11

(178)

-: HAND WRITTEN NOTES:-

OF

CIVIL ENGINEERING

(1)

-: SUBJECT:-

IRRIGATION & HYDROLOGY

11

2

Hydrology :-

Hydrology is the Science of Water which deals with the occurrens circulation & distribution of water on Earth Surface and its atmosphere.

3

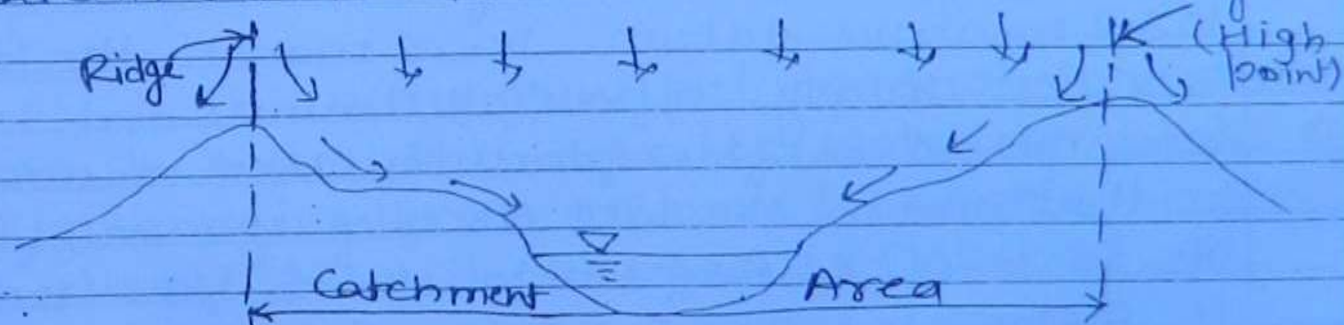
Hydrological Cycle :-

This is a cycle in which water moves from one phase to another face.

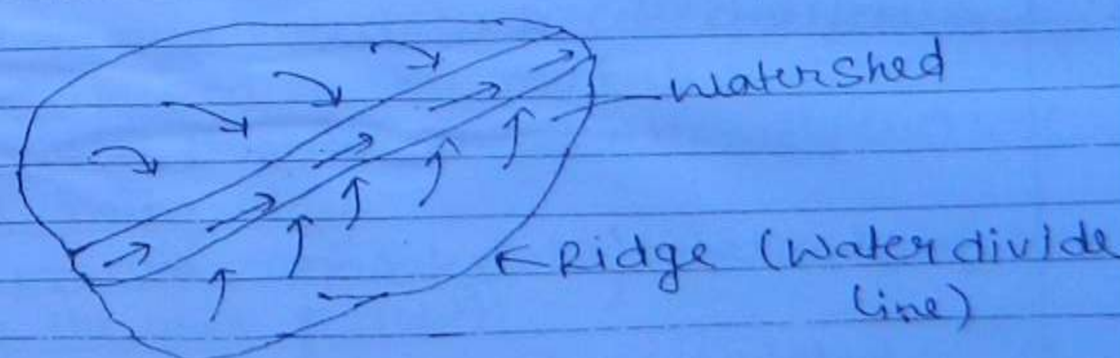
Residence time :-

This is the time taken by a water particle in ~~proces~~ crossing ~~one~~ particular face of hydrological cycle.

Catchment Area :-



The Area draining into a river or stream is called the Catchment area for that particular stream or river.



Catchment area is also called as water shed.

(4)

Ridge:- It is the line which demarkets one catchment area from its neighbouring area. This is also called as water divide line or divide line.

Water budget Equation :-

This Eq^n is based on the law of conservation of mass and according to it,

Mass Inflow − Mass outflow = Change in Storage.

A lake has a water surface elevation of 103.2 m above datum. In a month the lake receives an avg. inflow of 6 cumecs and in the same period outflow from the lake was 6.5 cumecs. In that month the lake receives the rainfall of 145 mm & evaporation from lake surface is estimated as 6.1 cm. Calculate the water surface of elevation at the end of month. Surface area of lake is 5000 hectare.

$$\text{Inflow} = 6 \text{ cumecs} + 145 \text{ mm}$$

$$\text{Outflow} = 6.5 \text{ cumecs} + 6.1 \text{ cm}$$

$$\text{Inflow} = (6 \, m^3/s \times 60 \times 60 \times 24 \times 30) / \text{Area} + .145 \, m$$

outflow = $6.5\ m^3/s \times (60\times60\times24\times30)$ / Area
(5) $+ .061\ m$

$= \left(\frac{6\times60\times60\times24\times30}{5000} + .145\right) - \left(\frac{6.5\times60\times60\times24\times30}{5000} + .061\right)$

$= .456 \qquad\qquad 0.398$

$103.2 + 0.456 - 0.398 = 103.258\ m$

Precipitation :-

Precipitation denotes all forms of water or moisture that reaches the Earth Surface.

following are the different forms of precipitation :-

1) Rain :-

This is the principal mode of precipitation in India. This denotes water droplets of size ranging from 0.5 mm to 6 mm. On the basis of Intensity rainfall is classified as

0 − 2.5 mm/hr ⟶ Light rain
2.5 − 7.5 mm/hr ⟶ Moderate
> 7.5 mm/hr ⟶ Heavy Rain

NOTE In India rainfall data is collected Every day at 8:30 A.M and If this rainfall is more than 2.5 mm on a perticular day then that day is called rainy day.

from mm → we can calulate total vol. by multiplying the area × depth.

and its discharge can be calulated m^3/s

Date:

Snow :- These are Ice Crystal having a density of ·1 gm/c.c.

6

drizzle :- These are fine droplets of water. whose size is less than ·5 mm and Intensity is less than 1mm/hr.

Glaze :- When droplets of water comes into contact with cold ground (appr. 0°c) then water droplet is converted into ice which is called glaze.

Sleet :- This denotes frozen raindrops of transparent grain.

Hail :- These are lumps of Ice. whose size is more than 8mm.

In India primary mode of precipitation is orographic.
cyclonic
convective
orographic → bcoz of himalya

A. Cloud measured in okta ($1/8^{th}$).
~~f~~ Sky is fully from cloud than 8 okta = 100%
only half ~~pa~~ part then — 4 okta = 50%

Amount of rain collected by a raingauge in the last 24 hours is called daily rain fall. and the amount collected in the one year is called Anual Rainfall. (7)

Avg. anual rainfall is the avg. value of antual rainfal for the last 35 years

Index of Wetness :-

This Index is used to find the rainfall variation for ~~a~~ a particu-lar year and it is calculated as.

$$\boxed{\text{Index of Wetness} = \frac{\text{Rainfall in a year}}{\text{Avg. Annual Rainfall}} \times 100}$$

eg. Index of Wetness $= \frac{90\,cm \times 100}{120\,cm} = 75\%$

This shows that the rainfall deficiency is 25%.

If rainfall deficiency is in b/w 30–45%. It is called large deficiency.

If it in b/w 45–60% It is called Serious deficiency

If it is more then 60% It is disaster deficiency.

If Index of Wetness is Equall to 100%.. It Indicates that rainfall is normal.

If this Index is less than 100% It is called a bad year.

If this Index is more than 100% then It is called a good year.

Drought :-
This is the climatic situation which is characterised by

Drought can be classified as follows:-

(8)

1 Metrological drought :-
This represents deficiency in precipitation. If the decrease in precipitation is more than 25%. It is called
If it is in b/w 25% - 50% it is called modrate drought
~~If it is more than 50% it is called~~

- An year is called drought year if area effected by drought is more than 25% of the total area of the country.

If drought occures in an area with a probability of ·2 to ·4 then that area is called drought prone area. and if this probability is greater than ·4 it is called chronologically drought ~~porne~~ prone area.

2. Hydrological drought :-
This refers to the below avg. value of stream flow, water content in lakes, reservoir, underground water

Agriculture drought :-
This type of drought is charactrised by deficiency of moisture available for a plant grows. This can be calculated using a

factor Aridity Index.

$$\text{Aridity Index} = \frac{PET - AET}{PET} \times 100$$

PET → Potential evapo-transpiration and this refers to the water consumed by the plants if sufficient moisture is available during the plant growth period. This is also called consumptive use.

AET → It denotes actual evapotranspiration and it is the actual moisture available under the prevailing condition.

If Aridity Index is in b/w 0-25%.. It is called mild Arid region.

0-25%	Mild
25-50%	Modrate Arid region
>50%	Sarious Arid region

Avg. rainfall and design of Rain gauge:-

In order to find Avg. rainfall over a basin or catchment area the rainfall is measured at a number of raingauge station which are suitabaly located.

The network density of rainguage depend upon

1. Magnitude of Rainfall.
2. Topography of the region
3. Accuracy desired.

Measurement of Rainfall :- (10)

Rainfall is measured in term of depth (vol. per unit area) to which water would stand on an area. If all the rainwater was to be collected on it.

The rainfall is measured by a rainguage which is also called Pluviometer or Ombrometer or Hyetometer.

A Rainguage essentially consist of a cylindrical vessel assembly which is kept in open to collect the rain water.

Requirments for a Rainguage :-

The rain measuring Instrument s/d be surrounded by open an open fenced area. of atleast 5.5 x 5.5 m and It s/d be kept atleast 30m away or twice the height of building or obstruction.

Rain guage may be brodly classified under two heads :-

Non Recording Rain guage :-

These are simple cylindrical vessel of known area and having graduations. In India the most commonly used non recording rain guage is Symon's Rain guage. Which has a collecting diameter of 127 mm. (5")

different type of Rain gauge see from text Book)

Date:

(11)

Recording

This Rain guage produces a Continious rainfall variation against time. With this data the rainfall Intensity V/s time curve Can be plotted and the accumulated rainfall V/s time curve Can also be plotted.

T

NOTE:- In India the float type rain guage also called as natural syphon type rain gauge is Ex

Latest Improvement in rainguage are as follows

1) Telemetring rainguage :-

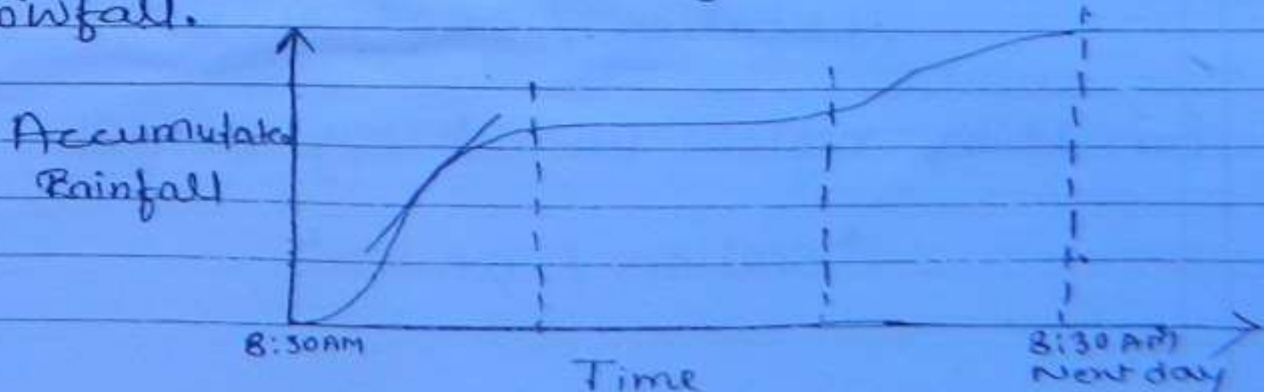
This is basically a recording type rainguage which contains Electronic unit to transfer data to the base station.

2) Radar technology :-

This is used to measure the areal Extent Location and movement of rainstorm.

Rainfall over a large area can be measured with a good accuracy.

Metrological radars operate at a wave length range of 3 to 10 cm. in which 10 cm wave length (λ) is to be used for heavy rain and 3 cm wave length is to be used for Light rainfall and Snowfall.



Rainguage Network :→

For andlysing rain strom finding peak flood discharge, forcasting flood drought a proper distribution of rain guage station is required.

Following are the recommandation of world metrological organisation :-

for ~~plak~~ flat region of temprate, modrate meditarion region

1 station / 600-900 km^2

for mountaine region of temprate,

1 station / 100 - 250 Km^2

For arid and polar zone the requirment is

1 station / 1500 - 10000 Km^2 (bcoz Rain fall ↓)

Indian meteriological dept. Recomandation.

for Plane region 1 Station / 520 Km^2

for regions having Avg. Elevation of

1 Station / 260 - 390 Km^2

for Hilly regions with heavy rainfall

1 Station / 130 Km^2

Q. A Catchment has 6 rainguage station and in a year the anual rainfal recorded by the diff. rain gauguages are as follows.

(13)

Station	P (cm)	$(x-\bar{x})^2$
A	82.6	$(118.6-82.6)^2$
B	102.9	$(118.6-102.9)^2$
C	180.3	$(118.6-180.3)^2$
D	110.3	$(118.6-110.3)^2$
E	98.8	$(118.6-98.8)^2$ 6451.6
F	136.7	$(118.6-136.7)^2$

For a 10% Error in the Estimation of mean rainfal calculate the optimum no of rain guage station?

Sol.

$$N = \left(\frac{C_v}{\epsilon}\right)^2$$

$\epsilon = 10\%$ $\qquad C_v = \frac{\sigma}{\bar{x}} \times 100$

$$\bar{x} = \frac{82.6+102.9+180.3+110.3+98.8+136}{6}$$

$$\bar{x} = 118.6$$

ΣP $\qquad \Sigma (x-\bar{x})^2 =$

$$\sigma = \sqrt{\frac{\Sigma(x-\bar{x})^2}{n-1}} = \sqrt{\frac{\quad}{6-1}}$$

Page No. ____ Date: ____

Catchment has 6 rainguage station and in a
year the anual rainfal recorded by the diff.
rain gauguages are as follows.
station

station	P (cm)	$(x-\bar{x})^2$
A	82.6	$(118.6-82.6)^2$
B	102.9	$(118.6-102.9)^2$
C	180.3	$(118.6-180.3)^2$
D	110.3	$(118.6-110.3)^2$
E	98.8	$(118.6-98.8)^2$
F	136.7	$(118.6-136.7)^2$

6851.6

(14)

a 10% error in the Estimation of mean
rainfal calculate the optimum no of rain
gauge station ?

$$N = \left(\frac{C_v}{\epsilon}\right)^2$$

$\epsilon = 10\%$.

$$C_v = \frac{\sigma}{\bar{x}} \times 100$$

$$\bar{x} = \frac{82.6 + 102.9 + 180.3 + 110.3 + 98.8 + 136.7}{6}$$

$\bar{x} = 118.6$

$$\Sigma (x-\bar{x})^2 =$$

$$\sigma = \sqrt{\frac{\Sigma(x-\bar{x})^2}{n-1}} = \sqrt{\frac{\quad}{6-1}}$$

Q. The Information available from an Isohytel ~~map~~ map of 1100 Km². basin area is as follows.

Zone	Area	Rain guage	Normal Anual Rainfall
I	85	A	120
II	290	B C	95 96
III	395	D E F	60 65 70
IV	230	G + 1	45
V	65	H	21
VI	35	— + 1	—

13

How many additional rainguage stations may be required if desired error in the mean value of rainfall is not to exceed 10 cm. Suggest how would you proposed to distribute these addition ~~rain~~ guage. Stations.

Sol

$$N = \left(\frac{C_v}{\epsilon}\right)^2$$

$$C_v = \frac{\sigma}{\bar{x}}$$

$$\bar{x} = \frac{120+95+96+60+65+70+45+21}{8}$$

$$\bar{x} = 71.5 \text{ cm}$$

$$\Sigma(x-\bar{x})^2 = (71.5-120)^2 + (71.5-95)^2 + (71.5-96)^2 + (71.5-60)^2 + (71.5-65)^2 + (71.5-70)^2 + (71.5-45)^2 + (71.5-21)^2$$

Date:

$$\Sigma(x-\bar{x})^2 =$$

$$\sigma = \sqrt{\frac{\Sigma(x-\bar{x})^2}{n-1}} = 31.47$$

$$C_v = \frac{\sigma}{\bar{x}} \times 100 = 44.01$$

16

$$\epsilon = 13.98 \qquad \epsilon = \frac{10}{71.5} \times 100$$

$$N = \left(\frac{C_v}{\epsilon}\right)^2$$

$$= \qquad = 9.9 = 10 \text{ Stations.}$$

Provide 2 additional Rainguage one at Zone VI & other at IVth.

Estimation of missing rainfall data :-
If on a particular day the data of rainfall at station x could not be recorded at it is required to find an approximate value of the missing data. then the following method is to be applied.

Few rainguage stations close to station 'x' (which is the defective station) are selected and the rainfall data $P_1, P_2 ------ P_n$ is noted

P_x is the rainfall data of station 'x' which is missing and has to be approximately calculated

Let $N_1, N_2 ------ N_m$ & N_x be the Normal Avg. anual precipitation of all the stations

Date:

Case 1 When N_x differs from $N_1, N_2 ---- N_m$ by 10% or less, then P_x given by

$$P_x = \frac{P_1 + P_2 + ---- + P_m}{m} \quad (17)$$

Case 2 When N_x differs from one or more of $N_1, N_2 ---- N_m$ by more than 10%, then P_x is calculated using the normal ratio method

$$P_x = \frac{N_x}{m}\left[\frac{P_1}{N_1} + \frac{P_2}{N_2} + ---- + \frac{P_m}{N_m}\right]$$

Q

A Normal Anual rainfal at station A, B, C & D In a basin are 80.97, 67.59, 76.28, 92.01 cm respectively. In a particular year the station D was Inoprated, and the station A, B & C recorded anual precipetation of 91.11 then 71.23 and 79.89 cm (respectively) find the rainfall value at station D for that year.

Sol

$$N_x = 92.01 \text{ cm} \begin{cases} \rightarrow 1.1 N_x = \\ \rightarrow 0.9 N_x = \end{cases}$$

use Normal ratio method (bcoz it is in range)

$P_x = 92$

$$P_x = \frac{92.01}{3}\left[\right.$$

$$P_x = 99.40$$

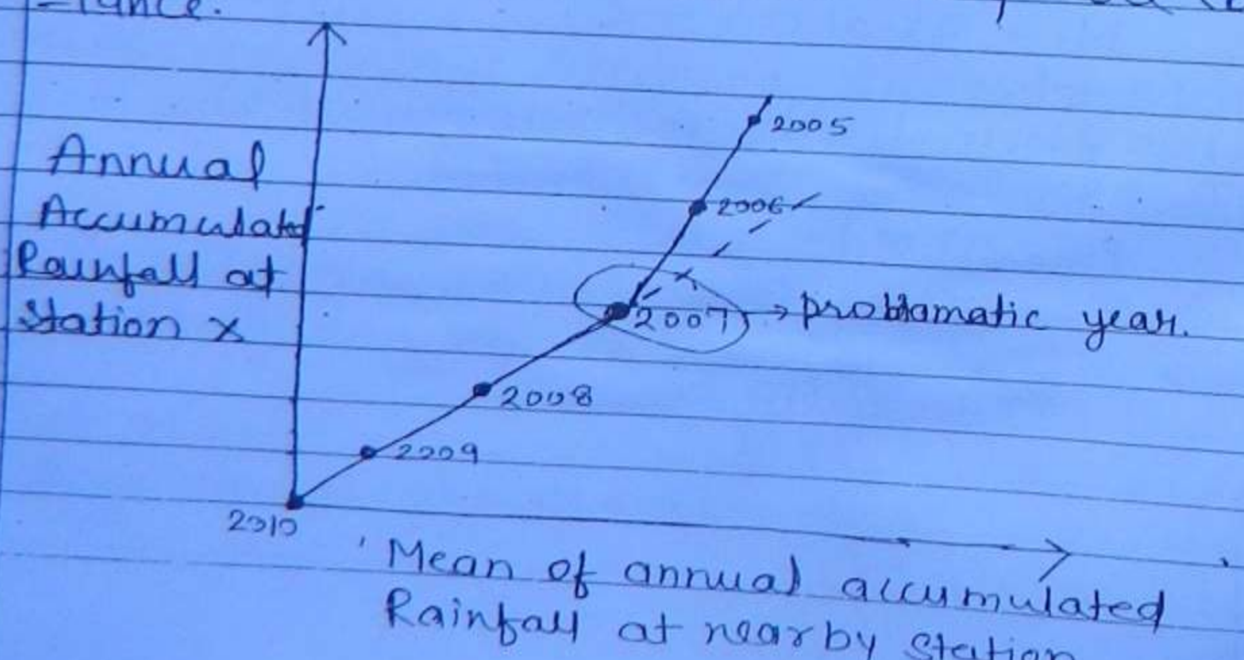
Page No. ____ Date:

Q. Consistency of Record :→ 18

Whenever their is change in the condition prevailing at any of the rainguage station ~~then~~ there will be Inconsistency in record available for that period. This Inconsistancy ~~be~~ may be due to the following factor.

1 Shifting of rainguage station to new station
2 Neighbour hood of the station under going ~~to~~ a significant change.
3 Change in the Ecosystem which may be due to factors like forset fire Land Slide Etc.
4 Due to observational Error.

The Inconsistancy of record can be found out by a technique called double mass curve technique.
This technique is based on the principal that when rainfall data comes from the same set of earlier observation they are consistance.



Method :→

→ few stations closed to the problamtic station is selected and a graph is plotted in b/w accumulated avg. rainfall at problamatic station and the mean of accumulated annual rainfall at the near by station.

NOTE:- bcoz on both side accumulated rainfall is taken

Double mass curve technique is also used for checking Airthmatical Error in trans-fering the rainfall data from one record to other.

Determination of Average Rainfall in an Area:-
If a catchment contain more than 1 raingauge station then the Avg. rainfall can be calculated using the following methods

1 Airthmatical mean Method :-
This method is suitable when rainfall is uniformly distributed and the area is not very large. ~~and~~

$P_1, P_2, ------ P_n$

$$P_{avg} = \frac{P_1 + P_2 + ----+ P_n}{n}$$

2. Theissen polygon method :-
This method is also called as weighted area method bcoz the rainfall data at every station is given a weightage on the basis

Date:

of an area close to the station. (20)
This method is suitable when area is large and rainfall varies from place to place.

→ This method is more accurate than the Airthmatical ~~of~~ avg. method.

$$P_1, P_2 \text{ ------ } P_n$$
$$A_1, A_2 \text{ ---- } A_n$$

$$\boxed{P_{Avg} = \frac{P_1 A_1 + P_2 A_2 + \text{----} + P_n A_n}{A_1 + A_2 + \text{---} + A_n}}$$

$$P_{Avg} = P_1\left(\frac{A_1}{A}\right) + P_2\left(\frac{A_2}{A}\right) + \text{------} + P_n\left(\frac{A_n}{A}\right)$$

weightage factor

ES 09

Q (i) The recorded annual rainfall from five rain gauge station in a catchment & the corresponding theisen polygon area are as follows

Polygon Area (cm^2)	P (cm)
25	125
30	175
30	275
10	275
5	325

If the scale of the map is 1:5000 then Estimate the Vol. and EUD (Equivalent uniform depth) (P_{avg})

(21)

$\Sigma A = 100\ cm^2$

$\therefore P_{avg} = 125 \times \frac{25}{100} + 175 \times \frac{30}{100} + 275 \times \frac{30}{100} + 275 \times \frac{10}{100} + 325 \times \frac{5}{100}$

$P_{Avg} = RUD = 210\ cm$

$Vol. = depth \times Area$

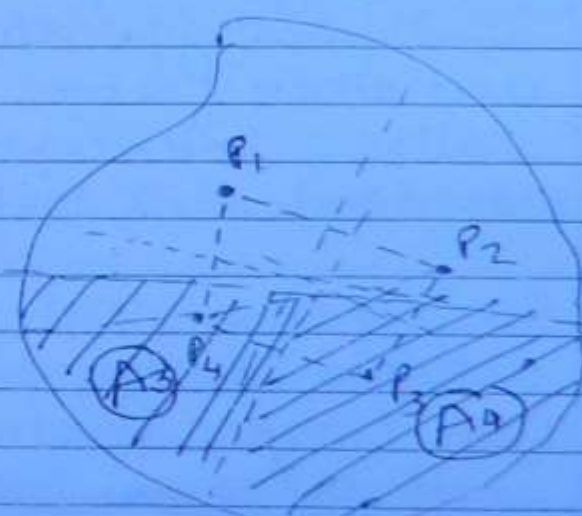
$= 2.1 \times [\underbrace{100\ cm^2 \times 5000^2}_{\text{Actual field Area } (cm^2)}] \times 10^{-4}$

$Vol. =$

(ii) Find the anual Avg. discharge at outlet If runoff Cofficient is 0.3?

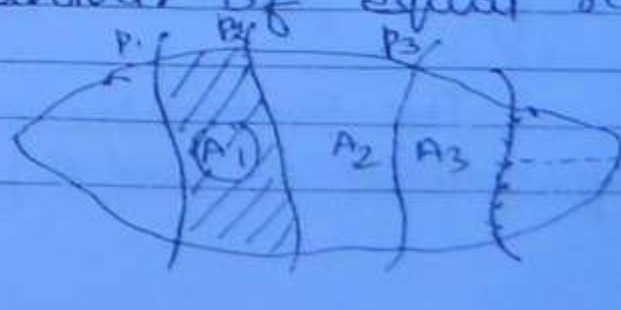
Sol

$Q_P = \frac{1}{36} K p_c A$



Q:3. Isohytal Method :-

An Isohyete is a line joining points of Equal rainfall magnitude and Isohytal maps are one which shows the countour of Equal rainfall



$$P_{avg} = \frac{\left(\frac{P_1+P_2}{2}\right)A_1 + \left(\frac{P_2+P_3}{2}\right)A_2 + \cdots + \left(\frac{P_{n-1}+P_n}{2}\right)A_{n-1}}{A_1+A_2+\cdots+A_{n-1}}$$

22

To Calculate Area We use Overlay Grid Method.
This method of calculating Avg. rainfall is Superior than the previous two methods if the number of stations are large. bcoz pricisenass of Contour is more for large no. of Statation.
We are Conversing in this method.

Losses from precipitation :-
In hydrology runoff is the primary Concern and other factors like Evaporation transpi-ration, Interception loss, depression losses are treated as losses.

Evaporation :- This is the process in which a liquid changes to gaseous state at the free surface of liquid.

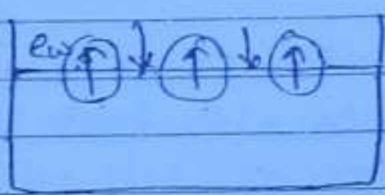
Evaporation is basically a colling process in which lattent heat of vapourisation must be provided by the water body

Factors affecting Evaporation rate :-
Vapour pressure :-
The Evaporatio rate is directly perpostional to the difference b/w

Actual Vapour pressure and the Vapour pressure of the saturated air,

$$\boxed{E \propto (e_w - e_a)}$$ (23)

e_a = Actual prevailing vapour pressure.
e_w = Sat. vapour pr. of water vapour.

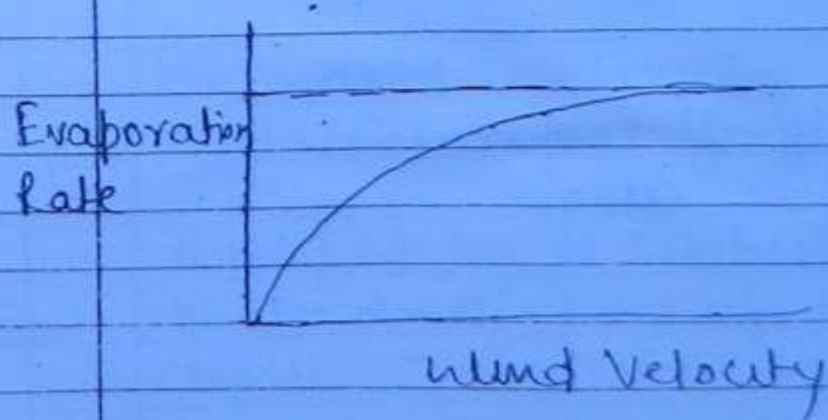


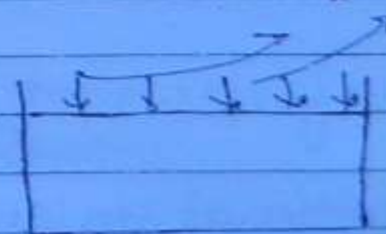
$e_w > e_a$ Evaporation will continiou.

