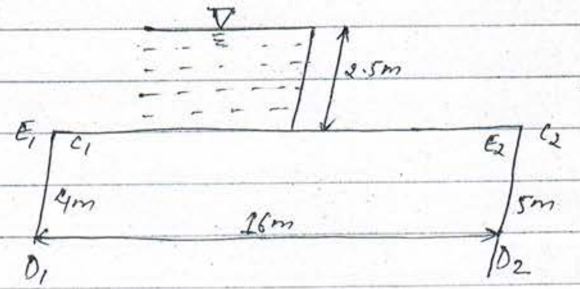
(C) Corrxn for Floor thickness

Corrxn for floor thickness at C_1 = $\frac{\text{observed } D_1 - \text{observed } C_1 \text{ ie } C_1'}{\text{Distance beth } C_1' D_1} \times$ thickness of floor



Example

(1)



$\phi C_1 = ?$ & $\phi E_2 = ?$

Soln.

we have,

$$\phi C_1 = 100 - \phi E \quad \text{--------- (1)}$$

$$\phi E = \frac{1}{\pi} \cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right) \quad \text{--------- (2)}$$

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} \quad \text{--------- (3)}$$

$$\alpha = \frac{b}{d} = \frac{16}{4} = 4$$

$b = 16m$
$d = 4m$

Then from eqn (3),

$$\lambda = \frac{1 + \sqrt{1 + 4^2}}{2}$$

$$\boxed{\lambda = 2.56m}$$

Check → mutual interference
→ slope
→ Floor thickness

from eqn (2)

$$\phi E = \frac{1}{\pi} \cos^{-1}\left(\frac{2.56-2}{2.56}\right)$$

$$= \frac{1}{\pi} \times 77.36^{\circ}$$

$\pi^c = 180^{\circ}$

$1^{\circ} = \frac{\pi}{180}$

$$= \frac{1}{\pi} \times \frac{\pi}{180} \times 77.36$$

$$= 0.429$$

$$\simeq 0.43$$

$$\boxed{\phi E = 43\%}$$

Now, from eqn (1)

$$\phi C = 100 - \phi E$$

$$= 100 - 43$$

$$\boxed{\phi C = 57\%}$$

Corrxn

(A) Corrxn due to Mutual interference

(a) Corrxn due to downstream pile

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

d = आफ्नो depth or length.

for C_1, C_2 is affected so $D = 5$ m

ie D is taken for affected part
↳ ie आफ्नो भन्दा अर्कोको

d = आफ्नो length

Here,

$D = 5m$

$d = 4m$

$b = b' = 16m$

$$\therefore C = 19\sqrt{\frac{5}{16}}\left(\frac{4+5}{16}\right)$$

$$\boxed{C = 5.97}$$

$\therefore$ Corrected $\phi C_1 = \phi C + C$

$= 57 + 5.97$

$= 62.97\%$

$\boxed{\text{Corrected } \phi C_1 \simeq 63\%}$ //Ans//

upstream ले तर्फ ↓stream मा affect गर्दा
corrxn is −ve & viceversa

down to upमा → affect E_1 गर्दा → +ve } always
upले तर्फ down मा affect E_1 गर्दा → −ve

Again,

$$\phi E_2 = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right)$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$$

$$\alpha = \frac{b}{d} = \frac{16}{5} = 3.2$$

$b = 16m$ } ie E_2 lies Near
$d = 5m$ } downstream so take
downstreamको value

then, $$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = \frac{1+\sqrt{1+3.2^2}}{2}$$

$$\boxed{\lambda = 2.17}$$

$$\phi E_2 = \frac{1}{\pi}\times\cos^{-1}\left(\frac{2.17-2}{2.17}\right)$$

$$= \frac{1}{\pi}\times 85.50^\circ$$

$$= \frac{1}{\pi}\times\frac{\pi}{180}\times 85.50$$

$$= 0.475$$

$$\boxed{\phi E_2 = 47.5\%}$$

Corrxn

(a) Corrxn due to upstream pile

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

$D = 4m$

$d = 5m$

$b' = b = 16m$

$\therefore \quad C = 19\sqrt{\frac{4}{16}}\left(\frac{5+4}{16}\right)$

$\boxed{C = 5.34\%}$

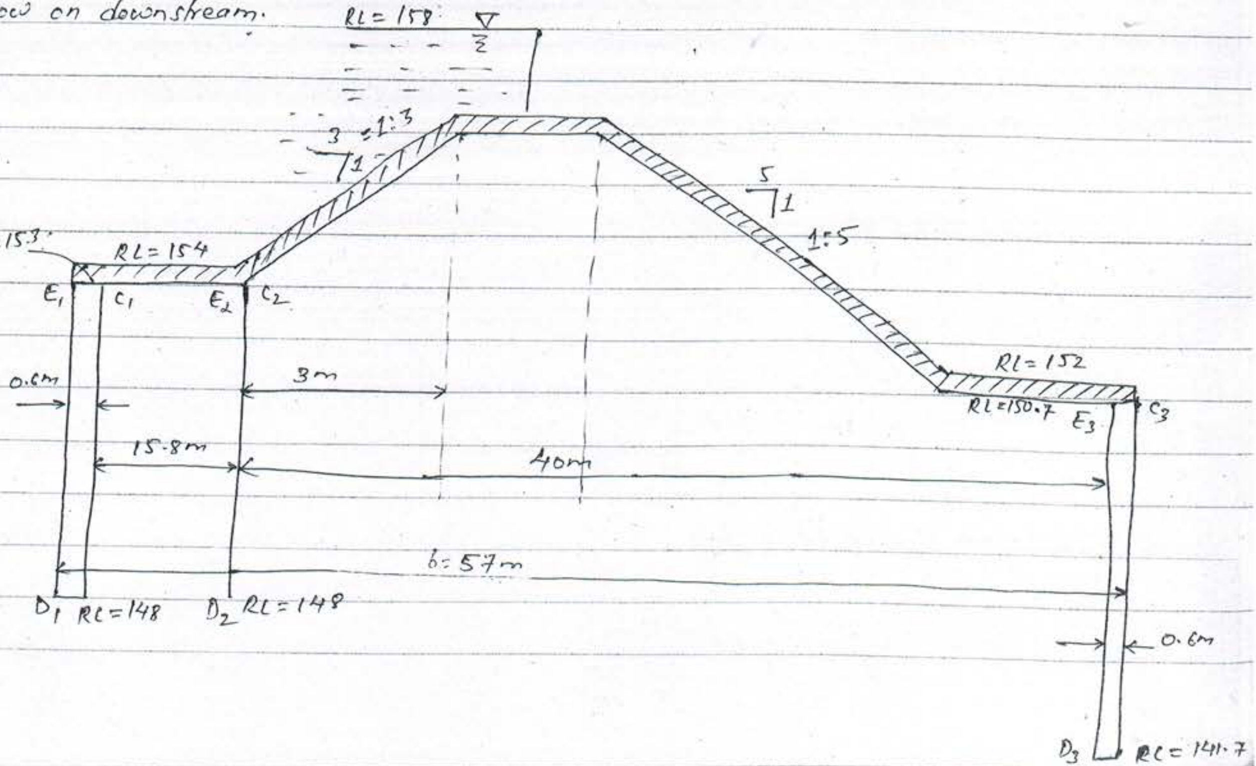
$\therefore$ Corrected $\phi E_2 = \phi E_2 - C$

$= 47.4 - 5.34$

$\boxed{\text{Corrected } \phi E_2 = 42\%}$ // Ans //.

IMP wst world //

(2) Determine the percentage pressR at Various key points in Fig. Also determine the Exist Gradient and plot the hydraulic gradient line for pond level on upstream and no Flow on downstream.



at 1st point → always 100% | ie decreasing
at last point → always zero ↓ order

Er. Umesh Raut

Soln. ① Percentage PressR at key points i.e
$E_1, D_1, C_1, E_2, D_2, C_2, E_3, D_3, C_3 = ?$

② Exist Gradient (G_E) = ?

③ Hydraulic gradient line (HGL) Graph = ?

① Percentage PressR at key points

we know, (A) for upstream pile No-1

For ϕC_1, $\phi C_1 = ?$

$b = 57m$

$d = 154 - 148 = 6m$

$\alpha = \frac{b}{d} = \frac{57}{6} = 9.5$

$\phi C_1 = 100 - \phi E$

$\phi E = \frac{1}{\pi} \cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$ where, $\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2}$

$= \frac{1}{\pi} \cos^{-1}\left(\frac{5.28 - 2}{5.28}\right)$ $= \frac{1 + \sqrt{1 + 9.5^2}}{2}$

$\boxed{\lambda = 5.28}$

$= \frac{1}{\pi} \cdot 51.59°$

$= \frac{1}{\pi} \times \frac{\pi}{180} \times 51.59°$

$= 0.286$

$\boxed{\phi E = 28.6\%}$

Now,

$\phi C_1 = 100 - 28.6$

$\boxed{\phi C_1 = 71.4\%}$ //

अगाडिको अर्को पछाडिका effect हुँदा +ve , for ϕ_{C_1} due to E_2 take +ve (i.e add)

b → length from upstream to downstream always

ⓐ Corrxn at C_1 for mutual interference (ϕ_{C_1} affected by intermediate pile 2)

– Corrxn is +ve because it is in dirxn of flow

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

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

$$\boxed{\text{Corrxn} = 1.87\%} \text{ (+ve)}$$

D = depth of pile 2 = 153 – 148 = 5m

d = depth of pile 1 = 153 – 148 = 5m

b' = distance betn 2 sheet pile = 15.8m

b = total floor length = 57m

$C_1 \longrightarrow E_2$ ie flow dirxn is in same line so +ve.

for ϕD_1,

$$\alpha = 9.5$$

$$\lambda = 5.28$$

$$\phi D_1 = 100 - \phi D$$

$$\phi D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right) = \frac{1}{\pi}\cos^{-1}\left(\frac{5.28-1}{5.28}\right)$$

$$= \frac{1}{\pi} \times 35.84^\circ$$

$$= \frac{1}{\cancel{\pi}} \times \frac{\cancel{\pi}}{180} \times 35.84$$

$$= 0.199$$

$$\boxed{\phi D = 19.9\%}$$

Now,

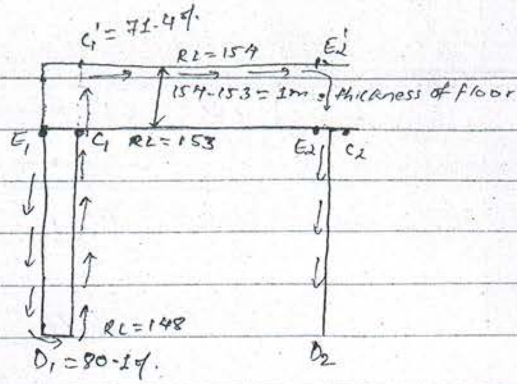
$$\phi D_1 = 100 - \phi D$$

$$= 100 - 19.9$$

$$\boxed{\phi D_1 = 80.1\%}$$

ⓑ Correction at C_1 due to thickness of floor

PressR Calculated from above RelatN, we have Calculated pressR at C_1'. But



we need pressR at C_1

PressR at C_1 shall be more than at C_1' as the dirxn of flow is from C_1 to C_1'.
Hence the Corrxn will be +ve

$\therefore$ Corrxn for floor thickness at C_1 $= \dfrac{\text{observed } D_1 - \text{observed } C_1 \text{ ie } C_1'}{\text{Distance beth } C_1' D_1} \times$ thickness of floor

$$= \frac{\phi D_1 - \phi C_1}{154-148} \times (154-153)$$

$$= \frac{80.1 - 71.4}{6} \times 1$$

$$\boxed{\text{Corrxn at } C_1 = 1.45\%}$$

$\therefore$ Corrected $\phi C_1 = \phi C_1 +$ Corrxn due to mutual interference + Corrxn due to floor thickness

$$= 71.4 + 1.87 + 1.45$$

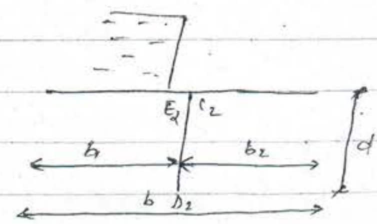
$$\boxed{\text{Corrected } \phi C_1 = 74.72\%}$$ // Ans//

and,

$$\boxed{\phi D_1 = 80.1\%}$$ // Ans//.

(B) for intermediate Pile No- 2

$$\phi E_2 = \frac{1}{\pi} \cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right)$$



$$\phi D_2 = \frac{1}{\pi} \cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$$

$$\phi C_2 = \frac{1}{\pi} \cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda}\right)$$

where,

$$\lambda = \frac{\sqrt{1+\alpha_1^2} + \sqrt{1+\alpha_2^2}}{2} \quad \& \quad \lambda_1 = \frac{\sqrt{1+\alpha_1^2} - \sqrt{1+\alpha_2^2}}{2}$$

$$\alpha_1 = \frac{b_1}{d} = \frac{16.4}{6} = 2.73$$

b_1 = distance from intermediate to upstream
$= 15.8 + 0.6 = 16.4m$

$$\alpha_2 = \frac{b_2}{d} = \frac{40.6}{6} = 6.77$$

b_2 = distance from intermediate to downstream
$= 40 + 0.6 = 40.6$

$d = 154 - 148 = 6m$

Now,

$$\lambda = \frac{\sqrt{1+2.73^2} + \sqrt{1+6.77^2}}{2} \quad \& \quad \lambda_1 = \frac{\sqrt{1+2.73^2} - \sqrt{1+6.77^2}}{2}$$

$\boxed{\lambda = 4.87}$ $\quad$ $\boxed{\lambda_1 = -1.97}$

Then,

$$\phi E_2 = \phi E_2' = \frac{1}{\pi} \cos^{-1}\left(\frac{-1.97-1}{4.87}\right)$$

$$= \frac{1}{\pi} \times \frac{\pi}{180} \times 127.57$$

$$= 0.708$$

$\boxed{\phi E_2' = 70.8\%}$

$$\phi C_2 = \frac{1}{\pi} \cos^{-1}\left(\frac{-1.97+1}{4.87}\right) = \frac{1}{\pi} \times \frac{\pi}{180} \times 101.488$$

$$= 0.5638$$

$\boxed{\phi C_2 = 56.38\%} = \phi C_2'$

$$\phi D_2 = \frac{1}{\pi} \cos^{-1}\left(\frac{-1.97}{4.87}\right) = \frac{1}{\pi} \times \frac{\pi}{180} \times 113.86$$

$$= 0.6325$$

$\boxed{\phi D_2 = 63.25\%}$

ϕE_2 & ϕC_2 should be Corrected for 3- Corrxn → Mutual interference
→ Floor thickness
→ slope

For ϕE_2

(a) Corrxn at E_2 due to sheet pile No. (1) ie mutual interference

D = depth of pile 1 = 153−148 = 5m

d = depth of pile-2 = 153−148 = 5m

b' = distance betn 2-pile = 15.8m

b = Total floor length = 57m

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

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

$$\boxed{\text{Corrxn} = 1.88\%}$$

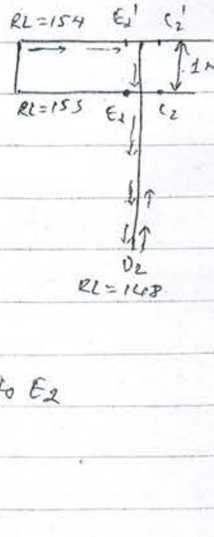
Since, E_2 is in the forward dirxn of flow from pile (1) So Corrxn is −ve

$C_1 \leftarrow E_2$

(ie flow dirxn is opposite so −ve.

$$\therefore \boxed{\text{Correction} = 1.88\% \ (-ve)}$$ //

(b) Corrxn at E_2 due to Floor thickness



$$\text{corrxn} = \frac{\text{observed } \phi E_2 - \text{observed } \phi D_2}{\text{Distance betn } E_2 D_2} \times \text{thickness of Floor}$$

$$= \frac{70.8 - 63.25}{153-148} \times (154-153)$$

$$\boxed{\text{Corrxn} = 1.26\%}$$

Here,

PressR at E_2' shall be more than E_2 as dirxn of flow is from E_2' to E_2

So Corrxn will be −ve

$$\therefore \boxed{\text{Corrxn} = 1.26\% \ (-ve)}$$ //

(c) Corrxn at E_2 due to slope

- Corrxn due to slope is zero. Since E_2 is neither at start nor at end of slope.

$\therefore$ Corrected ϕE_2 = observed ϕE_2 + Corrxn due to mutual interference + Corxn due to floor thick

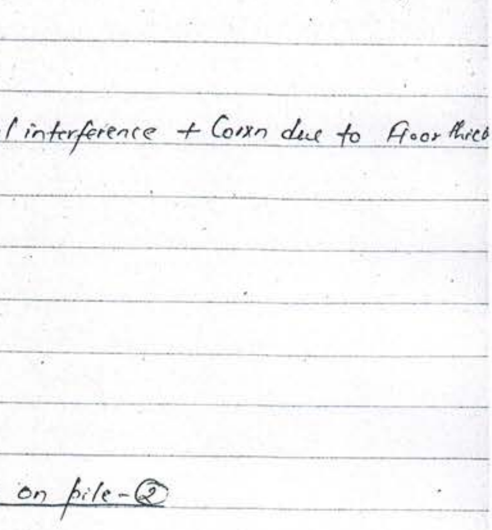
$= 70.8 + (-1.88) + (-1.26)$

$\boxed{\text{Corrected } \phi E_2 = 67.66\%}$ //Ans//

for ϕC_2

(a) Corrxn due to mutual interference of pile No-(3) on pile-(2)

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



D = depth of pile (3) $= 153 - 141.7 = 11.3m$

d = depth of pile-(2) $= 153 - 148 = 5m$

b' = distance betn 2 piles = 40m

b = Total floor length = 57m

Now,

$$\text{Corrxn} = 19\sqrt{\frac{11.3}{40}}\left(\frac{11.3+5}{57}\right)$$

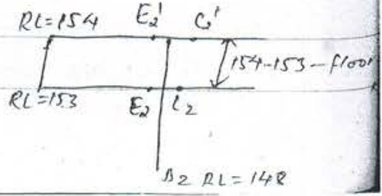
$\boxed{\text{Corrxn} = 2.88\%}$

Corrxn is +ve because in same flow dirxn.