Temperature :-
Rate of Evaporation is directly perportional to temperature.

Wind Speed :-
Vaporisation rate Increases ~~with~~ due to faster wind.
If the wind velocity is very high and ~~it~~ is sufficient to remove all the vapour above the water surface then any further Increase in wind velocity will not Increase the Evaporation rate.

$E \propto (e_w - e_a)$





(24)

5 Soluable Salts :-
W When Salts are present the vapour pressure is decrease and Consecutivly the rate of Evaporation also decreases.

NOTE Evaporation from Sea water is about 2-3 % less than that of fresh water.

In Summer the shallow depth of water permits the rapid increase of temprature during the day time and hence Evaporation rate is more in shallow water bodies.
In winter the temp. of Surrounding air falls but the temp. of deep lakes

Co

Surface Area :-
Rate of Evaporation is directly perportional to Surface area.

Conservation of Reservoir Water :-
Following methods may be adopted to reduce Evaporation loss.

(25)

1. Reduction in Surface Area :-
2. Providing mechanical Covers in the form of Shades.
3. Providing chemical films :-
Certain chemicals Such as hexadecanol (cetyl alcohal) ~~or~~ octadecanol (Stearyl alcohal) can be use to reduce the Evaporation.
A thin colourless, odourless and nontoxic film is formed on the water Surface which recovers Easily If punchered. and which allow free Circulation of oxygen and CO_2.

NOTE Evaporation rate can be reduce to a minimum If a film pressure of 0.04 N/m is maintained
Sur. tension

~~Two~~ Through Experiment it has been found that Evaporation can be controlled upto 60% by using cetyl alcohal and In field Conditions their is 20-50% reduction in Evaporation rate.

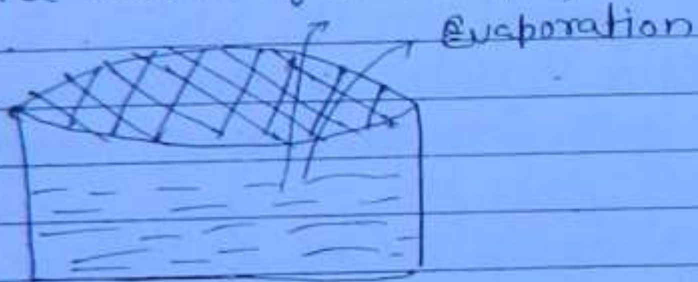
Methods to determine Evaporation :-
1) Class A Evaporation pan :-
This is a pan developed by U.S Wheather beauro and is a standard pan of 1210 mm dia - 255 mm depth.

The pan is normally made of unpainted G.I Sheet. In case of persistence corrosion monal metal is used.

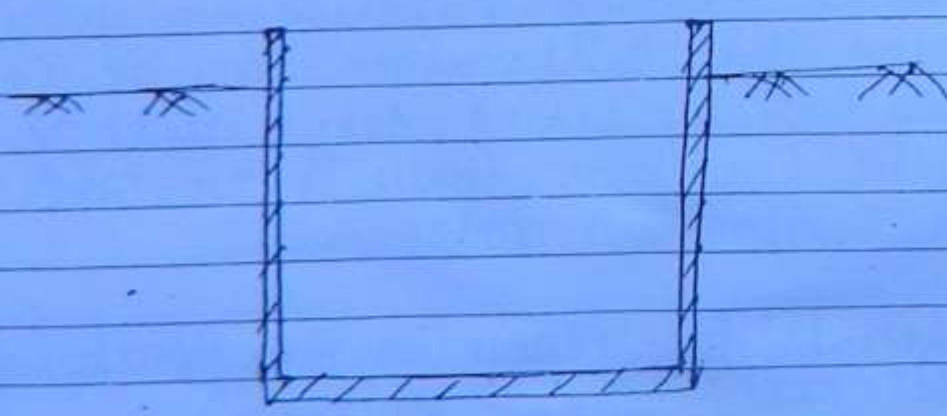
2 ISI Standard pan:-

This is modified form of Class-A Evaporation pan. in which diameter is 1220mm and depth is 255 mm.

This is made of Copper sheet of 0.9 mm thickness Top of the pan is fully covered with the hexagonal wire neting so as to ~~prevent~~ protect the water from any disturbance.



3. Colorado Sunken Pan:-



The main advantage of this method is that actual radiation and aerodynamic Conditions are Created.

NOTE To Convert the pan Evaporation rate into lake Evaporation a pan Coefficient called C_p is used.

$C_p \rightarrow 0.7 \rightarrow$ class A (27)

$0.8 \rightarrow$ ISI

$0.78 \rightarrow$ Colorado

$$\boxed{\text{Lake Evaporation} = C_p \times \text{Pan Evaporation}}$$

Emperical Method :-

This method is based on the Dalton's law. and the Equation derived is called Meyer's Equation. According to this

$$\boxed{E = K_m(e_w - e_a)\left[1 + \frac{V_9}{16}\right]}$$

E = Rate of Evaporation in mm per day.

V_9 = Monthly Wind velocity measured in Km/hr

K_m = Coefficient depend upon size of Water body.

$0.36 \rightarrow$ deep water body

$0.5 \rightarrow$ Shallow Water body

The Wind velocity at any height follows the $1/7^{th}$ power law.

According to which velocity at height h

$$\boxed{V_h = C\,h^{1/7}}$$

$V_{1m} = 16$ kmph

$$\frac{V_9}{V_1} = \frac{C(9)^{1/7}}{C(1)^{1/7}}$$

$$V_9 = V_1(9)^{1/7}$$

Date:

Q A Reservoir with a surface area of 250 hectares had the value of saturated vapour pressure as 17.54 mm of mercury (Hg) and actual vapour pressure of 7.02 mm of Hg. The wind velocity at 1m above ground is 16 Kmph. Find the avg. daily evaporation ~~from this~~ rate from this lake and vol. of water evaporated from the lake in one week. (Km = 0.36)

Sol.

$$E = K_m (e_w - e_a)\left[1 + \frac{V_9}{16}\right]$$

28

$$V_9 = V_1 (9)^{1/7}$$

$$V_9 = 1.368 \times 16$$

$$V_9 = 21.89$$

$$E = 0.36(17.54 - 7.02)\left[1 + \frac{21.89}{16}\right]$$

$$E = 8.97 \text{ mm/day}$$

$$Vol. = \frac{8.97}{10^3} \times 7 \times 250 \times 10^4$$

$$Vol. = \cancel{150}\ 156949 \text{ m}^3$$

Stream flow Measurement:-

The Science and practise of Water measurement is called hydrometry.

STAGE:-

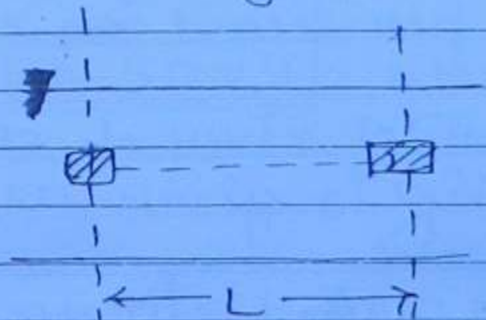
This is defined as the Water Surface Elevation measured above datum. datum can be mean Sea level or any other fixed level.

The process of measurement of stage is called gauging.

In order to find the discharge of Stream velocity is required which Can be measured using the following devices.

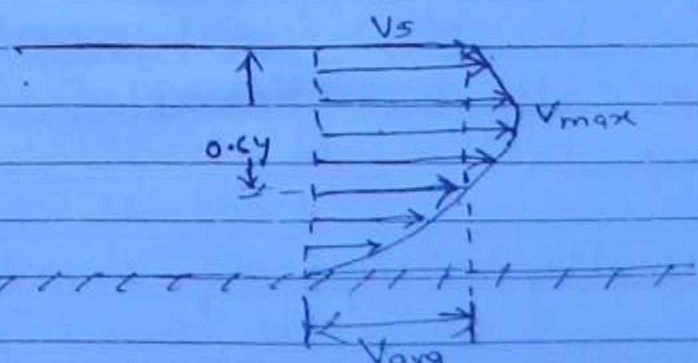
1 Float:-

This is generally used to measure the approximate velocity of river Surface. This is basically a floating device which is passed with Water along the flow.



$$V = \frac{L}{t}$$

Flow Vel.



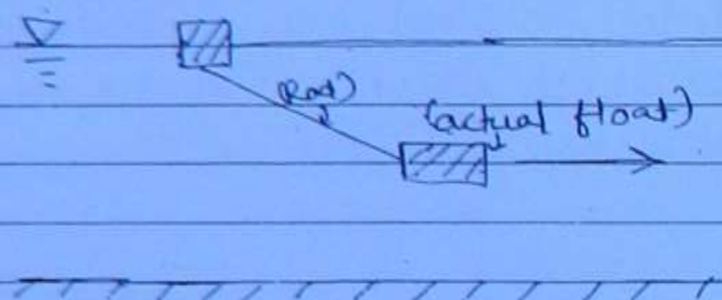
Vel. profile for given channel flow

→ The velocity at a d

The Avg. Velocity can also be calculated

$$\boxed{V_{avg} = \frac{V_{0.2y} + V_{0.8y}}{2}}$$

To over Come the deficiency of float Canister float is Introduce.



Current meter :→

This Consist of rotating Element Which rotates due to Rxn of stream and the No. of revolution per second are Counted. (RPS)

If V is the Velocity of river of stream at any depth then velocity is given as

$$\boxed{V = a N_s + b}$$

N_s - No. of revolution per second
a, b - Characteristic Constants
It is also called Characteristic Eqn of Current meter.

Theirs are basically 2 types of Current meter

1 Vertical Axis current meter :-

(3) This has a vertical Axis on which a rotating disk is mounted. It's major disadvantage is that it cannot be use in situations where there is an appreciable Inclined load.

2 Horizontal Axis current meter ;-

These are fairly rugged and are not affected by Inclined flow upto 15°

Sounding Weight :-

These are Standard weights attached to the Current meter in order to Keep the Current meter at a fixed location. In order to reduce drag force these are stream line.

The minimum weight required depends upon velocity and depth of flow and may be given by

$$\boxed{W = 50\bar{V}y}$$

Where,
$\bar{V}$ = Avg. velocity in m/sec
y = Avg. depth in m.
W = Weight in Newton.

Determination of discharge :-

Their are basically two set of method :-

1) Direct Method :-
i) Area Velocity Method
ii) Dilution Method
iii) Ultrasonic method
iv) Moving boat method
v) Electro magnetic method

2. Indirect method.
   i) Slope area method
   ii) Discharge measuring structure eg. (Weir, Notch etc)

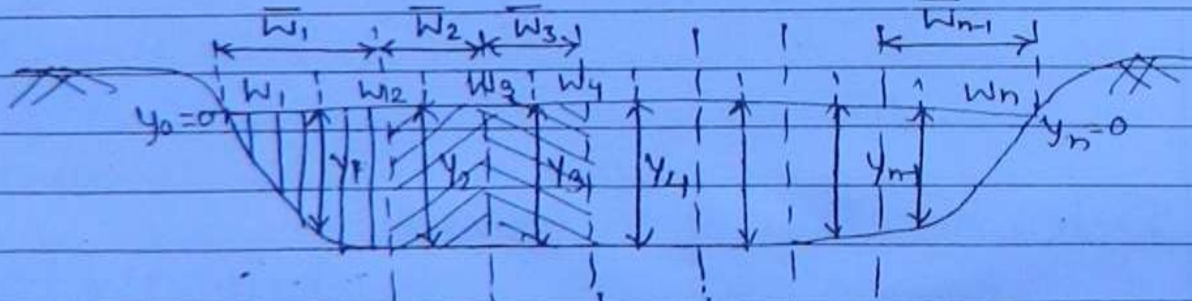
Area velocity method :-

For calculating discharge the section of river or stream is divided into number of segment according to the following condition.

1. Discharge in each segment must be less than 10% of the total discharge.
2. Difference in velocities for adjacent segment s/d not be more than 20%.
3. The segmental ~~width~~ width s/d not be more than $\frac{1}{15}^{th}$ to $\frac{1}{20}^{th}$ of the river bed.

Let $\Delta Q_1, \Delta Q_2$ ---- be the segmental discharge than total discharge for the stream is given by the summition of $\Delta Q_1, \Delta Q_2$ ----

$$\Sigma(\Delta Q_1 + \Delta Q_2 + \Delta Q_3 ----)$$

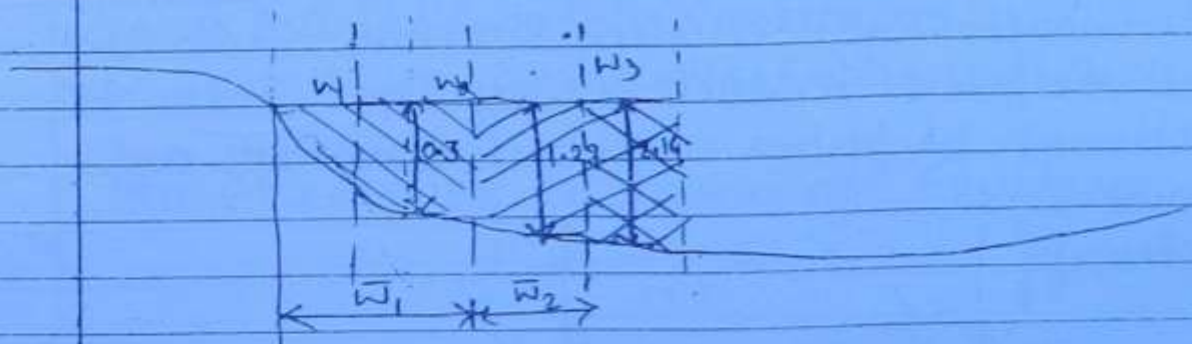


$$\bar{W}_1 = \frac{\left(W_1 + \frac{W_2}{2}\right)^2}{2W_1} ; \quad \bar{W}_2 = \left(\frac{W_2}{2} + \frac{W_3}{2}\right)$$

$$\bar{W}_{n-1} = \frac{\left(W_n + \frac{W_{n-1}}{2}\right)^2}{2W_n}$$

Q. Compute the ~~Screen~~ stream velocity and discharge for the following given data. (33)

Distance from Left Edge (m)	0	0.6	1.2	1.8	2.4	3.6	4.2	5.4	6.0	6.6
Depth (m)	0	0.3	1.29	2.16	2.55	1.68	1.47	0.63	0.42	0
Velocity at 0.2y (m/s)	0	0.42	0.57	0.78	0.87	0.75	1.05	0.54	0.45	0
Velocity at 0.8y (m/s)	0	0.21	0.36	0.54	0.60	0.51	0.68	0.33	0.3	0
$Vel = \frac{V_{0.2y} + V_{0.8y}}{2}$	0	0.315	0.465	0.66	0.735	0.63	0.865	0.435	0.379	0



$$\bar{w}_1 = \frac{\left(w_1 + \frac{w_2}{2}\right)^2}{2w_1} = 0.675\,m$$

$$\Delta Q_1 = (\bar{w}_1 \times y_1) \times v_1 \Rightarrow (0.675 \times 0.3) \times 0.315$$
$$= \cancel{0.0567} = .0637$$

$$\Delta Q_2 = (\bar{w}_2 \times y_2) \times v_2 \Rightarrow (0.675 \times 1.29) \times 0.315$$
$$= 0.24$$

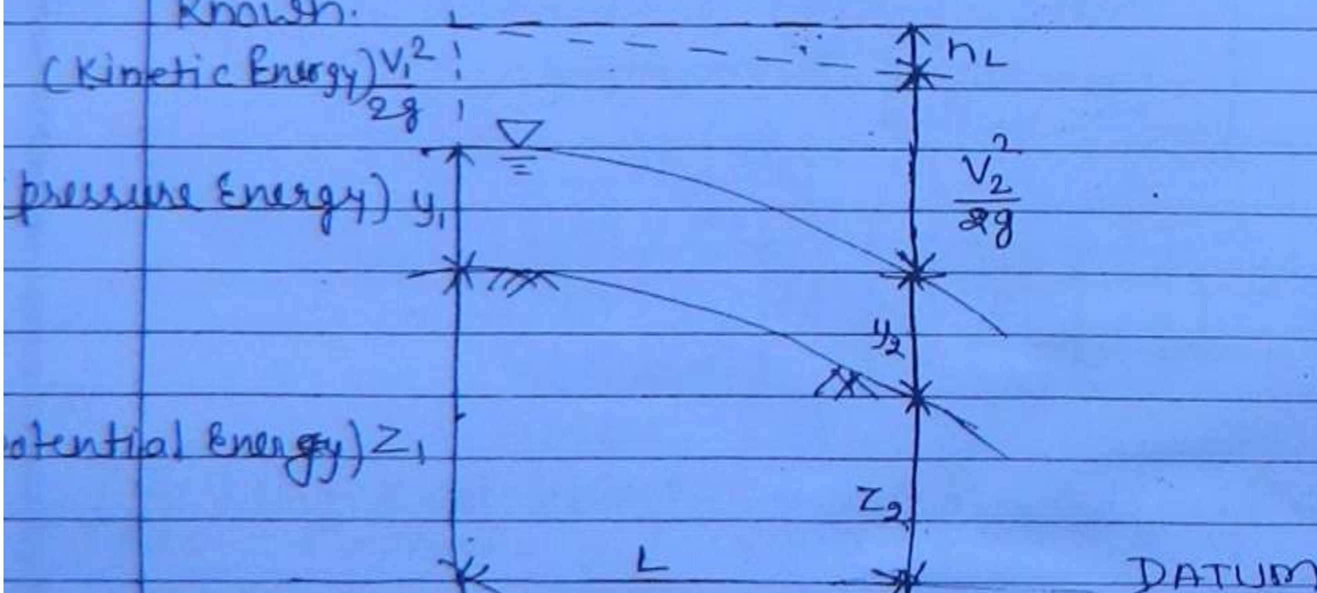
$$\Delta Q_8 =$$

$$\Delta Q = 5.39\,m^3/s$$

34

Slope Area Method:-

This method is based on the Law of Conservation of Energy for fluids (Bernoulli's Eqn) in this method two Sections are Selected in such a way that their cross sectional properties and bed Elevations are Known.



$$Z_1 + y_1 + \frac{V_1^2}{2g} = Z_2 + y_2 + \frac{V_2^2}{2g} + h_L$$

$$h_L = \underbrace{(Z_1 + y_1)}_{h_1} - \underbrace{(Z_2 + y_2)}_{h_2} + \left(\frac{V_1^2 - V_2^2}{2g}\right)$$

Date:

$$h_L = (h_1 - h_2) + \left(\frac{V_1^2 - V_2^2}{2g}\right)$$

$$h_L \begin{cases} \rightarrow h_f \text{ (frictional head loss)} \\ \rightarrow h_e \text{ (Eddy head loss)} \end{cases}$$

$$h_f = (h_1 - h_2) + \left(\frac{V_1^2 - V_2^2}{2g}\right) - h_e$$

$$h_e = K_e\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right)$$

$$h_f = (h_1 - h_2) + (1 - K_e)\left(\frac{V_1^2 - V_2^2}{2g}\right)$$

K_e = Eddy head loss constant.

K_e is 0.14 for gradually contracting section

K_e is 0.3 for gradually Expanding section

K_e is 0.6 for suddenly contracting

K_e is 0.8 for suddenly expanding section.

According to manning's Equation

$$Q = \underbrace{\frac{1}{n} A R^{2/3}}_{\text{Conveyance (K)}} S^{1/2}$$

$$\boxed{Q = K\sqrt{S}}$$

$$S = \frac{h_f}{L}$$

STEPS :-

The value of h_f depends upon discharge hence the following method is adopted.

STEP 1 Assume $V_1 = V_2$ and then calculate h_f.

STEP 2 Using this value of h_f calculate of discharge $\boxed{Q = K\sqrt{\frac{h_f}{L}}}$

STEP 3 Using this Value of discharge find V_1 & V_2

STEP 4 Again Calculate the Value of h_f. using the Value of V_1 & V_2.

This process is continious until two values of discharge come out to be the same.

for 2 section $K \rightarrow K_1, K_2$

36

$$K = \sqrt{K_1 K_2}$$

for 3 section $K \rightarrow K_1, K_2, K_3$

$$K = \sqrt[3]{K_1 K_2 K_3}$$

Q. ~~Due to~~ During the passes of a flood the following data was Estimated at two Section Seprated by 500 m.

Section	Water Surface Elevation	Area of flow	R
U/S	85.23 h_1	91.746	2.835
D/S	85.176 h_2	84.354	2.917

The Eddy loss coefficient for gradual ~~Ex~~ Contration is 0.1 & gradual Expansion is 0.35. Estimate the flood discharge passing through the channel If manning coeff. is 0.022.

Sol) $K = \frac{1}{n} A R^{2/3}$

$$K_{u/s} = \frac{1}{0.022}(91.746)(2.835)^{2/3}$$

$$K = 8353.4 \ m^3/s$$ (37)

$$K_{d/s} = \frac{1}{0.022}(84.354)(2.917)^{2/3}$$

$$K_{d/s} = 7827.8 \ m^3/s$$

$$K = \sqrt{K_1 K_2}$$

$$K = 8086.36$$

$$Q = K\sqrt{\frac{h_f}{L}} = 8086.36\sqrt{\frac{h_f}{L}}$$

$$h_f = (h_1 - h_2) + K(1 - K_e)\cancel{\left(\frac{V_1^2 - V_2^2}{2g}\right)}$$

$$h_f = 0.054$$

$$Q = 84.035 \ m^3/sec.$$

$$V_1 = \frac{Q}{A_1} = .91 \ m/s$$

$$V_2 = \frac{Q}{A_2} = .99 \ m/s$$

$$\Rightarrow (85.23 - 85.176) + (1 - \overset{0.1}{\cancel{8086.36}})\left(\frac{V_1^2 - V_2^2}{2g}\right)$$

$$h_f \cancel{Q} = \cancel{61.50 \ m^3/sec.} \ 0.04716$$

$$\cancel{V_1 = \frac{Q'}{A_1}} \qquad Q' = 8086.3\sqrt{\frac{61.5}{500}}$$

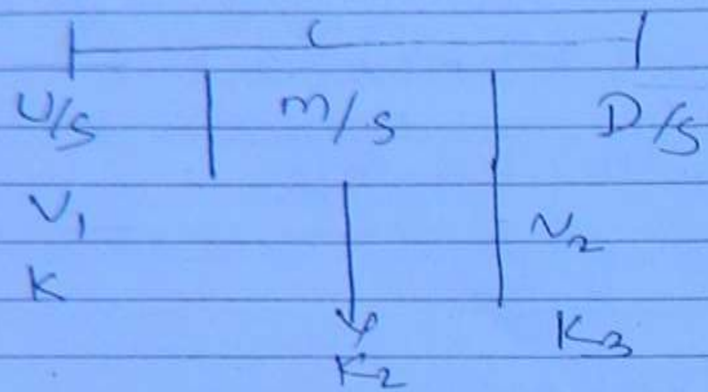
$$Q' = 78.53$$

$$V_1 = \frac{Q'}{A_1} \ 0.855$$

$$V_2 = \frac{Q'}{A} = 0.93$$

$$= (85.23 - 85.176) + (1 - 0.1)\left(\frac{.855^2 - 0.93^2}{2g}\right)$$

$Q = 72.91$ Ans.



$$K = 3\sqrt{K_1 K_2 K_3}$$

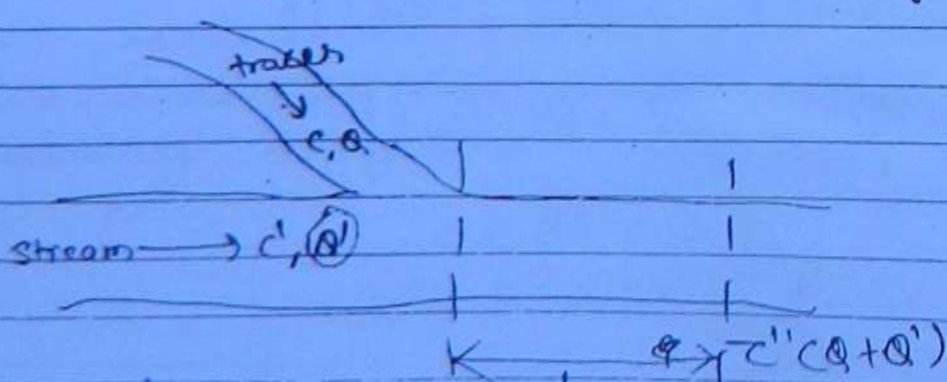
$$h_f = (h_1 - h_2) + (1 - K_e)\left(\frac{V_1^2 - V_2^2}{2g}\right)$$

Dilution Method :-

In this method a chemical compound called tracer such as sodium chloride, sodium dichromate colour dye or some other radio active material is introduced in the stream at the given location.

Na_2CrO_4

Let 'c' be the concentration of the tracer which is being introduce at a rate Q. Let 'c'' be the concentration of tracer in the stream whose discharge is Q'



$c' = 0$

According to the law of conservation of mass

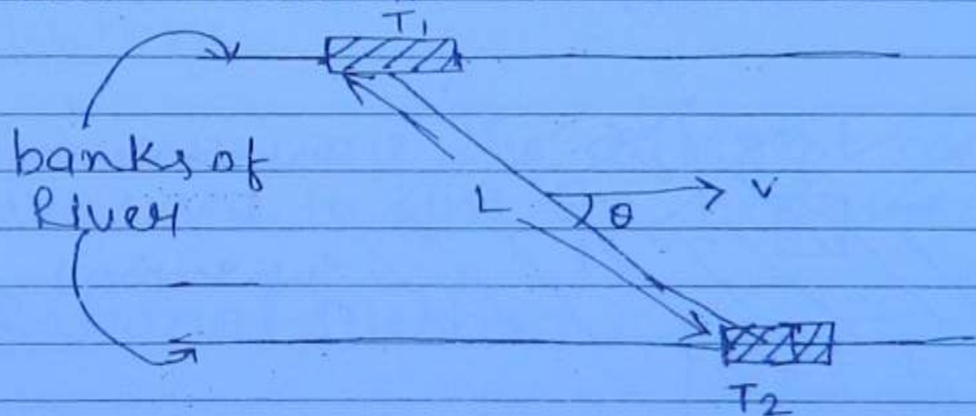
$$CQ + C'Q' = C''(Q + Q')$$

$$Q' = ?$$

The minimum length which is required for sufficient mixing depends on roughness of surface, width of channel and the depth of flow.

Ultrasonic Method :-

In this method two transducers are introduced on both the sides of river or stream.



In the first phase signal is emitted from transducer T_1 and is received by transducer T_2. Therefore the time taken will be given by

$$\boxed{t_1 = \frac{L}{C + v\cos\theta}} \quad \text{---(1)}$$

C = velocity of wave which is equal to the velocity of sound wave.

In the next phase signal is emitted from transducer T_2 and is received by transducer T_1

$$\boxed{t_2 = \frac{L}{C - v\cos\theta}} \quad \text{---(2)}$$

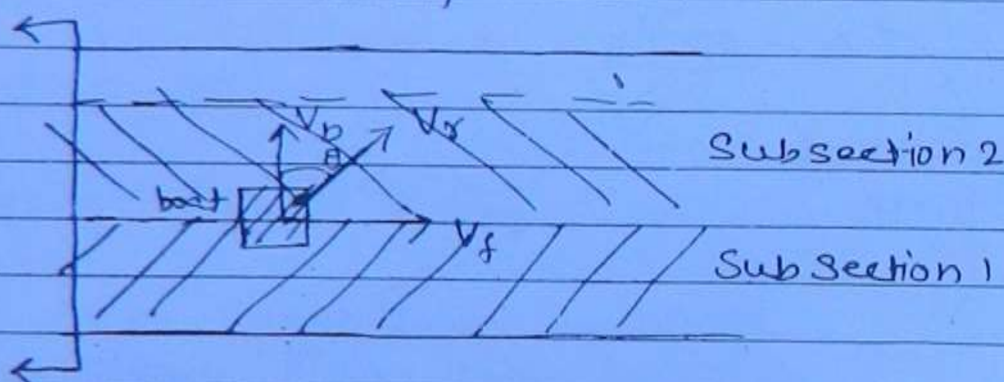
40

Using Eq (1) & (2) Calculate V

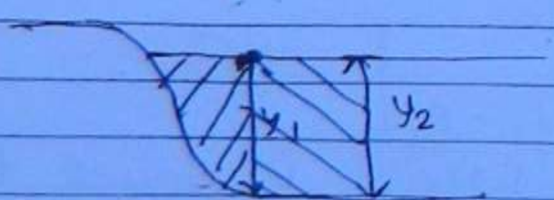
$$\boxed{V = \frac{L}{2\cos\theta}\left(\frac{1}{t_1} - \frac{1}{t_2}\right)}$$

Moving Boat Method :-

This is a Special type of Area Velocity method In which Current meter is Installed in the boat Itself. and angle Indicator is also Installed In the boat which records the direction of resultant velocity.



In this method, the Entire Section is divided into different Sub Section and the time taken by the boat to Cross these Sub Section is recorded as Δt_1, Δt_2 ----



ΔQ_1 = Area x V_f

= length x Avg width x V_f

$$\Delta Q_1 = V_b \Delta t_1 \times \left(\frac{0+y_1}{2}\right) \times V_f$$

$$V_b = V_r \cos\theta$$
$$V_f = V_r \sin\theta$$

41

$$\Delta Q_1 = V_r^2 \sin\theta \cos\theta \frac{y_1 \Delta t_1}{2}$$

Discharge Measurement :-

Flood :-

Flood is a saturation characterised by high ~~space~~ stage in river In which water over the banks and surrounds the adjoining area of the different characterstics of the flood ~~to~~ the most Important is peak flood discharge.

Following are the different method which are used to find the peak discharge.

① Rational Method :- $\boxed{Q_p = \frac{1}{36} k p_c A}$

A → hectare , p_c = cm/hr (rainfall Intensity)

This method is suitable for small size catch-ment where area is less than 50 km^2

(2) Empirical formula:- These are region specific and given as follows.

(i) Dicken's formula :- $\boxed{Q_P = CA^{3/4}}$

This is suitable for north & central India

(ii) Ryve's formula :- $\boxed{Q_P = CA^{2/3}}$

This is applicable in South India

(iii) Inglis formula :- $\boxed{Q_P = 123\sqrt{A}} \simeq \frac{124\,A}{\sqrt{A+10.4}}$

This is suitable for Westren Ghhats and maharashtra region.

NOTE Some other technique such as fuller's eqn and Envelope curve may also be used to find peak flood discharge.

(3) Use of Hydrograph & Unit hydrograph This method is suitable for moderate size catchment whose area is less than 5000 Km^2.

(4) Probability method :- In this category their are 2 types of method.

(i) <u>Statistical probability Method :-</u>

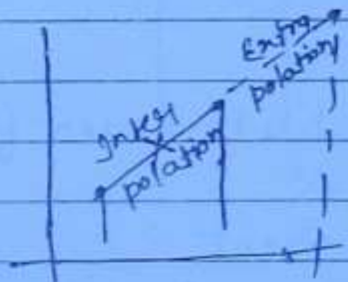
This method is suitable when the sample size or data given is large over a long period of time and it is required to find the peak flood discharge at any given point of time.

Imp.

(ii) <u>Theoritical probability Method (Gumbell's Method)</u>

This method is used when sample size is small and extrapolation is required to be done over a long period of time.

43



Statistical probability method :-

In this method following steps are adopted.

(California Method)

Q	Order No. (m)	Q	Recurrence Interval $T = N/m$	$P = \frac{1}{T}$
60	1	110	6	$p = \frac{\text{fav. Event}}{\text{Total Event}}$
85	2	90	3	$= \frac{1}{6}$
25	3	85	2	
90	4	60	1.5	
110	5	52	1.2	
52	6	25	1.0	

$N = 6$

STEP 1: Arrange all the discharge values in decreasing order of magnitude and give order number to all of them.

Recurrence Interval :- (44)

This is the time period on an average after which peak flood discharge is likely to be Equal or Exceeded. This is also called return period.

$$\boxed{T = \frac{N}{m}}$$ --- California formula

$$\boxed{T = \frac{N+1}{m}}$$ --- Weibull's formula

$$\boxed{T = \frac{N}{m+1}}$$ --- U.S Corps method

$p = \frac{m}{N} \rightarrow 1$ will never fix value

$p = \frac{m}{N+1} \rightarrow < 1$ best

$p = \frac{m+1}{N} \rightarrow > 1$ can't

Suppose a max^m flood discharge recorded at a bridge site is 200 cumecs and on an average this peak value occurs once in 50 years then the recurrence Interval is 50 years and the probability of this flood being Equal or Exceed $p = \frac{1}{T} = \frac{1}{50}$

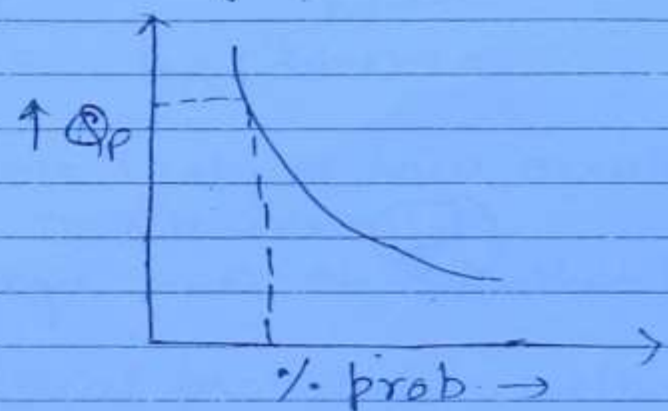
Recurrence Interval doesnot mean that 200 cumecs discharge will take place Every 50 years but this mean that 200 cumecs discharge will take place once in 50 year on an Avg. which may be at any point of time.

STEP 2 After calculating recurrence interval by any of 3 method the probability is calculated as

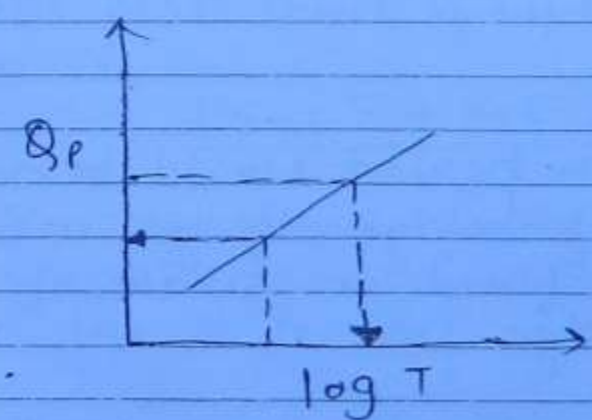
$$P = \frac{1}{T}$$

45

STEP 3 A graph is plotted b/w peak flood discharge and percentage probability



A graph can also be plotted b/w peak discharge and recurrence Interval



The probability of an Event occuring r times out of n trials is given as

$$\boxed{p = {}^{n}C_{r}\, p^{r} q^{n-r}}$$

- p = probability of occurance
  q = probability of non-occurance or failure

In 50 year discharge will never more than 100 m³/s

$$^{50}C_0\, p^0 (1-p)^{50} \longrightarrow \text{Reliability}$$

$$^nC_0\, p^0 (1-p)^n = (1-p)^n \quad \text{Reliability}$$

$$\left[{}^nC_1\, p^1 q^{n-1} + {}^nC_2\, p^2 q^{n-2} + {}^nC_3\, p^3 q^{n-3} + \dots + {}^nC_n\, p^n q^0\right] + {}^nC_0\, p^0 q^n = (p+q)^n$$

$$\rightarrow 1-q^n \qquad\qquad = 1$$

$$= 1-(1-p)^n$$

46

Q. What Return period would be adapted in the design of bridge, if you are allow only 5% risk of flood in 25 years of expected life of bridge

Sol) $n = 25$, $\quad \cancel{1-q^n = 5\%}$

$$1 - .05 = q^n$$

$$\text{Risk} = 1-(1-p)^n$$

$$0.05 = 1-(1-p)^{25}$$

$$(1-p)^{25} = 0.95$$

$$\left(1-\frac{1}{T}\right)^{25} = 0.95$$

$\left(1-\frac{1}{T}\right) = .997$

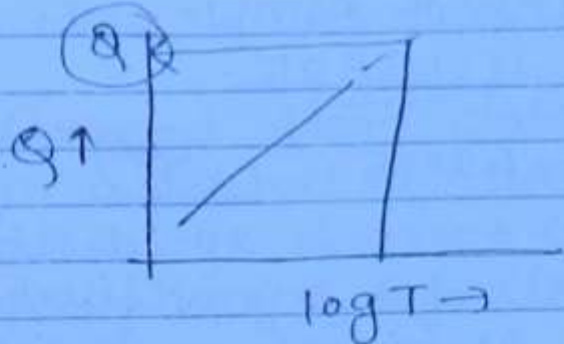
$T = 487.83$ year

or.

$(1+x)^n \simeq 1+nx$

$1 - \frac{25}{T} \simeq 0.95$

$T = 500$



(47)

you are likely to have assurance of 75%. that 75% is relability.
danger level of 5% that means Risk.

Theoritical probabelity method :-
Also known as Gumbell's method.
If the $\bar{x}$ is mean of hydrological data and σ ~~be~~ be the standard deviation then the peak value of hydrological data is given as

$$\boxed{X_T = \bar{x} + K_T \sigma}$$

Where

K_T = frequency factor.

$$\boxed{K_T = \frac{Y_T - \bar{y}_n}{S_n}}$$

$Y_T \rightarrow$ Reduced variate which is a function of Recurrance Interval and is given as

$$\boxed{Y_T = -\ln \ln \left(\frac{T}{T-1}\right)}$$

$\bar{y}_n$ = Reduced mean which depends on ~~at~~ available size of sample data and
..-

Maxm value of $\bar{y}_n$ is 0.557

S_n = Reduced standard deviation which depends on the size of sample data and its maxm value is 1.2825

$$(S_n)_{max} = 1.2825$$

48

Q. The Regression analysis of a 80 year flood data at a point on a River gives mean discharge as 1200 cumecs and standard deviation 650 cumecs. for what value discharge would you design a structure at this point so as to provide 95% assurance that the structure would not fail in the next 50 years? $\bar{y}_n = 0.53622$, $S_n = 1.11238$

Sol

$\sigma = 650$

$\bar{x} = 1200$

$$X_T = \bar{x} + K_T \sigma$$

$$X_T = 1200 + K_T \times 650$$

$$K_T = \frac{y_T - \bar{y}_n}{S_n} = \frac{y_T - 0.53622}{1.11238}$$

$$y_T = \ln \ln \left(\frac{T}{T-1}\right)$$

$$^{50}C_0 (.95)^0 (1 - .95)^{50} = \cancel{(1 - .95)}^{50}$$

$$\left(1 - \frac{1}{T}\right)^{50} = .95$$

$$T = 975.28 \text{ years}$$

$$y_T = \ln \ln \left(\frac{975.28}{975.28 - 1}\right)$$

$y_T = 6.88$

$K_T = 5.702$ (49)

$x_T = 4906.3 \ m^3/s$

Q Annual flood data of a river at a Section for a period of 32 years gives a mean flood discharge of 29000 cumecs and a Standard deviation of ~~486~~ 14860 cumecs for a proposed bridge on this river. It is decided to have an acceptable risk of 10% In Its Expected life of 50 years.

Estimate the flood discharge to be used in the design of the structure. for 32 years $\bar{y}_n = .5380$ and reduced standard deviation is 1.1193

Sol)

$$1-\left(1-\frac{1}{T}\right)^{50} = .10$$

$$.9 = \left(1-\frac{1}{T}\right)^{50}$$

$$.996 = 1-\frac{1}{T}$$

$$T = \cancel{304.02}\ 475.06$$

$$Y_T = -\ln\ln\left(\frac{T}{T-1}\right)$$

$$Y_T = 6.162$$

$$K_T = 5.02 \quad ; \quad x_T = 29000 + 5.02 \times 14860$$

$$x_T = 103665 \ m^3/sec$$

(ii) If actual flood value adopted in design is 1 lack 25 thousand cumecs (1,25,000) then what is the safety factor and safety margin for the structure.

Sol)

$$\boxed{\text{Safety factor} = \frac{Q_{\text{design value}}}{Q_{\text{Estimated value}}}}$$

50

$$= \frac{125000}{103665}$$

$$S.F = 1.205$$

Safety margin = Design Value - Estimated Value

Safety margin = 21,335 cumecs.

Confidence limit :-

The value of X_T determined by gumbells method can have errors due to limited sample data and hence an estimate of the confidence limit in the value of x_T is required. This means that x_T will lie b/w these confidence limit.

→ For a confidence probability 'c' the confidence interval X_T is bounded by x_1 & X_2

$$\boxed{x_1/x_2 = X_T \pm f(c)\, S_e}$$

Where f(c) is function of confidence probability c and whose value is given as .

C	f(c)
50%	0.674
68%	1.0
80%	1.282
90%	1.648
95%	1.96
98%	2.58

57

S_e = probability error.

$$\boxed{S_e = \frac{b\sigma}{\sqrt{N}}}$$

$N \rightarrow$ Number of Sample data.

$$\boxed{b = \sqrt{1 + 1.3K_T + 1.1K_T^2}}$$

Q Data covering a period of 92 years for a river gives standard deviation of 2951 cumecs and a mean flood discharge of 6437 m^3/sec. using gumbles method find the flood discharge with a return period of 500 years. What are 80% confidence limits for this estimate. $\bar{Y}_n = 0.5589$ & $S_n = 1.202$

Sol $n = 92$ years $\sigma = 2951\ m^3/s$ $\bar{X} = 6437\ m^3/sec$

$$Y_T = 6.21$$

$$K_T =$$

$$K_T = 4.7044$$

$$X_T =$$

$$X_T = 20319.7\ m^3/s$$

$x_r =$
x_2

$Se = \sqrt{}$

$b = \sqrt{}$

52

$b = 5.608$

$Se = \frac{}{\sqrt{92}}$

$Se = 1725.66$

x_1
$x_2 = 22532$
$(18107, 22532) \rightarrow 80\%.$

Infultration

1. Initial loss :-

(i) Interception loss :-
This is the amount of water caught by vegitation and other obstruction which is subsequently evaporated :-

(ii) Depression storage :-
This is the amount of water which stored in ditches & sumps.
This depends on type of soil, surface condition and slope of the catchment

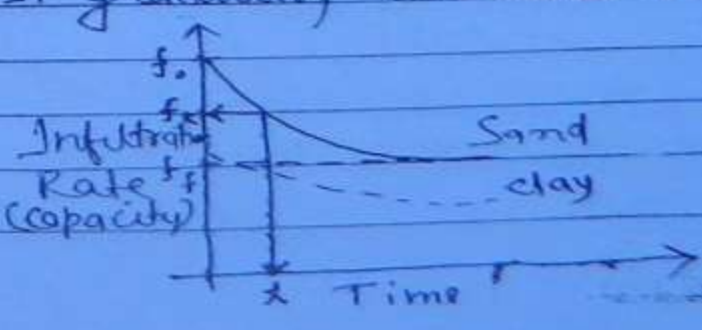
NOTE Interception loss going back to atmosphere may be as high as 20% of the rainfall.
Some of the water obstructed by the plant run along the branches goes down the stem and is called Stemfall

Infultration :-

Infultration is the movement of water through soil into the ground water table.

→ The max^m rate at which water can be absorbed by the soil is called Infultration capacity

→ This capacity is max^m at the begining of a storm and it gradually decreases to a constant value.



The Equation of this Infiltration curve is given in a exponential form.

$$\boxed{f_t = f_f + (f_0 - f_f)e^{-kt}}$$ Horton's Eq^n

f_t = Infiltration rate at any time t.
f_f = final Infiltration rate.
f_0 = Initial Infiltration rate
k = Constant and It depends on the type of Soil (per hour, per day)

Q. Assuming an Initial Infiltration rate of 10 mm/hr and the final Infiltration rate of 5 mm/hr the value of constant (discribing the rate of decay of the difference b/w the Initial & final Infiltration rate) is 0.95/hr. Calculate the total Infiltration depth for a storm lasting 6 hours.

Sol $f_0 = 10$ mm/hr, $f_f = 5$ mm/hr.

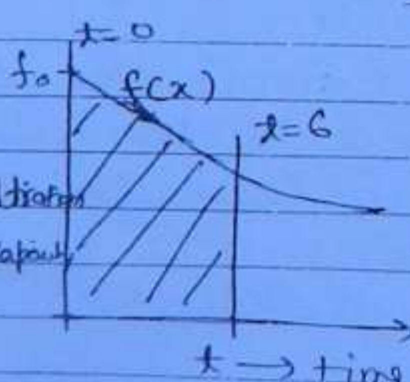
$$f_t = f_f + (f_0 - f_f)e^{-kt}$$
$$f_t = 5 + (10-5)e^{-.95 \times t}$$

[Rate of filteration in 6 hourse then $t = 6$]

$$f_t = \int_0^6 (5 + 5e^{-.95t})\,dt$$

$$\left[5t - \frac{5e^{0.95t}}{0.95}\right]_0^6$$

Infiltration Rate/capacity



f_t = 35.24 mm Ans.

Infiltration Constants :-

ϕ - Index :- This is the avg. rainfall above which rainfall Vol. is Equal to the runoff Vol.

→ In the Calculation of ϕ Index Initial loss is also Considered as Infiltration Quantity.

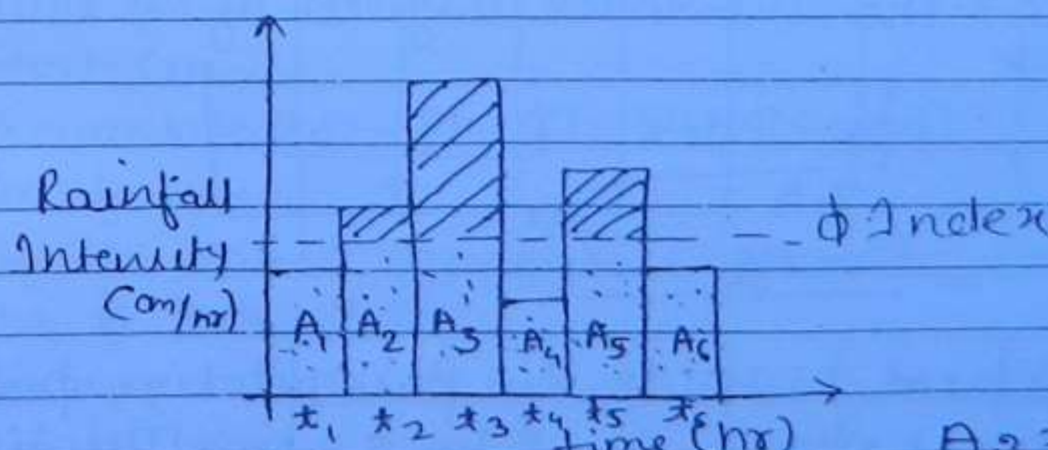
→ "ϕ - Index basically represents avg. Infiltration rate during the period of rainfall Excess."

→ Rainfall Excess is that Rainfall Which Contributes to runoff and the period during which such a rainfall takes place is called the period of rainfall Excess.

× 1^{st} hr → 3 mm/hr
✓ 2^{nd} hr → 8 mm/hr
(8-5) = 3

Hyetograph :-

This is the graphical representation of rainfall Intensity with time.



$$A_2 = \phi \times t_2$$
$$A_3 = \phi \times t_3$$
$$A_5 = \phi \times t_5$$

$$\phi\text{-Index} = \frac{A_2 + A_3 + A_5}{t_2 + t_3 + t_5}$$

NOTE:- If rainfall is less than ϕ-Index then Infiltration rate will be Equal to Rainfall and If rainfall is greater than ϕ-Index then Infiltration rate will be ϕ-Index only

ϕ-Index $\rightarrow$ Max^m possible rate of Infiltration.

56

W - Index

This is the Avg. Infiltration rate during the Whole period of Storm.

$$W = \frac{A_1 + A_2 + A_3 + A_4 + A_5 + A_6}{t}$$

In the Calculation of W-Index Initial Losses are seprated from total Infiltration, Whereas In the Calculation of ϕ-Index Initial loss is also Considered as Infiltration Quantity.

$$\boxed{\phi\text{ Index} \geq W\text{-Index}}$$

Let P be the total amount of rainfall (in cm or mm) and let Q be the total runoff during a period 't' in Which their is an Initial loss of Δl then W-Index is given by

$$\boxed{W\text{-Index} = \frac{P - Q - \Delta l}{t}}$$

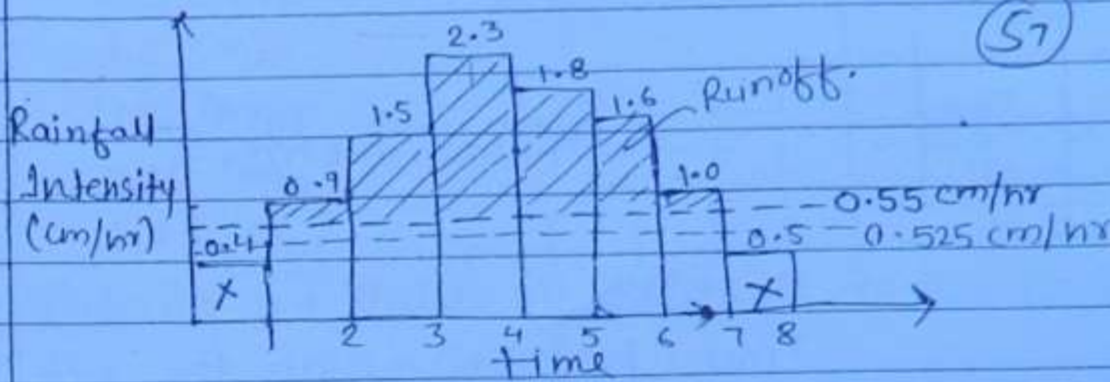
$$= \frac{\overset{P-Q}{\text{Runoff}} - \overset{\Delta l}{\text{losses}}}{t}$$

Q A Storm with 10 cm precipitation produces a direct runoff equal to 5.8 cm. The time distribution of rainfall is given as follows.

Time from Start	1	2	3	4	5	6	7	8
Incremental Rainfall (cm)	0.4	0.9	1.5	2.3	1.8	1.6	1.0	0.5

57

Sol



$$W\text{-Index} = \frac{P - Q - \cancel{S}^{0}}{t} = \frac{10 - 5.8}{8} = 0.525\ \text{cm/hr}$$

$$\phi \text{ Index} = \frac{10 - 5.8 - 0.4 - 0.5}{8 - 1 - 1}$$

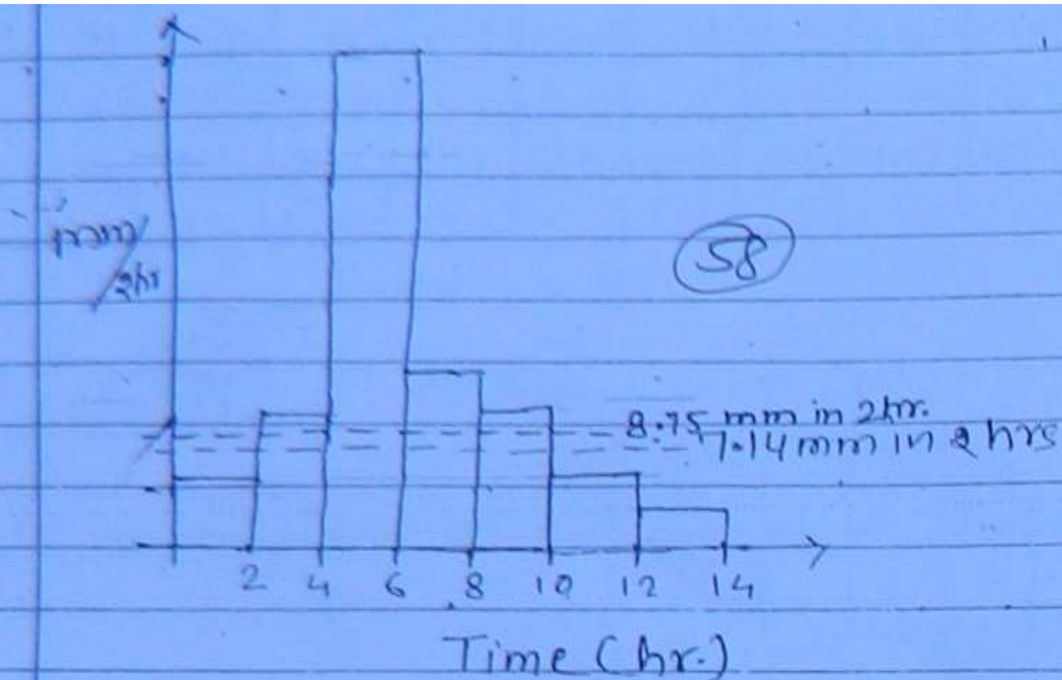
GATE 17

Q. A Catchment Area of 30 km^2 has one recording raingauge during the period of storm the following values of total Rainfall were Calculated.

Time from Start (hr.)	0	2	4	6	8	10	12	14
Accumulated Rainfall (mm)	0	6	17	57	70	81	87	90

If the Vol. of runoff is $1.2 \times 10^6\ m^3$ then Estimate the ϕ Index?

Incremental Value (mm)	0	6	11	40	13	11	6	3



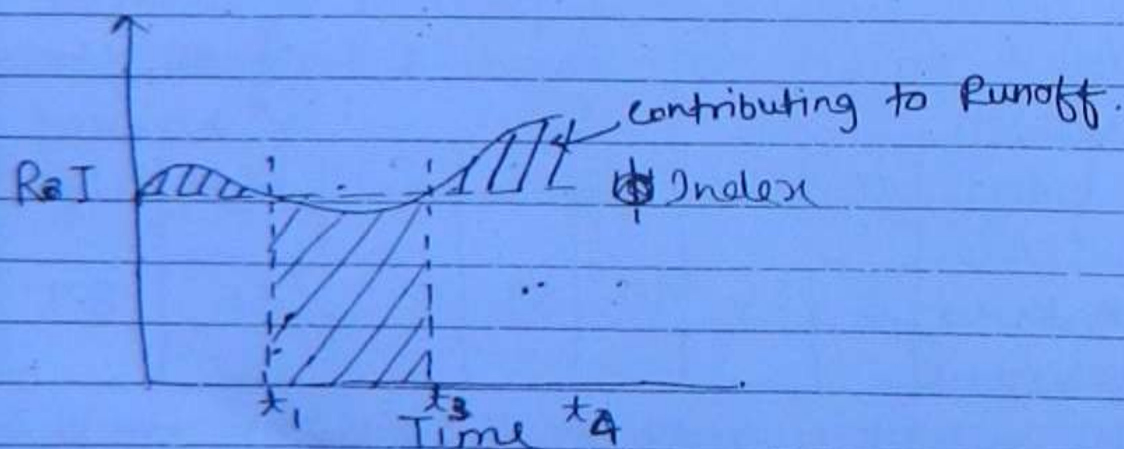
$$W\text{-Index} = \frac{P - Q - \Delta S^{\,0}}{t} = \frac{90 - 40}{14} = 3.57\ mm/hr$$

$$Q = \frac{Vol.}{A} \qquad = 7.14\ mm/2hr$$

$$\phi\text{-Index} = \frac{90 - 40 - 6 - 6 - 3}{14 - 2 - 2 - 2}$$

$$\phi\text{-Index} = 4.375\ mm/hr$$

Q.



$$\text{Runoff} = \int_0^{t_1} (f(x) - \phi)\,dx + \int_{t_3}^{t_4} (f(x) - \phi)\,dx$$

$$\text{Infiltration} = \int_{t_1}^{t_3} f(x)\,dx$$

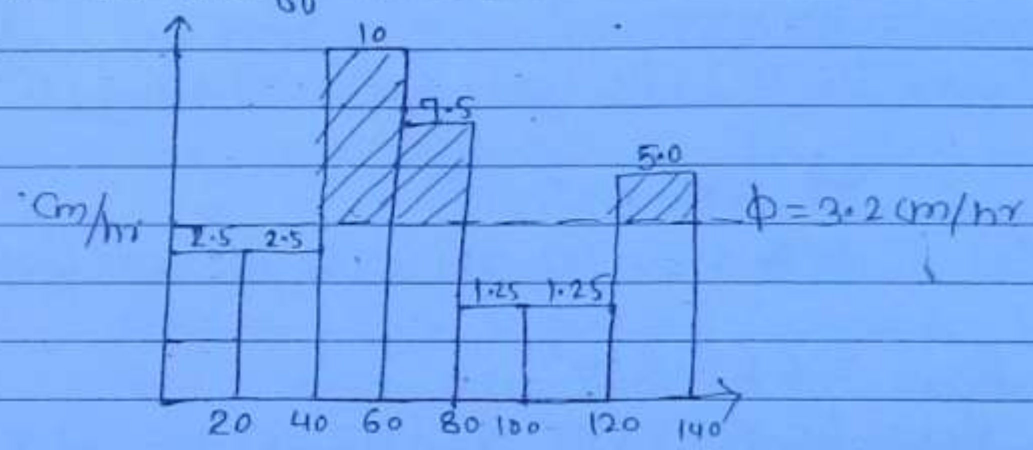
Q Following are the rates of rainfall for successive 20 minute period for a 140 minute storm.