$\therefore$ $\boxed{\text{Corrxn} = 2.88\% \ (+ve)}$ //

(b) Corrxn due to floor thickness

$$\text{Correction} = \frac{\text{obs. } \phi D_2 - \text{obs. } \phi C_2}{\text{distance bet}^n \ D_2 C_2'} \times \text{floor thickness}$$



Note:- slope Corrxn is essential only at begining & end of slope

$$Corrxn = \frac{63.25 - 56.38}{154 - 148} \times (154 - 153)$$

$$\boxed{Corrxn = 1.145\%}$$

Here, dirxn of flow is from C_2 to C_1', So Corrxn is +ve

$$\therefore \boxed{Corrxn = 1.145\% \ (+ve)}$$

© Corrxn for slope at C_2

Given, slope = 3:1

Since, C_2 is at start point of slope 3:1 So slope Corrxn is needed

ie slope is in dirxn of flow - so the Corrxn is -ve

Here,

Corrxn factor for 3:1 is 4.5

Horizontal length of slope = 3m = b_s

distance betn 2-pile beth which slope floor is located = 40m = b'

Table

slope (H:V)	Corrxn factor (C.F.)
1:1	11.2
2:1	6.5
3:1	4.5
4:1	3.3
5:1	2.8
6:1	2.5

$$\therefore \text{Actual Corrxn} = \text{corrxn factor (CF)} \times \frac{\text{Horizontal distance } (b_s)}{\text{distance betn 2-pile } (b')}$$

$$= 4.5 \times \frac{3}{40}$$

$$\boxed{Corrxn = 0.3375\%}$$

Corrxn is -ve because slope is upward.

$$\therefore \boxed{Corrxn = 0.3375\% \ (-ve)}$$

Now,

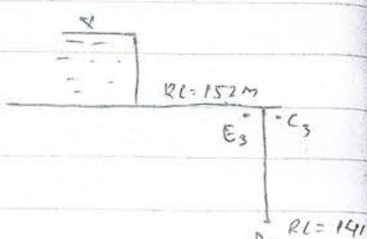
Corrected ϕC_2 = ϕC_2 + Corrxn due to mutual Interference + Corrxn due to floor thickness + Corrxn for slope

$= 56.38 + 2.88 + 1.145 + (-0.3375)$

$$\boxed{\text{Corrected } \phi C_2 = 60.06\%}$$

// Ans //

Ⓒ For Downstream pile No. 3



$$\phi E_3 = \frac{1}{\pi} \cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$$

$$\phi D_3 = \frac{1}{\pi} \cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right)$$

$$\alpha = \frac{b}{d} = \frac{57}{10.3} = 5.53$$

$b = 57\,m$ = total floor thicknes

$d = 152 - 141.7 = 10.3$

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} = \frac{1 + \sqrt{1 + 5.53^2}}{2}$$

$$\boxed{\lambda = 3.31}$$

Now,

$$\phi E_3 = \frac{1}{\pi} \cos^{-1}\left(\frac{3.31 - 2}{3.31}\right) = \frac{1}{\pi} \times \frac{\pi}{180} \times 66.68$$

$$\phi E_3 = 0.3704$$

$$\boxed{\phi E_3 = 37.04\%}$$

and,

$$\phi D_3 = \frac{1}{\pi} \cos^{-1}\left(\frac{3.31 - 1}{3.31}\right) = \frac{1}{\pi} \times \frac{\pi}{180} \times 45.74$$

$$= 0.2541$$

$$\boxed{\phi D_3 = 25.41\%}$$

↳ ϕE_3 is to be Corrected for Corrxn
- → mutual interference
- → Thickness of floor
- ↳ slope

Corrxn for ϕE_3

ⓐ Corrxn for mutual interference of pile-② on pile-③

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

D = depth of pile-② whose effect is Considered

$= 150.7 - 148$

$= 2.7m$

d = depth of pile-③ $= 150.7 - 141.7 = 9m$

b = Total floor thickness $= 57m$

b' = distance betn 2-pile = $40m$

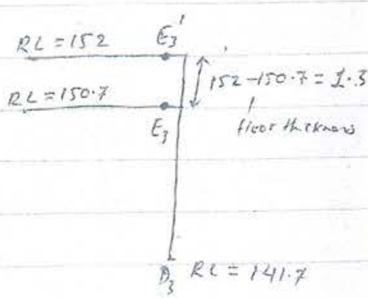
$$\therefore \quad Corrxn = 19\sqrt{\frac{2.7}{40}}\left(\frac{9+2.7}{57}\right)$$

$$\boxed{Corrxn = 1.01\%}$$

Corrxn is -ve because flow dirxn is opposite

$$\therefore \quad \boxed{Corrxn = 1.01\% \; (-ve)}$$

ⓑ Corrxn for floor thickness



$$Corrxn = \frac{obs.\ \phi E_3' - obs.\ \phi D_3}{\text{distance betn } E_3'\ D_3} \times \text{floor thickness}$$

$$= \frac{37.04 - 25.41}{152 - 141.7}(152 - 150.7)$$

$$\boxed{Corrxn = 1.47\%}$$

Flow dirxn is from E_3' to E_3 so -ve Corrxn

$$\therefore \quad \boxed{Corrxn = 1.47\% \; (-ve)}$$

© Corrxn for slope

Slope Corrxn is zero (i.e not needed) because sheet pile is in horizontal plane not in slope.

No slope corrxn needed (due to horizontal)

Now,

Corrected $\phi E_3 = \phi E_3$ + Corrxns

$= 37.04 + (-1.01) + (-1.47)$

$$\boxed{\text{Corrected } \phi E_3 = 34.56\%}$$

// Ans//

② Exist Gradient (G_E)

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

where,

$$\boxed{\lambda = \frac{1 + \sqrt{1+\alpha^2}}{2}}$$

and, $\boxed{\alpha = \frac{b}{d}}$

H = Total head difference from HFL to downstream slab or max^m seepage head

d = depth of downstream cutoff (sheet pile with slab)

Here,

$H = 158 - 152$

$H = 6m$

b = Total floor length = 57m

$d = 152 - 141.7 = 10.3m$

$\alpha = \frac{b}{d} = \frac{57}{152 - 141.7}$

$= \frac{57}{10.3}$

$$\boxed{\alpha = 5.53}$$

$$\lambda = \frac{1+\sqrt{1+5.53^2}}{2}$$

$$\boxed{\lambda = 3.30}$$

Now,

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

$$= \frac{6}{10.3} \times \frac{1}{\pi\sqrt{3.30}}$$

$$\boxed{G_E = 0.102}$$ //

Khosla's Safe Exist Gradient

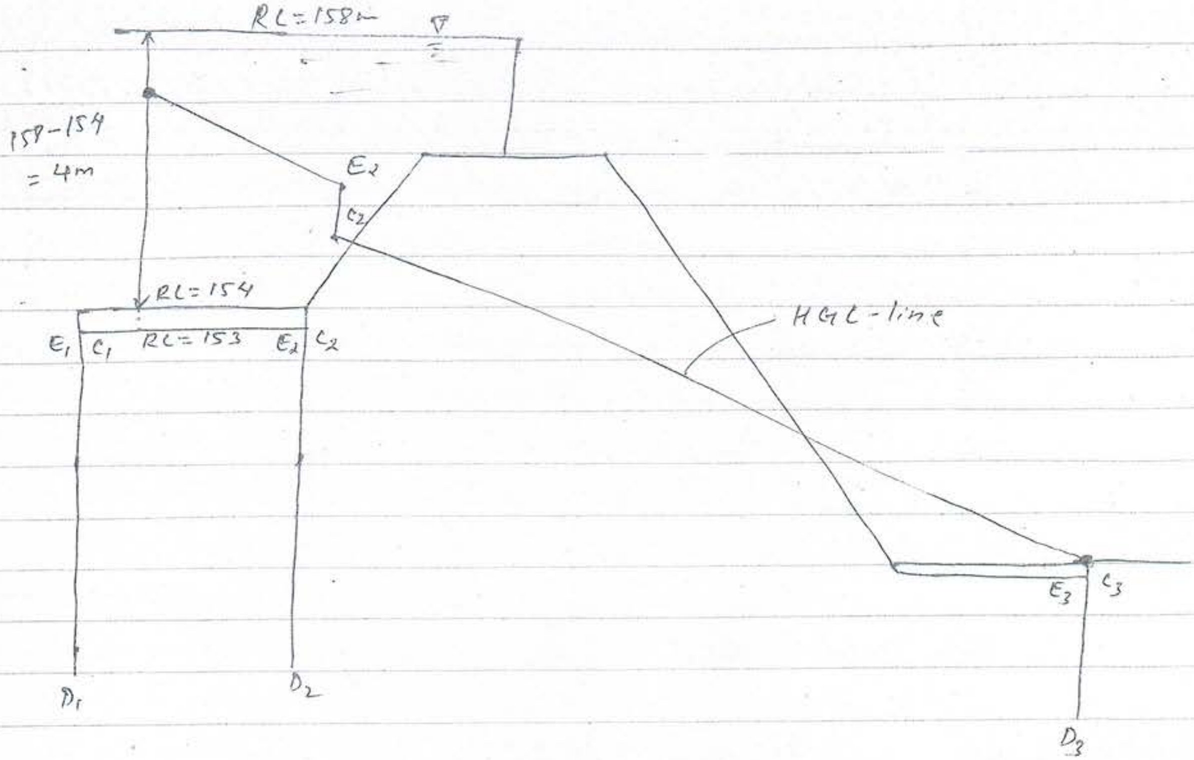
Type of Soil	Khosla safe gradient
(Boulder) shingle	$\frac{1}{4}$ to $\frac{1}{5}$ (0.25 to 0.20)
Coarse Sand	$\frac{1}{5}$ to $\frac{1}{6}$ (0.20 to 0.17)
fine Sand	$\frac{1}{6}$ to $\frac{1}{7}$ (0.17 to 0.14

Here, $G_E = 0.102 \ll 0.14$ (ie Small value)

so safe // ie Very much safe Comparing to above values

ok //.

(3) Hydraulic gradient line (HGL) - Graph



Here,

$$\phi C_1 = 71.31$$

Press R head at $C_1 = H \times \phi C_1$

$$= 4 \times \frac{71.4}{100}$$

$$= 4 \times 0.714$$

$$= 2.85 \text{ sec.}$$

Different types of Canal outlets OR Modules

- A Canal outlets or module is a small structure built at the head of watercourse so a to connect it with a minor or a distributary channel.

Requirements of Good Module

(1) The module should fit well to the decided principlas of water distributn.
(2) The module should be simple. so that it can be easily Constructed or fabricated by local masons or ~~techni~~ technicians.
(3) Should Work efficiently with small working heast.
(4) Should be Cheaper
(5) should be strong
(6) should draw its fair share of silt.

Types of Outlets or Modules - 3 types

(1) Non-modular outlets - These are those whose discharge depends upon the head difference of distributary & water Course.
eg. open sluice, drowned pipe outlet.

(2) Semi-modules OR Flexible modules - These are those whose discharge depends only upon water level in distributary & independent of water Course.
eg. Pipe outlet, Venturi flume, open flume, orfice semi-module.

(3) Rigid Module or Modular outlets - These are those whose dicharge is Constant and fixed within limits.
eg. Gibb's module

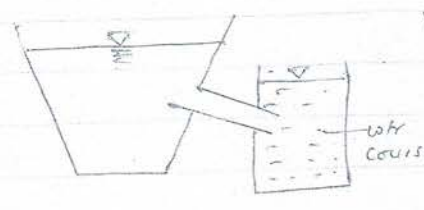


Fig:- Non-modular

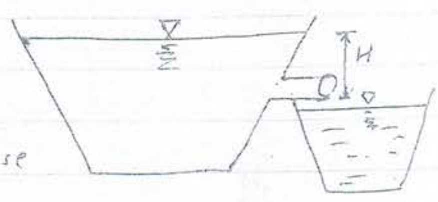


Fig :- Semi-modules

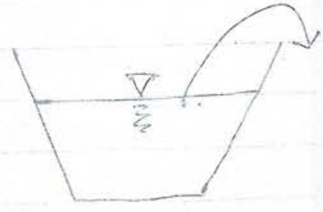
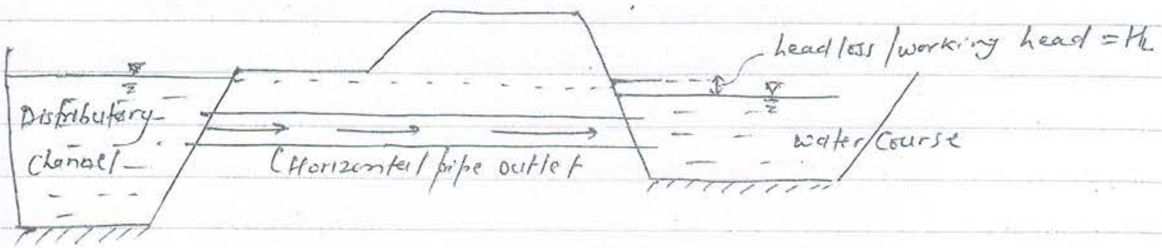


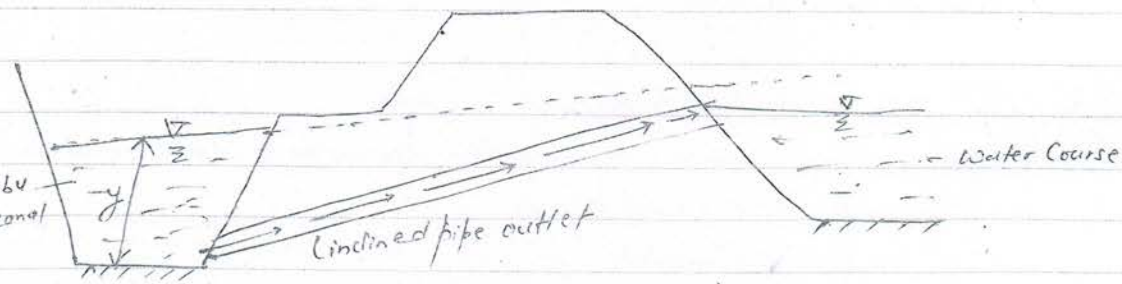
Fig :- Rigid module

Design of pipe outlet → Free and → Submerged

① Design of Sub-merged pipe outlet

H_L = Head loss





Here,

H_L = Total loss of head

① H_L = Entry loss + frxnl loss + Velocity head at exist

$$= \frac{0.5V^2}{2g} + \frac{flV^2}{2gd} + \frac{V^2}{2g}$$

$$\boxed{H_L = \frac{V^2}{2g}\left[0.5 + \frac{fl}{d} + 1\right]}$$

H_L = ~~head~~ difference in wtr level of distributary and wtr Course

l = length of pipe

d = diamtr of pipe

f = Coefficient of frxn

(2) Discharge (q) = $V \times A$ $\qquad$ A = Area of outlet pipe $= \frac{\pi d^2}{4}$

For all practical purposes discharge is given as,

$$\boxed{q = C_d \cdot A \sqrt{2 g H_L}}$$

where, $\qquad$ A = Area of pipe $= \frac{\pi d^2}{4}$

q = Discharge through the outlet

C_d = Cofficient of discharge.

$= 0.73$

(3) Design of free pipe outlet

- Pipe outlet discharging freely into atmosphere
- Simplest & oldest type of outlet
- Discharge is given by,

$$\boxed{Q = C_d\, A \sqrt{2 g H_0}}$$

$C_d = 0.62$ = Coefficient of discharge

H_0 = Head on upstream side measured from FSL of distributary upto centre of pipe.

Numericals

(1) Design an irrigatn outlet for the following data

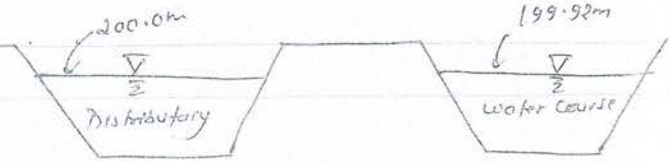
Full supply discharge of outlet (FSQ) = 50 lit/sec

FSL in distributary on u/s side of outlet = 200.0m

FSL in water course on d/s side of outlet = 199.92m

Full supply Depth (FSD) in distributary on u/s side of outlet = 1.05m

Soln.



we know, $\boxed{Q = C_d A \sqrt{2 g H_L}}$ ——— (1)

Available head at outlet $(H_L) = 200 - 199.92$

$= 0.08m$

Since available head is very small No-modular outlet is suitable.

$C_d = 0.73$

$Q = 50$ lit/sec $= 0.05 m^3/s$

then, from eqn (1)

$$0.05 = 0.73 \times \frac{\pi d^2}{4} \sqrt{2 \times 9.81 \times 0.08}$$

$$A = \frac{\pi d^2}{4}$$

$\boxed{d = 0.263m}$

∴ Use a pipe of diameter $(d) = 0.3m$

$= 30cm$ // Ans //

RL bed of distributary $= 200 - 1.05$

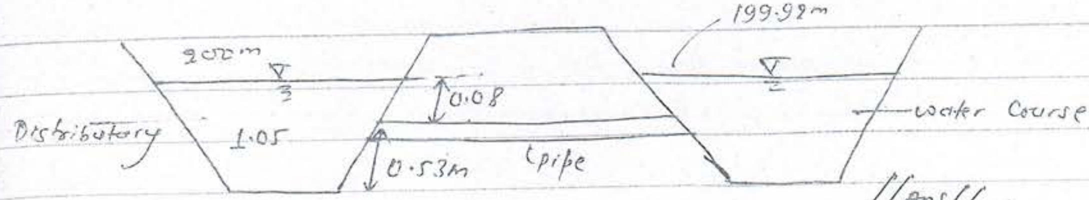
$= 198.95m$

Pipe can be laid horizontal at $= 200 - 0.22 - 0.30$

$= 199.48m$

Pipe top fixed at 22
$= 0.22m$ below FSL

ie at $199.48 - 198.95 = 0.53m$ above the bed of distributary



//Ans//.

② Design a pipe outlet for following data
Full supply discharge at the head of water Course = 90 lit/sec
FSL in distributary = 205.0m
FSL in water Course = 204m

Soln.

we have,

$$\boxed{Q = C_d A \cdot \sqrt{2gH_o}} \quad \text{———} \quad (1)$$

Available head (H_L) = 205 − 204
= 1m

$C_d = 0.62$

Assume,

diameter of pipe (d) = 25cm = 0.25m

Then,

Area of pipe (A) = $\frac{\pi d^2}{4} = \frac{\pi \times 0.25^2}{4}$

= 0.049 m²

$Q = 90 \text{ lit/sec} = \frac{90}{1000} \frac{m^3}{sec}$

$Q = 0.09 m^3/s$

Now from eqn (1),

$$0.09 = 0.62 \times 0.049 \sqrt{2 \times 9.81 \times H_o}$$

$$\boxed{H_o = 0.44m}$$

RL of Centre of outlet pipe = FSL of distributary $- H_o$

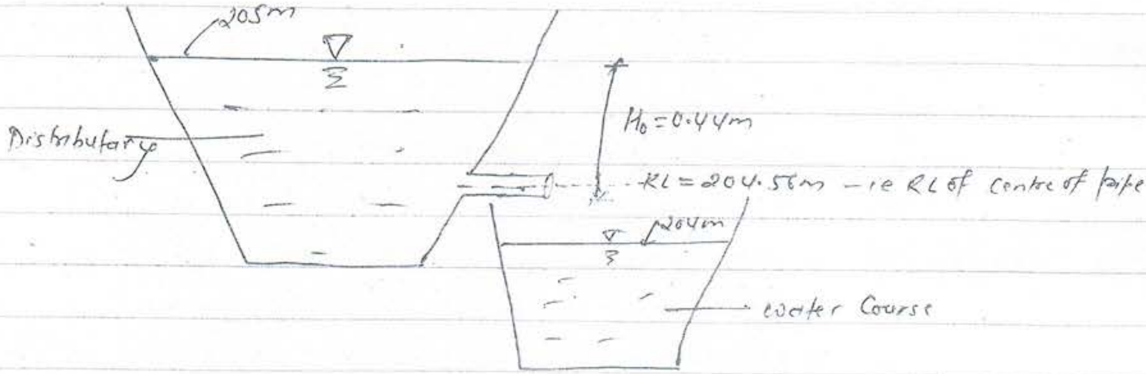
$= 205 - 0.44$

$= 204.56m$

RL. of invert of outlet pipe $= 204.56 - \frac{0.25}{2}$

$= 204.435 \gg$ FSL of wtr Course (204m).

ok//.



2072/08/20 Sunday

⇒ Er. Umesh Raut

(1) Cross Drainage structures (C/D-strxtrs)

A cross drainage is a structure constructed at the crossing of a canal and the natur drains to dispose drainage water without disturbing the continuous canal supply.

Types of Cross-Drainage (C/D) works

Depending upon relative level, cross-sxn discharges, C/D works are:

(A) Irrigation Canal passes over drainage
(1) Aqueduct
(2) Syphon-aqueduct

(B) Irrigation Canal passes below Drainage
(1) Super passage
(2) Syphon or Canal Syphon

(C) Drainage and Canal intersection at same level
(1) Level Crossing
(2) Inlet and outlet

(A)
(1) Aqueduct :- when bed level of the canal is above High Flood level (HFL) of the drain, the structure is known as aqueduct.

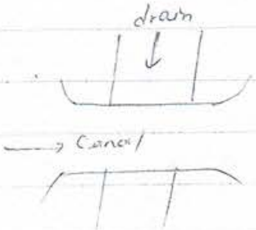


Fig: Aqueduct
(when drain HFL is lower than the canal bed level)

Fig: Syphon aqueduct
(Drain HFL is higher than the canal bed level)

(i) Factors to be taken into account while selecting the suitable type of cross drainage works;

(2) Syphon Aqueduct :- When HFL of the drain is higher than the Canal bed level then Syphon aqueduct is provided.

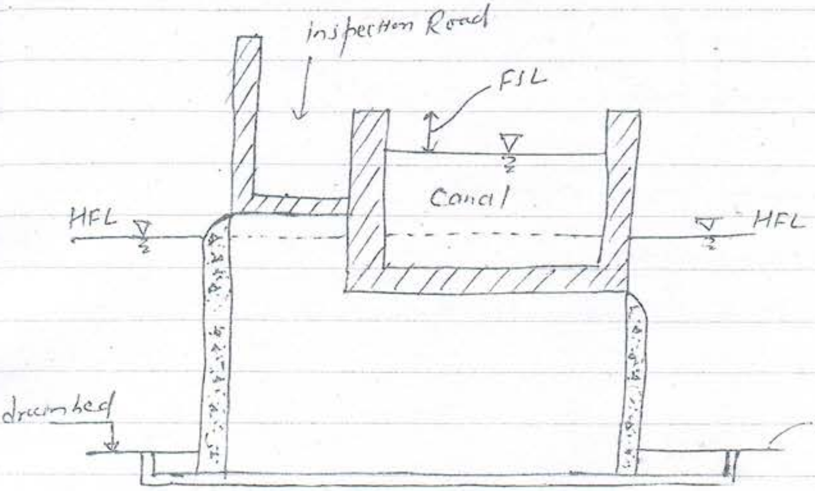
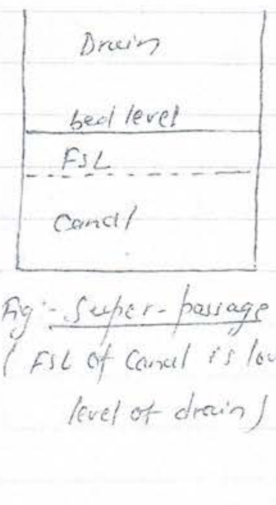
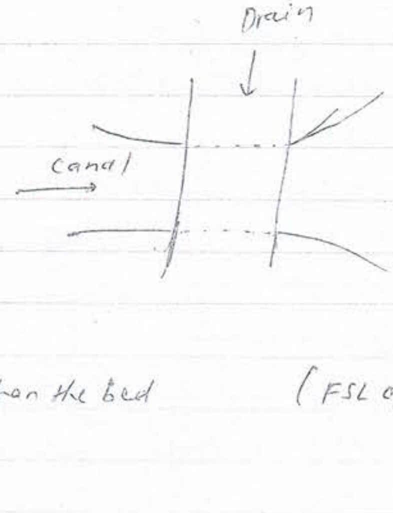


Fig:- Syphon aqueduct

(B) Irrigatal Canal passes below drainage

(i) Super-passage :- when FSL of Canal is lower than the bed level of drain, then it is Super passage.





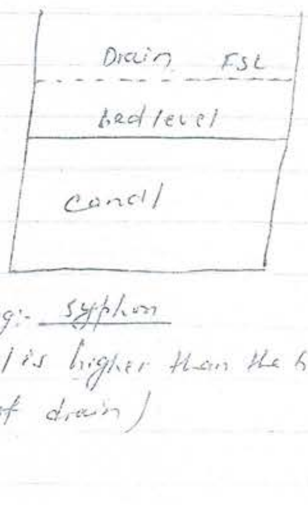


Fig:- Super-passage
(FSL of Canal is lower than the bed level of drain)

Fig:- syphon
(FSL of Canal is higher than the bed level of drain)

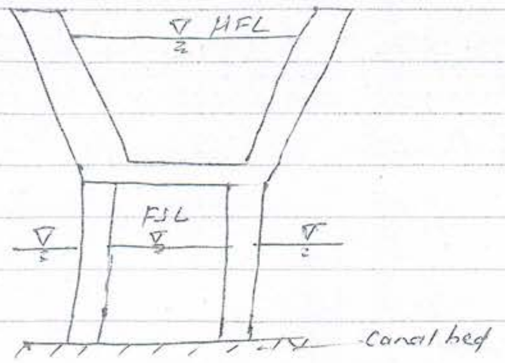


Fig:- Super-passage

(ii) Syphon / Canal syphon :- In this structure, FSL of canal is higher than the bed level of drain.

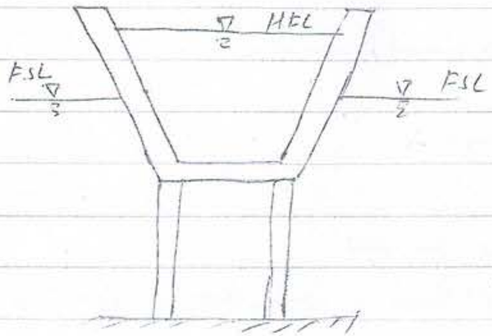
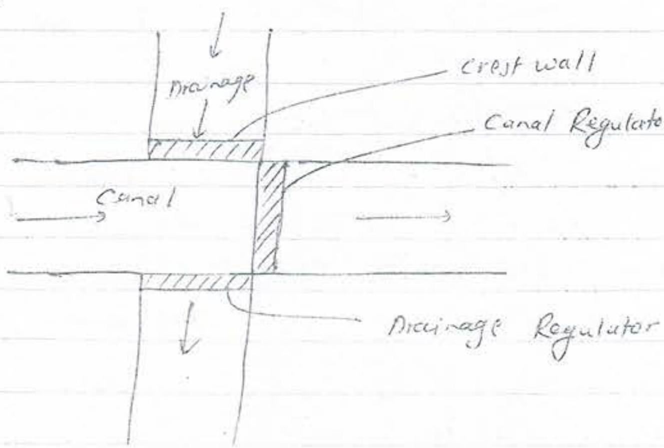


Fig:- Syphon

(C) Drainage and Canal intersxn at same level

(i) Level crossing - It is provided when canal bed level and drain bed level is same.



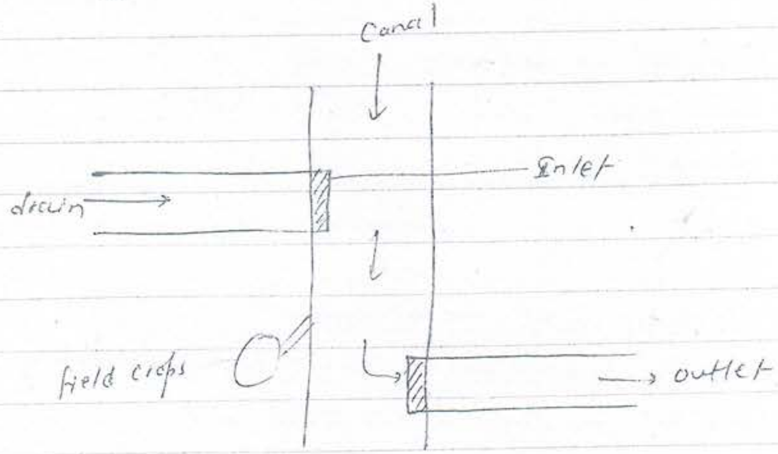
Advantage

- low initial Cost
- The FSL of Canal Can be maintained from the discharge through drain in Dry weather flow
- Additional discharge can be supplied to Canal from drainage.

Disadvantage

- Regulation of work is difficult
- Faulty regulatl of gates may damage Canal.
- There is additional expenditure for silt clearance.

(2) Inlet and outlet :- If the admitted drainage wtr in the Canal is discharged at a suitable site opposite to inlet or on downstream the arrangement is Called inlet and outlet.



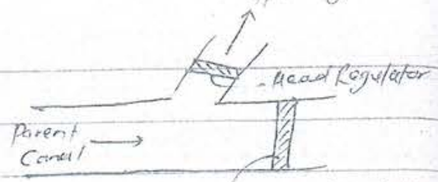
(2) Canal Regulators
- (1) Cross - Regulators
- (2) Distributory Load Regulator

(3) Drop structure - Canal falls

(4) Canal escape (used)
- (1) weir type
- (2) Regulator type

(5) Canal outlets as Modules

Distributary Head Regulator and Cross Regulator



Distributary head Regulator

A head Regulator provided at the head of distributary and Controls the supply entering the distributary is called Distributary head Regulator.

Function

1. Regulate and Control the supply of water into the distributary
2. Serve as a meter for measuring discharge
3. Controls silt entry into the off-taking Canal.
4. help in shutting of supplies when not needed

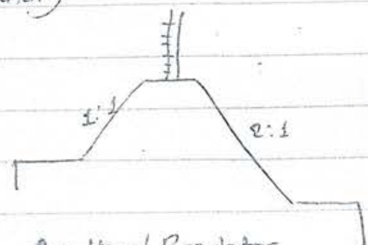


fig: Head Regulator

CROSS Regulator

A Cross Regulator is provided at downstream of main Canal to head up the water level and to enable the off-taking canal to draw the required supply.

- Controls the supply of parent Canal.

Function

1. Effectively Control the entire Canal Irrigation System
2. when the water level in the parent Canal is low, it helps in heading up water on the upstream & to feed the off take Canal to their full demand
3. helps in closing the supply to down stream for Repairs.
4. Bridge, and other Communication's work Can be Combined with it.
5. Controls silt entry into the downstream

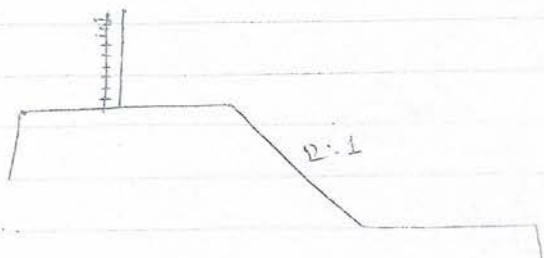


fig:- Cross Regulator

Numericals : Design of Cross Regulator & Head Regulators

(I) Design a Cross Regulator (Cross Regulator) and head Regulator for following data

(1) Discharge of parent channel (canal) = $140 m^3/sec$

(2) Discharge of distributary = 15 cumec

(3) FSL of parent u/s = 210 m

(4) FSL of parent d/s = 209.8 m

(5) Depth of water in parent canal u/s and d/s = 2.5 m

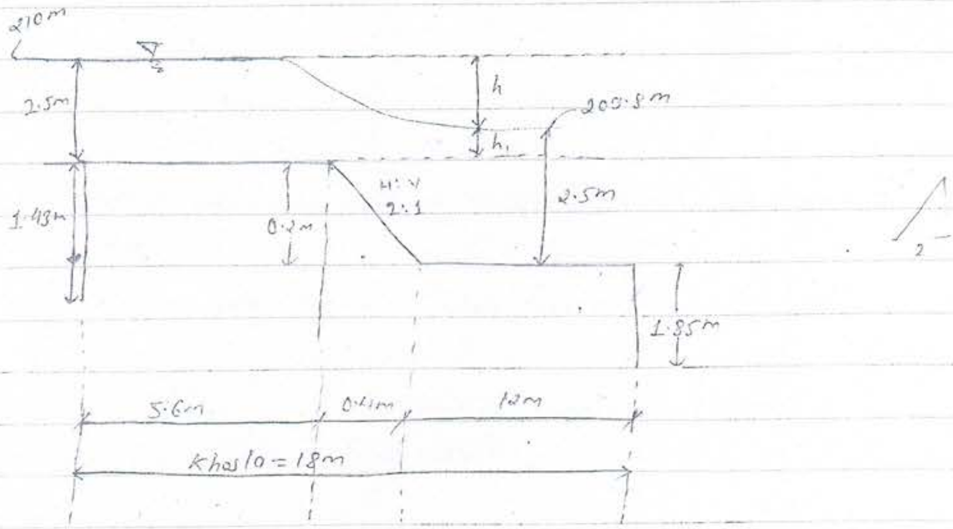
(6) Silt factor (f) = 0.8 m

(7) Assume safe exist gradient = 1/5

(8) Bed width of parent channel (canal) u/s = 52 m

(9) " " " " " d/s = 46 m

(10) FSL of distributary = 209.1 m



(A) Design of Cross Regulator

Crest levels : Crest level of cross regulator is kept the same as bed level of parent canal which is equal to = 210 − 2.5

= 207.5 m

Provide crest at RL = 207.5 m //

For width, water way,

$$\boxed{Q = B\sqrt{h}\,(1.69h + 3.54h_1)} \quad \text{——— (1)}$$

$h =$ u/s FSL $-$ d/s FSL $= 210 - 209.8 = 0.2m$

$h_1 =$ d/s FSL $-$ crest level $= 209.8 - 207.5 = 2.3m$

or $2.5 - 0.2 = 2.3$

$B =$ clear waterway required $= ?$

$Q = 140 m^3/s$

Then from eqn (1),

$$140 = B\sqrt{0.2}\,(1.69 \times 0.2 + 3.54 \times 2.3)$$

$$B = 36.91m$$

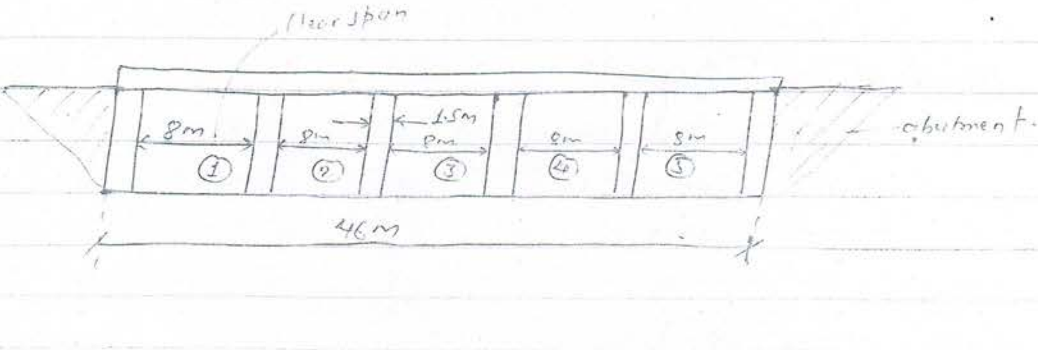
$$\boxed{B \simeq 40m}$$

Provide,

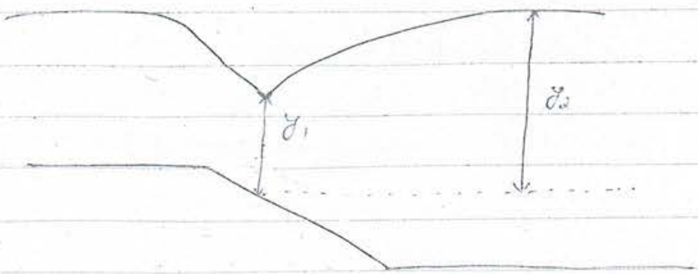
5 bays of 8m each with a clear water way $= 5 \times 8$
$= 40m$

Provide 4 piers of 1.5m width each $= 4 \times 1.5 = 6m$

$\therefore$ Overall water way $= 40 + 6$
$= 46m$ //



From the Concept of hydraulic Jump



length of Jump / length of d/s Floor required = $5(y_2 - y_1)$

Head loss $(H_L) = 0.2$

Discharge per unit width, (q) or downstream floor level

$$q = \frac{Q}{B} = \frac{140}{40}$$

$B = 40m$

$Q = 140m^3/s$

$$\boxed{q = 3.5 \text{ cumecs/m}}$$

Here,

$$\boxed{y_1 y_2 (y_2 + y_1) = \frac{2q^2}{g}} \quad \text{———— (2)}$$

and,

$$\boxed{H_L = \frac{(y_2 - y_1)^3}{4 y_1 y_2}} \quad \text{———— (3)}$$

from eqn (2)

$$y_1 y_2 (y_2 + y_1) = \frac{2 \times 3.5^2}{9.81}$$

$$\boxed{y_1 y_2 (y_2 + y_1) = 2.497} \quad \text{———— (4)}$$

from eqn (3)

$$\boxed{0.2 = \frac{(y_2 - y_1)^3}{4 y_1 y_2}} \quad \text{———— (5)}$$

let, $\frac{y_2}{y_1} = x$

$y_2 = xy_1$

then from eqn (4)

$$y_1 \cdot xy_1 \, (xy_1 + y_1) = 2.497 \qquad (y_2 = xy_1)$$

$$\boxed{xy_1^3 \, (x+1) = 2.497} \quad \text{——— (6)}$$

from eqn (5)

$$0.2 = \frac{(xy_1 - y_1)^3}{4y_1 \cdot xy_1}$$

$$0.2 = \frac{y_1^3 \, (x-1)^3}{4y_1^2 \cdot x}$$

$$\boxed{y_1 = \frac{0.8x}{(x-1)^3}} \quad \text{——— (7)}$$

Now from eqn (6) & (7) we get,

$$x \left(\frac{0.8x}{(x-1)^3}\right)^3 (x+1) = 2.497$$

$$\boxed{x = 2.425}$$

Putting value of x in eqn (7),

$$y_1 = \frac{0.8 \times 2.425}{(2.425 - 1)^3}$$

$$\boxed{y_1 = 0.67 m}$$

and,

$$y_2 = xy_1 = 2.425 \times 0.67 = 1.625$$

Now,

length of d/s Floor Required $= 5(y_2 - y_1)$

$= 5(1.625 - 0.67)$

∴ length of d/s floor required $\cong$ 5m

If this length Comes Smaller in Comparison to $\frac{2}{3}$ of total floor length worked out by exist gradient Consideration ($b = \alpha . d$)

Then, the length of downstream floor is kept equal to $\frac{2}{3}$ of total floor length

Now,

Depth of Vertical Cutoffs

For upstream (u/s)

$$depth = \frac{y_u}{3} + 0.6 \qquad (y_u = \text{upstream wtr depth})$$

$$= \frac{2.5}{3} + 0.6$$

$$= 1.43m$$

For downstream (d/s)

$$depth = \frac{y_d}{2} + 0.6 \qquad (y_d = \text{d/s wtr depth})$$

$$= \frac{2.5}{2} + 0.6$$

$$= 1.85m$$

Total floor length from Exist gradient Consideration,

$$\boxed{G_E = \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}}} \quad \text{——— (8)}$$

H = u/s FSL − d/s bed level

$= 210 - 207.3$

$= 2.7m$

d = depth of d/s cutoff = 1.85m

$G_E = \frac{1}{5}$ (Given)

Then eqn (8) becomes,

$$\frac{1}{5} = \frac{2.7}{1.85} \times \frac{1}{\pi\sqrt{\lambda}}$$

$$\lambda = 5.395$$

$$\boxed{\lambda \simeq 5.4}$$

Now, we know,

$$\lambda = \frac{1 + \sqrt{1+\alpha^2}}{2}$$

$$5.4 = \frac{1 + \sqrt{1+\alpha^2}}{2}$$

$$\boxed{\alpha = 9.74}$$

and,

$$\alpha = \frac{b}{d}$$

$$b = \alpha \cdot d = 9.74 \times 1.85$$

$$b = 18.01$$

$$\boxed{b \simeq 18m}$$

Minimum d/s floor length required $= \frac{2}{3} \times b$

$= \frac{2}{3} \times 18$

$= 12m \gg 5(y_2 - y_1) = 5m$

ok//.