(59)

2.5 cm/hr
2.5 cm/hr
10 cm/hr
7.5 cm/hr
1.25 cm/hr
1.25 cm/hr
5 cm/hr

Taking the value of ϕ Index as 3.2 cm/hr. find the net runoff in c.m.

Sol



$$\text{Runoff} = (10-3.2)\times\frac{20}{60} + (7.5-3.2)\times\frac{20}{60} + (5.0-3.2)\frac{20}{60}$$

$$= 4.3 \text{ cm}$$

$$\text{Total Rainfall} = \frac{\qquad}{\qquad} = \frac{30}{}\times\frac{20}{60}$$

$$= 10 \text{ cm/hr}$$

Q A 4 hours storm occurs over a $80\,km^2$ water-sheed. the details of catchment are as follows

Date:

Sub Area (km^2)	15	25	35	5
ϕ (mm/hr)	10	15	21	16
Hourly Rain (mm)				
1^{st}	16	16	12	15
2^{nd}	48	42	40	42
3^{rd}	22	20	18	18
4^{th}	10	8	6	8

66

$R_1 = [(16-10)+(48-10)+(22-10)+(10-10)] \times 10^{-3} \times 15 \times 10^6$

$= 840 \times 10^3\ m^3$

$R_2 = [(16-15)+(42-15)+(20-15)+0] \times 10^{-3} \times 15$

$= 825 \times 10^3$

$R_3 = [(12-21)+(40-21)+(18-21)+(6-21)] \times 35$

$R_3 = 665 \times 10^3$

$R_4 = (15-15)+(42-16)+(18-16)+(8-16) \times 5$

$= 140$

Total Run off $= R_1 + R_2 + R_3 + R_4 = 2470 \times 10^3\ m^3$

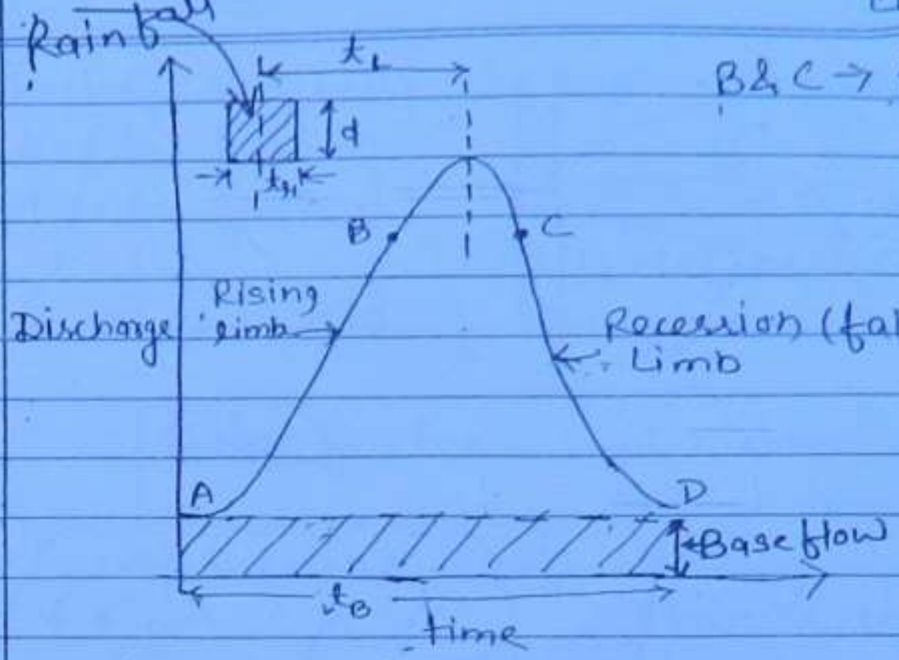
Runoff depth $= 30.87$ mm (V/A)

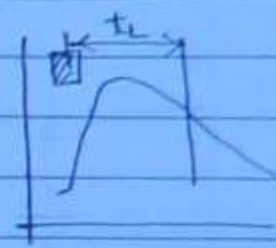
NOTE:- Effective rainfall is that part of total rainfall which is responsible for direct runoff.

Hydrograph :-

It is the graphical variation of discharge with time at a particular point on the stream.

B & C → points of Inflection





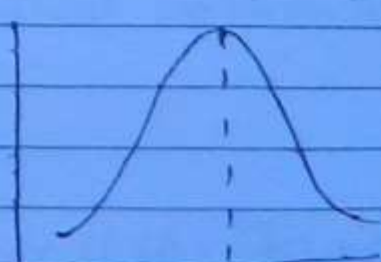
t_L = leg time and this is the time Interval from the Centre of mass of strom or Rain-fall to the Centre of mass of the given hydrograph

t_B = It is the Base time and is the total time during Which Catchment Contributes flow at the Section where hydrograph is plotted.

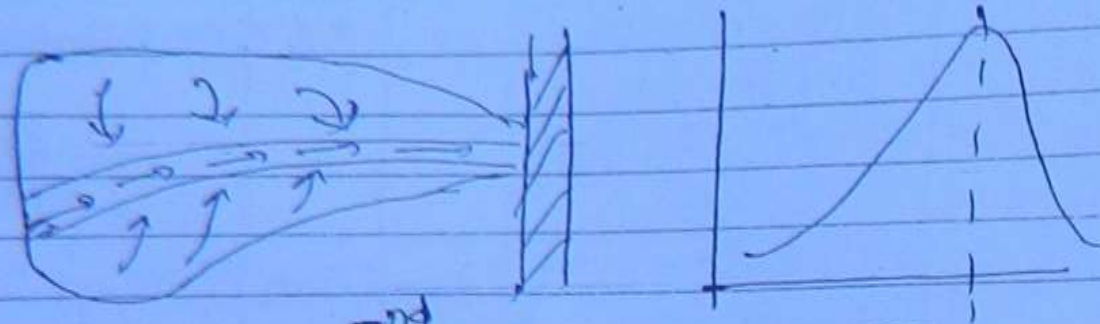
t_H = duration of Rainfall
d = depth of Rainfall

factors affecting Hydrograph :→

1 Shape of the Basin :→

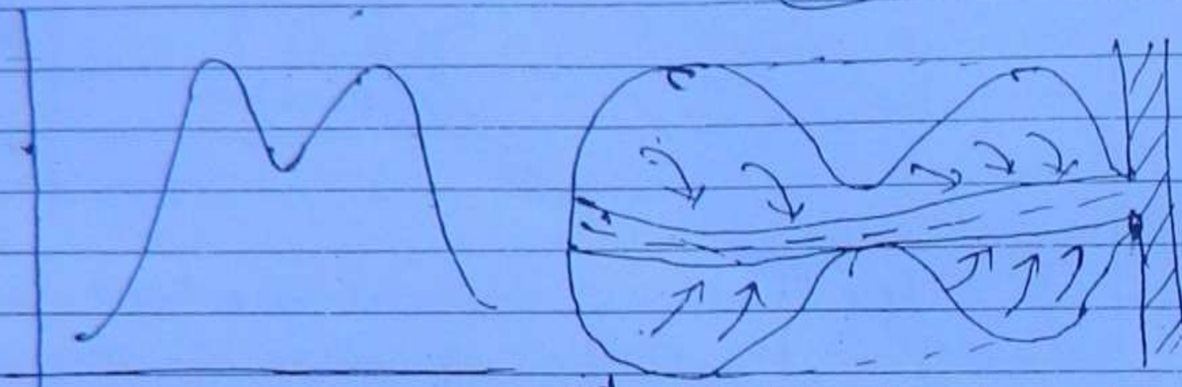


All area Contributing in discharges Immidetly



IInd

62



IIIrd Complex hydrograph

Shape of Basin Influences the time taken by water to reach to location from the farthest point & Hence it Influences the time after which peak discharge takes place.

In Case I peak is reach Early becoz the time taken by water to reach the outlet from farthest point is less.

In Case II time taken by water to reach

In Case IIIrd It is a Composite Area In Which we get a Complex hydrograph having more than one peak.

Factors affecting the hydrograph base on shape are as follows

i. Form factor :-

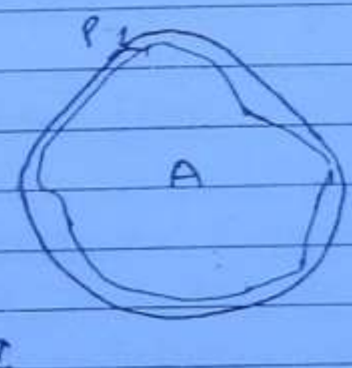
This is defined as the ratio of Avg. width of the basin to the axial length

$$\boxed{\text{Form factor} = \frac{\text{Width}}{\text{Axial length}} = \frac{B_{avg.}}{L_{axial}}}$$

ii. Compactness Coefficient :-

This is defined as the ratio of perimeter of the basin to the perimeter of a circle whose are is same as that of the basin.

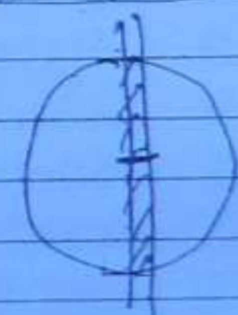
$$\text{Compactness coefficient} = \frac{P}{2\pi r_e} = \frac{P}{2\pi\sqrt{A/\pi}} = \frac{P}{2\sqrt{A\pi}}$$



$\pi r_e^2 = A$

$r_e = \sqrt{\frac{A}{\pi}}$

$C_c = 1$



If C_c (Compactness coefficient) tends to 1 It means that the Catchment area is closer to a Circular Area and hence the hydrograph will be symmetrical.

iii. Stream density

This denotes the number of streams per unit of catchment area.

(iv) Drainage Density :-
This is the ratio of total length to the stream to the area of catchment

2. Size of basin :-
If catchment area is more then total flow time will be more and hence base time (t_b) of the hydrograph will be more.

The peak discharge varies with catchment Area as

$$\boxed{Q_P = KA^n} \qquad Q_P \propto A^n$$

Where
n = Index whose value less than 1

* Other factors.
3 Effect of wind
4 Direction of storm
5 Rainfall Intensity
6 Duration of Rainfall
7. Soil moisture Condion

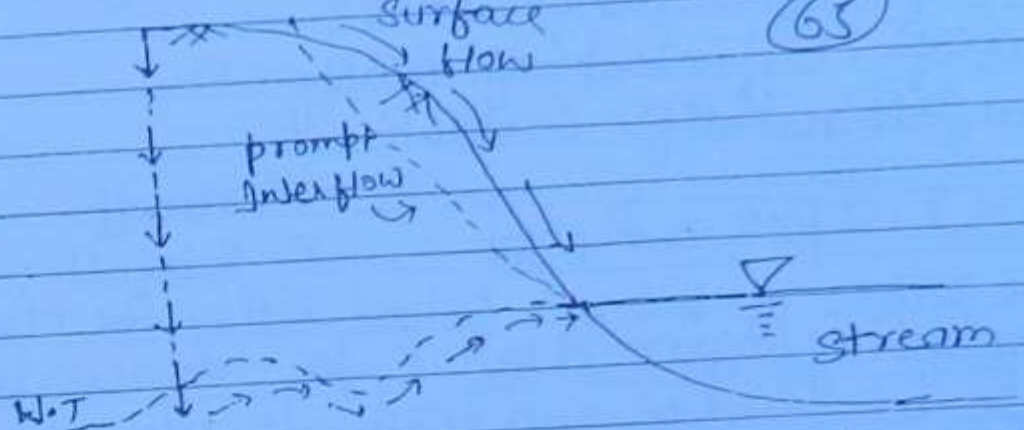
Runoff :-
Runoff is defined as a the flowing off of precipitation from a catchment Area

The flow which takes place all the time over the surface as over land flow and through the channel as open channel flow is called Surface runoff.

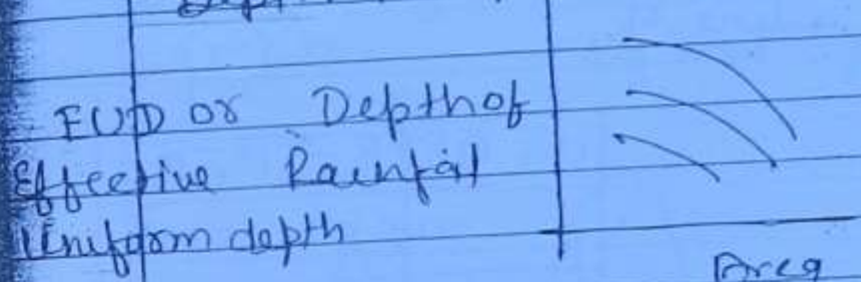
Runoff is brodly classified under two heads

1 Direct Runoff:
2 Base flow.

(63)



Depth Area duration curve (D D A Curve)



Direct flow :-
This is the Part of Runoff which Immediatly Reaches the stream after precipitation this Includes Surface runoff, Prompt Interflow & precipitation on the Stream Surface.

Base flow :-
The delayed flow which Enters the Stream Essentially as ground water flow is called Base flow.

$$\boxed{\text{Direct Runoff} = \text{Total Runoff} - \text{Base flow}}$$

NOTE: In India water year is taken from 1 June to 31st may

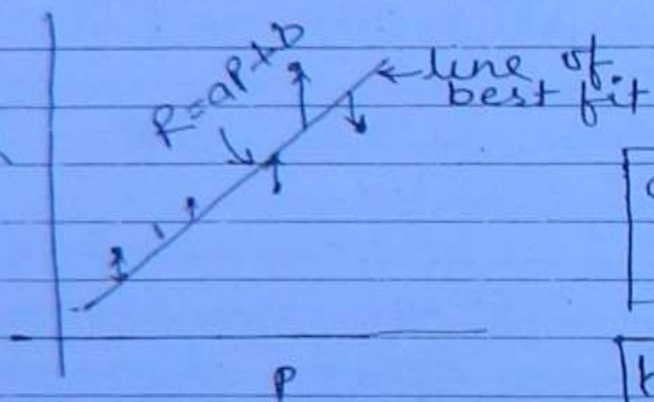
Determination of Runoff :-

1 Use of Emperical formula
2 Use of Hydrograph & unit hydrograph.
3 Use of Infilteration Curve
4 Use of Coefficient of runoff.
5 Rainfall runoff Co-relation

$$\boxed{R = aP + b}$$

R = Runoff.
P = Rainfall.

The value of a & b is found using the Method of least square.



$$\boxed{a = \frac{n\Sigma PR - \Sigma P \Sigma R}{n\Sigma P^2 - (\Sigma P)^2}}$$

$$\boxed{b = \frac{\Sigma R - a\Sigma P}{n}}$$

If R & P Coordinate change than $P = aR + b$ and a & b Values will also be accordingly Change.

If Value of R is not given then eg $R = 4.7P - 6.2$
get R put P

Given below are the monthly rainfall & Corresponding runoff Value for period of 10 months Develop a corelation b/w R & P.

P	R	PR	P	P	R
4	0.2			8	1.3
22	7.1			4	0.4
28	10.9			15	4.1
15	4	—		10	2.0
12	3			5	0.3

$\Sigma P = 123 \quad \Sigma P^2 = 15129 \quad \Sigma R = 33.3$

$\Sigma PR = 653.2 \quad n = 10$

$$a = \frac{10 \times 653.2 - 123 \times 33.3}{10 \times 15129 - 15129}$$

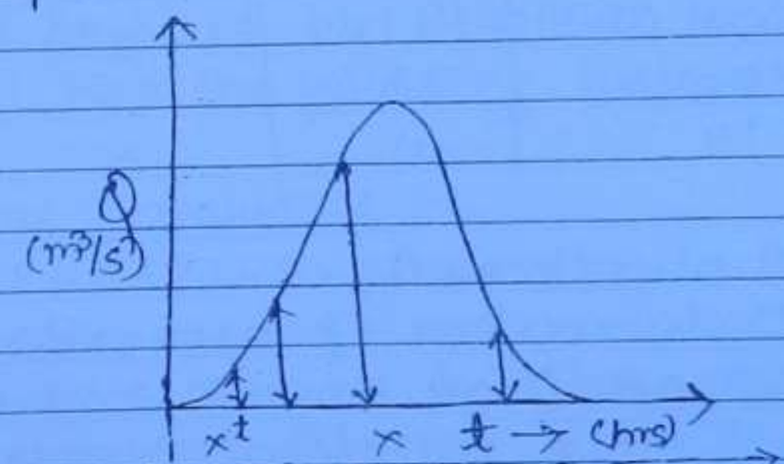
$a = .017$

$R = 0.427P - 1.92$ Ans

$b =$

Calculation of Direct runoff Hydrograph:-

STEP 1 Ordinates of DRH are obtain by subtracting base flow from the ordinate of given hydro-graph



2 Let the ordinates of this DRH be $O_1, O_2 \ldots O_n$

3 Direct runoff depth can be calculated by dividing the total runoff volume with the area of catchment.

Area of hydrograph $= \Sigma O \cdot t \times 60 \times 60$ m^3

$$\text{Direct runoff depth} = \frac{\Sigma O \cdot t \times 60 \times 60}{A \times 10^6} \text{ (meter)}$$

$$= \frac{\Sigma O t \times 60 \times 60}{10^6} \times 100 \text{ (cm)}$$

$$\boxed{\text{Direct Runoff Depth} = \frac{0.36\,\Sigma O.t}{A}} \quad c(m)^{*}$$

~~This~~

(68)

Area in km time in hr.

Q Find the direct runoff in c.m for a given storm. in 30 km^2

Time	ordinates (cumec) of hydrograph	Base flow	Direct runoffs of DRH
5 AM	14	14	0/5.97
8 AM	25	12	13/5.97
11 AM	51	11	40/5.97
2 PM	65	10	55
5 PM	54	11	43
8 PM	28	13	15
11 PM	14	14	0

$$\Sigma O t = 0 + 13\times3 + 40\times3 + 55\times3 + 43\times3 + 15\times3 + 0\times3$$

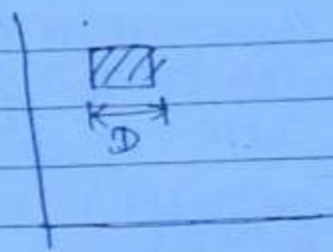
$$\Sigma O t = \frac{498 \times 0.36}{30}$$

$$n = 5.976$$

$$n' = 1\ \text{cm}$$

Unit Hydrograph :-
This is a direct runoff hydrograph which gives a runoff depth of 1cm this 1 cm may also be some other units like 1m or 1 inch etc.

→ The duration of Rainfall which produces such a direct runoff hydrograph is called unit duration



D hr UH

4 hr UH

unit duration

D hr. UH Indicates a DRH resulting in a runoff depth of 1 cm from a Storm or rainfall which lasts for D hours.

This D hour is called unit duration of this unit hydrograph.

Theory of unit Hydrograph :-

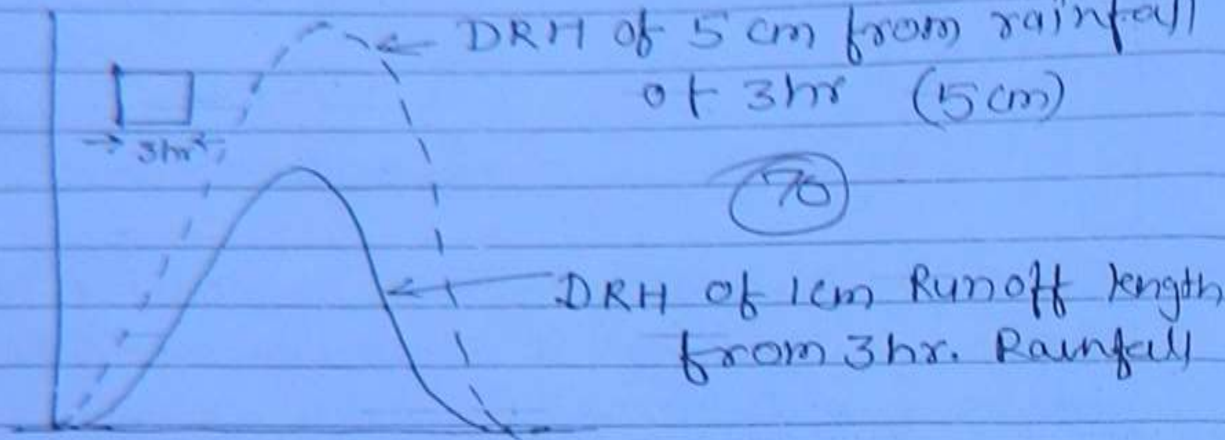
The theory of unit hydrograph was proposed by Sherman in the year 1932 and has the following basic assumption.

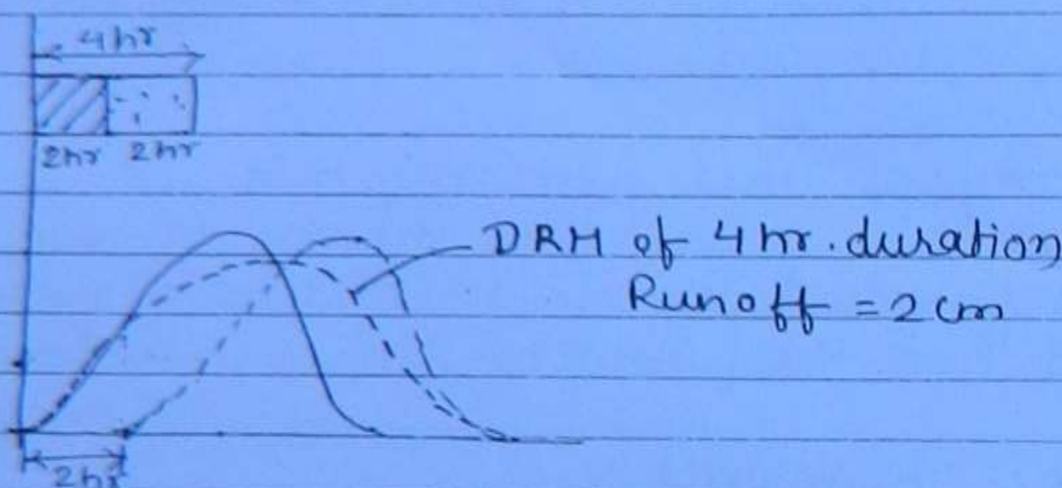
1 Time Invariance :-

According to this assumption the direct runoff hydrograph (DRH) for a given effective rainfall is always the same in the catchment irrespective of the time when rainfall takes place.

2 Linear Responce :-

This is the most Important Assumption in the theory of unit hydrograph. According to this Assumption any Variation in the Input value is proportionately reflected in the output Value.





3. Rainfall is uniformly distributed over the whole Catchment Area.

4. Intensity of Excess Rainfall is assume to be Constant for the given unit duration of unit hydrograph.

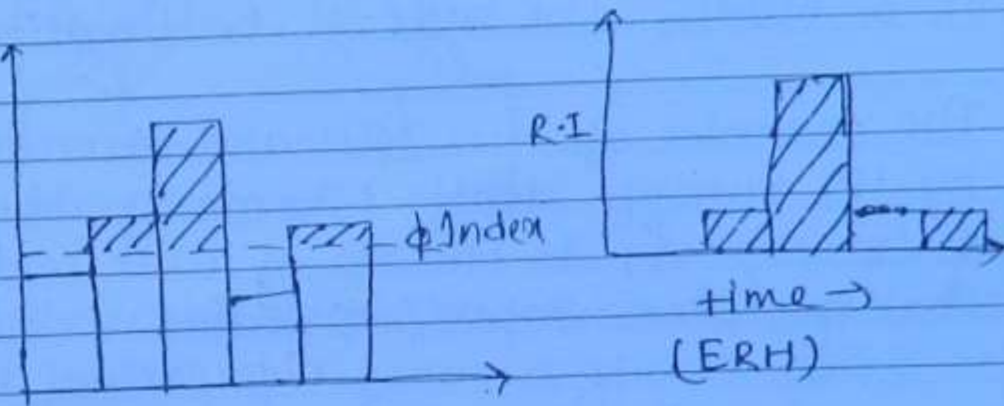
D hr UH
↳ 1cm Direct runoff

for Excessive rain $\left(\frac{1}{D}\right) \times D = 1$ cm

$\frac{cm}{hr} \times D$ hr

5. Unit hydrograph represents relation b/w direct run-off and rainfall excess.

NOTE: ERH (Effective Rainfall hyetograph and DRH both represent the same quantity but in different units.



(ERH)

Applications of unit hydrograph :→

1 Construction of flood Hydrograph :→
Suppose a 4 hour unit hydrograph is given and it is required to obtain a flood hydrograph which results in a runoff depth of x cm for a rainfall of the same 4 hour duration.

→ For this the ordinates of unit hydrograph are multiplied by x to obtain the DRH of 4 hour duration and having a runoff depth of x cm. To this at the base flow to obtain the ordinates of 4 hour duration flood hydro...

2. Construction other unit hydrograph of different unit duration which may be used to find the flood hydrograph for same duration.

Case 1st · When $t_0 = nT_0$

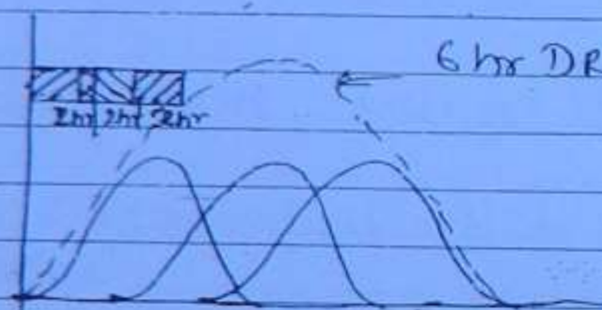
T_0 = duration of given hydrograph

t_0 = duration of unit hydrograph to be obtain.

n = positive Integer.

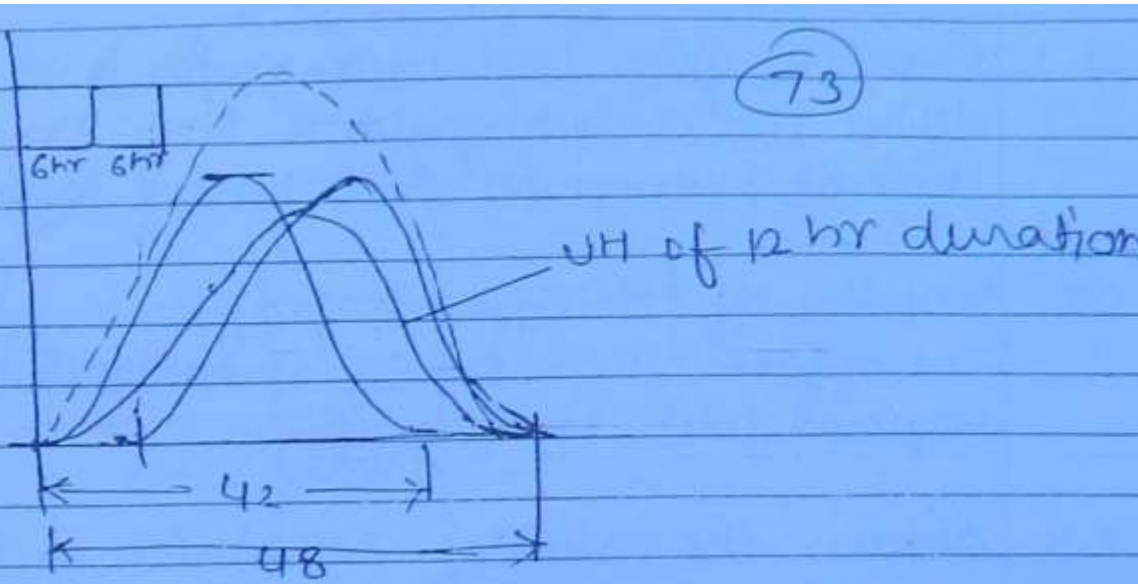
The given unit hydrograph of T_0 duration are lagged and Super Imposed in Such a way So as to obtain the DRH of t_0 duration.

The ordinates of this DRH are then divided by the runoff depth (The runoff depth for a Corresponds to the no. of unit hydrograph that has been Super Imposed).



Q Ordinate of 6 Hour Unit hydrograph are given below. find ordinate of 12 hr unit hydrograph.

Time	Ordinate of 6hr UH	Ordi. of 6hr Unit H. lagged by 6 Hour	12 hr DRH	UH of 12hr durn.
0	0	—	0	0
3	9	—	9	4.5.
6	20	0	20	10
9	35	9	44	22
12	49	20	69	34.5
15	43	35		[illegible]
18	35	49		
21	28	43		
24	22	35		
27	17	28		
30	12			
33	9			
36	6			
39	3			

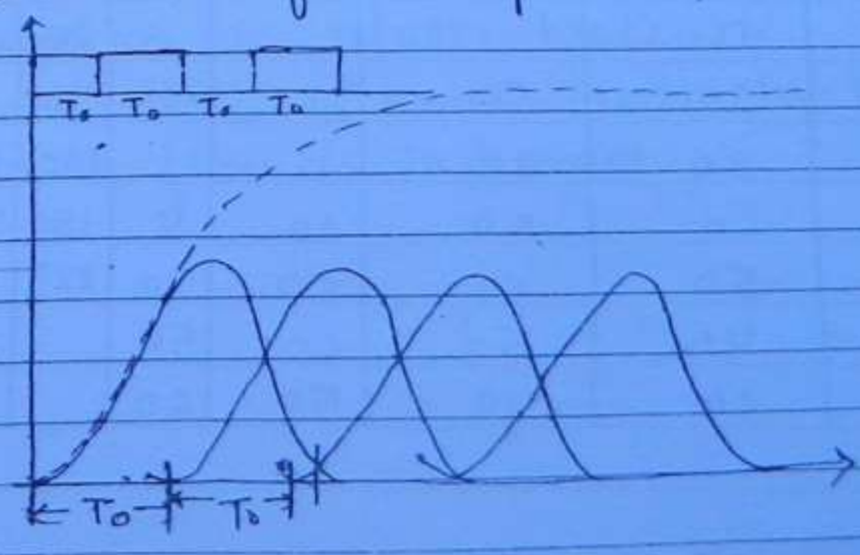


Case II Construction of a shorter and longer duration Unit Hydrograph from a given Unit hydrograph ~~from~~ of duration T_0

$$\boxed{t_0 = mT_0}$$ m = fraction value.

This can be done by using a different technique which is known as S-curve technique & S-curve method.

→ The S-curve is also known as S-Hydrograph and its produced by continious effective rainfall at a constant rate for an Infinite period.



STEP 1 From the given Unit Hydrograph ~~to~~ of T_0 duration derive the S-Hydrograph a large no. of same unit of hydrograph each having lagged of 'T_0'

74

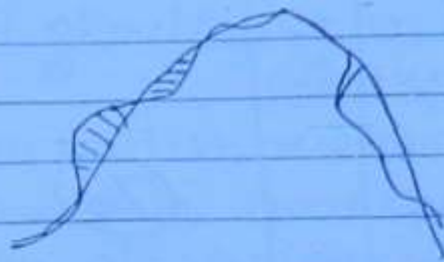
STEP 2 Find the offset S hydrograph.
The offset given is equal to the duration of required unit hydrograph.

STEP 3 Obtain the value of Δy which is the difference of S curve ordinate and offset S-curve ordinate

Ordinates of ~~direct~~ desired unit hydrograph of duration t_0 is given by $\frac{\Delta y \times T_0}{t_0}$

Q. Ordinates of 3hr. unit hydrograph are given below find the ordinates of 6 hr U.H using S-Curve?

Time	Ordinate of 3hr UH	1st lagging by 3 hr.	2nd lagging	3rd lagging	4th lagging	S curve	offset S-curve	Δy
0	0					0		0
3	20	0				20		20
6	50	20	0			70	0	70
9	60	50	20	0		130	20	110
12	50	60	50	20	0	180	70	110
15	40	50	60	50	20	220	130	90
18	10	40	50	60	50			
21	0	10	40	50	60			



Time	ordi. of 3hr UH	S-curve addition	S curve	off set	ΔY
0	0		0	—	
3	20	0	20	—	
6	50	20	70	0	
9	60	70	130	20	
12	50	130	180	70	
15	40	180	220	130	
18	10	220	230	180	
21	0	230	230		

Application :-

Derivation of unit Hydrograph from a given Complex hydrograph :→

$\frac{\Delta y \times T_0}{t_0} = \frac{\Delta y}{2}$

Q. The ordinates of flood hydrograph resulting from 2 successive storms each of 2 cm rainfall excess and each of 6hr duration is given below? find the 6 hour unit hydrograph. Base flow is 10 cumecs.

0
10
35
55
55
45

Time	Ordi. of 12hr flood Hydr. graph	DRH	$R_1 U$ $2 \times U$	$R_2 U$ $2 \times U$
0	10	0	$2U_1$	
6	30	20	$2U_2$	$2U_1$
12	90	80	$2U_3$	$2U_2$
18	220	210	$2U_4$	$2U_3$
24	280	270	$2U_5$	$2U_4$
30	220	210	$2U_6$	$2U_5$
36	166	156	$2U_7$	$2U_6$
42	126	116	$2U_8$	$2U_7$
48	92	82	$2U_9$	$2U_8$
54	62	52	$2U_{10}$	$2U_9$
60	40	30	$2U_{11}$	$2U_{10}$
66	20	10	$2U_{12}$	$2U_{11}$
72	10	0	$2U_{13}$	$2U_{12}$

76

$$2U_1 = 0 \Rightarrow U_1 = 0$$

$$2(U_2 + U_1) = 20$$

$$U_2 = 10$$

$$2(U_2 + U_3) = 80$$

$$U_3 = 30$$

Instantaneous Unit Hydrograph :→

a) $d = 0$ → bcoz ~~dept~~ 1cm

b) $T = 0$

c)

d)

IUH $\lim_{D \to 0}$

Flood gRouting :→

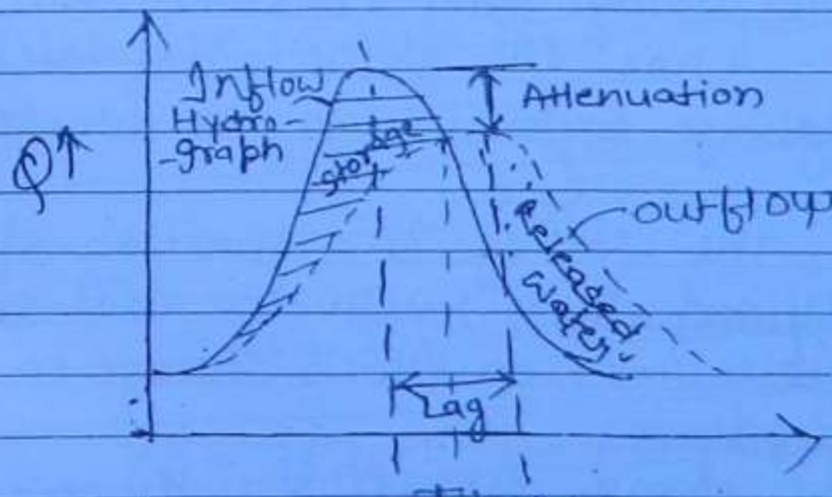
This is the process of Calculating Water level in reservoir, storage quantity and outflow rate corresponding to a Inflow hydro-graph at varience Instance. (77)

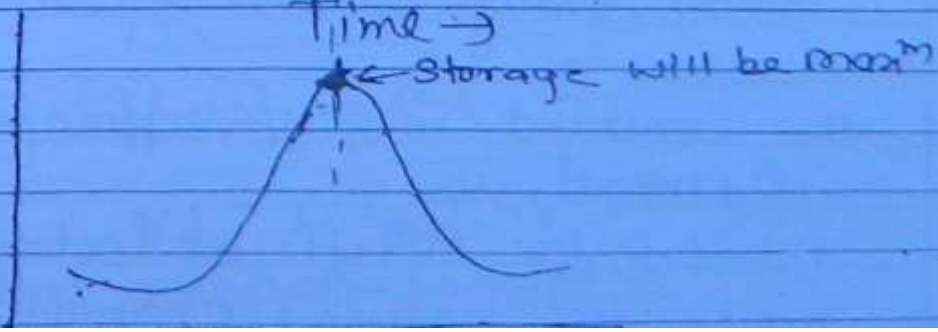
→ Flood g Routing is an Important technique necessary for the Complete Solution of a flood problem.

When flood passes through a reservoir its peak reduces and Its requires a bigger time base.

In this process a lag is Introduce. The reduction in peak flow is called attenuation and the delay in occurance of peak is called lag.

Through the process of flood routing The attenuation and lag may be determined





NOTE When outflow from a reservoir is uncontrolled Such as in the case of freely oprating Spill-ways the peak of outflow hydrograph ~~with~~ will occure at the point of Inter-section of Inflow and outflow hydrograph.

78

Muskingum

Through the process of routing for a given Inflow and outflow hydrograph the attenuation and lag may be determined.

→ In case of channels the total Storage is a function of both Inflow & ~~In~~ outflow.

According to muskingum the total Storage 'S' is given as

$$S = K[xI^m + (1-x)Q^m]$$

Where,

K = Storage time constant which has the dimension of time

x = Weightage factor which lies b/w 0 & 0.5

If x = 0, the storage is a function of out flow only and Such reservoirs are called linear reservoirs. on which Storage is f^n of outflow only.

x = 0.5, Storage depends Equally on out flow & Inflow

m = Constant and It depend on type of channel. ~~so~~

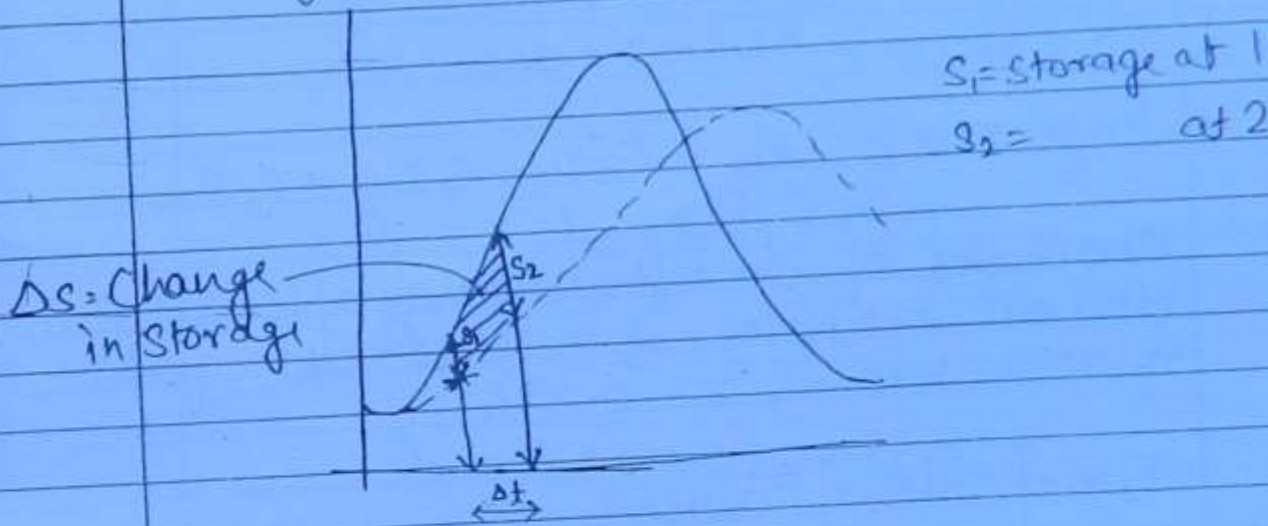
m = 0.6 for Artificial Rectangular Channel

m = 1 for natural channels

That mean for a natural channel musking-um Eq^n is

$$S = K[xI + (1-x)Q] \quad (79)$$

For a given channel by selecting a suitable routing interval. Δt



$$\Delta S = K[x(I_2 - I_1) + (1-x)(Q_2 - Q_1)] \quad — (1)$$

Also ~~for~~ Change in storage can be written as

(Change in Inflow Rate − change in outflow Rate) Δt

Or

= (Avg Inflow − Avg outflow) Δt

$$\Delta S = \left[\left(\frac{I_1 + I_2}{2}\right) - \left(\frac{Q_1 + Q_2}{2}\right)\right]\Delta t \quad — (2)$$

using Eq^n (1) & (2) Q_2 can be written as

$$Q_2 = C_0 I_2 + C_1 I_1 + C_2 Q_1$$

In this relation $C_0 + C_1 + C_2 = 1$

$$C_0 = \frac{0.5\Delta t - Kx}{K(1-x) + 0.5\Delta t}$$

$$C_1 = \frac{0.5\Delta t + Kx}{K(1-x) + 0.5\Delta t}$$

80

$$C_2 = \frac{K(1-x) - 0.5\Delta t}{K(1-x) + 0.5\Delta t}$$

In general $Q_n = C_0 I_n + C_1 I_{n-1} + C_2 Q_{n-1}$

Q Ordinates of an inflow hydrograph are given find out ordinates of outflow hydrograph If $K = 12$ hrs & $x = 0.278$

Time	Inflow	Outflow
0	42 $-I_1$	$Q_1 = 42$
4	68	
8	116 $-I_2$	$Q_2 = 45.87$
12	164	
16	194 $-I_3$	$Q_3 = 94.13$
20	$\boxed{200}$	
24	192 $-I_4$	$Q_4 = 157.1$
28	170	
32	150 $-I_5$	$Q_5 = \boxed{176.96} \rightarrow$ max outflow.
36	128	
40	106 $-I_6$	$Q_6 = 157.62$

Max^m Inflow

NOTE The value of Δt is selected such that Δt lies b/w $(2Kx$ & $K)$
This however is not a compulsion

Date:

$2Kx \Rightarrow 6.672$

$\Delta Kx \Rightarrow 12$, So Choose $\Delta t = 8$

81

$$C_0 = \frac{0.5 \times 8 - 3336}{12(1-.278)+.5\times 8} = 0.05243$$

$$C_1 = \qquad = 0.579$$

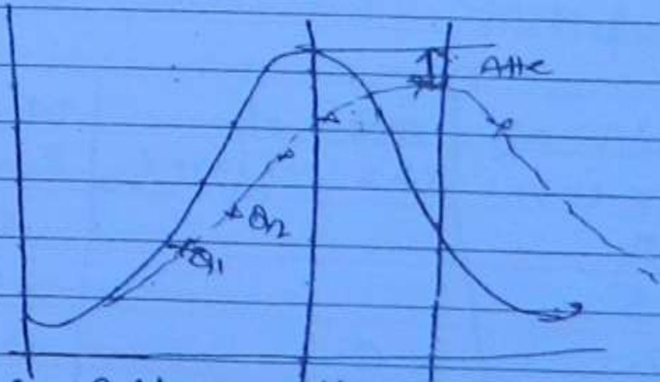
$$C_2 = \qquad = 0.3683$$

$$Q_n = C_0 I_2 + C_1 I_1 + C_2 Q_1$$

$$Q_2 =$$

$$Q_3 = C_0 I_3 + C_1 I_2 + C_2 Q_2$$

$$Q_4 =$$



Attenuation $= 200 - 176.96 = 23.04$

Lag $= 12$ hr 20 (12) 32

$32 - 20 = 12$

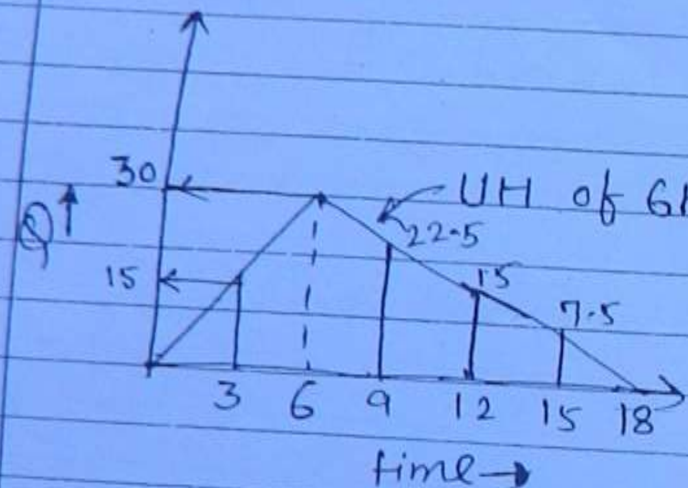
(18–24) then Select 20 → then 40 then it gives only two outflow point so curve cant be drawn so Calculate by direct choosen.

Date:

Q. An Avg. rainfall of 16 cm occures over a catchment during a period of 12 hours. Unit hydrograph of 6 hour duration rises linearly from 0 to 30 cumecs. in 6 hours and then falls from 30 to 0 in the next 12 hours. φ-Index of catchment is 0.5 cm/hr. and baseflow in the river is 5 cumecs. Find out the peak discharge of direct run off hydrograph.

Sol

(82)



→ DRH of 12 hr. having dept 10 cm

Run of depth (n) = 16 − 0.5 × 12
n = 10 cm

Time	ordi. of 6hr UH.	ordi. of 6hr. UH lagged by 6 hrs	DRH of 12 hr Runoff = 2 cm	DRH
0	0		0	
3	15		15	
6	30	0	30	
9	22.5	15	37.5	
12	15	30	45 Peak	45 × 5 = 225 Ans
15	7.5	22.5	30	
18	0	15	15	
		7.5	7.5	
		0	0	

What will be the Catchment Area :-

$$\frac{0.36\, \Sigma O t}{A} = 1 cm \quad (\text{for ord of 6hr UH})$$

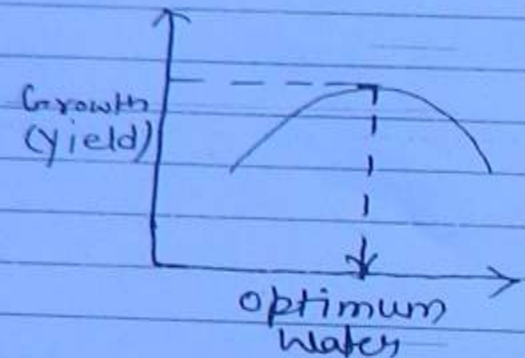
$$0.36$$

$$= 97.2 \text{ hect.}$$

IRRIGATION

Page No. 84
Date:

Irrigation may be defined as the science of artificial application of water to the agricult-ure land in accordance with the requirement of the crop through the crop period.



Advantages of Irrigation :-

1. Increase in the food production.
2. Optimum growth of or yield.
3. It Elimination of mixed croping.
4. Generation of Hydroelectric power.
5. Domestic water supply
6. Facility of communication.
7. Inland Navigation. (In India - Nation Water Ways - 6 NW-1 → 1620 Km Allahabad to Haldia → Longest) N.H-7 Varanasi to Kanyam
8. Afforestation (Increasing the forest)

Disadvantages of Irrigation :-

- Problem of water polution which is due to percolation of unwanted Elements like Nitrates Etc. which are present in fertiliser applied in crop.

Contamination → Not necessary to harmfull
→ If chalk mix in water then it will conta-minat but not polluted

2. Irrigation may result in creation of Cold & damp Enviromental Conditions which is favourable for out break of dearease like malaria, dengu etc

(85)

3 Over Irrigation may lead to problems of water logging. which reduce Crop yield.

Types of Irrigation :-

Irrigation may be brodly classified under two heads

1 Surface Irrigation :- water directly to the Surface

(i) Flow Irrigation

(ii) Lift Irrigation

Flow Irrigation :-

In this System water is available at a higher level and it is Conveyed to the field by mear action of gravity.

Lift Irrigation :-

In this System water is lifted up by Some mechanical means for Examples pumps, tube-wells etc.

2. Sub-Surface Irrigation :-

In this system of Irrigation water does not wet the soil Surface and Irrigation is primarily achived by the process of Capillarity.

Techeniques of Water distribution :-

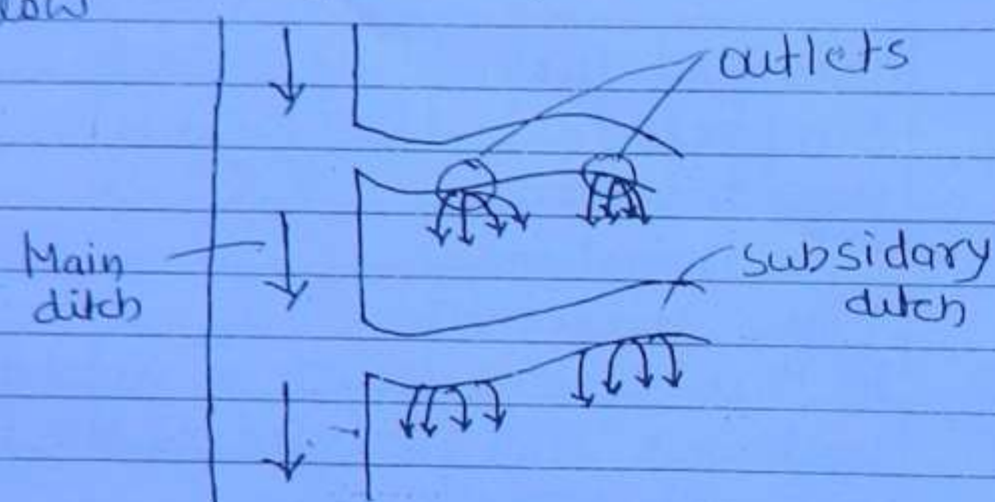
86

1. Free flooding :-

This is also called as ordinary flooding or wild flooding.

In this method the movement of water is not restricted. and this method is suitable for slopy lands.

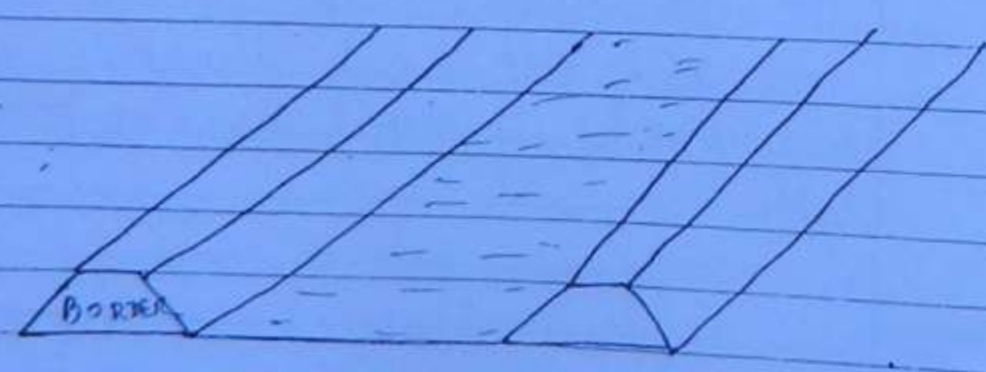
The Irrigation efficiency of this method is quit low



As the Irrigation efficiency is slow this method is not adopted for Important crops.

2. Border flooding :-

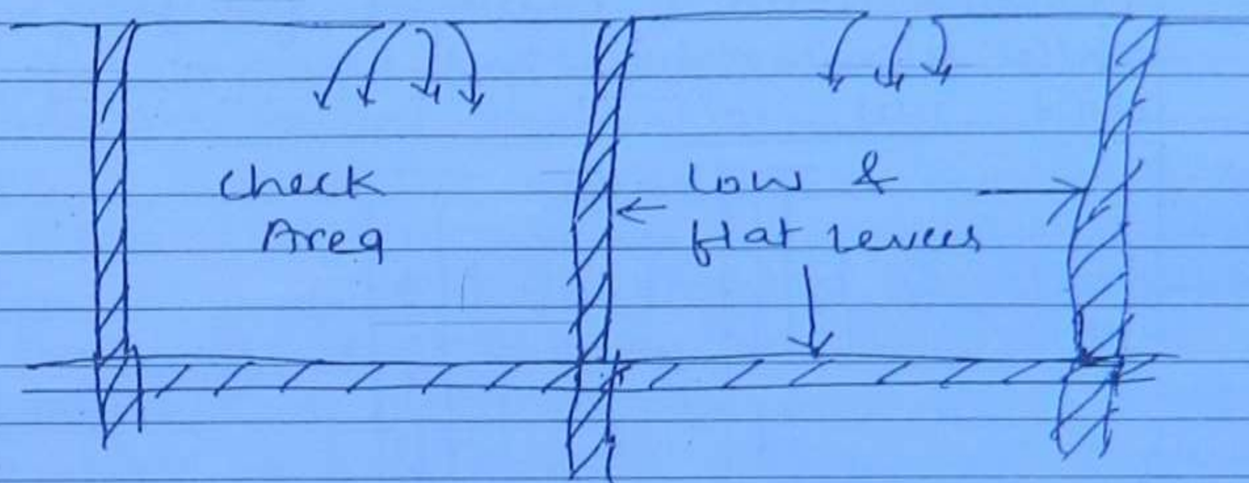
In this method the Entire area is divided into a number of Strips Which are sepsrated by low levees called border.



(87)

3 Check flooding :-

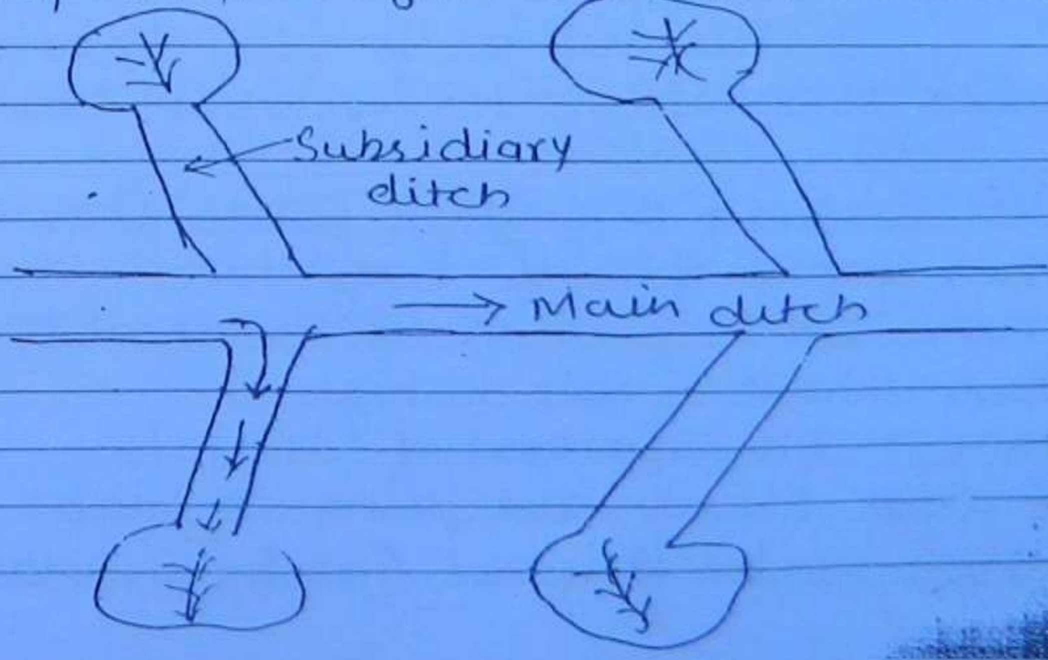
This method of Irrigation is similar to ordinary flooding except that the flow of water is controlled in the check area with low & flat levees.