Hence,

Provide, 12m as d/s floor length //Ans//.

(B) Design of Head Regulator

$Q = 15m^3$

$f = 0.8$

Sol^n.

$$V = \left(\frac{Qf^2}{140}\right)^{1/6} = \left(\frac{15 \times 0.8^2}{140}\right)^{1/6}$$

$$\boxed{V = 0.64 m/s}$$

$$A = Q/V = \frac{15}{0.64}$$

$$\boxed{A = 23.43 m^2}$$

$$P = 4.75\sqrt{Q} = 4.75\sqrt{15}$$

$$\boxed{P = 18.4 m}$$

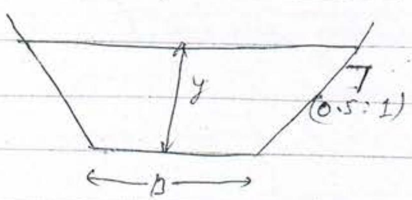
For Trapezoidal Canal with 0.5H to 1V (0.5:1)

$$P = b + \sqrt{5} \cdot y$$

$$18.4 = b + \sqrt{5} \cdot y$$

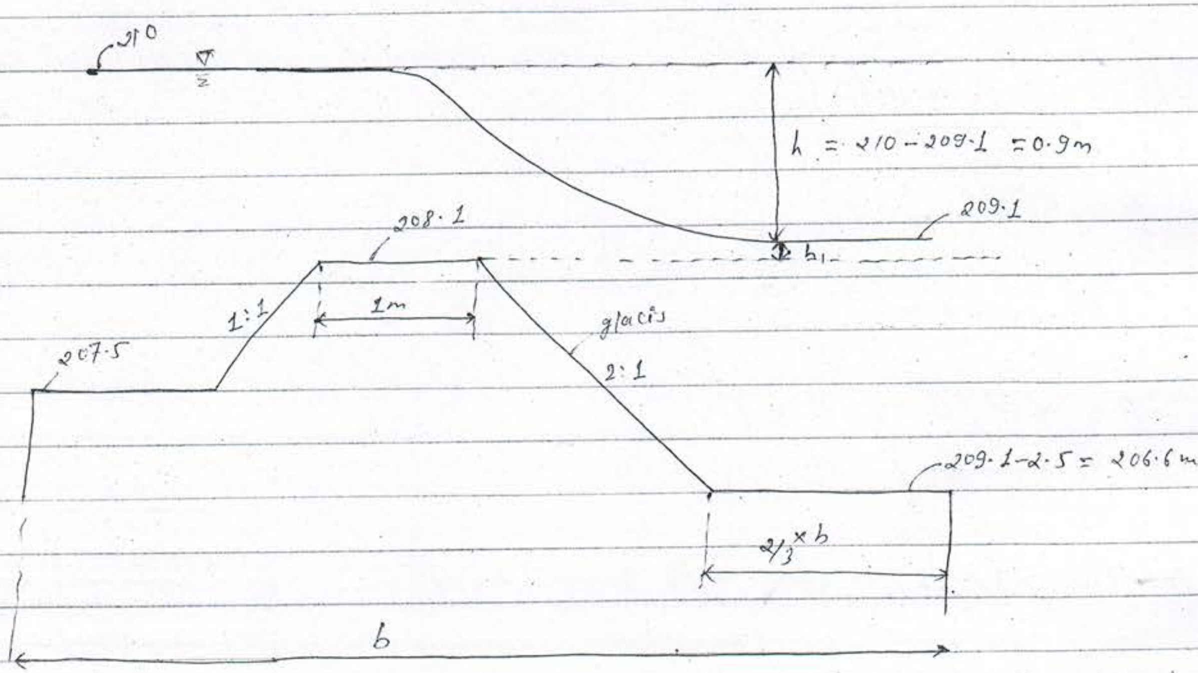
And,

$$A = \left(b + \frac{y}{2}\right) \cdot y$$



Bed level of distributary Head Regulator at u/s = 207.5

The crest level of distributory head is generally kept 0.3 to 1m higher than the bed level of parent Canal



Let us keep it 0.6 m higher

$\therefore$ Crest level $= 207.5 + 0.6$

$= 208.1m$

Hydraulic Jump [illegible] slope [illegible] glacis

H : V

Provide slope of upstream glacis 1 : 1 & downstream glacis 2 : 1

$\therefore$

$$\boxed{\text{Water way } (Q) = B\sqrt{h}\,(1.69h + 3.54\sqrt{h_1})} \quad \text{——— } (1)$$

Here, $Q = 15\,m^3/s$

$h = 210 - 209.1 = 0.9m$

$h_1 = 209.1 - 208.1 = 1m$

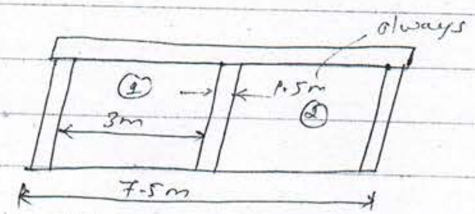
then from eqn (1),

$$15 = B\sqrt{0.9}\,(1.69 \times 0.9 + 3.54 \times 1)$$

$$\boxed{B = 3.124\,m}$$

if 3m भन्दा ठूलो आयो भने दुई वटा Bays लिने But spacing of Canal को width 3m भन्दा कम लिन पाइँन

Provide, 2 bays of 3m width each

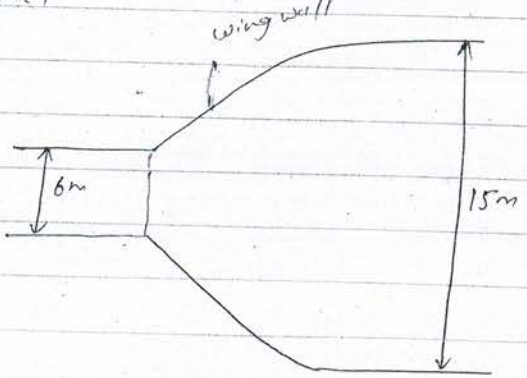


Clear water way = 2 × 3 = 6m

Provide 1 pier of 1.5m width each = 1 × 1.5 = 1.5m

∴ Overall water way = 6m + 1.5m
= 7.5m

The wing wall shall be expanded with proper divergence, so as to provide the normal width of channel



Depth of u/s vertical cut off $= \frac{y_4}{3} + 0.6$

$= \frac{2.5}{3} + 0.6$

Depth of d/s vertical Cutoff $= \frac{Y_d}{2} + 0.6$

$$= \frac{2.5}{2} + 0.6$$

$$= 1.85\,m$$

Total floor length from exist Gradient Considertn,

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

where,

$$H = 210 - 206.6 = 3.4\,m$$

$$G_E = \frac{1}{5} \quad , \; d = 1.85\,m$$

Then,

$$\frac{1}{5} = \frac{3.4}{1.85} \times \frac{1}{\pi\sqrt{\lambda}}$$

$$\boxed{\lambda = 8.55}$$

Now, $\lambda = \frac{1 + \sqrt{1+\alpha^2}}{2}$

$$8.55 = \frac{1 + \sqrt{1+\alpha^2}}{2}$$

$$\boxed{\alpha = 16.06}$$

Now, $\alpha = \frac{b}{d}$

$$b = \alpha \cdot d = 16.06 \times 1.85$$

$$\boxed{b = 29.72\,m}$$

$\therefore$ Provide 29.72m as d/s floor length. //Ans//.

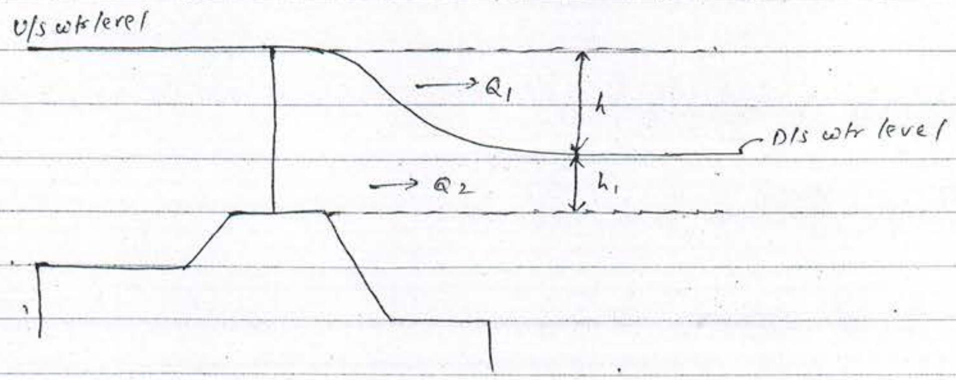
Design of Cross Regulator and Distributary Head Regulator

① Crest level - The crest of **cross Regulator** is generally kept at the upstream bed level of Canal.

- Crest level of distributary **Head** Regulator is generally kept 0.3 to 1.0m higher than the crest level of cross Regulator

② Water way

$$\boxed{Q = B\sqrt{h}\,(1.69h + 3.54h_1)}$$



③ Downstream Floor level OR Cistern level

$$q = \frac{Q}{B}$$

$$H_L = U/s\ FSL - D/s\ FSL$$

also, from hydraulic Jump,

$$y_1 y_2 (y_1 + y_2) = \frac{2q^2}{g} \quad \text{———— ①}$$

&

$$H_L = \frac{(y_2 - y_1)^3}{4y_1 y_2} \quad \text{———— ②}$$

From eqn ① & ② find Value of y_1 & y_2

Then,

$$E_f = y_1 + \frac{V_1^2}{2g}$$

and, $E_{f_2} = y_2 + \frac{V_2^2}{2g}$

where,

$$V_1 = \frac{q}{y_1} \quad \&$$

$$V_2 = \frac{q}{y_2}$$

Now,

$\therefore$ $\boxed{\text{length of d/s floor or RL of cistern} = \text{d/s FSL} - E_{f_2}}$

④ Length of d/s Floor $= 5(y_2 - y_1)$

⑤ Vertical Cut-offs / Depth of Vertical Cut-offs

ⓐ upstream Cut-off depth $= \frac{y_u}{3} + 0.6$

ⓑ downstream Cut-off depth $= \frac{y_d}{2} + 0.6$

y_u = depth of water in upstream channel

y_d : depth of water in downstream channel

⑥ Total floor length from Exist Gradient (b)

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

$$\lambda = \frac{1 + \sqrt{1+\alpha^2}}{2}$$

$$\alpha = \frac{b}{d}$$

Minimum length of d/s floor (Cistern) $= \frac{2}{3}$ of b

Er. Umesh Raut

Escapes and Their types

Canal Escapes / Escapes - These are the structure constructed to remove surplus water from an irrigation channel onto a natural drain are called Canal escapes.

Types of Escapes - 3 types

① weir type / Tail escape

- in this type, crest of weir = level of crest or canal FSL
- Provided at tail end of canal so called Tail escape & is useful in maintaining the required FSL in tail end.

② Regulator type (sluice type) / scouring escape

- Used for the purpose of scouring of excess silt from time to time.

③ Surplus escapes

- Used to remove excess or surplus water from the canal.

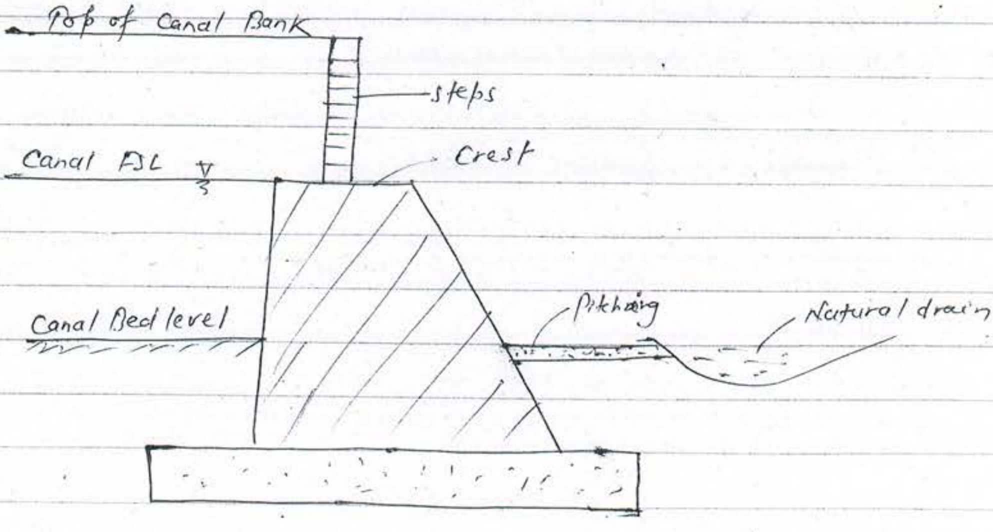


Fig:- Weir type escape or Tail escape

Selection of Suitable Type of Drainage works (Condtns of appliktn)

Primary factors affecting suitable s/xns are :-

(1) Relative bed levels, and water levels of Canals and drainage

(2) Relative discharge of Canal and drainage (ie size of Canal and drainage)

Following Consideratns are important for selection of suitable type of cross drainage s/xn

) When bed level of Canal is much above the HFL of drainage so that sufficient headway is available for floating rubbish etc. then aqueduct is selected

) If the bed level of drainage is well above the FSL of Canal, then super passage is provided

) When drainage bed is much lower but HFL of drainage is higher than the bed level of canal, Syphon aqueduct is provided.

) When the bed level of Canal is much lower than that of drainage and FSL of Canal is ?er than the bed level of drainage, Canal Syphon is provided.

) When drainage and the Canal Crosses practically at the same level then level crossing may be used.

) If $Q_{canal} \gg Q_{drainage}$ → syphon (Canal syphon)

) If $Q_{canal} \ll Q_{drainage}$ → Syphon aqueduct

However in Actual field, Such ideal Condtns may not be available So other factors are :-

(1) Suitable Canal alignment

(2) Topography

(3) Ground water Table position

(4) Foundation strata

(5) Sediment characteristics

) Loss of head in Canal

) Availability of fund

Different Types of hydraulic Structures

(1) Dam
(2) Weir
(3) Barrage
(4) Cross-drainage works (Aqueduct & syphon)
(5) head Regulator
(6) Escapes
(7) falls etc.

falls / Canal falls / Canal drops

A fall or a drop is an irrigatnl structure Constructed across the canal to lower down it water level and destroy the Surplus energy in order to prevent the scour of downstream bed and banks of the Canal.

Hence Canal Fall is a type of energy dissipating structure.

Location of a Fall

↳ Fall should be located at a place where cost of excavatnl of channel will balance the Cost of filling.

↳ Before the bed of canal Comes in filling

↳ At a place where it may serve the Combining service of regulator and a bridge as far as possible

↳ Economic feasibility - wheather large no. of small falls or small no. of large falls will be economically Viable.

Types of Falls

(1) Ogee falls
(2) Rapid Fall
(3) Trapezoidal Notch fall

4) Glacis type fall
5) Simple vertical drop. /sarda type fall
~~6) well type fall~~
7) well type falls or syphon well Drops
8) Stepped fall
9) Meter and Non-meter falls
10) Inglis Falls or Baffle falls

1) Ogee Falls - old technology

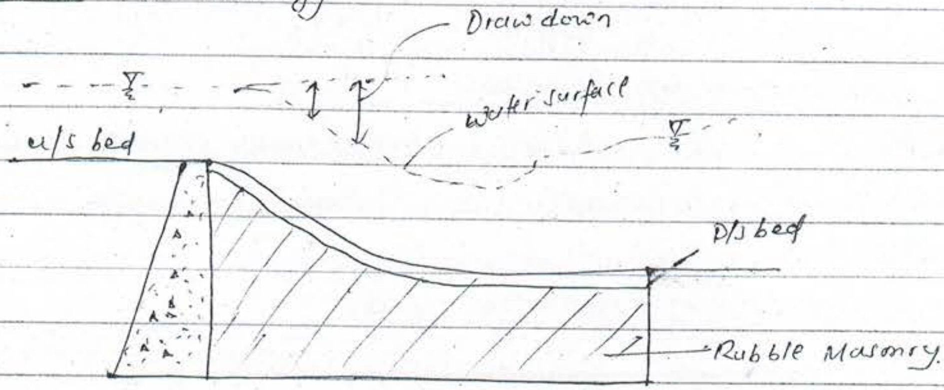


Fig:- Ogee falls

2) Rapid Falls

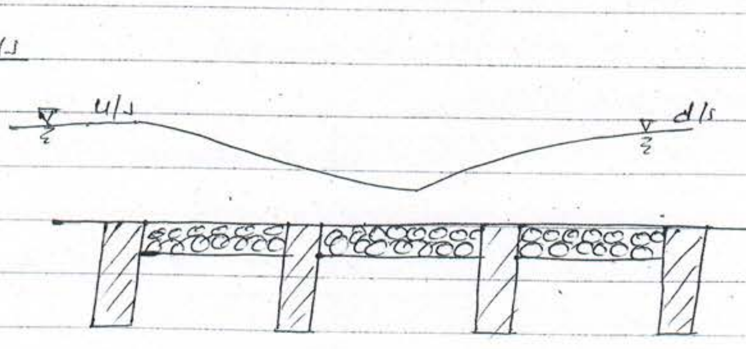
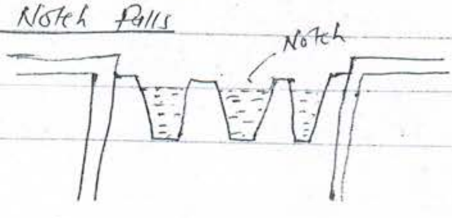


Fig - Rapid falls

3) Trapezoidal Notch falls



(4) <u>well Types fall or Syphon well fall</u> - This type of Fall Consists of an inlet well with a pipe at its bottom. Carrying water from the inlet well to a d/s well.

(5) <u>Glacis type fall</u> - effeciency of fall increases by creating hydraulic Jump

(6) <u>Stepped Fall</u> - Suitable for hill area

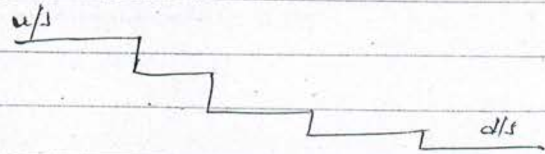
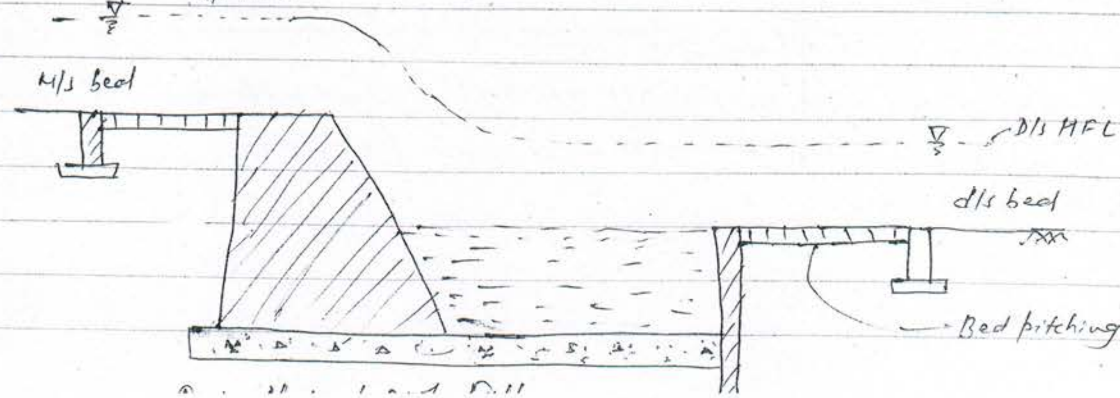


Fig:- stepped fall

(7) <u>Meter and Non - Meters fall</u>
- Those falls that measure Q are called Meter falls
- Non- meter falls donot measure Q.