→ This method is suitable for more permeable as well as less permeable soil.

(4) Basin flooding :-

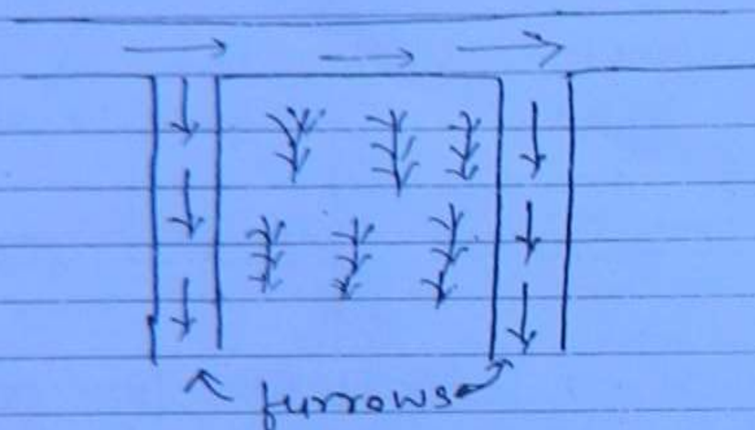
This method is similar to the check flooding method and this is particularly adopted for Orchard trees. (Garden of any tree).



5 Furrow Irrigation :-

In this method only 20-50% of the land area is wetted by water and thus evaporation losses are considerably reduce.

Furrows are basically narrow field ditches in which water is conveyed for Irrigation. These furrows are laid b/w rows of plants.



6 Sprinkler Method :-

This method of Irrigation resembles artificial rains in which water is sprayed on plant through a network of pipes and pumps.

This method is very coastly and hence is adopted in developed country.

This method is suitable when / Advantage.

1 When topography is Irregular
2 When soil more or less permeable
3 When water table is high

(NOTE :- This method of Irrigation is suitable for areas which are prone to water logging when water is not easily available

4. Irrigation Efficiency is very high.
5. Speepage losses are very less
6. less labour is required

Disadvantage:-

1. Skilled man power is required
2. Coastly method.
3. This is not suitable for Crops which require heavy Irrigation like Rice

Drip Irrigation:-

This method is also called trickling Irrigation & in this method, water is drip nozzles. As such Evaporation and percolation losses are Considerably reduced ~~it~~ is these are the main disadvantage of this method

Quality of Irrigation Water:-

(1) Sediment Concentration in Water:-

The precence of fine Sediment in Irrigation water may Increase or decrease the fertility of land If Sediments is obtained from fine Sandy Soils the fertility will Increase However If the Sediments are obtained from Eroded area the fertility of land is reduced.

2) Salt Concentration :-

Salts of Sodium, pottasium, Mg, Ca are harmful for plants and these are mainly found in water.

The salinity conc. of the soil solution after consumptive use has been extracted from the soil is given by

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

where,

Q = Quantity of water applied

C = Conc. of salts in Irrigation water

C_u = Consumptive use of water.

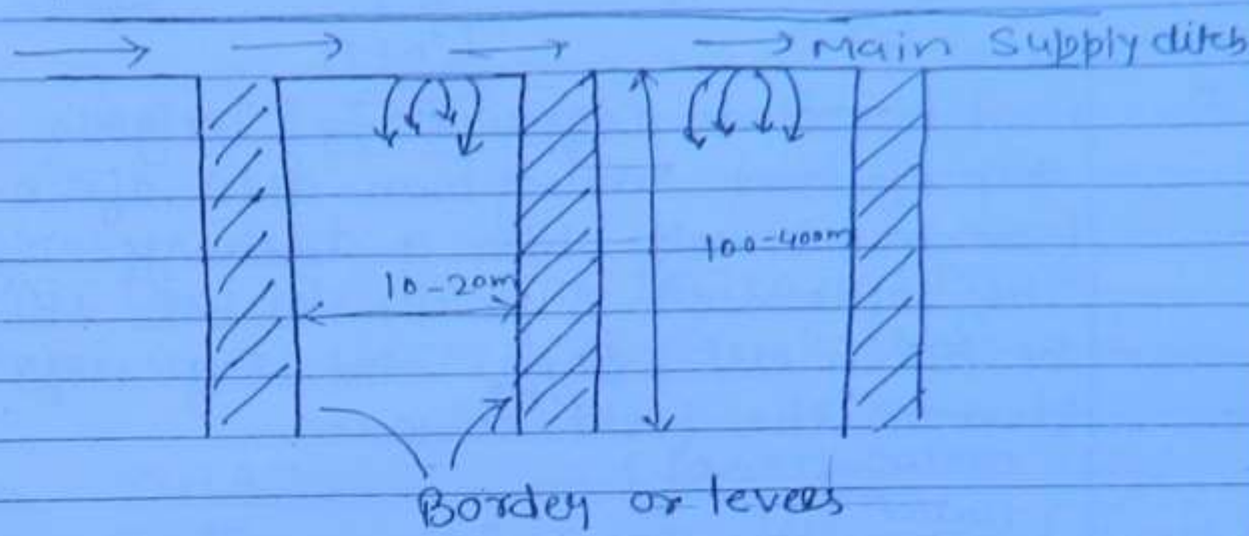
P_{eff} = useful Rainfall (during growing period of crop)

If this concentration is more than 2000 ppm then it is Injurious to all crops.

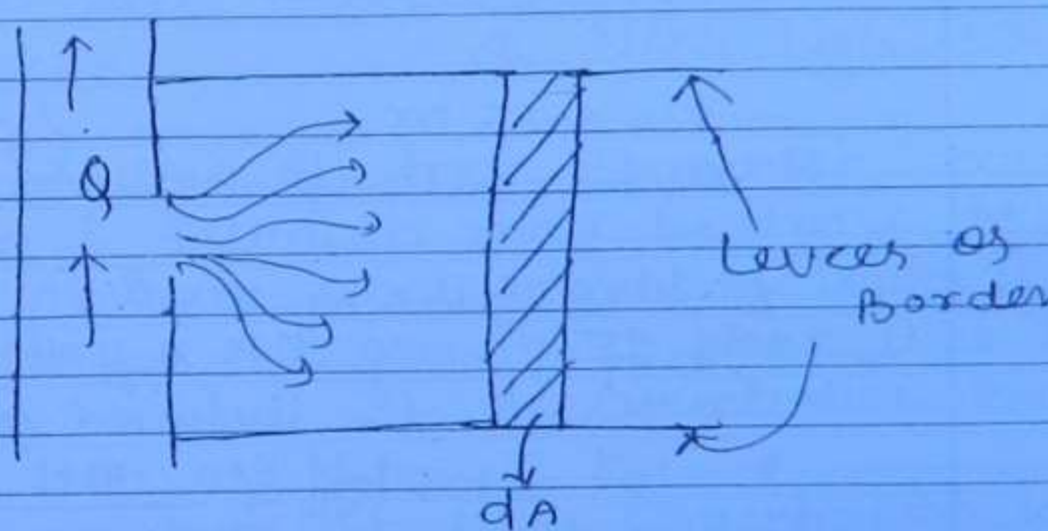
The Salt Concentration is Indirectly measured by finding the Electrical Conductivity of water [μ Mhos/cm]

Electric Conductivity (μ Mhos/cm)	Classification
0 - 250	C_1 = Suitable for Irrigation
250 - 750	C_2 = Suitable when leaching is done. (to reduce salt)
750 - 2250	C_3 = Not used
> 2250	C_4 = Never used (for Irrigation)

In this method the land area is confined in b/w 10-20 m width and its streches over a length of 100 - 400 m. 91



Time taken to irrigate particular area



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

$$(Q - fA)dt = y\,dA$$

$$\int dt = \int \frac{y}{(Q - fA)}\,dA \quad t \Rightarrow \frac{y}{f} \ln \frac{Q}{Q - fA}$$

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

Date:

At $t = \infty$ $\qquad$ ∵ because y, f, Q const so $f \neq 0$

$$Q - fA = 0$$

$$\boxed{A_{max} = \frac{Q}{f}}$$

92

Q Find the time required to Irrigate the strip of land having an area of 0.04 hectares from a tubewell with a discharge of 0.02 cumec. The Infiltration Capacity of the soil may be taken as 5 cm/hr and Avg. depth of flow on the field is 10 cm.

Sol)

$$\log_{10}\left(\frac{.02}{.02 - \frac{5(0.04 \times 10^4) \times 10^{-2}}{3600}}\right)$$

$$\left.\begin{array}{l} t = 0.65\ hr \\ A_{max} = 0.144\ hactare \end{array}\right\} \text{Ans} \quad \text{Or } 1440\ m^2$$

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

$$t = \frac{10^{-1}}{5 \times 10^{-2}}\, 2.303 \log_{10}\left(\frac{72}{72 - 5 \times 10^{-2} \times 0.04 \times 10^4}\right)$$

$$t = 0.650$$

$$A_{max} = \frac{Q}{f} = \frac{72\ m^3/hr}{5 \times 10^{-2}\ m/hr} = 1440\ m^2$$

Or 0.144 hare

Concentration of Sodium :-

The Concentration of Sodium Ion is Expressed by a factor Called Sodium absorption Ratio (SAR).

93

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

The Concentration of Na, Ca and Mg is Expressed in terms of Equivalents.

SAR		Classification
0 - 10		S_1 low salinity
10 - 18	Coarse soil ←	S_2 Medium salinity
18 - 26	leaching ←	S_3 High salinity
>26		S_4 Very high

Concentration of toxic Elements :-

Boron & Selinaium may be toxic to plant however small amount of boron is Essential for the growth of plant but It becomes harmfull If its Concentration Exceeds more than 0.3 ppm.

Selinium Even at low Concentration is toxic to plants.

Bacterial Concentration :- This is harm[illegible] particularly for the food crops and Vegitables.

Design of Canal

Regime Channel :-
A Channel is set to be in a state of regime if there is no silting and no scouring

For the design of Canals two theories are available (unlined or Earthen channels)

B.

Kennedy Theory :-
Kennedy gives the theory in 1895. based on the research conducted by him in the upper Bari doab Canal. From the observation conducted by him. He Concluded that Silt supporting power of a Channel depends mainly uppon generation of Eddies from the base of the channels.

Bari:- where 2 [illegible] [illegible]
Sangam:- where 2 rivers meet (each)

These Eddies are primarily generated due to friction. If velocity of flow is sufficient to keep the sediments in suspension silting may be avoided and If this velocity is not very high then scouring may also be prevented. Such a Channel will be called a Regime channel.

During the analysis Kennedy has assumed that sediments are Incoherrent and discreate.

(C=0) cohesionless

$$\tau = \cancel{C} + \sigma \tan\phi$$

$\rightarrow 0$

STEPS IN THE DESIGN OF CANAL :-

1. Assuming a trial depth y find the velocity V_0 which keeps the sediments in suspension. Such a velocity is called Critical Velocity and is given as $\boxed{V_0 = 0.55\, m\, y^{0.64}}$

Where, - m = Critical Velocity Ratio and it depends on type and size of silt grains.

m is the function of soil particle diameter and its value is 1 - 1.3 for coarse particles
m = 0.7 mm for fine sands
(0.7 to 1.3) Range

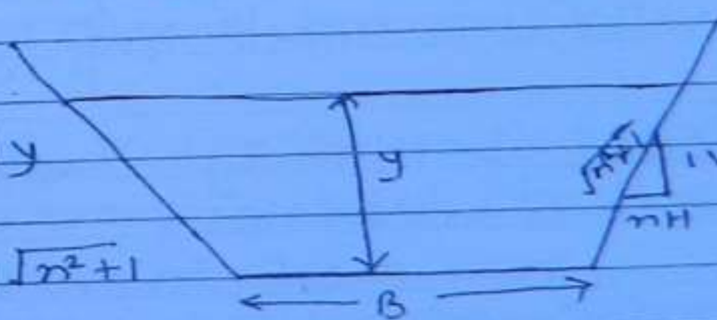
2. Find the area of channel for the given discharge.

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

3. Find the channel dimensions by assuming a trapezoidal section whose side slope is $\frac{1}{2}H : 1V$

$$A = (B + ny)\,y$$

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



4. Find the hydraulic Radius (R) :

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

5 Using the value of R find the actual mean velocity.

$$\boxed{V = C\sqrt{RS}} \quad \text{or} \quad \boxed{V = \frac{1}{n} R^{2/3} S^{1/2}}$$

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

Though velocity can be calculated by either of the method Kennedy has recommended the use of Chezy's and Kutter's Eqn for finding the Actual Velocity

6 If the actual mean velocity obtained is equal to V_0 then design is ok or else repeat the calculation using the different trial depth y.

NOTE If channel bottom slope is not given then its value is assumed b/w $\boxed{S = 1/3500 \text{ to } 1/5000}$

If value of manning's (n) is not given then it is taken as

$n = 0.015$ to 0.018 for Conc. channel

$n = 0.022$ to 0.025 for good earthen

$n = 0.025$ to 0.030 for poor earthen channel

~~Guide Line~~ :-

NOTE:- Garret's diagram can also be used to find the solution for design of canal using Kennedy theory graphical.

K · Chezzy → open channel ...
other formula → pipe line supply

Page No.
Date:

Guide line :-

Q	y
0 - 20	1 m
20 - 40	2 m
40 - 80	2.5 m
80 - 100	3 m
≥ 100	≥ 3.5 m

(97)

Limitations :-

(1) If Kennedys has assumed that eddies are generated from the base of channel where as in real pratice eddies are generated both from base as well as side of channels.

(2) Kennedy has not given his own Equation for finding the actual mean velocity and instead he has used Cheezy's and Kutter's Equation. As such all the limitations of Cheezy's and Kutter's Eqn's becomes a limitation of Kennedy's theory.

(3) Kennedy has not given any Equation for finding the channel bottom Slope.

(4) The value of m (i.e critical Vel ratio) has been arbitarory related to the silt grade.

(5) Kennedy has not given any Importance to width to depth ratio.

(6) He has not taken into account silt concentration and bed load.

NOTE :-

Lacey's ~~form~~ silt theory :-
Lacey's found many drawbacks in the Kennedy's theory for design of Earthen channel (98)

Lacey proposed his own theory in 1939 for the design of stable channel In alluvial soil

Lacey stated that a channel may be three regime conditions
i) Initial Regime
ii) True Regime
iii) Final Regime

Initial Regime :-
If the channel is such that only bed slope of channel varies and Its wetted perimeter remains unaffected then a condition of non silting and non-scouring may exist which is called Initial Regime.
During the subsequent flow and change in condition the regime condition may change. Under such a condition lacey's silt theory is not valid.

True Regime :-
There can be only one channel section and one bed slope at which a channel having a given discharge under particular type & quantity of silt would be in regime.
Such channels are artificial which

have fixed slope, discharge at a constant rate and have fixed silt amount and rate. Such conditions are Ideal in nature and do not exist in practice.

Lacey's theory is valid for such regime conditions.

Final Regime :-

If all the variables of the channels such as perimeter depth slope etc are free to vary and can adjust according to discharge and rate of silt and their occurs no siltation and no erosion than such channel are said to have achived permanent stability and are said to be in final Regime.

- Design steps in Lacey's silt theory

1 Find the velocity of flow using Equation

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

Q = discharge in cumecs
V = Velocity in m/sec
f = silt factor

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

2 Find the hydraulic radius using this Equation

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

Date:

3 Find the area of the channel

$$\boxed{A = \frac{Q}{V} = (B+ny)y} \quad — (1)$$

100

4 Find the Wetted perimeter of channel

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

$$\boxed{P = B + 2y\sqrt{n^2+1}} \quad — (2)$$

5 Here also we assume side slope is $\frac{1}{2}H : 1V$

$$n = \frac{1}{2}$$

5 The channel bottom slope is given as

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

Comparisions of Kennedy's & Laceys theory:-

1 Kennedy had neglected the Eddies generated from sides of the channel which was taken into account by lacey.

2 Kennedy Stated that all channels are in Regime if they do not Silt or scoure. Where as lacye differentiated b/w the two regime conditions i.e Initial Regime and final

3 Kennedy has not provided a relation b/w Silt factor and Size of grain which has

been provided by Laceys $S = \frac{f^{5/3}}{3340\, Q^{1/6}}$ $f = 1.76\sqrt{d}$

4 Kennedy has not given his own Equation for channel Water flow which has been provided by Laceys (T01)

5 Kennedy has used Chezy's and Kutter's Eq. for stablishing a relation b/w Velocity and hydraulic radius where as lacy has given his own Eqn for Establish the relation b/w vel. & hydraulic radius.

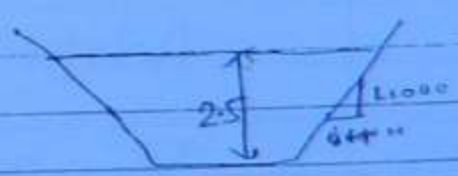
Q Design an Irrigation channel to carry 50 cumecs of discharge. Channel is to be laid on the slope of 1 in 4000 the Critical velocity ratio for soil is 1.1 and manning's n = 0.023.

Sol $Q = 50$, $b \cos 3$ $y = 2.5$

$$V_0 = 0.55 \times 1.1 \times 2.5^{0.64}$$
$$= 1.087$$

$$A = \frac{Q}{V_0} = \frac{50 \text{ m}^3/\text{sec}}{}$$

$$A = 45.97 \text{ m}^2$$



∴ Adopt $n = 1/2$

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

$$45.97 = (B + \tfrac{1}{2} \times 2.5)2.5$$

$$B = 17.138$$

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

$P = 22.72$

102

$R = A/P = 2.022$

$V = 1.099$ by manning but cont use

$$C = \frac{23 + \frac{0.00155}{(1/4000)} + \frac{1}{0.023}}{1 + \left(23 + \frac{0.00155}{1/4000}\right)\frac{0.023}{\sqrt{2.022}}}$$

$\cancel{V = 1.100}$ $C = 49.35$

$V = C\sqrt{RS}$

$V = 49.35\sqrt{2.022 \times 1/4000}$

$V = 1.109$

assume $y = 2.7$ m

$V_0 = 0.55\, m y^{0.64} = 1.142$ m/s

$A = \frac{Q}{V_0} = 43.76\ m^2 = (B + ny)y$

Adopt $n = 1/2$, $\Rightarrow B = 14.86$ m

$P = B + 2y\sqrt{n^2+1} = 20.89$

$R = A/P = 2.09$m

$V = C\sqrt{RS}$ $C = 49.61$

$V = 1.1356$ m/sec

$y = 2.6$ m

Q. Design an Irrigation channel in alluvial soil according to laceys theory following data is given. - Q = 15 cumecs
Lacey's silt factor is one and channel side slope is ½H : 1V.

103

Sol.

$Q = 15\ m^3/sec$

$f = 1$

$V = .689$

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

$R = 1.186$

$$A = \frac{Q}{A} = 21.77 = (B + ny)y$$

$\cancel{B = 21.77}$ $P = 4.75\sqrt{Q}$

$P = 18.39$

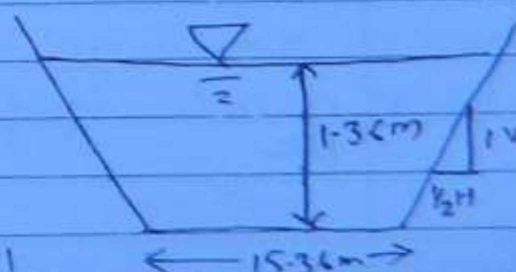
$P = B + 2y\sqrt{n^2+1}$

$\cancel{18.39 = B + (2\sqrt{.5^2+1})\,y}$

$\cancel{B = 16.15}$

$18.39 = B + 2y\sqrt{n^2+1}$

$21.77 = By + ny^2$



$B = 15.36m$

$y = 1.356m$

$$S = \frac{f^{5/3}}{3340\,Q^{1/6}} = \frac{1}{5245}$$

Q Design an Irrigation channel to carry 40 cumecs discharge in which $B/y = 2.5$ and CVR $= 1.0$.
Assume mannings $n = 0.023$

104

Sol

$Q = 40\ m^3/s$

$B/y = 2.5$

$m = 1.0$

$$V_0 = 0.55 \times 1 \times \left(\frac{B}{2.5}\right)^{.64}$$

$$\cancel{= .988}$$

$$V_0 = .305 \times B^{.64}$$

$$A = \frac{Q}{V_0} = \frac{40}{.305 \times B^{.64}}$$

$$A = \frac{130.73}{B^{.64}}$$

$$\frac{130.73}{B^{.64}} = (B + my)y$$

$$(2.5y + my)y = \frac{130.73}{B^{.64}}$$

$$2.5y^2 + .5y^2 = \frac{130.73}{B^{.64}}$$

$$3y^2 = \frac{130.73}{(2.5y)^{.64}} \qquad y =$$

$$y^{2.64} = 24.24$$

$$y = 3.345$$

$$B = 8.3625$$

$$V = C\sqrt{RS} \qquad \text{Let } S = 1/4000$$

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

$A = 33.579$

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

$P = \cancel{15.249}\ 15.842$

$R = \frac{A}{P} = \cancel{3.102}\ 2.1196$

$$C = \frac{23 + \frac{0.00155}{1/4000} + \frac{1}{0.023}}{1 + \left(23 + \frac{0.00155}{1/4000}\right)\frac{0.023}{\sqrt{3.102}}}$$

$C = 52.61$

$V = C\sqrt{RS}$

$V = 5860.2$

Designs of Lined Canal :-
Lining of canal means that the Earthen surface of channel is lined with the stable and Inerodable lining surface such as tile, concrete, asphalt etc.

Advantages of lining :-

1 Seepage control :-
3 to 30% of water in Earthen channel is lost due to seepage which may be saved by lining

2 Prevention of Water logging :-

3 Increase in channel capacity :-
by permi[illegible]
greatest velocity. $V \propto \frac{1}{n}$. n will d[illegible]

4 Reduction in maintainance Cost by about 40-50%

106

5 Increase in Command Area.

6. Elimination of flood ~~dang~~ danger.

Disadvantage of lining :-

1 Large Initial Investment which is about 2-3 time of that of an unlined canal

2 Greater Construction period (Return of amount)

Economic Justification of lining

If average annual benefit is greater than avg. annual Investment then lining of the Canal is desirable.

$$\left(\frac{B}{C}\right) = \frac{\text{Benefit in one year}}{\text{Cost}}$$

$$\left(\frac{B}{C}\right) > 1 \text{ Lining}$$

Average Annual benefit :-

Monitory benefit is achieved in the following two ways.

1 Reduction in Seepage :-

Let m Cumecs of Water we save per year by preventing seepage and

It is sold to farmers at the Rate of Rupees R_1 per Cumec than total annual saving will be mR_1

(107)

$$\left.\begin{matrix} m \\ R_1 \end{matrix}\right\rangle mR_1$$

2 If ₹ R_2 per Annum is the maintance cost for unlined cannal and if by lining p fraction of it save. then total saving per annum is pR_2

Total Annual benefit $= mR_1 + pR_2$

Annual Expenses :-

(i) Let C be the total Amount of capital required for lining the canal and let y years the service life of canals.
Therefor Avg. annual Expense of lining is (C/y).

$$\left.\begin{matrix} C \\ y \end{matrix}\right\rangle \rightarrow C/y$$

ii) $r \longrightarrow C \times \frac{r}{100}$

$$\left(\frac{C+0}{2}\right) \times \frac{r}{100} \Rightarrow \frac{1}{2}\frac{Cr}{100}$$

Due to Investment of capital there is loss of Intrest.
In the begining loss is on Rs C where as in the End capital reduces to zero.
So We take Avg value $\frac{1}{2}\frac{Cr}{100}$

Annual Expenses $= \frac{c}{y} + \frac{1}{2}\frac{c\,r}{100}$

Total Cost

108

$C \rightarrow$ Cost $= \frac{c}{y} + \frac{1}{2}\frac{c\,r}{100}$

$B \rightarrow$ Benfit $= mR_1 + pR_2$

if $\frac{B}{C} > 1$, than go for lining.

Assumption :-

1 Make the Calculation for 1 km and find the Cost and benfit for 1 year

2 Interest rate may be assumed in b/w 5 to 8%

3 Saving in maintance cost approximately 40%

4 Life of Cannal after lining is 40 to 50 years

5 Seepage looses from unlined Canal approximately 30 to 35% where as from a lined Canal it is b/w 1 to 3%

Q. A unlined canal giving a seepage loss of 3.3 cumecs/million m² of wetted area is proposed to be lined by 10 cm thick cement concrete lining which costs Rs 180/10 m² of area. Now following data is given based on which find out benefit to cost ratio?

a. Annual Revenue per cumec of water for all crop = 3.5 laks

b. Wetted perimeter of lining = 18.5 m (109)

c. Discharge in the channel $Q = 83.5 \ m^3/s$

d. Area of the channel is 40.8 m² than wetted perimeter of channel when unlined is 18.8 m.

f. Annual maintainance cost of unlined channel is Rs 1 per 10 m² of area assume % saving in maintainance by lining is 40%. take $r = 5\%$ seepage through lined cannal is ~~0.01 cumec~~ 0.01 cumecs per million m² of wetted perimeter

Sol

B:

Reduction in Seepage = $(\text{Seepage})_{\text{unlined}} - (\text{Seepage})_{\text{Lined}}$

Wetted Area = Wetted perimeter × Length

$= 18.8 \times 1000$ (1 km)

$= 18800 \ m^2$

Seepage in Unlined $= \frac{3.3}{10^6} \times 18800$

$= 0.062 \ m^3/sec$

Area of Wetted ~~p~~ lined canal $= 18.5 \times 1000$

$= 18500 \ m^2$

Seepage in lined $= .000185 \ m^3/s \leftarrow \frac{18500 \times .01}{10^6}$

Seepage loss $= 0.062 - 0.000185 = .06188$

Anual Revenue = 6.0618 × 3.5
= 21630

(110)

Maintainance, given that ⇒ $1/10\ m^2$
cost for unlined canal

$= \frac{1}{10} \times 18800 = 1880$ Rs.

Saving = 40% of 1880
= .4 × 1880
= 752

Total benefit = 21630 + 752

B = 22382.

C:

Wetted Area of lining = 18.5 × 1000
= 18500

For lined Canal $= \frac{180}{10} \times 18500$

C = 333000 Rs

Service life of canal y = 40 yrs

Annual Expense $= \frac{C}{y} = \frac{333000}{40} = 8325$

(ii) $\frac{1}{2} C \frac{r}{100} = 8325$

Total Cost = 8325 + 8325 = 16650

$\frac{B}{C}$ ratio = 1.34 > 1

So Lining is Economically Justified.

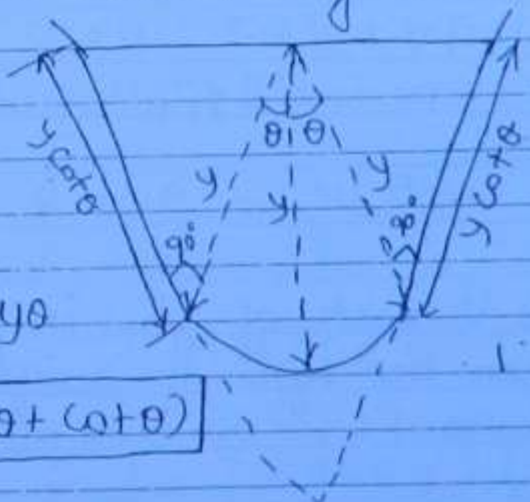
Design of Lined Canal :-

In lined Canals higer Velocity may be permi-
-tted by taking advantage of hard wearing
surface.