(8) <u>Inglis or Baffle falls</u> - when in glacis type fall Baffle is added it is called Baffle falls or Inglis falls.
- devloped by Englis so called Inglis Fall.
- Suitable for all discharges.

(9) <u>Simple Vertical drop / Sarda Type Falls</u>

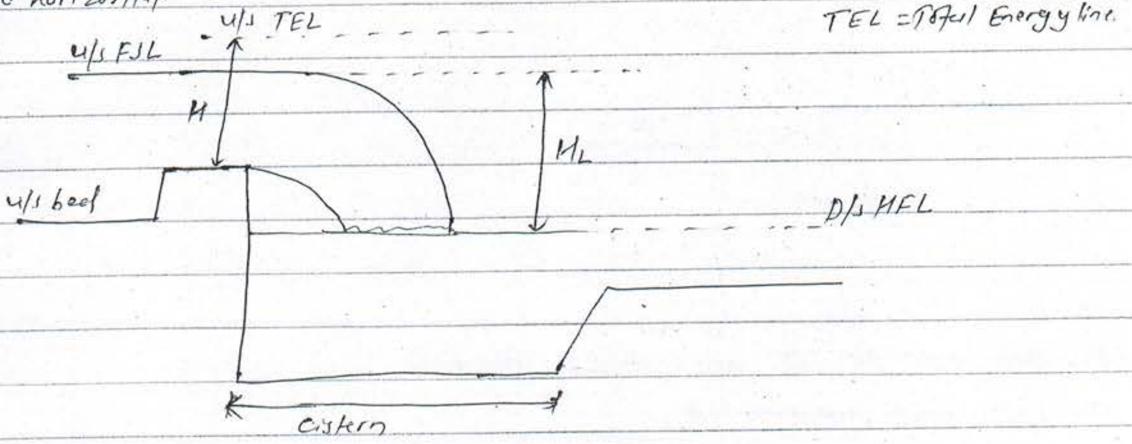


A Raised crest fall with a vertical impact was first of all introduced on Sarda Canal UP, India.

Sarda type fall is high crested fall.

Design of Vertical drop fall

Energy is dissipated by means of impact and by sudden defxns of velocity from vertical to horizontal.



Cistern length $\boxed{(L_c) = 5\sqrt{H \cdot H_L}}$

Cistern depth $\boxed{(x) = \frac{1}{4}\left(H_L \times H\right)^{2/3}}$

L_c = length of Cistern (m)

x = Cistern depression below d/s bed (m)

H_L = drop in meter = u/s water level − D/s wtr level = head difference

H = Depth of water over crest or head of water above the crest (m)

= u/s TEL − crest level

Design of Sarda Type fall

(1) length of crest :- The length of crest is equal to bed width of Canal.

(2) Shape of crest

(a) $Q \ll 15$ cumecs → Rectangular with both faces Vertical

Top width $= 0.55\sqrt{d} = B_t$

Base width $= \frac{d}{L}$ or $\frac{h+d}{G}$ ($G = 2$ for masonry)

d = ht. of crest above the downstream bed level

h = head over the crest

$$\boxed{Q = 1.84\, L H^{3/2} \left(\frac{H}{B_t}\right)^{1/6}}$$

(b) $Q \gg 15$ cumecs → Trapezoidal crest

Top width $= 0.55\sqrt{H+d} = B_t$

U/s side slope 1:3 & d/s 1:8

Base width $= H/2$

H = ht. of TEL from crest

$$\boxed{Q = 1.99\, L H^{3/2} \left(\frac{H}{B_t}\right)^{1/6}}$$

(3) Crest level

$$\boxed{Q = C_d \cdot \sqrt{2g}\, L \cdot H^{3/2} \left(\frac{H}{B_t}\right)^{1/6}}$$

$C_d = 0.415$ for Rectangular Crest

$= 0.45$ " trapezoidal "

L = length of crest

B_t = top width of crest

H = depth of crest below TEL

∴ Height of crest above bed = $y - h$ $\quad$ y = Nrml depth of channel (upstream
$= y - H$ $\quad$ $h = H$ assuming.

) Impervious Concrete Floor

otal Floor length - determined by Bligh's Theory, for small works & by Khosla's theory large works.

Minimum length of floor on d/s = 2 (waterdepth + 1.2m) + drop

D/s Floor $= 10.53\, d_c + 4.877 - 1.5 H_L$

$$d_c = \left(\frac{q^2}{g}\right)^{1/3} \qquad q = \text{discharge per m} = \frac{Q}{L}$$

$$= 6.77\, d_c + 5.182 + H_L$$

Thickness : The minimum thickness of floor for practical considerations is 0.30 m in u/s, /s floor for uplift pressure = (0.4 - 0.6) m for large work & 0.3m for minor work.

Cistern length

length of Cistern $= 3.8\, d_c + 0.415 + H_L$ → for clear & sub-mergence upto 33%.
$= 5.2\, d_c + 1.067 + H_L$ → for submergence above 33%.

∴ depth of Cistern $= \frac{d_c}{3}$ in all case

Length of Cistern (L_c) $= 5\sqrt{H \cdot H_L}$

Depth of stern (x) $= \frac{1}{4}(H \cdot H_L)^{2/3}$ //

Aqueduct

Er. Umesh Raut

Design of Aqueduct & Syphon Aqueduct

Design is done from a Numerical

① Design a suitable cross drainage structure

Canal data

Full supply discharge = 32 cumecs

Full supply level = RL 213.5

Canal bed level = RL 212

Canal bed width = 20m

Trapezoidal Canal sxn = 1.5H to 1V

Canal water depth = 1.5m

Drainage

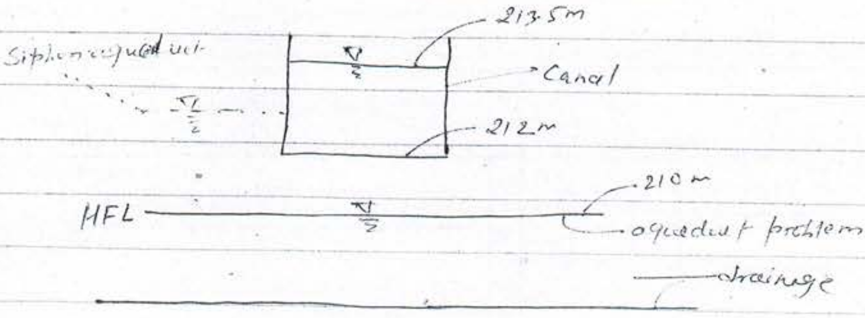
High flood discharge = 300 cumecs

HFL = 210m

High flood depth = 2.5m

General grnd level = 212.5m

Soln.



Here the Canal Bed level (212m) is much above than HFL of Drainage (210m). So Aqueduct is Constructed.

Canal Bed level $\gg$ HFL $\Rightarrow$ So aqueduct

Step-1: <u>Design of Drainage waterway</u>

Lacey's Regime perimeter $(P) = 4.75\sqrt{Q}$ — Q = High flood discharge of drain = 300 cumecs

$= 4.75\sqrt{300}$

$\boxed{P = 82.3\,m}$

Let, Clear span between piers be 9m and pier thickness be 1.5m

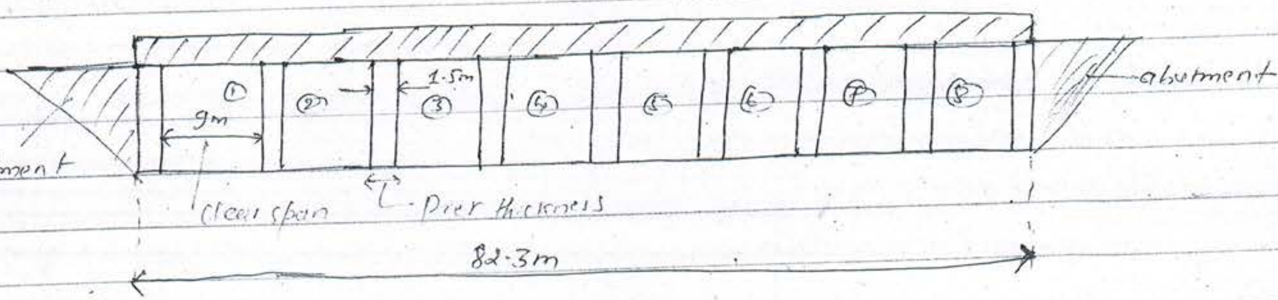
Using 8 bays of 9m each

$\therefore$ Clear water way $= 9 \times 8 = 72\,m$ and

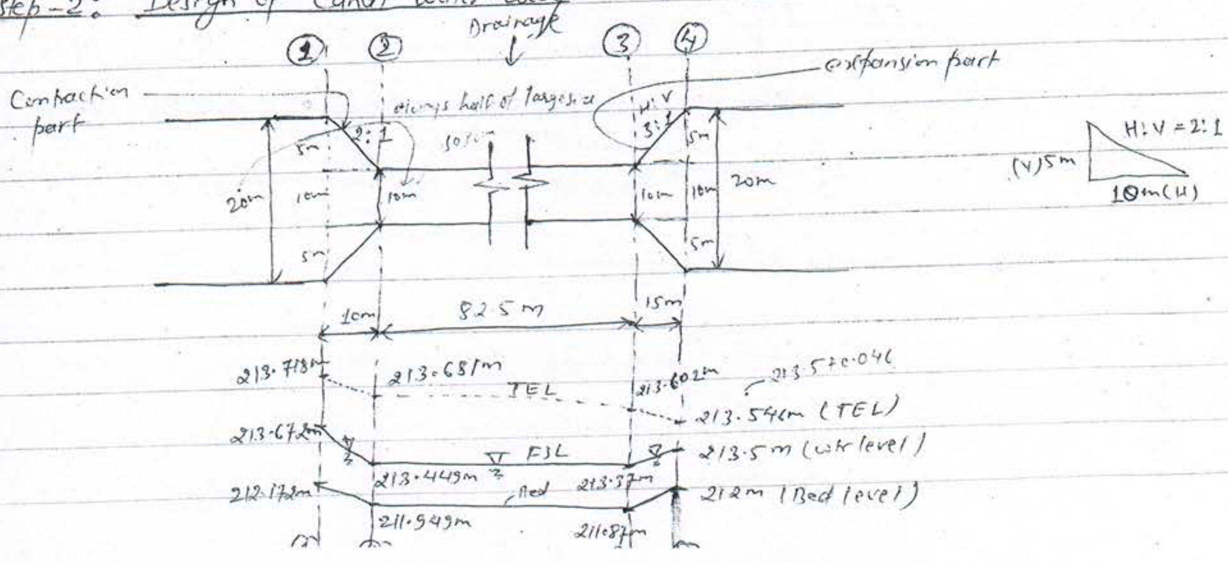
Pier width $= 1.5 \times 7 = 10.5$

82.5

balanced.



Step-2: <u>Design of Canal water-way</u>



Let width of Flumed = 10m

Providing a splay 2:1 in Contraction

$\therefore$ length of Contraction $= \frac{20-10}{2} \times 2$

$= 10m$

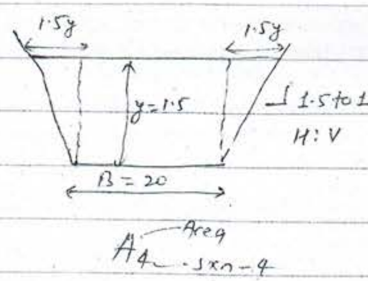
Provide a splay 3:1 in expansion

$\therefore$ length of expansion $= \frac{20-10}{2} \times 3$

$= 15m$

Step-3: Head loss and Bed levels at Different sxn's

(a) From figure, At sxn (4)-(4) → Trapezoidal sxn



$\therefore$ Area of Trapezoidal sxn $= (B+1.5y)y$

$= (20+1.5\times1.5)*1.5$

$\boxed{A_4 = 33.75 m^2}$

Now,

Velocity $(V_4) = \frac{Q_4}{A_4}$

$= \frac{32}{33.75}$

$\boxed{V_4 = 0.947 m/sec}$

Velocity head at (4)-(4) $= \frac{V_4^2}{2g} = \frac{0.947^2}{2\times9.81}$

$= 0.046m$

∴ At sxn (4)-(4) → Bed level = 212 m
→ water level = 213.5 m
→ TEL = 213.5 + Velocity head (0.046)
= 213.546

TEL = Total Energy line

5) for sxn (3)-(3)

Keeping water depth = 1.5 m

Bed width = 10 m

$\boxed{\text{Area} = 10 \times 1.5 = 15 \text{ m}^2}$

breadth = 10
wtr height/depth = 1.5 } always

Velocity at (3)-(3) $(V_{3-3}) = \frac{Q_{3-3}}{A_{3-3}} = \frac{32}{15}$

$$\boxed{V_{33} = 2.13 \text{ m/s}}$$

Velocity head at (3)-(3) $= \frac{V_{33}^2}{2g} = \frac{2.13^2}{2 \times 9.81} = 0.232 \text{ m}$

Assuming loss of head in Expansion from (3)-(3) to (4)-(4)

$$= 0.3 \left(\frac{V_{3-3}^2 - V_{4-4}^2}{2g} \right)$$

$$= 0.3 \left| \frac{2.13^2 - 0.947^2}{2 \times 9.81} \right|$$

$$= 0.056 \text{ m}$$

RL of TEL at (3)-(3) = RL of TEL at (4)-(4) + loss of head in expansion

= 213.546 + 0.056

= 213.602 m

RL of wtR Surface at (3)-(3) = 213.602 − 0.232 = 213.37 m

RL of Bed level at (3)-(3) = 213.37 − 1.5 = 211.87 m

(c) for sxn (2)-(2)

From sxn (2)-(2) to (3)-(3) the sxn is Constant. So Area and Velocity at (2)-(2) same as (3)-(3)

$n = 0.016$

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

$V = 2.13 m/s$

$A = 15 m^2$

$$V = \frac{1}{n} R^{2/3} \left(\frac{H_L}{L}\right)^{1/2}$$

$L = 82.5 m$

$B = 10 m$

$y = 1.5 m$

$$H_L = \frac{n^2 V^2 . L}{R^{4/3}}$$

$$S = \frac{H_L}{L}$$

$$= \frac{0.016^2 \times 2.13^2 \times 82.5}{(1.15)^{4/3}}$$

$$R = \frac{A}{P} = \frac{By}{B+2y} = \frac{10 \times 1.5}{10 + 2 \times 1.5}$$

$$\boxed{H_L = 0.079 m}$$

$$\boxed{R = 1.15 m}$$

Now,

RL of TEL at (2)-(2) = 213.602 + 0.079 = 213.681 m

RL of wtr surface at (2)-(2) = 213.681 − 0.232 = 213.449 m

RL of Bed level at (2)-(2) = 213.449 − 1.5 = 211.949 m

{ Velocity head at (3)-(3) = Velocity head at (2)-(2) = 0.232 m.

(d) for sxn (1)-(1)

loss of head in Contraction from (1)-(1) to (2)-(2) $= 0.2 \left| \frac{V_2^2 - V_1^2}{2g} \right|$

$$= 0.2 \left| \frac{2.13^2 - 0.947^2}{2 \times 9.81} \right|$$

$V_2 = V_3 = 2.13 m/s$

$V_1 = V_4 = 0.947 m/s$

$= 0.037 m$

Now,

RL of TEL at (1)-(1) = RL of TEL at (2)-(2) + Loss in contrxn

= 213.681 + 0.037 = 213.718 m

RL of water Surface at (1)-(1) = 213.718 − 0.046 = 213.672 m

{ Velocity head of 1 = V. head of 4 = 0.046 m

RL of Bed level at (1)-(1) = 213.672 − 1.5 = 212.172 m

// Ans //

Syphon Aqueduct

(2) Design a Suitable Cross drainage Strxtr

Canal data

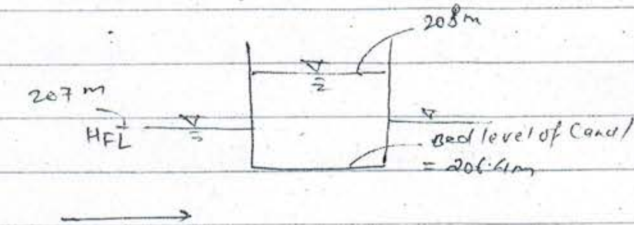
Discharge of Canal = 40 cumec

Bed width = 30m

Full supply depth = 1.6m

Bed level of Canal = 206.4m

Side slope of Canal = 1.5H to 1V



Drainage data

High Flood discharge = 450 cumecs

HFL of drainage = 207m

Bed level of drainage = 204.5m

General ground level = 206.5m

n.

Here,

Canal bed level (206.4m) is Sightly below drainage HFL (207m). So Syphon
ueduct is Constructed.

ie,

Canal Bed level 206.4m <<< Drainage HFL ⇒ So Syphon aqueduct

ep 1:- Design of Drainage water way

Q = 450 cumec for drainage

Lacey's Regime perimeter $(P) = 4.75\sqrt{Q}$

$= 4.75\sqrt{450}$

$\boxed{P = 100.8m}$

always.