Generally two type of Sections are designed
i) Triangular Section with round bottom
(This is prefered when discharge < 150 cumecs
ii) Trapezoidal Section with round Corners
(This is prefered when discharge > 150 cumecs

Triangular Section :

To Increase the discha-
-rge Carring Capacity with minimum Cost
and to avoid silting base of the Section
is modified into a Circular arc so that
Hydraulic radius may Increase.



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

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

$$A = \left(\tfrac{1}{2} \times y\cot\theta \times y\right) \times 2 + \frac{\pi y^2}{2\pi} \times 2\theta$$

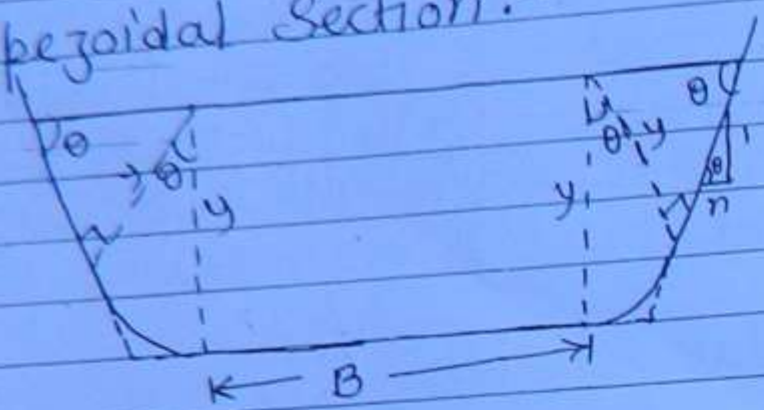
$$= y^2\cot\theta + y^2\theta$$

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

NOTE :- Value of θ is in Radian.

NOTE For designing the most efficient section the circular arc is made in such a way that radius of circle at tangent makes an angle of ~~90°~~ 90° with the side slope.

2. Trapezoidal Section :- (112)



$$\boxed{A = By + y^2(\theta + \cot\theta)}$$

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

nH : 1V

1.5 H : 1V   n is usually ≃ 1.2 to 1.5

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

Permissible Velocity of flow in the channel depends upon the type of surface ~~[illegible]~~

① Cement Concrete channel = 2 – 2.5 m/sec permissible velocity

② burnt clay lining = 1.8 m/sec per. vel.

③ Boulder velocity = 1.5 m/sec. per. vel.

Date

Q. Designed a lined canal to carry a discharge of 15 cumecs. The available Slope is 1 in 9000. assume suitable values of side slopes and good brick work in lining. (113)

Sol $Q = 15\ m^3/s < 150$ So design triangular section)

$S = 1/9000$

1.5H : 1V

$n = 1.5$

$n = \cot\theta = 1.5$

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

$\cot\theta = 1.5$

$\cot\theta = .026$

$\theta = 0.588$

$\theta = .67$

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

$A = y^2(.588 + 1.5)$ —— (1)

$= y^2 \times 2.088$

$P = 2y(.588 + 1.5)$

$P = 4.176y$ —— (2)

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

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

$15 = \frac{1}{.018}(2.088y^2)\left(\frac{y}{2}\right)^{2/3}\left(\frac{1}{9000}\right)^{1/2}$

$\times .629\, y^{2/3} \times (.0105)$

$15 = .767 \times y^{8/3}$

$(y^{2+2/3}) \times \frac{6+2}{3}$

$y = 3.04$ Ans

Q Designed a Conc. lined channel to carry a discharge of 350 cumecs at a slope of 1 in 5000. The side slope of the channel may be taken as 1.5 H : 1 V. The values of mannings n is 0.014 and assume the limiting velocity in the channel as 2 m/sec

114

Sol

$Q = 350\ m^3/sec$

$S = 1/5000$

$n = 1.5$

$n = .014$ (manning.

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

$$\cot\theta = 1.5$$
$$\theta = 0.588$$

$$A = By + y^2(.588 + 1.5)$$

$$P = B + 2y(.588 + 1.5)$$

$$R = \frac{A}{P} = \frac{By + y^2(2.088)}{B + 4.176y}$$

$Q = AV$

$$V = \frac{Q}{A} \qquad A = \frac{Q}{V} = \frac{350\ m^3/sec}{2\ m/sec}$$

$$A = 175\ m^2$$

$$175 = By + y^2 \times 2.088$$

$$\frac{175 - y^2 \times 2.088}{y} = B$$

Date:

$$P = \frac{175}{y} - y \times 2.088 + 2y(2.088)$$

$$P = \frac{175}{y} - y \times 2.088 + 4.176y$$

115

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

$$2 = \frac{1}{.014} R^{2/3} \left(\frac{1}{5000}\right)^{1/2}$$

$$R^{2/3} = 1.97$$

$$R = 2.785$$

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

$$P = \cancel{487.53}\ 62.83$$

$$62.83\ \cancel{487.53} = \frac{175}{y} + 2.088y$$

$$175 + 2.088y^2 - \cancel{487.53}\,62.83\,y = 0$$

$$2.088y^2 - 62.83y + 175 = 0$$

$$\frac{\cancel{487.53}\,62.83 + \cancel{\ldots}\ 49.85}{2 \times 2.088}$$

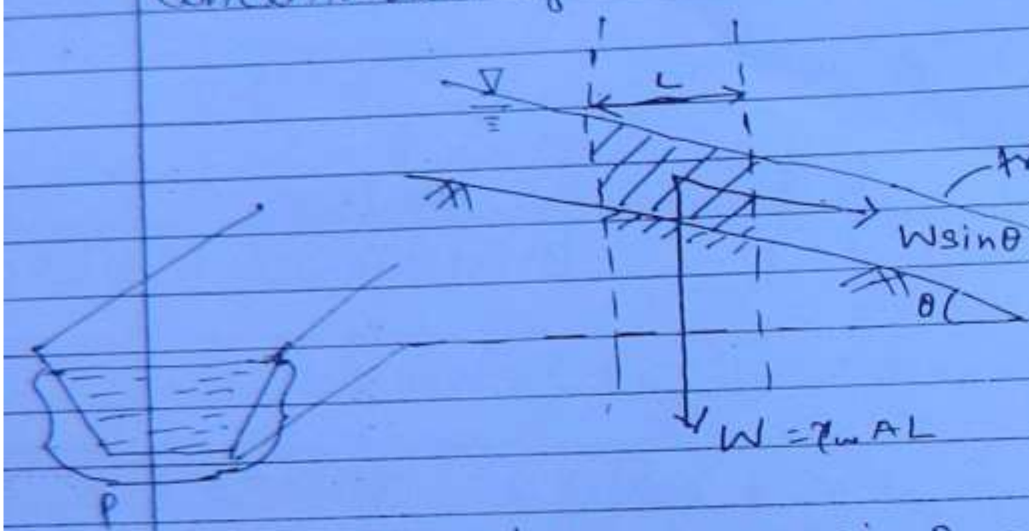
$$y = 26.98 \text{ or } 3.108\ m$$

$$B = 49.85\ m$$

Sediment transport Mechanism :-
(for unlined ~~canal~~ canal)

This mechanism is for an unlined canal. and It Involves the following assumption.

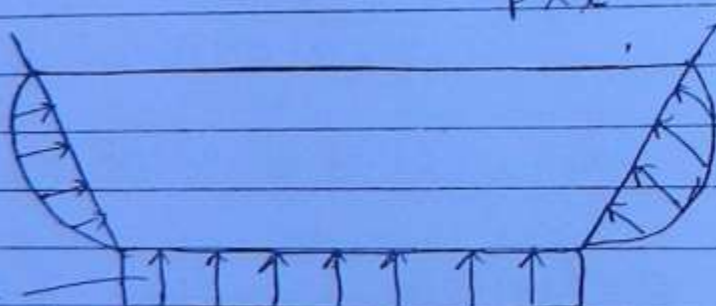
1. Soil is Incoherent (cohessionless) Sands & gravels
2. Sediment load is uniform which means that concentration of Sediment remains Constant



$W\sin\theta = \gamma_w A L \sin\theta$
$= \gamma_w A L \tan\theta$
$= \gamma_w A L S$
$\theta \downarrow \ \sin\theta \simeq \tan\theta$

Tractive force $= W\sin\theta = \gamma_w A L S$

Tractive stress $= \dfrac{\gamma_w A \cancel{L} S}{P \times \cancel{L}} = \gamma_w R S$



$\tau_o = \gamma_w R S$
$= 0.75\, \gamma_w R S$

τ_o = Actual ~~shear~~ tractive stress

Date:

Shield has given his own theory for the non-scouring condition in course alluvium. According to his theory If τ_c is the shear stress required to move the soil grains of diameter d on the bed of the channels then τ_c is given as

117

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

S_s = specific gravity of soil grains
usually the value of $S_s = 2.65$

$$\tau_c = 0.056\,\gamma_w d\,(2.65 - 1)$$
$$= 0.0924\,\gamma_w d$$

$$\boxed{\tau_c = \frac{\gamma_w d}{11}}$$

τ_c = is the stress at which soil particles starts moving.

For no Erosion from the bed of the Channel that is to Ensure the non scouring condition $\tau_o \le \tau_c$

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

$$\boxed{d \geq 11 RS}$$

The min^m shear stress required for Erosion of grains from sides of channel is Equal to

$$\boxed{\tau_c' = \tau_c \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}}}$$

For no Errosion (Scouring) from Sides of the Channel

$$\boxed{\tau_0' \le \tau_c'}$$

$$0.75\gamma_w RS \le \tau_c \sqrt{1-\frac{\sin^2\theta}{\sin^2\phi}} \quad (118)$$

Where,

ϕ = friction angle of Soil

θ = Angle of channel side from horizontal

Q An unlined Irrigation Canal has its bed and side composed of Cohession less malerial having mean diameter of 6mm angle of repose is 40° width of channel is 5m and side slope is 1.5H : 1V. determine the discharge that can be admitted into the canal without any sediment movement the channel bottom slope is 1 in 5000 and manning coeff $n = .025$.

Sol

$d = 6m$

$\theta = \phi = 40°$

$B = 5m$

Condition ① :-

For no Scouring from the bed of the channel

$$\gamma_w RS \le \tau_c \sqrt{1-\frac{\sin^2\theta}{\sin^2\phi}} \text{ - for } 2^{nd} \text{ cond}$$

$$\gamma_w RS \le \frac{\gamma_w d}{11} \text{ — for } 1^{st} \text{ Cond}$$

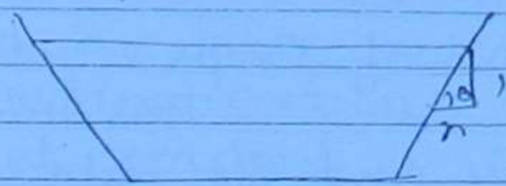
$$R \le \frac{d}{11S}$$

$$R \le 2.72m$$

Date:

Cond. 2^{nd} $0.75\,\gamma_w R S \le \frac{\gamma_w d}{11}\sqrt{1-\frac{\sin^2\theta}{\sin^2\phi}}$

(119)



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

$$\sin\theta = \frac{1}{\sqrt{n^2+1}}$$

$$0.75 \times R \times \frac{1}{5000} \le \frac{6}{11}\sqrt{1-\frac{\sin^2\theta}{\sin^2\phi}} \qquad \sin\theta = 0.554$$

$$R \le 1844.1 \text{ mm}$$

$$R \le 1.844 \text{ m}$$

$$R = \frac{A}{P} = \frac{(B+ny)y}{B+2y\sqrt{n^2+1}}$$

$$1.844 = \frac{(5+1.5\times y)y}{5+2y\sqrt{1.5^2+1}}$$

$$1.844 = \frac{5y+1.5y^2}{5+3.6y}$$

$$y = 3.06 \text{ m}$$

$$Q_{max} = \frac{1}{n} A R_{max}^{2/3} S^{1/2}$$

$$Q_{max} = \frac{1}{0.025} \times (5+1.5\times 3.06)\,3.06 \times 1.844^{2/3} \times \left(\frac{1}{5000}\right)^{1/2}$$

$$Q_{max} = 24.96,\ 18.81$$

If the discharge is slightly increase then scouring will start to take place from side of the channel whereas no scouring will take place from bed of the channel as the no Scouring condition for the channel bed is still Ensured.

Water requirement of Crops :-

This Indicates the total quantity and the technique by which a crop requires water from the time it is shown to the time of harvesting.

→ Soil moisture is Essentially made up of four Components

1. Gravity Water
2. Capillary Water
3. Hygroscopic Water.
4. Chemically bound Water

Gravity Water :-

If heavy Irrigation or rainfall takes place then moisture Content reaches 100% of Saturation.

obj PSU If this Saturated Sample is subjected to free drainage then a part of water is removed by gravity force in 5 to 15 days (2 to 5 days)

This Water Which Can be removed by gravity force is Called gravity water and the remaining water is at field Capacity.

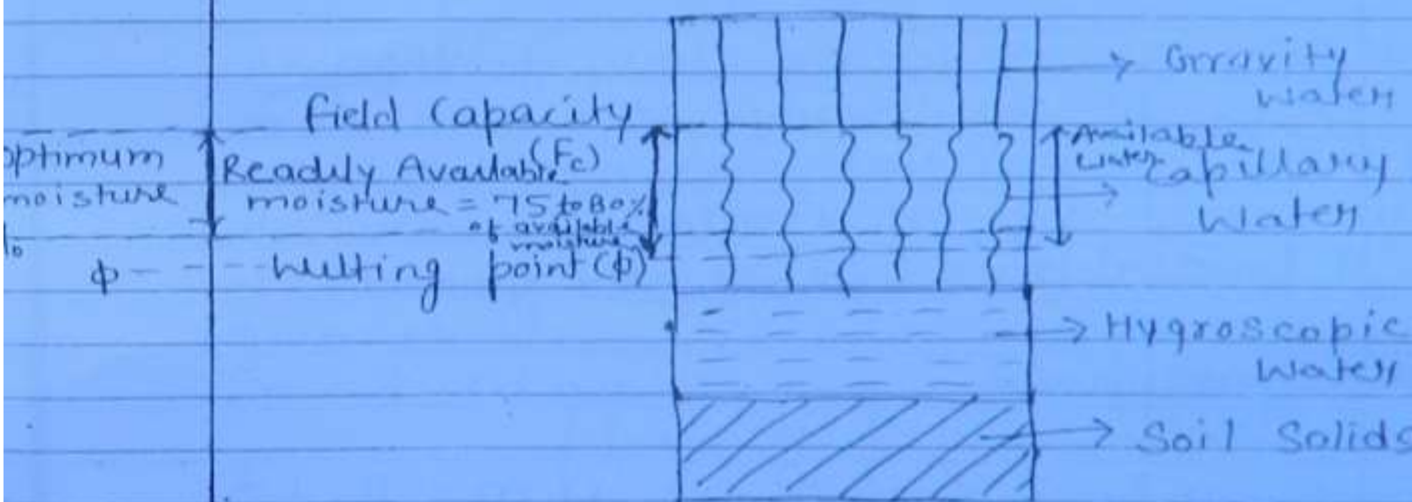
Capillary Water :-

This is the water between hygroscopic and field capacity and is held in position due to the Intermolecular attractio

Out of this water entire amount cannot be extracted by plants and therefore the water which is removed by plants is called available moisture and after extracting available moisture, the moisture content reached is called wilting point.
(wilting → dry (सूखना))

121

If water content falls below wilting point plants cannot survive and it will wilt up or dry up.



Hygroscopic water :-
When an oven dried sample is kept in free atmosphere it absorbs some moisture from the surrounding which is called hygroscopic water.
This is max^m for clay and Min^m for sand.

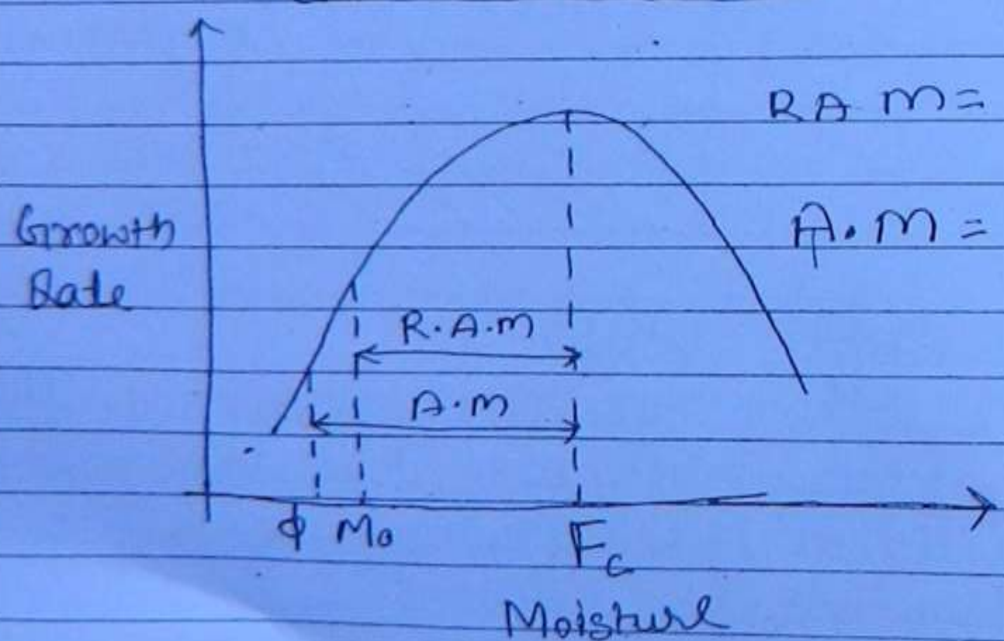
Chemically bound water :- Structural water that can not be extracted.

Field Capacity :- This is the moisture content of the soil after free drainage has taken place for 5 to 15 days and (2 to 5 days) and no more water can be extracted by the action of gravity. It includes hygroscopic and capillary water.

It is expressed in percentage and it is given by the weight of water retained in certain vol. of soil to the weight of ~~tot~~ that vol. of vol.

Field Capacity $= F_c = \dfrac{W_w}{W_{soil}}$ $\qquad \omega = \dfrac{W_w}{W_s}$

$$\boxed{F_c = \frac{W_w}{W_{soil}} \times 100}$$



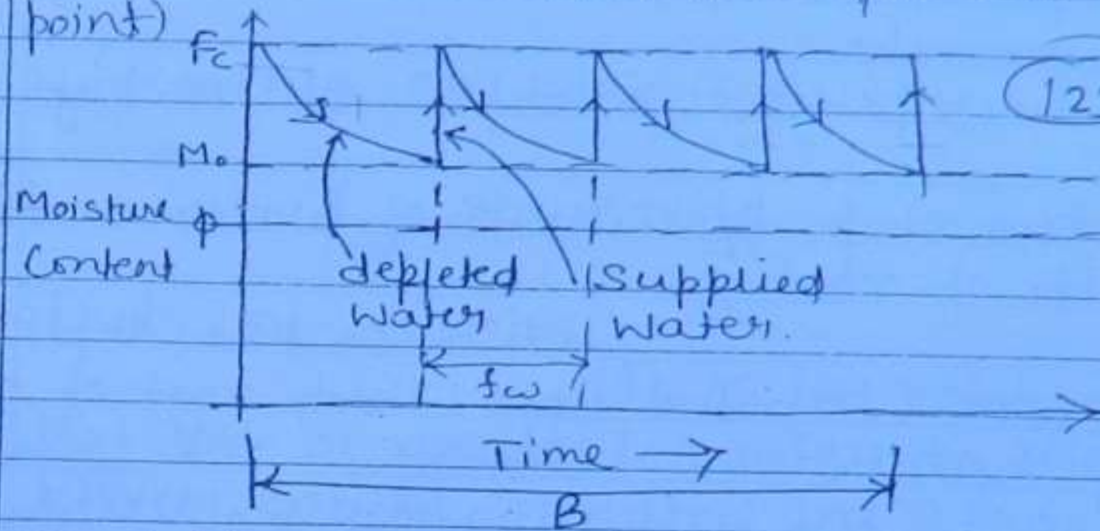
$RAM = F_c - M_o$

$A.M = F_c - \phi$

RAM = Readily available moisture

AM = Available moisture

NOTE:- For optimum growth of plant the moisture content is allow to fall only upto M_o (optimum moisture content) and not upto ~~said~~ ϕ (wilting point)



If depth of water stored in root zone is d_w (which is b/w optimum moisture and field capacity) and C_u is the rate of consumptive use (Evapotranspiration Rate) then frequency of watering ~~and~~ or watering interval is given by

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

f_w = frequency of water

Crop period or base period :-

The time that elapses from the instant of sowing of a crop to the instant of harvesting is called the crop period.

The time interval between the first watering of a crop to its last watering is called base period of the crop.

NOTE :→

In Reality Crop period is slightly more than the base period however the two term (i.e base period & crop period) are used ~~synonimosly~~ synonimously.

The base period is Expressed in days

Relationship b/w depth of water and depth of root zone :-

Let d be the depth of root zone (in meter) upto which root of plant may extent.

Let f_c be the field capacity expressed in fraction and γ_w, γ be the unit wt. of water and soil respectively.

d_w is the depth of water stored in the root zone.

$$F_c = \frac{W_w}{W_s} = \frac{\gamma_w \times Vol}{\gamma_{soil} \times Vol} = \frac{\gamma_w \times A \times d_w}{\gamma \times A \times d}$$

$$F_c = \frac{\gamma_w d_w}{\gamma d}$$

$$\boxed{d_w = \frac{\gamma}{\gamma_w} d F_c}$$

Available depth of water/moisture $= \frac{\gamma}{\gamma_w} d (F_c - \phi)$

depth of water that can't be extracted $= \frac{\gamma}{\gamma_w} d \phi$

Date:

Readily Available moisture depth :-

$$\boxed{d_w = \frac{\gamma}{\gamma_w} d (F_c - M_o)}$$ (125)

Frequency of watering $f_w = \frac{d_w}{C_u}$

$\frac{\gamma}{\gamma_w}$ = sp. gravity.

Duty of Crops :-

It is the relationship b/w Vol. of available water to the area which can be Irrigated for a particular crop.

The Area which can be Irrigated by 1 cumecs of discharge when it is available throughout the base period is called the duty of crop and it is indicated by D (hecto/cumecs)

Delta (Δ) :-

This is the total depth of water required by the crop during the entire base period and it is represented as Δ.

Relation b/w duty & delta :-

If 1 cumecs of discharge is available for the entire base period (B days) then total Vol. of water supplied will be

$$Vol. = 1 \, m^3/sec \times B \times 24 \times 60 \times 60$$

Volume = $D \times 10^4 \times \Delta$

$1 \frac{m^3}{sec} \times B \times 24 \times 60 \times 60 = D \times 10^4 \times \Delta$

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

Where

Δ in meter

B in days

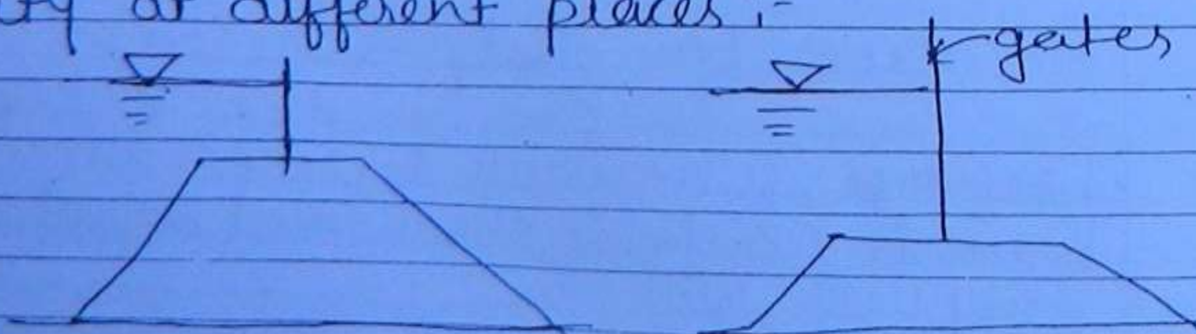
D in ~~~~ hectare/cumecs.

$$\boxed{\Delta = 2 \frac{B}{D}}$$ in FPS

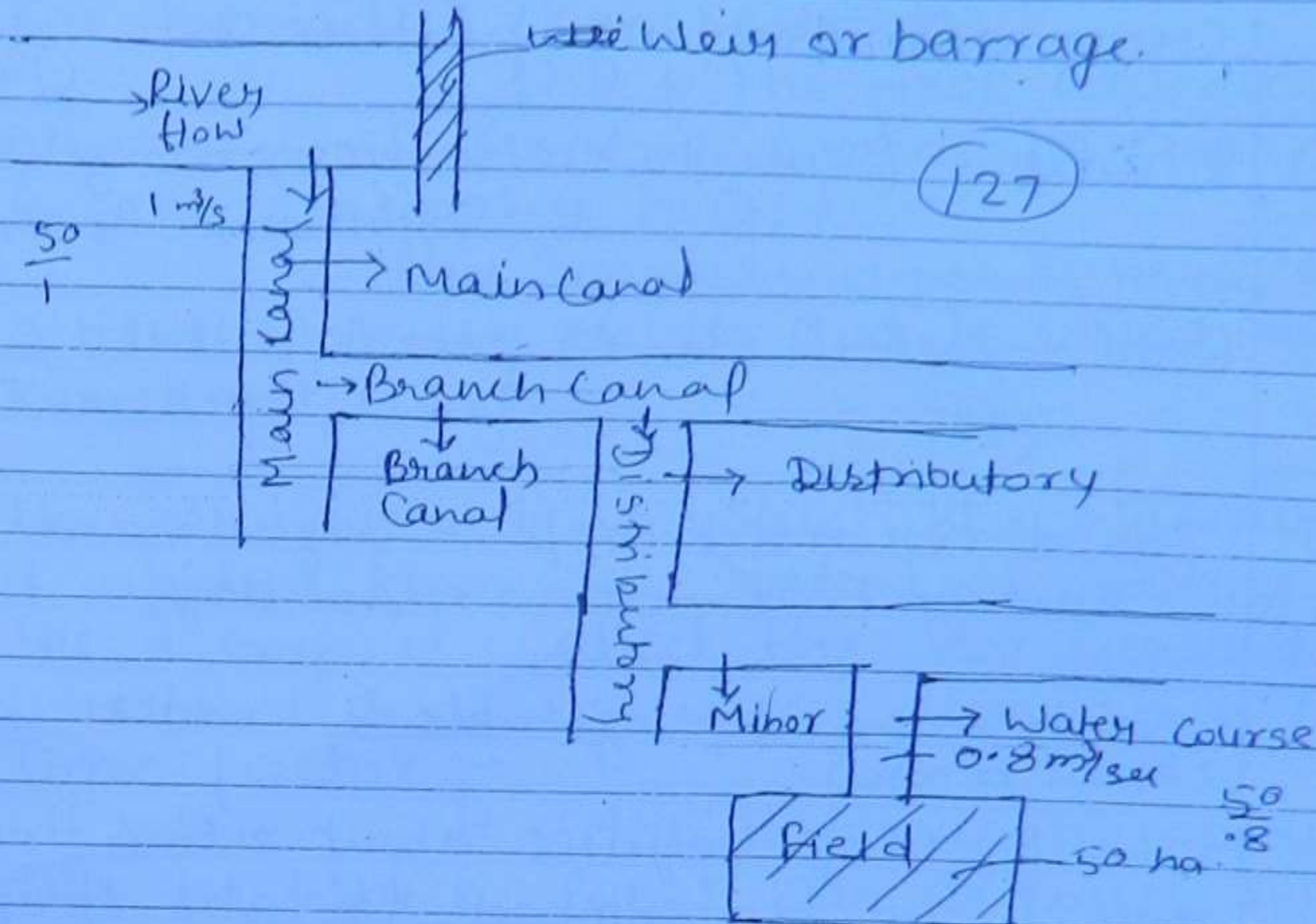
Factors affecting duty :-

1 Type of Crop
2 Base period of Crop
3 Method and System of Irrigation
4 Type of Soil
5 Time and frequency of Irrigation.
6 Climatic Condition of Area   wet area duty ↓
7 Nature of ground
8 Quality of Water.