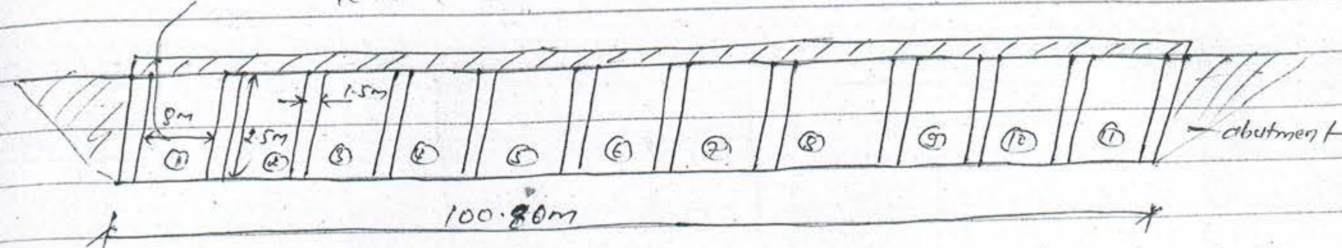
ovide 11 Clear span of 8m each with pier width 1.5 each

∴ length occupied by 11 bays = 11×8 = 88m

length occupied by 10 piers of 1.5 width = 10×1.5 = 15m

∴ total length of water way = 103m

3m भन्दा सानो न राख्ने & 14m भन्दा ठूलो पनि नराख्ने
2nd width
i.e. betn (3-14) take any value & balance length of perimeter But 1.5 is always constant



assume same always.

let us. limit the velocity through Syphon barrel to a value of 2m/s

$$\therefore \text{Height of barrel required } (H) = \frac{\text{Discharge } (Q)}{\text{Velocity} * \text{clear width of water way}}$$

$$= \frac{450}{2 \times 88}$$

$$\boxed{H = 2.56m}$$

Hence,

Provide 11 Rectangulr barrel, each 8m wide & 2.56m high.

$$\therefore \text{Actual velocity through barrel} = \frac{450}{88 \times 2.5}$$

$$= 2.05 m/sec$$

Step-2: Design of Canal watrway

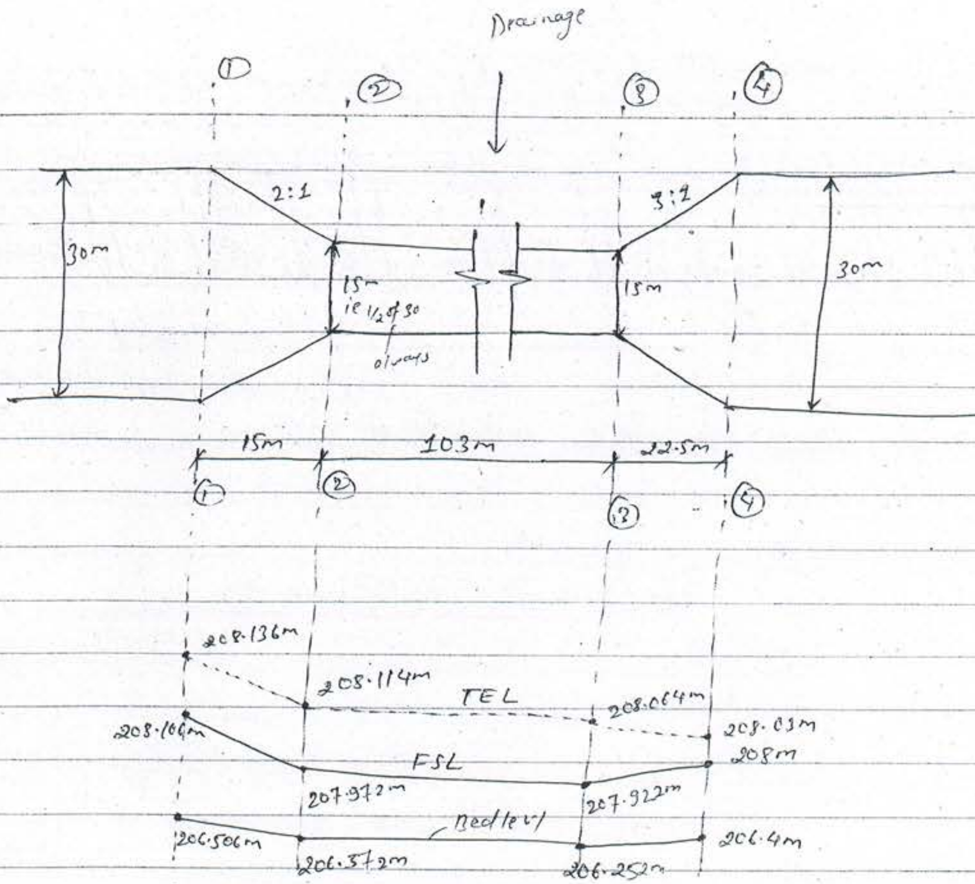
Normal bed width of Canal = 30m.

let the width Reduced to 15m.

Providing a splay 2:1 in Contraction

$$\therefore \text{length of Contraction} = \frac{30-15}{2} \times 2$$

$$= 15m$$



Providing a splay of 3:1 in expansion
the length of expansion $= \frac{30-15}{2} \times 3$

$= 22.5m$

Step-3

(a) for sxn ④-④ → Trapezoidal sxn

Area of trapezoidal Canal sxn $= (B + 1.5y)y$
$= (30 + 1.5 \times 1.6) \times 1.6$

$\boxed{A = 51.84 m^2}$

B = bed width = 30m
y = depth = 1.6m

Velocity of sxn ④-④ $(V_{④-④}) = \frac{Q_{4-4}}{A_{4-4}} = \frac{40}{51.84}$

$V_{4-4} = 0.77 \text{ m/sec}$ $\qquad\qquad$ $Q = 40 \text{ cumec}$

Velocity head at ④-④ $= \frac{V_{4-4}^2}{2g} = \frac{0.77^2}{2\times 9.81}$

$= 0.030\text{m}$

RL of Bed level of Canal at ④-④ $= 206.4\text{m}$ (Given)

water depth $= 1.6\text{m}$ (Given)

RL of water surface at $4-4 = 206.4 + 1.6 = 208.0\text{m}$

RL of TEL at $4-4 = 208.0 + 0.03 = 208.03\text{m}$

(b) for sxn ③-③

Keeping Constant depth $= 1.6\text{m}$

Bed width $= 15\text{m}$

$\therefore$ Area $= 1.6 \times 15$

$\boxed{A_{33} = 24\text{m}^2}$

Velocity at ③-③ $(V_{3-3}) = \frac{Q_{3-3}}{A_{3-3}} = \frac{40}{24} = 1.67 \text{ m/sec.}$

Velocity head $= \frac{V_{3-3}^2}{2g} = \frac{1.67^2}{2\times 9.81} = 0.142\text{m}$

Assuming loss of head in expansion from ③-③ to ④-④

$= 0.3 \left| \frac{V_{3-3}^2 - V_{4-4}^2}{2g} \right|$

$= 0.3 \left| \frac{1.67^2 - 0.77^2}{2\times 9.81} \right|$

$= 0.034\text{m}$

RL of TEL at ③-③ = RL of TEL at ④-④ + loss in expansion

$= 208.03 + 0.034$

$= 208.064\,m$

RL of water surface at ③-③ $= 208.064 - 0.142$

$= 207.922\,m$

RL of Bed level of ③-③ $= 207.922 - 1.67 = 206.252\,m$

© For sxn ②-②

For sxn ② & ③ is same so,

$Q = 40\,m^3/s$

$n = 0.016$

$L = 103\,m$

$B = 15\,m$

$y = 1.6\,m$

Area $(A_{2-2}) = 15 \times 1.6 = 24\,m^2$

Velocity $(V_{2-2}) = \frac{40}{24} = 1.67\,m/sec.$

Head loss $(H_L) = \frac{n^2 V^2 L}{R^{4/3}}$

$R = \frac{A}{P} = \frac{By}{B+2y} = \frac{15 \times 1.6}{15 + 2 \times 1.6} = 1.32\,m$

$= \frac{0.016^2 \times 1.67^2 \times 103}{1.32^{4/3}}$

$\boxed{H_L = 0.05\,m}$

RL of TEL at ②-② $= 208.064 + 0.05 = 208.114\,m$

RL of water surface at ②-② $= 208.114 - 0.142 = 207.972\,m$

RL of Bed level at ②-② $= 207.972 - 1.6 = 206.372\,m$

velocity head

$V_{22} = V_{3-3} = 0.142\,m$

$V_{22} = V_{33} = 1.67\,m/s$

velocity

(d) For section ①-①

loss of head in Contraction from ①-① to ②-② $= 0.2 \left| \frac{V_{2-2}^2 - V_{1-1}^2}{2g} \right|$

$= 0.2 \left| \frac{1.67^2 - 0.77^2}{2 \times 9.81} \right|$

$V_1 = V_4$

$V_2 = V_3$

$= 0.022\,m$

RL of TEL at ①-① = RL of TEL at ②-② + loss in Contrxn

$= 208.114 + 0.022$

$= 208.136m$

RL of water Surface at ①-① = $208.136 - 0.03 = 208.106m$

RL of Bed level at ①-① = $208.106 - 1.6 = 206.506m$

Step-4:- Afflux OR Head loss through Syphon Barrels

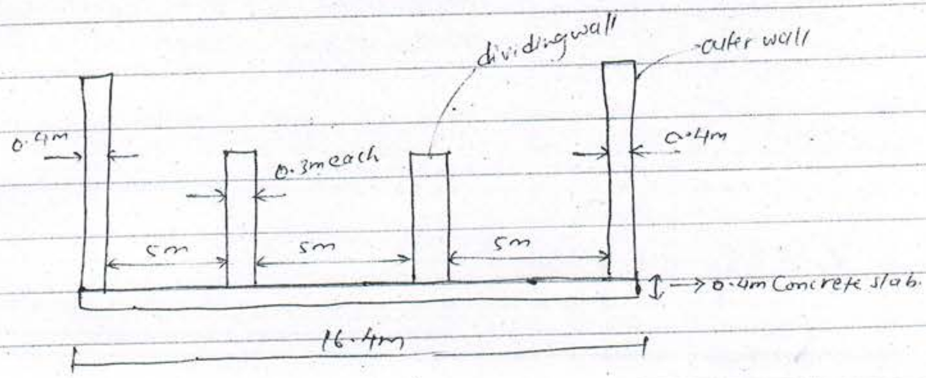


Fig:- width of trough

Width of Trough -: The trough is divided into 3-equal Compartments each 5m wide separated by 0.3m thick partition wall with 0.4m thick outer wall

Overall width of Trough = $(5+5+5) + (0.3+0.3) + (0.4+0.4)$

$$\boxed{L = 16.4m}$$

length of Syphon barrel = 16.4m

The head loss through Syphon barrels is given by Unwin's formula

$$h = \left[1 + f_1 + f_2 * \frac{L}{R}\right] \frac{V^2}{2g} \quad \text{(a)}$$

सुरक्षा निकालेको

V = Velocity through barrels = 2.05 m/s.

F_1 = coefficient of loss of head at entry

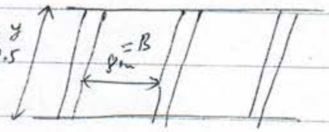
= 0.505

$f_a = a\left(1 + \frac{b}{R}\right)$ where a & b are different for different mtrls

<u>for cement plastered</u>

$a = 0.00316$

$b = 0.030$



$R = \frac{A}{P} = \frac{B \cdot y}{2(B+y)}$

Area of Rectangle = $B \times y = 8 \times 2.5$

Perimeter of Rectangle = $2(l+B) = 2(y+B)$

$= \frac{8 \times 2.5}{2(8+2.5)}$

$\boxed{R = 0.952\,m}$

$\therefore f_a = a\left(1 + \frac{b}{R}\right) = 0.00316\left(1 + \frac{0.030}{0.952}\right)$

$= 0.00326$

Now from eqn (a), $L = 16.4\,m$

$h = \left[1 + 0.505 + 0.00326 * \frac{16.4}{0.952}\right] \frac{2.05^2}{2 \times 9.81}$

$\boxed{h = 0.334\,m}$ = Afflux.

High Flood level of Drainage (HFL) = 207 m (Given) (d/s)

HFL of drainage (u/s) = d/s HFL + Afflux or (loss of head)

$= 207 + 0.334$

$= 207.334\,m$ //

Step-5 : Uplift Pressure on Roof of Barrels

RL of bottom of Trough = RL of canal bed − slab thickness

$= 206.4 - 0.4$

$= 206m$

Head loss at entry of barrel $= 0.505 \frac{V^2}{2g}$

$= 0.505 \times \frac{2.05^2}{2 \times 9.81}$

$= 0.108m$

Uplift on the Roof = u/s HFL − Loss at entry − level of Underside of Roof slab

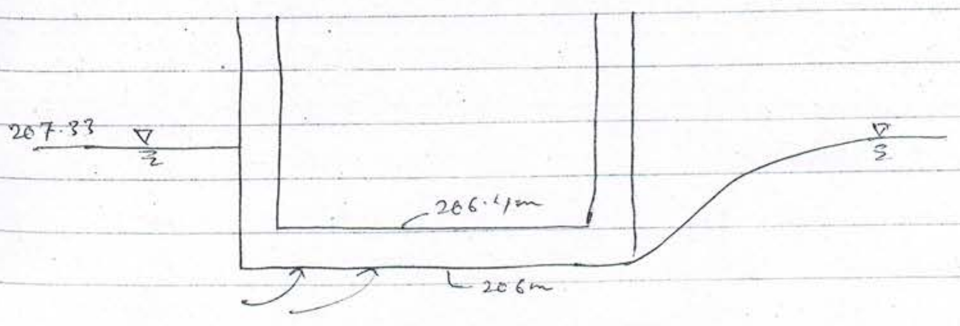
$= 207.33 - 0.108 - 206$

$= 1.225m$ of water

Assuming Unit wt. of water $= 10 kN/m^2$

∴ Uplift on Roof $= 1.225 \times 10$

$= 12.25 kN/m^2$ //Ans//



//

2072/08/18 friday

Chapter-7: Water Logging and Drainage

⇒ Er. Umesh Raut

Water logging

The Cultivable land is called water-logged if it's productivity decreases due to rise in water table. The soil pores within the root zone of the crops gets saturated and normal Circulation of air is Cut off. It Can be prevented by providing effective drainage system.

Causes of water logging

1. Over irrigation - due to over irrigation water table rises.

2. Inadequated drainage system
3. seepage through Canal
4. Impervious obstruction - wtr seeping below soil moves horizontally. it is obstructed by an impervious layer, then wtr level will Rise up.

5. Constrxn of Reservoir - The area near & under reservoir may remains wtr logged due to seepage.

6. Defective irrigation practise
7. Intensive Rainfall
8. Flat Topography

Effects of water logging

1. inadequate Circulation of air in Root zone of Crops leading less productivity
2. Fall in Soil temperature
3. Delay in Cultivation operation.
   - Difficult to ploughing.
   - sowing of crops & growth are also delayed

4. Breeding of insects & Mosquito.
5. Growth of unwanted plants, grasses

(6) Rise of salt in top soil.
(7) Absence of Aeratn of soil
(8) Plant disease

Preventive Measures of wtR logging.

(1) Decrease inflow
(2) increases outflow

(1) Decreases inflow
- Canal lining
- Vies intensity of irrigtn
- Crop Rotatn
- improve in irrigtn System
- lowering the free surface level of canal
- Provide the catch drain along high land.

(2) Increase outflow
- Provision of Surface & Sub-surface drainage
- improving natural drainage
- Removing the obstrn
- Adopting well irrigtn.

Types of Drainage

(1) Surface drainage
(2) Sub-Surface drainage

(1) Surface drainage System
- Removal of excess water by Constructing open ditches is Called Surface drainage.

Types of Surface drainage

① Shallow surface drainage

② Deep surface drainage

① Shallow surface drainage

- To remove excess irrigtn water
- To remove excesss Rain water
- Nrmally trapezoidal in shape
- Manning's & Cutters formula Can be used for design.

② Deep Surface drainage

- Relatively deeper than shallow drainage & effective for draining sub-soil water (i.e grnd wtr)
- dug upto grnd wtr table (GWT)

Design of shallow Surface drainage

Assumption (for plane land)

① Rainfall for 5 or 10 year Return period is Considered.

② Initial water level in field is Considered 40mm

③ Maximum water level is 300mm, which may persist for 1 day

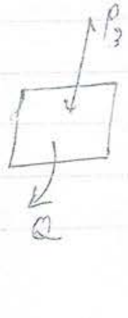
④ Depth in excess of 200mm may persist for 3 day.

⑤ No Rainfall follows design Rainfall for more than 3 days

⑥ Irrigtn inflows are neglected

P_3 = yearly maxm Rainfall for Consecutive 3 days

Q = drainage in mm/day



$$\boxed{h = 40 + \frac{P_3 \times t}{3} - Q \times t}$$

h = ht. of wtr

Numerical

① Design the surface Drainage for the field having Rainfall intensity of $P_3 = 450mm$, for area $= 40ha$.

Soln.

(1) Assume,

P_3 = for 3 day = 450mm

$Q = 50 mm/day$

1 day = 450/3

= 150mm

Trial - 1

always 40 - initial depth assumed

(t) Day	initial depth	$P_3/3$	Q	Final water depth (h)
1	40	150	50	140
2	140	150	50	240
3	240	150	50	340

$$h_1 = 40 + \frac{P_3 \times t}{3} - Q \cdot t = 40 + \frac{450 \times 1}{3} - 50 \times 1$$

for (t) = 1 day

$$\boxed{h_1 = 140mm}$$

$$h_2 = 140 + \frac{450 \times 1}{3} - 50 \times 1 = 240m$$

$$h_3 = 240 + \frac{450 \times 1}{3} - 50 \times 1 = 340m$$

Here water depth (h) is greater >>> than 300 so 2nd trial is required

2nd trial

$Q = 75mm/day$

Day	initial depth	$P_3/3$	Q	Final water depth (h)
1	40	150	75	115
2	115	150	75	190
3	190	150	75	265
4	265	-	75	190

$$h_1 = 40 + \frac{450 \times 1}{3} - 75 \times 1 = 115m$$

$h_2 = 115 + \frac{450 \times 1}{3} - 75 \times 1 = 190m$

$h_3 = 190 + \frac{450 \times 1}{3} - 75 \times 1 = 265$

$h_4 = 265 + 0 - 75 = 190$

Here, No water depth is greater than >> 300mm. Depth excess of 200mm occur in 1 day only So, theoritically the design is ok//. But uneconomical.

3rd Trial

$Q = 65 mm/day$

Day	initial depth	$P_3/3$	Q	h
1	40	150	65	125
2	125	150	65	210
3	210	150	65	295
4	295	–	65	230
5	230	–	65	165

$h_1 = 40 + \frac{450 \times 1}{3} - 65 \times 1 = 125m$

$h_2 = 125 + 150 - 65 = 210m$

$h_3 = 210 + 150 - 65 = 295m$

$h_4 = 295 + 0 - 65 = 230m$

$h_5 = 230 + 0 - 65 = 165m$

Here water level greater than >> 200mm occur for 3 days & No wtr depth is greater than 300mm. Hence design is ok// & also economical

Now, $Q = 65 mm/day = \frac{65}{1000 \times 24 \times 60 \times 60}$

ie $\boxed{V = 7.52 \times 10^{-7} m/s}$

and,

Area (A) = 40 ha = $40 \times 10^4 m^2$

we know,

$Q' = AV$

$= 40 \times 10^4 \times 7.52 \times 10^{-7}$

$\boxed{Q' = 0.3 m^3/s}$ // Ans//

(5) Assumptions for shallow drainage in hilly Regions

(1) Rainfall for 5 or 10 yrs Return period is Considered.

(2) Initial water level should be considered 40mm

(3) Max^m water level is 100mm, which may persist for 1 day.

(4) Evaporation & transpiration is neglected

(5) Irrigation inflows are neglected

(6) No Rainfall follows design Rainfall for more than 1 day.

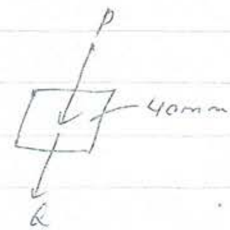
P = Max^m yearly Rainfall

Q = drainage discharge (mm/day)

Then,

Water Balance equation,

$\boxed{40 + P - Q = 100}$

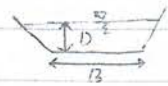


Design criteria for Surface drainage

(1) Bed slope - 1:300 to 1:500

(2) velocity $(V) \leq 0.85\,m/sec$

(3) $\frac{B}{D}$ ratio = 3:1



(4) Capacity of drain should be designed for storm water

(5) The drain alignment should follow Natural drain as far as possible.

(6) Side slope - 1:1

(2) Question - Taken from previous Question,

Here,

$$Q = 0.3\,m^3/s$$

Bed slope = 1:500

Side slope = 1:1

$n = 0.025$ — Manning's Constant

Then,

$$Q = \frac{1}{n} \cdot A\, R^{2/3} \cdot S^{1/2}$$

$$\boxed{Q = \frac{1}{n} \times \frac{A^{5/3}}{P^{2/3}} \cdot S^{1/2}}$$

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

always

$A = BD + D^2$ and $P = B + 2\sqrt{2}\,D$

$= 3D^2 + D^2$ $\quad = 3D + 2\sqrt{2}\,D$

$A = 4D^2$ $\quad P = 3D + 2\sqrt{2}\,D$

$B = 3D$ i.e. $\frac{B}{D} = 3$

then,

$$0.3 = \frac{1}{0.025} \times \frac{(4D^2)^{5/3}}{(3D + 2\sqrt{2}D)^{2/3}} \times \left(\frac{1}{500}\right)^{1/2}$$

$$\boxed{D = 0.33\,m}$$

Now,

$$\frac{B}{D} = \frac{3}{1}$$

$$B = 3 \times 0.33$$

$$\boxed{B \doteq 0.99m}$$ // Ans //.

② Sub-Surface drainage System (Tile drainage System & their Design)

Advantage of Sub-Surface

① It removes free gravity water

② It provides air Circulation

③ Provides more Volume of Soil for Root Zone.

④ It increases the bacterial activity due to good supply of air.

⑤ It decreases soil erosion.

⑥ Effective in Removal of toxic substances as sodium and other Soluble salt.

↳ Sub-Surface drainage or tile drainage generally made of earther wore

↳ Circular in sxn

↳ diameter 10 to 30cm or more

↳ Butting each other with open Joint

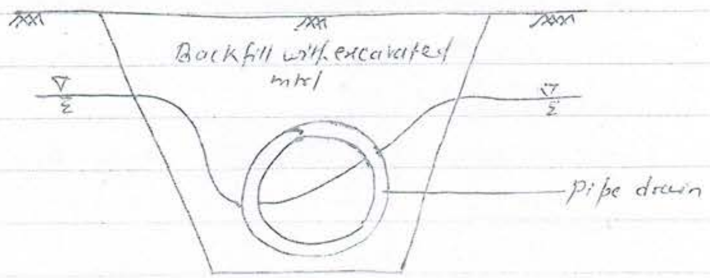


fig:- Tile drain in pervious soil

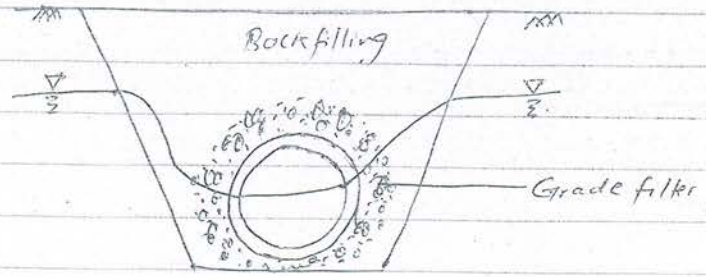
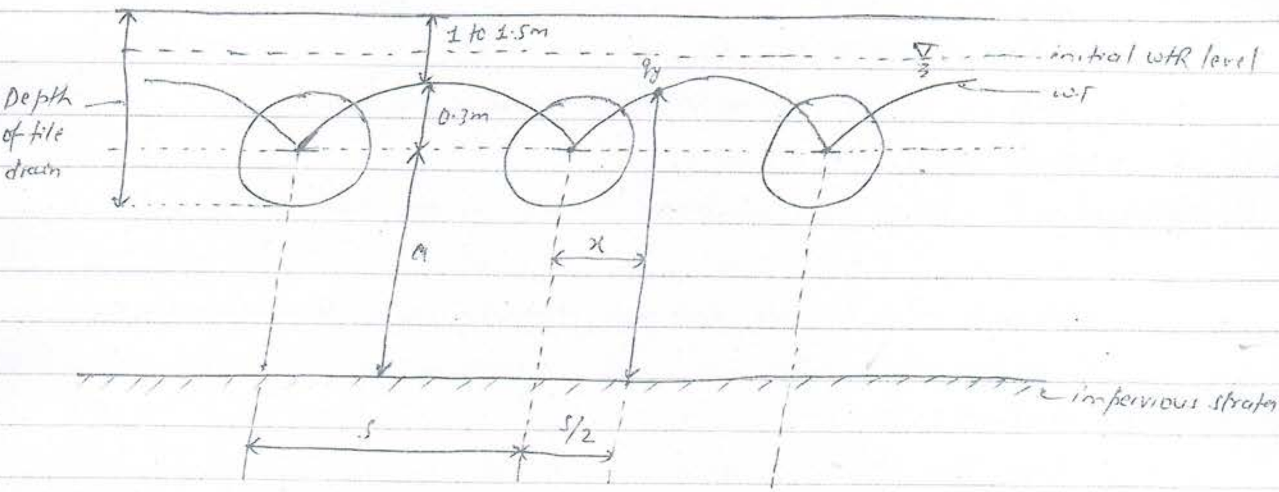


Fig:- Tile drain in less pervious soil

Depth and spacing of Tile drains



The depth and spacing of sub-surface drain should be sufficient enough to lower the wtr table from Rootzone of the plant. For most of the plant the top point of the water table must be at the level of 1m to 1.5m below the ground level. It may vary from 0.7 to 2.5m depending upon types of crops and type of soil.

The tile drain may be placed at about 0.3m below the design heighest level of water table.

From Darcy law,

$$\boxed{Q = KIA}$$

K = Coefficient of permeability
I = hydraulic gradient

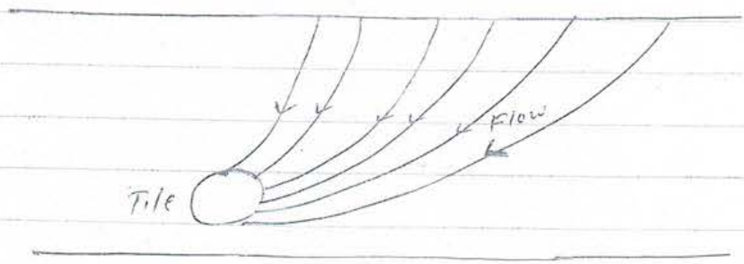
Discharge per unit length of drain passing the sxn y ie q_y is given by,

$$\boxed{q_y = K \cdot \frac{dy}{dx} \cdot y \cdot 1} \quad \text{——— (1)} \quad \text{ie } i = \frac{dy}{dx} = \text{slope}$$

$A = y \times 1$ for unit sxn

If 'q' is the total discharge per unit length Carried by 1-drain, a dischrage $\frac{q}{2}$ will enter the grain from either side.

From Assumptn, Discharge toward drain is inversly proportnl to the distance from the drain.



when,

$$x = \frac{s}{2} \quad ; \quad q_y = 0$$

$$x = 0 \quad ; \quad q_y = \frac{q}{2}$$

At distance x, by linear interpolatn,

$$q_y = \frac{q}{2} - \left(\frac{q/2}{s/2}\right) x$$

$$q_y = \frac{q}{2} - \left(\frac{q}{s}\right) x$$

$$\boxed{q_y = \frac{q}{2s}(s - 2x)} \quad \text{——— (2)}$$

Equating (1) & (2),

$$\frac{q}{2s}(s-2x) = k \cdot y\frac{dy}{dx}$$

$$y\,dy = \frac{q}{2sk}(s-2x)\,dx$$

Integrating we get,

$$\boxed{\frac{y^2}{2} + c = \frac{q}{2sk}\left(sx - \frac{2x^2}{2}\right)} \quad \text{——— (3)}$$

when,

$x = 0$, $y = a$

$$\frac{a^2}{2} + c = \cancel{\frac{q}{2sk}} \times 0$$

$$\boxed{c = \frac{-a^2}{2}}$$

Substituting value of 'c' in eqn (3),

$$\frac{y^2}{2} - \frac{a^2}{2} = \frac{q}{2sk}\left(sx - \frac{2x^2}{2}\right)$$

$$\frac{q}{2sk}(sx - x^2) = \frac{y^2}{2} - \frac{a^2}{2}$$

$$\boxed{q = \frac{ks(y^2-a^2)}{x(s-x)}}$$

when,

$x = \frac{s}{2}$, $y = h$

$$q = \frac{ks(h^2-a^2)}{\left(s-\frac{s}{2}\right)\cdot\frac{s}{2}}$$

$$\boxed{q = \frac{4k}{s}\left(b^2 - a^2\right)}$$

and,

$$\boxed{s = \frac{4k}{q}\left(b^2 - a^2\right)}$$

This eqn can be used to predict spacing 's' between grains, if q is known.
Different value has been suggested. Generally 1% of average annual Rainfall is considered to be drained by the tile drains in 24 hrs.

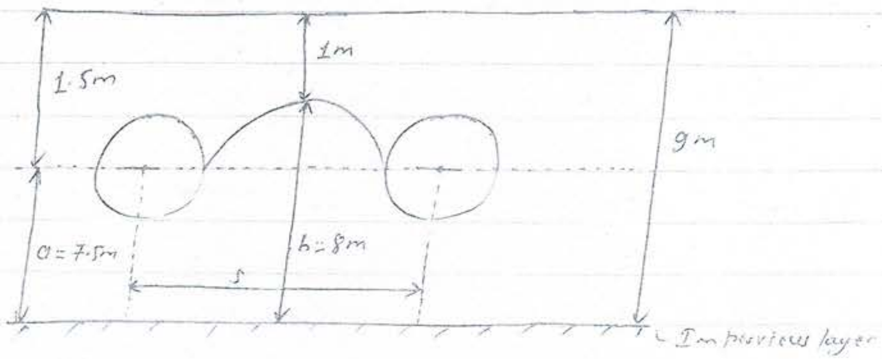
if average annual rainfall is P_{AA} meters, then,

$$\boxed{q = \frac{0.01 \times P_{AA}}{24 \times 3600} \times s \times 1}$$ cumec/m length of drain

Numericals

Q1) In a tile drainage system, the drains are laid with their centre 1.5m below the ground level, the impervious layer is 9m below the ground level and the average annual rainfall in the area is 80cm. If 1% of the annual rainfall is to be drained in 24hrs to keep the highest position of the watertable to 1m below ground level, Determine the spacing of the drain pipes. Coefficient of permeability may be taken as 0.001 cm/sec.

soln.



We have,

$$q = \frac{0.01 \times P_{AA}}{24 \times 3600} (S * 1)$$

$P_{AA} = 80cm = 0.8m$

$$= \frac{0.01 \times 0.8}{24 \times 3600} \times S$$

$$= 92.59 \times 10^{-9} S \quad m^3/s/m \text{ length of pipe}$$

Also,

$$S = \frac{4k}{q} (b^2 - a^2)$$

$k = 0.001 \, cm/s = \frac{0.001}{100} m/s$

$$= \frac{4 \times 1 \times 10^{-5}}{92.59 \times 10^{-9} S} (8^2 - 7.5^2)$$

$k = 1 \times 10^{-5}$

b = ht. of WT above imperrious layer

$= 9m - 1m = 8m$

$$S^2 = 3348.097$$

a = depth of imperrious stratum below the centre of drains

$a = 9 - 1.5 = 7.5m$

$$S = \sqrt{3348.097}$$

$$\boxed{S = 57.86m}$$

// Ans //.

Reclamation of water logged area by different methods

Restoring the productivity and fertility of a land which has become unculturable because of water logging is called Reclamation of water logged area.

Methods applying for Reclamation of land

(1) Lowering of water Table - By making drainage
(2) Leaching Method - Removing salt or by adding Chemicals
(3) Use of Chemicals
- Gypsum - Commonly used chemical for alkaline

(4) Crop Rotation
(5) Addition of agricultural waste products
(6) Green manuring

Chapter-8 : River Training

2072/08/16 wed

⇒ Er. Umesh Raut

River Training and its Necessity

The River generally takes off from mountain and flow through the hilly region before traversing the plain. The upper reach of the River may be termed as River in hills. They are further divided into

① Incised or Rockey stage

- erosion
- highly steep
- Bed and bank Consist of Rockey Strata

② Boulder stage

- River passes through Rocky to boulder stage
- River bed Consists of mixture of boulder, gravel & shingles.

In Alluvial Flood plain the River follows Zig-Zag path which is known as ~~Mend~~ Mandering. The River in Flood plain Can be divided into

① Aggrading → थुप्रिनु → थुप्रिनु
② Degrading → बगाउनु
③ stable
④ Braided

Delta formtn (Δ) :- It is the last stage of River just discharging into sea.

River Training :- River training work is defined as Various measures adopted on a river to direct and guide river Flow to train and regulate the River bed and Control change or minimize stream stability Condtn.

Objectives and Necessity of River Training

① To achieve safe passage of flood through the River

② To prevent flooding of surrounding Area.

③ To prevent River from changing its path and outflanking of Bridge, weirs, dams etc.

④ To provide the minimum wtr depth Required for Navigtn.

⑤ To make River Courses stable and Reduce bank erosion.

⑥ To protect the River bank by deflecting the River away from attacked bank.

⑦ To ensure the effective disposal of sediment load.

Types of River Training - 3 types

keeping in view the objects of River training it can be Classified as follow.

① High water Training - for Purpose of Flood Control

② Low water Training - for purpose of Navigatn

③ Mean water training - for efficient disposal of sediment loads & bed loads.

Methods of River Training

The main aim of River Training is to achieve ultimate stability of River with the aid of River-training measures. The stability means the equilibrium stage.

Methods - 6

① Marginal embankment OR Levees

② Guide Banks (Bell's bund)

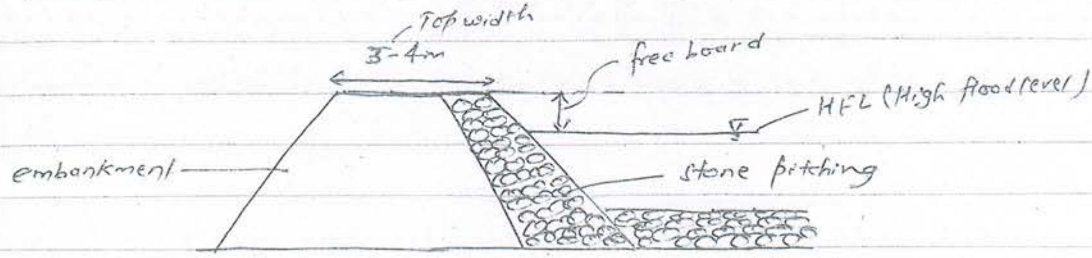
③ Artificial Cut-offs

④ Pitching of bank and Provision of launching aprons / Revetments

⑤ Pitched islands

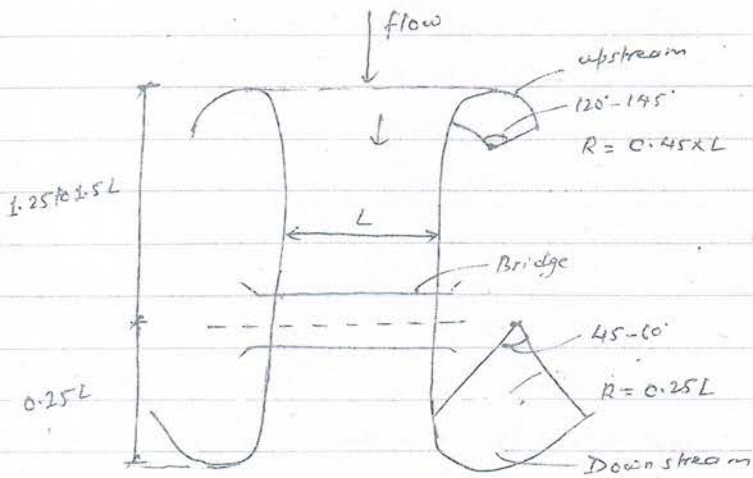
⑥ Groynes or Spurs

① Marginal embankment or levees



These are the earthen embankment which are Constructed parallel to the river bank at some distance upstream (u/s) from it. These embankment wall retains the flood wtr and prevent from spreading into near by area.

② Guide bund/Bank (Bell's bund)



These are the earthen embankment Constructed for the Confining the alluvial river flow within a resonable waterway and guiding it to the hydraulic strctr. Such as Dam, weir, barrage, bridge.

Design of Guide banks

① Water way

$\boxed{P = 4.75\sqrt{Q}}$ P = Laceys wetted perimeter

$$\boxed{L = 1.2P}$$

② Top width level :- Top level of guide bank is kept equal to upstream total energy level plus (+) adequate free board.

$$\boxed{\text{A upstream total energy level} = \text{high flood level before Constrxn} + \text{Afflux}}$$

③ Length of guide bank

Discharge	length of upstream guide bank
< 20 cumec	$1.25L$
$20 - 40\ m^3/s$	$1.25L$ to $1.5L$
$>> 40\ m^3/s$	$1.5L$

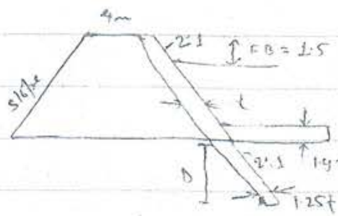
cumec = m^3/s

④ Radius of guide bank

Upstream = $0.45L$

Downstream = $0.25L$

⑤ Cross-sxn of bend



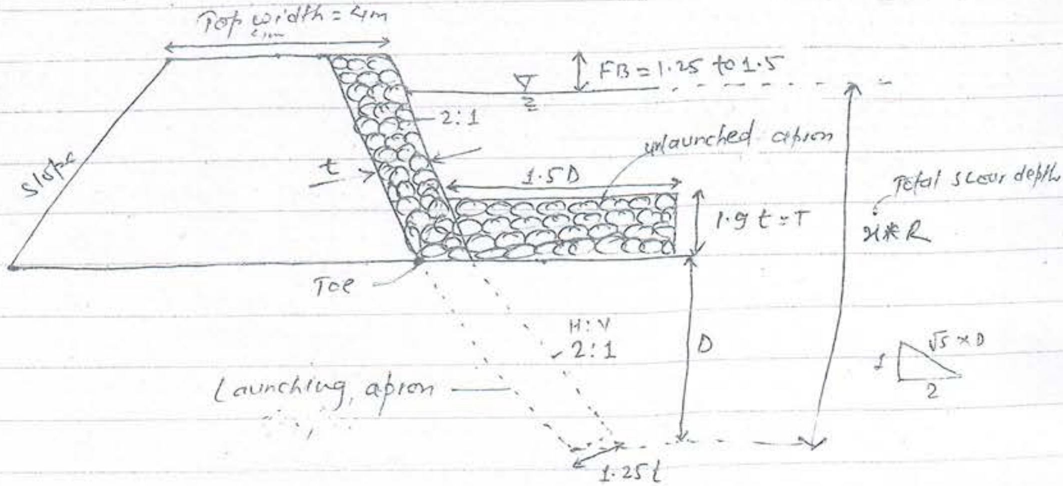
↳ Top width of guide bank should not be less than 4m.

↳ Side slope should not be steeper than 2:1

↳ Minimum free Board of 1.25 to 1.5m

⑥ Slope protxn :- The water face of guide bank is protected by stone pitching to withstand erosive axn of cofr Current.

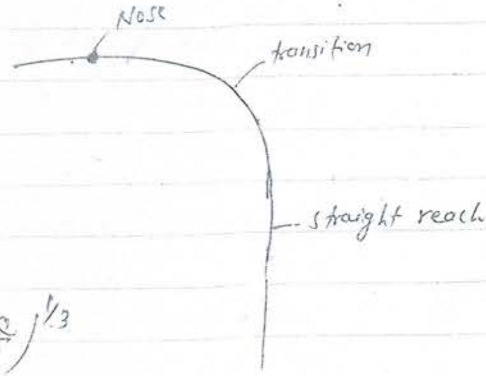
$$\boxed{\text{Thickness of pitching } (t) = 0.06\, Q^{1/3}}$$



⑦ <u>Launching apron</u> :- The stone pitching is extended beyond the toe of the stone pitching in the form of packed stone which is called Launching apron.

- The launching apron is generally laid in width of 1.5 D
  where, D = Scour depth

- Total scour depth below HFL is taken as $x \cdot R$.



<u>Value of x</u>

Nose → 2.25

transition → 1.5

Straight reach = 1.25

and,

R = lacy Normal scour depth $= 0.47\left(\frac{Q}{f}\right)^{1/3}$

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

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

Volume of stone pitching of launched apron per meter length

$= \sqrt{2^2+1^2} \times D \times 1.25t$

$= \sqrt{5}D \times 1.25t$

if width of unlaunched apron is 1.5D

Then,

$$\text{Thickness of Unlaunched apron } (T) = \frac{\sqrt{2^2+1^2} \times D \times 1.25}{1.5D}$$

$$\boxed{T = 1.9t}$$

Numerical VVI

① Design guide bund where, $Q = 6000\,m^3/s$

HFL = 104m

River bed level = 100m

$d_{mm} = 0.1mm$

Soln.

① By Lacey's Regime waterway for plan

$$P = 4.75\sqrt{Q}$$
$$= 4.75 \times \sqrt{6000}$$
$$\boxed{P = 367.93m}$$

② $L = 1.2P$
$= 1.2 \times 367.93$
$\boxed{L = 441.52m}$

Here,

$Q >> 40m^3/s$ So,

③ length of guide bund = 1.5L
= 1.5 × 441.52
= 662.28m = upstream length

④ downstream length = 0.25L = 0.25 × 441.52
= 110.38m

(4) Radius of guide bund

For upstream $= 0.45L$
$= 0.45 \times 441.52$
$= 198.684\,m$

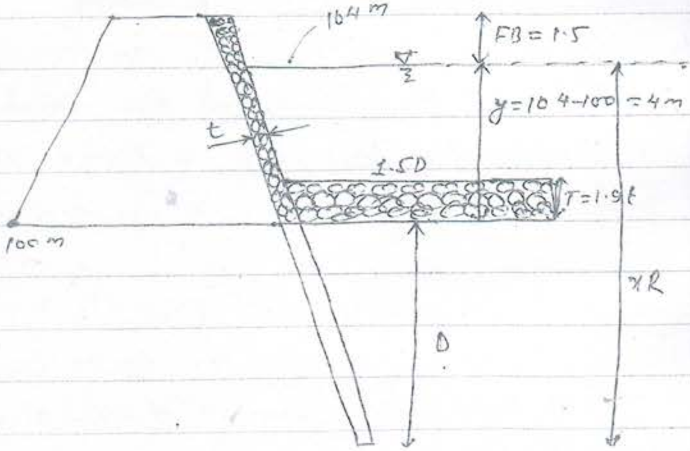
for downstream $= 0.25L$
$= 0.25 \times 441.52$
$= 110.38$

For cross-sxn

Here,

$t = 0.06\,Q^{1/3}$
$= 0.06 \times 6000^{1/3}$

$\boxed{t = 1.09m}$



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

$f = 1.76\sqrt{d_{mm}}$
$= 1.76\sqrt{0.1}$
$= 0.55$

$R = 0.47 \times \left(\frac{6000}{0.55}\right)^{1/3}$

$\boxed{R = 10.42\,m}$

Nose $= 2.25$
$x \cdot R = 2.25\,R$

$HFL = 104m$
$FB = 1.5$

$\therefore$ Top level of guide bund $= HFL + FB$
$= 104 + 1.5$
$= 105.5$

(6) Spurs / Groynes / dikes or transverse dikes

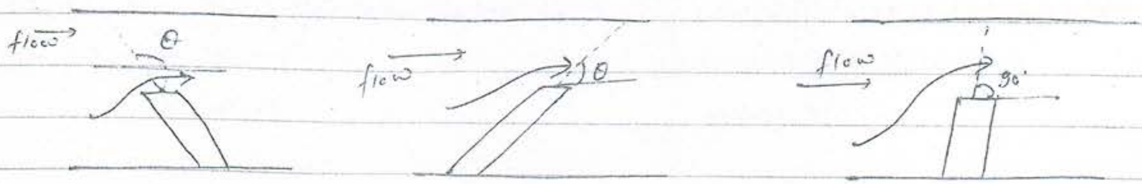


Fig:- Repelling spur $\boxed{\theta > 90^\circ}$ Fig:- Attracting spur $\boxed{\theta < 90^\circ}$ Fig:- Normal spur $\boxed{\theta = 90^\circ}$

θ = Spur orientation

These are the embankment type structure (spur axis) Constructed transverse to the river flow extending from bank into the River. These are constructed in order to protect the bank from which they are extended by deflecting the current away from the bank.

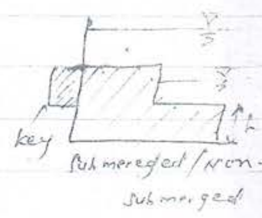
Objectives of Spurs / Purpose

(1) To train River along the desired Course
(2) To Reduce the Concentration of flow at particular point of bank.
(3) To protect the River bank
(4) To Contract River channel to improve its depth

General guidelines for Spur Design

(1) Permeability :- Spur may be permeable or impermeable
- Impermeable spur built of local soil, gravel, stones & Rocks and gabbion while permeable spur generally Consists Timber, bamboo etc.

(2) Spur height :- Spur may be sub-merged or non-submerged
- The height of Non-submerged spur should not exceed the bank height
- Sub-merged spur should have height between $\frac{1}{3}$ to $\frac{1}{2}$ of water depth



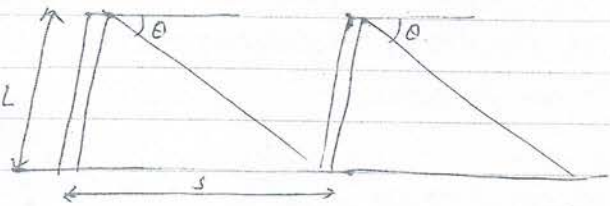
Submerged / non-submerged

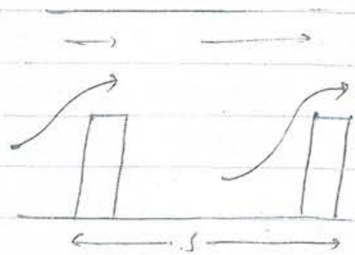
Er. Umesh Raut

(3) Spur orientation : $\theta_1 \rightarrow 90^\circ$ less, grtr or equal.

(4) Spur length :- while choosing the length of spur it is important to consider the safety of opposite bank. As a general Rule the length of spur should not be more than 20% of River width and not less than 2.5 times scour depth.

(5) Spur spacing





$\theta \rightarrow$ expansion angle

$= 15$ to 20°

$= 17^\circ$ is best

$\tan\phi = \frac{L}{S}$

$S = L \cot\phi$

$= L \cot 17^\circ$

$\boxed{S = 3.27L}$

Types of Spurs

(A) Based on Materials of Constrxn

(1) Impermeable spurs - built of local soil, gravel, stones, Rocks.

(2) Permeable Spurs - Timber, bamboo

(B) Based on Sub-mergence

(1) Sub-merged spur - height should be in betn 1/3 to 1/2 of wtr depth

(2) Non-sub-merged spur - heigh should not exceed the bank ht.

(C) Based on function
(1) Attracting spur
(2) Deflecting spur
(3) Repelling spur
(4) Sedimenting spur

(3) Artificial Cut-offs :- A Cut off channel may devlop by itself or may be induced artificially. when a meander goes on increasing and may some valuable land or property become danger than the River Course may be straightened by inducing an artificial Cut-off.
- Cut off help in Reducing flood height and flood periods

(4) Pitching of banks and Provision of launching aprons / Revetments
Pitching of bank & Provision of launching is Called Revetments.
- stone pitching is done to protect Banks of River & other like concrete blocks, brick lining or by growing vegetative Cover also Bank of River Can be protected.
- launching apron is done to prevent scour at toe & fall of slope pitching

(5) Pitched Islands :- Pitched island is an artificially Constructed island in the River bed & is protected by stone pitching on all sides.
- It help to attract Current towards themselves & thus Reduce Concentrat on opposite banks.

(VI) Stages of Rivers and Their Meandering Process

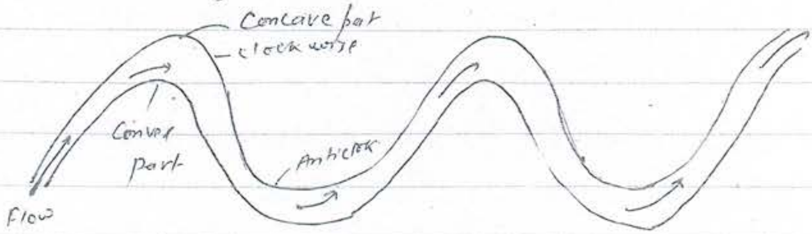
- A River has various stages in its path of flow from higher to lower level. In Nepal, generally River starts at Mountains & ends at plain

Stages of River - 2 stage

(1) Upper Reaches → (a) Mountainous River
→ (b) Sub-mountainous River

(2) Lower Reaches → (a) Alluvial River
→ (b) Delta

Meandering Process



Meandering process

A River following Zig-Zag path is Called Meandering process.

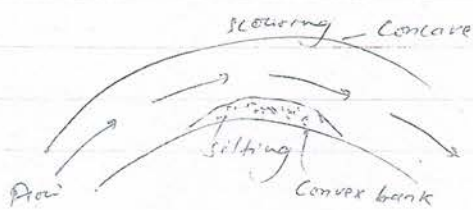
Causes of Meanderring

- Local bank erosion & over quantity of silt Causes River meandering.
- Primary cause of meandering is excess of total charge during floods when excess of turbu-nce is devloped

Causes

(1) Local bank erosion
(2) Excess of total change during floods
(3) Excess of turbulence devloped
(4) High Concentratn of sediment.

↳ Due to Cutting in one side & depostn on another side also meandering occur.
↳ The process by which a River changes its original path by deviation due to high Concentration of silt & excess floods is Called Meandering process.



Meandering process

Factor Controlling Process of Mendering

(1) Valley slope - Change in valley slope always produces a Change in meandering pattern

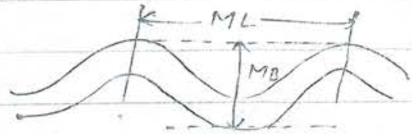
(2) Stream load : Compostal of stream load as well as its movement affect meandering process

(3) Discharge : There is Relatalship betn rate of discharge to Rate of bed load movement

(4) Bed and side Resistance :- with the soil characteristics, the bed & side resistance affects & that affects meandering process.

Meandering parameter

(1) Meandering length (ML) :- It is distance between the tangent point of one curve to another curve.



(2) Meandering width (MB) :- The transverse distance between the apex point of one Curve to apex point of next reverse Curve is Meandering width.

(3) Meander Ratio $= \dfrac{\text{width of Meandering}}{\text{length of Meandering}}$

$= \dfrac{MB}{ML}$

Effects of Degradation on hydraulic (River) structure

7. The process of loosing of River bed (scouring axn) is called Degradatn of River.

(1) The degrading rivers increase the slope of the River

(2) When cut-off forms the River, that will be degrading river.

(3) As sediment load decreases suddenly, degradatn occurs in downstream (u/s)

(4) when a sediment load is held up by a dam upstream, a degrading reach may devlop
in downstream (d/s)

(5) increase in scour depth.

(6) structural instability

12/08/25 friday

CHAPTER-9: Planning and Management of irrigatn system

⇒ Er. Umesh Raut

General Irrigation System planning

Planning is necessary for any project design. In the planning we have to first analyse the water requirements and the water Resources

- secondly the availability of land
- Third
  - → cropping pattern
  - → Croping intensity
  - → Types of crop
  - → Type of soil

Stages of planing

1. Preliminary planning including feasibility study
2. Detail planning of water and water use
3. Design of irrigtn structures and Canals.

Factors to be Considered in planning stage

1. Type of project and general plan of irrigtn wrks.
2. Location & type of irrigtn land
3. Crop water Requirement
4. Culturable area
5. Availability of water
6. Needs of immediate and future drainage
7. Cost of works
8. Evaluatn of benefits
9. Methods of financing
10. Annual Cost of water to the farmer
11. Cropping pattern
12. Environmental aspects

Organisation and irrigtn Management

There is mainly only one organisation involved in the field of irrigtn ie "Department of Irrigtn" under Ministry of Water Resource. All type of projects are monitored by DOI (Departm of Irrigtn) of Nepal.

Director General
(chief engineer)
↓
Deputy Director General
↓
Regional Director
↓

Divisional Engineer
↓
Sub-dvsNl
↓
Sub-Engineer
↓
Associate Engineer

Sub-divisional engineer
↓
Sub-engineer
↓
Associate organiser

Fig:- Administration organisation Flowchart

Irrigation Management :- Irrigatn Management is a key for efficient and timely distributn of water in Canal, effective operation and maintenance of irrigatn system.

- The main objectives of irrigtn management is to supply and apply the right amount of wtr at right place and at right time.

Management of irrigatn system includes

① Land devlopment
② Field applicatn of water
③ Availability of water and its optimum use
④ Irrigtn by Rotatn
⑤ cropping pattern
⑥ Educatn to farmers
⑦ New technology of farming
⑧ Method of charging.

Aims of Irrigtn Management

① Economical use of water to achieve maximum benefits.
② Reliable & efficient service to the farmers at right time, right place in right quantity.
③ Protecn and Maintenance of irrigtn works.
④ Transport of water without any maxm loss
⑤ Participtn of farmers in organistn & management
⑥ Use of new techniques of irrigtn.
⑦ To manage the field irrigtn of water
⑧ To Regulate and Control the flow of water as per desired
⑨ Protection from water-logging
⑩ Methods of charging so as to minimize the loss of water due to careless handling of farmers.

① Operation and Maintenance of Irrigtn System

Operatn is defined as the process of storing, conveying, delivery, regulating, measuring, distributing & applying the right amount of water at right time in right place for a certain required duratn.

Objectives of opertn

↳ To distribute the availabe irrigtn water to the crops
- at Right time
- at right place

Er. Umesh Raut

- for required durtn
- at adequate quantity
- with least cost
- effectively & efficiently

Maintenance of irrigtn System

Objectives of Maintenance

- To operate the planned system efficiently to fulfill their operational requirement
- To minimize the rate of deterioratn of structures, canals etc.
- To ensure the continued efficient opertn.

Impoartance of Maintenance

If an irrigtn system is maintained properly & timely the following defects may not results

1. Deterioratn of irrigtn system
2. Reduced water supply
3. Poor canal opertn

Maintenance Problems in Nepal

Broadly in Nepal, two types of irrigtn system

1. Agency managed irrigtn system
2. farmed managed irrigtn system

Types of Maintenance

1. Regular maintenance → almost all day maintenance
2. Periodic maintenance - maintenance once in a year to prevent possible major damages
3. Emergency maintenance - done in emergency condtn

   Deffered mainten- -poned maintenance is call
4. Deffered maintenance - Some maintenance works are not much critical & could be postponed this post
5. Specific maintenance - done for specific components like Drainage network, Road network, irrigtn structure & channel maintenance etc.

<u>Institutional Aspects of irrigtn/ System Management</u>

Needs of institutional devlopment for irrigtn/ system

1. Registration of users institutn/
2. Users Constitution
3. Awareness development & training needed.
4. Awareness for Responsibility & duty
5. Awareness of Constrxn. system involvement of users during this period.
6. Responsibility of Constrxn. maintenance and operatn/ management.
7. Training need for irrigtn/
8. Ownership Responsibility
9. Effective management & effective Relatnship devloped betn users & irrgtn/ office.

2072/08/25 friday

Chapter-10: Introdxn to farmer Managed irrigtn System (FMIS)

⇒ Er. Umesh Raut

Introdxn to FMIS (= farmer Managed Irrigtn System)

From early days, water is impoartant part for farmers for agriculture. In Nepal irrigtn devlopment is in the hand of people for thousand of years which give rise to FMIS.

Out of Total irrigated land 70%. is managed and operated by farmers and 30%. by government.

- It is estimated that there are abt. 1700 FMIS Units in Terai and over 15000 in hills of Nepal. They include Command area Ranging from 10ha to 15000ha.

- In FMIS. System, Non-structural Components are given more impoortance.

A/c to Norman uphoff irrigtn system are:- — 3 activities

① Water use activity - water acquisition, allocation, Distributn, drainage

② Controlled Structure activities - Design, Constrxn, opertn, Maintenance

③ Organizational activities - Decision making, Resource Mobliztn, Communicatn, Conflict resolutn.

Characteristics of FMIS

① Pluralism: FMIS Comprises diversity of caste/ethenic group in variable locatn of the Country.

② Common Resource base: FMIS Common property is water which help them to specify water right to Community. Common Resourse base help to Unite Community members for Collective axns.

③ Indigenous governance: FMIS represent an indigenous autonomous model of gover--nance. They exist separetly from Non-govermental agency.

④ Collective self management: FMIS enforces Collective self management.

Farmers participation (Participatory approach)

Farmers participation means the involvement of members of farmers' organisation in the decision of organisation that affect irrigated agriculture based on livelihood for the member of farmers.

The main objectives of farmers participation is to manage irrigation system.

Water Allocation

The assignment of water from an irrigation system to an individual farmer or farmers group with specification that who will get how much water and when?. As a principle agreed upon by farmers, It also reflects the Right of a farmer to irrigation water.

- The principal of water allocation may differ from crop to crop and may change within the same croping season.

Some of widely adopted Principles

- In proportion to the size of land holding in the Command area.
- on the basis of initial investment made
  - → Purchased or inherited share
  - → labour contributed for maintenance
  - → farmer's demand or request
  - → Time

Objectives of irrigation policy

① To develop irrigation service and increase agricultural prodxn.

② To bring uniformity in all institutions & organisations in development and extension of irrigt

③ To decrease GOV's involvement in the constrxn, maintenance & operation of irrigation scheme increasing participation of organised user.

④ To continue the Nepali farmer's tradition of constrxn & managing irrigation system more stable & extensive.

Er. Umesh Raut

(5) To increase efficiency of governmental and Non-govermental institutions involved in irrigatal devlopment.
(6) To increase Research Capability in irrigtal technology & management system.

Water Resource strategy

Water is one of the precious gift of Nature for human Survival because No life without water.

water uses for - drinking, Cooking, bathing, washing etc.
- irrigtal
- Hydropower
- for industry

At the microlevel, there is already crisis and Conflicts on water use. As time passes, such Conflicts will become more serrous & more Common. So Scientific Sustainable and ~~don~~ agreement based mechanism (आविष्कारको इलाज) to manage the available water Resources should be found to manage the impending water Crisis.

Nepal's Content

HMG started the process of water Resource strategy formultal in 1955 with a one year problems in the water sector. This was followed by Current phase of strategy formultal study phase to identify issues and.

In January 1999. A team of Consultants the WRSF (water Resource strategy formultal) Consortium was engaged on a 2-year Contract basis. The work of Consultants was utilised by Core team of experts in water and energy Commission secretariat (WECS) to further analyze, test & devlop strategy statements through professional work, experts group meeting & stake holder Consultantions.