Duty at different places :-



weir   barrage



As Water moves from main canal to the Water course its duty of Various places, duty Increase

Duty at the head of Water Course is Called outlet factor.

Duty at the head of field is called Net quantity.

→ Duty at the head of branch canal is called lateral quantity.

→ Duty at the head of main Canal is called gross quantity.

Crops and Cropping pattern :-

(1) Kharif :- (June to Oct) Rice, maize, Cotton Soyabeen, Jhwar, bazra.

(2) Rabi :- (Oct to march) Wheat, pattato, pulses, gram.

(3) ~~Zaid~~ Perineal Crop :- Sugar Cane
Base period - 300 to 360 days.

(4) Zaid :- fruit, vegitable & fodder
(Aprail to June)

Kharif Crop require about 2 to 3 times more water as compair to rabi Crop

Crop Ratio :-
It is the ratio of proposed area to be Irrigated in Kharif Season to that in Rabi season is called crop ratio.
This ratio is generally 1:2

Paleo Irrigation :-
When october month is dry and soil is deficient in moisture then Irrigation is done to make the soil fit for Crops.
This watering which is done just before Sowing the Crop is called paleo Irrigation

Date:

Kor period & Kor Watering :-

This is the first watering after sowing the crop when the crop is few centimeter height.

This Irrigation depth is maxm of all the watering depth and is called the Kor depth.

(129)

The period during which Kor watering is applied from the instant of sowing the crop is called the Kor period

Time factor :-

It is the Ratio of number of days in which canal has actually run to the number of days of Estimated Irrigation period.

Transpiration Ratio :-

This is the ratio of weight of dry matter produce to the weight of water used by the plants exclusive of roots.

The Average values of transpiration ratio for wheat and rice are 560 and 680 respectively.

Root zone depth (d) :-

This is the depth of soil strata in which the plant spereds its root system and derives water from the soil.

Irrigation Efficiencies :→

1 Water Conveyance Efficency :- (η_c) :- This Efficiency take into account the conveyance ~~and~~ or transit losses such as seepage and Evaporation ~~to~~ through the canal. This can be represented as

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

Where,

W_f = Water delivered to the field
W_r = Water diverted from river or Reservoir

2 Water application Efficiency (η_a) :- If Irrigation technique is not good then water may be lost in surface runoff and depercolation. This loss is taken into account while calculating water application efficiency. This is also called as farm efficiency.

$$\eta_a = \frac{W_s}{W_f} \times 100$$

Where,

W_s = Water stored in the root zone
W_f = Water delivered to the field.

Water Used Efficiency (η_u) :- This is the ratio of water used benificealy ~~and~~ or consumptively to the water delivered to the field

$$\eta_u = \frac{W_u}{W_f} \times 100$$

(131)

Where,

W_u = Water used beneficialy

W_f = Water divert to field

Water storage Efficiency (η_s) :- This is the ratio of actual water stored in the root zone to the water needed to the store to bring the moisture content upto field capacity.

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

Where,

W_s = Water stored

W_n = Water needed

Water distribution Efficiency (η_d) :- This efficiency takes into accout the degree to which water is uiformly distributed in the root zone through out the field area.

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

Where,

d = Avg. depth

y = Avg. numerical deviation in the depth of water stored from the Avg. depth. Stored during Irrigation.

6. Consumptive use Efficiency :- (η_{cu})
This is the ratio of water that is use consumptively to the net ~~amou~~ amount of water depleted from ~~Root~~ Root zone.

$$\eta_{cu} = \frac{W_{cu}}{W_{depleted}} \times 100$$

(132)

Q. A stream of 130 ltr/sec was delivered from a Canal and 100 ltr/sec water reach the field which was applied on an area of 1.6 hectare in 8 hours.
The effective depth of root zone is 1.7 m. The runoff loss in the field is 420 m³. The depth of water penetration varies linearly from 1.7 m at the head of field to 1.1 m at the tail end of the field. Available moisture holding capacity of soil is 20 cm/m depth of soil.
Find water conveyance (η_c), η_a, η_s, η_d.
Assume Irrigation water stored at a moisture extraction level of 50% of the available moisture

Sol

$Q_{stream} = 130$ ltr/sec

$Q_{canal} = 100$ ltr/sec

(i)

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

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

ii) $\eta_a = \frac{W_s}{W_f} \times 100$

$= \cancel{\frac{420}{100 \times 8 \times 60 \times 60}}$ 133

$W_f \Rightarrow 100\ l/s = .100 \times 8 \times 60 \times 60 = 2880\ m^3$

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

$$\eta_a = \frac{2460}{2880} \times 100$$

$$= 85.41\ \%$$

iii) $\eta_s = \frac{W_s}{W_n} \times 100$

$W_s = 2460\ m^3$

$20\ cm/m \Rightarrow 1.7\ m \times 20\ \frac{cm}{m} = 34\ cm$

50% of $34\ cm = 17\ cm$

$$= 1.6 \times 10^4 \times 17 \times 10^{-2}$$

$W_n = 2720\ m^3$

$$\eta_s = \frac{2460}{2720} \times 100 = 90.44\ \%$$

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

If ... are given then take Avg depth

$d = \frac{1.7 + 1.1}{2} = 1.4$ $\quad \eta_d = (1 - \ldots$

$= 78\ldots$

$y = 0.3 + 0.3 = 0.3$

Irrigation requirement of Crop:-

Effective Rainfall :-

It is that part of precipitation Which falls during the growing period of Crops that is It is the Rainfall Which is available to meet the Evapotranspiration needs of the Crop

134

Consumptive Irrigation requirement

This is Equal to total Irrigation Water required for consumptive use ~~sp~~ minus Effective (usefull) rainfall

$$CIR = \begin{pmatrix} \text{Total Irrigation Water} \\ \text{for Consumptive use} \end{pmatrix} - P_{eff}$$

$$CIR = C_u - P_{eff}$$

Net Irrigation requirment :-

This is the total amount of water required to field to meet the need of Consumptive use as well as leaching.

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

Field Irrigation requirment :-

This takes into account losses occuring in the field.

$$FIR = \frac{NIR}{\eta}$$

Gross Irrigation requirement :-

This takes into account the Conveyance or transit losses and it is given as.

$$GIR = \frac{FIR}{\eta_c}$$ 135

Consumptive Use of Water :-

This is the depth of water consumed by Evaporation and transpiration during the growth of plants.

→ This Includes Water Consumed for the growth of weeds (paracides).

→ Water deposited by ~~due~~ dew or rainfall on leaves of plants also gets Evaporated and hence this is also a part of consumptive use.

Factors Effecting Consumptive Use :-

1. Humidity & Climatic Condition
2. Mean monthly temperature
3. Crop period
4. Rainfall during Crop period
5. Irrigation depth & supply of Water
6. Soil and its topography
7. Irrigation technique
8. Wind velocity & locality

Methods to find Consumptive:-

Direct Method | Indirect Method

Direct Method

1) Tank Lysimeter
2) Field Experimental Plots
3) Inflow outflow studies
4) Soil moisture method

} field or Lab Methods

Indirect Method

1) Blaney Criddle method
2) Hargreave's class A pan method
3) Penman's method.

} Emperical Equational Method

Tank Lysimeter:-

Tanks are cylindrical Container with an area of $10 m^2$ and a depth of 3m in which soil is filled and plants are grown.

This tank rests on perforated base plate. Water is applied to this ~~pan~~ tank through the metering system.

The difference in the total amount of water applied and the water that comes out of the perforated base plate gives the consumptive use of water.

OTE Phytometer is a closed glass chamber is used to find the transpiration of water.

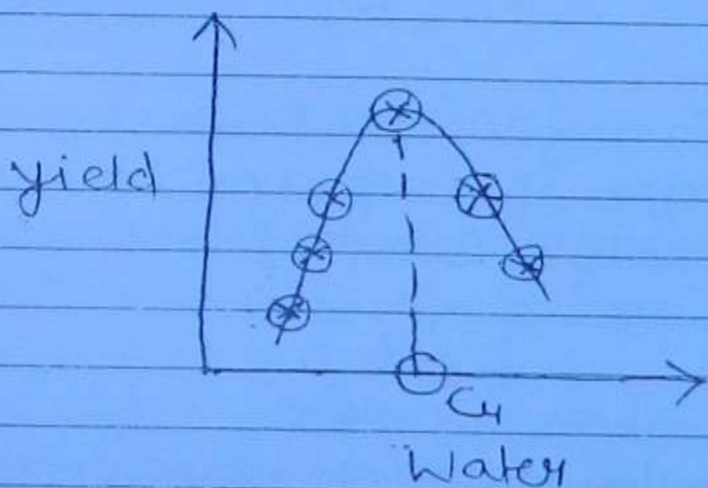
Field Experimental plots :-

In this method Irrigation water is applied to selected field plots in such a way that there is neither deep percolation nor surface runoff.

137

→ Yield obtain from different crops is plotted graphically against the total water used and that water is selected which gives the max^m yield.

This corresponds to the Consumptive use of water



Inflow outflow studies :-

This is an Approximat method which is used for large areas to find the Anual consumptive use.

Let U be the Consumptive Use per annum then U is given as

$$\boxed{U = I + P + (G_s - G_e) - R}$$

Where,

I = Total Annual Inflow

P = Annual precipitation

G_s = Total Ground storage at the starting of an year

G_e = Total Ground storage at the End of a year.

R = Annual out flow which Includes Surface and Subsurface Outflow.

Indirect Method :-

Blaney Criddle method :-

The monthly Consumptive use is given as

$$C_u = Kf$$

Where,

K = Crop factor which depends on type of Crop and Season.

f = Monthly Consumptive use factor and is given as

$$f = t \frac{p}{100}$$

Where,

t = Mean temperature of month in degree farhenhite

p = percentage of the day time hour of the year occuring during that period

C_u Unit = Inches/month

$$C_u = K \times \frac{p}{40}(1.8t + 32) \rightarrow \text{In M.K.S}$$

cm/month

$$\frac{C}{5} = \frac{F-32}{9} = \frac{R}{4}.$$

$$\frac{\text{Reading} - LFP}{UFP - LFP} = \text{Const}$$

$$\frac{C-0}{100-0} = \frac{F-32}{212-32}$$

LFP → Lower fixed point.
UFP → Upper fixed point.

This method does not account for humidity wind velocity and elevation from mean sea level.

Hargreave's Class A pan Evapotranspiration Method :—
Class A pan is a standard pan having a diameter of 1.2 m and a depth of 25 cm.

→ The water depth maintain in this pan during evapotranspiration is b/w 5 - 7.5 cm.

$$C_u = K \times \text{pan evapotranspiration}$$

Where;
K = Consumptive use coefficient and depends on type of crop.
K = 0.9 for Wheat
K = 0.8 to 1.3 for Rice.

Q. The base period, Intensity of Irrigation, duty of water under a canal section is given. find the reservoir and capacity and canal capacity. If CCA (cultarable command area) is 40000 ha. assume canal losses as 25% and reservoir loss is 15%.

GCA :- Gross Command Area :
This is the total Area which is under the perview or control of an Irrigation project. This includes culturable as well as non culturable area. for Eg:- ponds lakes, roads, residential complex Etc.

CCA (Culturable Command Area) :-
(Cultivable Command Area)
This is that part of GCA where actual Cultivation can be done.

Intensity of Irrigation :-
The percentage of CCA Proposed to be Irrigated in a given Season is called the Intensity of Irrigation of that Season.

	Crop	Base period (days)	Duty ha/cumecs	Intensity (%) of Irrigation
Rabi	Wheat	120	1800	20
	Sugar Cane	360	1700	20
Kharif	Rice	120	800	15
	Cotton	180	1400	10
	Vegetable	120	700	15

$$\Delta' = 8.64 \frac{B}{D} \qquad \Delta \times \text{Area}$$

$$\Delta_1 = 0.576 \quad \Rightarrow$$

$$\Delta_2 = 1.829 \qquad 141$$

$$\Delta_3 = 1.1296$$

$$\Delta_4 = 1.1108$$

$$\Delta_5 = 1.481 \quad \Rightarrow \quad 59245$$

Δ Vol. = 40308 ha-m

$$\text{Actual Vol.} = \frac{40308}{0.75 \times 0.85} = 63228.2 \text{ ha-m}$$

(ii) Canal Capacity :-

$$Q = \frac{\text{Area}}{\text{duty}} = \frac{A}{D}$$

$Q_{rabi} = 9.145 \, m^3/s + 8.57 = 17.7 \, m^3/s$ (for Economical design)

$Q_{kharif} = 15.06 \, m^3/s + 8.57 = 23.63 \, m^3/s$

$Q_{veg.} = 8.57 \, m^3/s$ (for Conservative design)

$\Delta = 8.64 \frac{B}{D}$	Area = I.I × CCA	Vol. of water = $\Delta \times A$ (ha.m)	$Q = \frac{A}{D}$
0.57	8000	4608	4.44
1.829	8000	14640	4.705
1.296	6000	7740	807.5
1.110	4000	4440	12.86
1.481	6000	8880	8.57

Σ = 40308 ha-m

$$Q_{act} = \frac{17.7}{0.75} \;,\; \frac{23.6}{0.75} \; m^3/sec$$

$$Q_{act} = \frac{17.7}{0.75} = 23.6 \; m^3/sec \text{ for Economic design}$$

$$Q_{act} = \frac{23.6}{0.75} = 31.46 \; m^3/sec \text{ for Safety design}$$

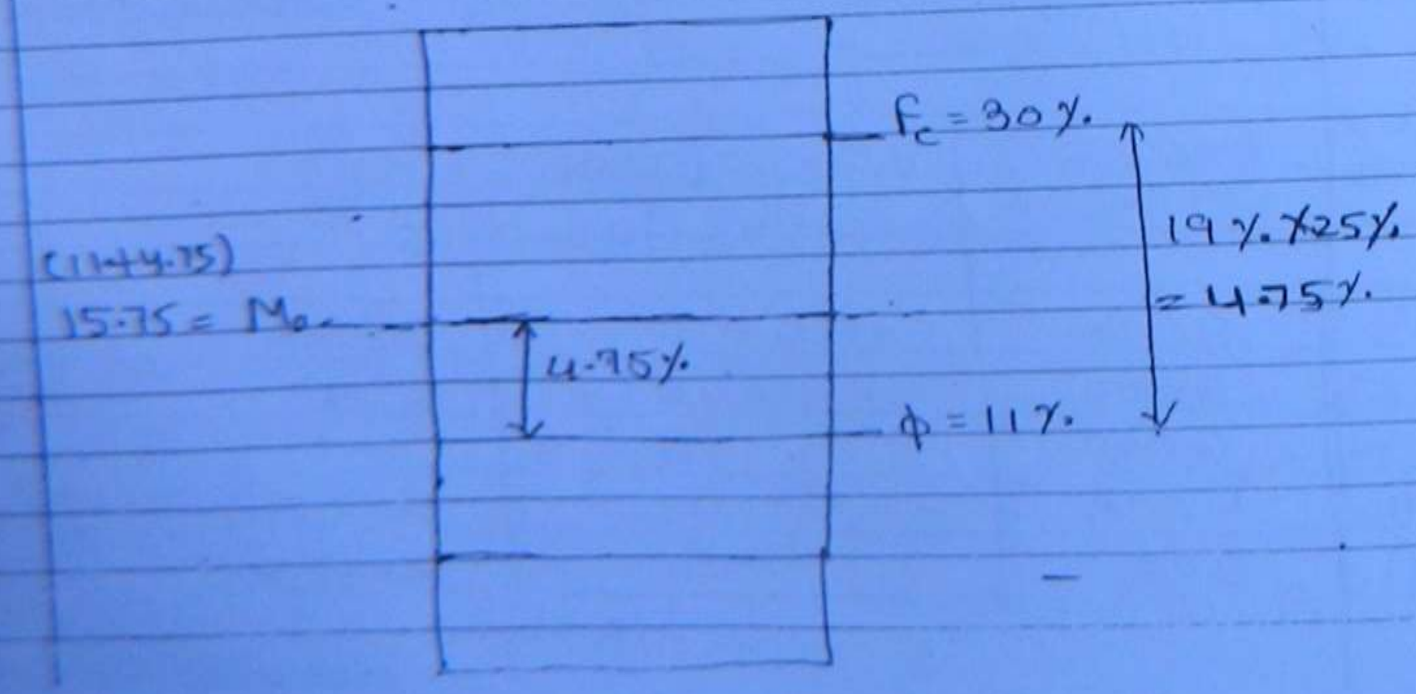
Conservative

Q. The following data is related to healthy growth of Crop.

142

Field Capacity of Soil = 30%
Permanent wilting point = 11%
density of soil = 1300 kg/m³
Effective depth of root zone = 700 mm
daily Consumptive use of water = 12 mm
For healthy growth moisture content must not fall below 25% of water holding capacity b/w field Capacity and permanent wilting point. Find the watering Interval of Crop in days.

Sol



$$d_w = \frac{\gamma}{\gamma_w} d (F_c - M_o)$$

$$= \frac{1300}{1000} \% \qquad = 129.67 \text{ m}$$

$$f_w = \frac{129.67}{12 \text{ mm/day}} = 10.8 \text{ days}$$

$$= 10 \text{ days}$$

Q A certain crop is grown in a area of 3000 ha. which is fead by a canal system. The data pertaining to Irrigatation are as
Field capacity of soil = 26%
optimum moisture is = 12%
Permanent wilting point = 10%
Effective depth of root zone = 80 cm
Sp. gr. of soil is = 1.4
If the frequency of Irrigation is 10 days and over all Irrigation Efficiency is 22%. then find
i) daily consumptive use
ii) discharge require in canal Feeding the area

$$d_w = \frac{\gamma}{\gamma_w} d (F_c - M_o)$$

$$d_w = 1.4 \times 80 (.26 - .12)$$

$$d_w = 15.68$$

$$f_w = \frac{d_w}{C_u}$$

$$C_u = \frac{d_w}{f_w} = \frac{15.68}{10} = 1.568 \text{ cm/day}$$

ii) $d_w = 0.1568\ m$

$$V = d_w \times A = \left(\frac{0.1568 \times 3000 \times 10^4}{0.22 \times 10 \times 24 \times 60 \times 60}\right) m^3$$

144

$$= 24.7\ m^3/sec.$$

Imp Q.

Q. 800 m^3 of water is applied to a farmers rice field whose area is 0.6 hectare. When the moisture content in the soil falls to the 40% of available water b/w field capacity (36%) and permanent wilting point (15%) find the field application efficiency if the root zone depth is 60 cm. Assume porosity of field as 0.4.

Sol)

$$\eta_a = \frac{W_s}{W_f} \times 100$$

$W_f = 800\ m^3$

W_s = Area of field $\times d_w$

$$d_w = \frac{\gamma}{\gamma_w} d\,(f_c - M_o)$$

$M_o = 23.4\%$ $\left((36-15) \times .40 = 8.4 \right.$

$15 + 8.4 = 23.4$

$$d_w = \frac{\gamma}{\gamma_w} \times 60\,(36 - 23.4)$$

$$F_c = \frac{W_w}{W_s} = \frac{\gamma_w \times V_w}{\gamma \times V}$$

-

at field capacity $V_w = V_v$

$$F_c = \frac{\gamma_w}{\gamma} \times \left(\frac{V_v}{V}\right)$$

$$\frac{\gamma}{\gamma_w} = \frac{n}{f_c} = 1.11$$

$$0.6 \times (.36 - .234) \times$$

$d_w = .084$

$= 504\ m^3$

$$n_a = 63\% \Rightarrow \frac{504}{800} \times 100$$

Water logging :

An Area is said to be water logged if its productivity of the land gets affected by the high water table.
This happen bcoz the root zone is flooded and is consequently becomes

Causes / Factors of water logging

1 Over Irrigation and Intensive Irrigation

2. Seepage from Canal

3 Inadequate natural drainage system

4 Excessive Rain

5 Submergence due to flood

6 Irregular topography.

Control Measures of water logging :

1 Lining of canal

2 Reduction in Intensity of Irrigation

3 Development of underground drainage system

4 Introduction of lift Irrigation

5 By Crop rotation.

Effect of Water logging :-

1. Salinity and alkalinity of the Soil which may lead to decline in productivity.

2. Out break of Epidemics like like malaria dengue Etc.

Reclamanation of Saline & ~~B~~alkaline Soil

→ Reclamanation is process by which an uncultrable land is made fit for cultivation. This is done by the ~~#~~ process of leaching.

→ In this process ground is flooded by water and it is stirred mechanically and then the water is drained out. There after Salt resistant Crop Such as Jhwar, bazra Fodder (चारा) in rotation.

NOTE:- In Every Crop rotation gram s/d be present bcoz it is nitrogen fixing crop (liguminoous crop).

eg :- Maize, Rice, gram, Wheat

→ The Salts responsible for Salinity are

$$Na^{+} \begin{cases} \rightarrow Cl^{-} \\ \rightarrow SO_4^{2-} \end{cases} \text{White Alkalinity}$$

$$Na^{+} \rightarrow CO_3^{2-} \; \} \text{ Black Alkalinity}$$

→ Among all these salts Sodium chloride ($NaCl$) is least harmful and Sodium Carbonate (Na_2CO_3) (Black alkali) is most harmful & it is also less soluble and hence is difficult to remove by leaching In order to remove it gypsum is added and then mechanical stirring is done.

148

Gypsum → $CaSO_4$

$$CaSO_4 + Na_2CO_3 \rightarrow \underbrace{Na_2SO_4}_{\text{more Soluble.}}$$

Na_2SO_4 is more soluble & can be easily removed.

→ The process of salts coming up in the solution and forming the thin layer of 5 - 7.5 cm on the surface of soil after Evaporation is called Efflorescence.

If this process continious for longer period in clayey soil it makes the soil Impermeable or unprotective and such soils are called

Alkaline Soil, usher or banzar.

The depth of water required for the process of bleaching is Expressed leaching requirment which is given as depth of water drained out per unit area to the depth of water ~~required~~ applied for Irrigation per unit area.

$$L \cdot R = \frac{D_d}{D_i}$$

$$D_i = C_u + D_d$$

$D_i \rightarrow$ NIR, $C_u \rightarrow$ CIR, $D_d \rightarrow$ leaching water

leaching requirement Can also be Expressed in terms of Salt Concentration

$$L.R = \frac{EC_i}{EC_d} = \frac{C_i}{C_d}$$

EC_i = Electrical Conductivity of Irrigation water

$$\boxed{EC_d = 2\,(EC_e)}$$

$EC_e \rightarrow$ Electrical Conductivity of a Soil water Extract of Soil Solution

Design of land drainage System :-

Drainage System may be made up of two parts

Surface or open drainage

Subsurface drainage

In Surface drainage System open ditches running along the land Slope are provided. These are normally trapezoidal in shape.

Subsurface or Tile drainage System :-

These drainage System are required for Soils having poor Internal drainage and having high water table Condition. These remove water through the action of gravity and improves the air circulation which helps in removing Toxic Element.

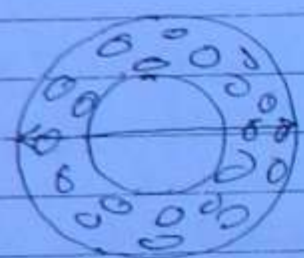
If soil is less pervious then these tile are surrounded by graded gravel filter which are called Envelop filter.

Envelop filter serves the following purpose:

1. It prevents the Inflow of soil into drain.

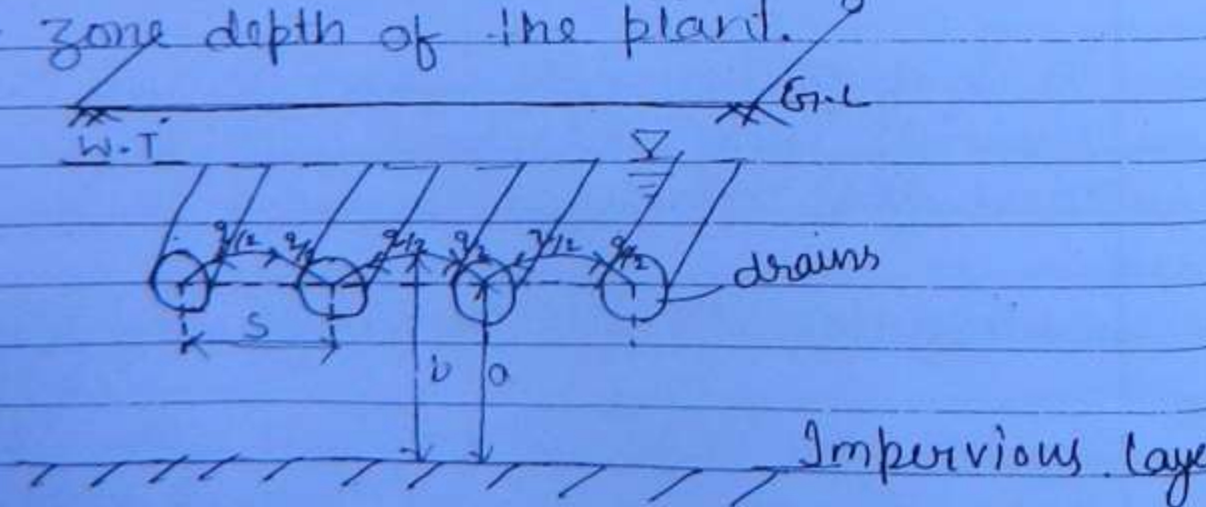
2. It Increases the Effective tile diameter and hence Increase the Inflow rate.

150



Depth and Spacing of tile drains :->

Tile drains are placed in such a manner that they are able to lower the Water table Sufficiently below the root zone depth of the plant.



S = Centre to Centre spacing b/w drains
a = depth of Impervious stratum from the centre of drain.
b = Max^m height of drained water table above the Impervious layer

then the Value of S is given,

$$S = \frac{4K}{q}(b^2 - a^2) \quad (157)$$

Where,
K = Coeff. of permeability
q = discharge per unit length of tile drains.

q/2 discharge Enters the drain from Either sides.

Value of q mainly depends on Infilteration of the ground and generally a value of 1% of Avg. anual rainfal at a place is Considered to be removed by tile drains in 24 hours.
This is called drainage coefficient (the rate at which water is removed by the drain).

$$q = \frac{0.01\bar{p}}{24 \times 60 \times 60} \times (S \times 1) \qquad \frac{cm}{s} \times cm^2$$

$\bar{p}$ = Avg annual rainfall.
(S×1) = Area ~~cm~~ cm²

The diameter of tile drain is design according to the mannings formula to carry the discharge decided by the drainage coeff of that area. (152)

The drains are laid on a certain longitudinal slope which varies b/w 0.05% to 3% However a desirable min^m working grade is 0.2%

Q. In a tile drainage system the drains are laid with there centre 1.5m below the G.L. The impervious layer is 9m below the ground and the avg. annual rainfall in the area is 80 cm. If 1% of annual rainfall is to be drained in 24 hours to keep the heighest position of water table 1m below the ground level find the spacing of drains. $K = 1 \times 10^{-3}$ cm/sec

Solution

$$a = 9 - 1.5 = 7.5m$$
$$b = 8 \ m$$
$$\bar{p} = 80 \ cm$$
$$K = 1 \times 10^{-3} \ cm/sec$$

$$q = \frac{0.01 \times 80 \times 10^{-2} \times (S \times 1)}{24 \times 60 \times 60}$$

$$S = \frac{4K}{q}(b^2 - a^2)$$

$$= \frac{4 \times 1 \times 10^{-3} \times 1}{}$$

153

$= 57.86\,m$

(ii) Find the size of the outlet of a 6 ha. drainage system. If the drainage coeff. is 1cm and tile grade is 0.3% value of mannings n is .011

NOTE ~~So~~ drainage coefficient of 1cm means that 1cm depth of water is to be drained out from an area of 6 ha. and is entering the tile drains in 24 hours)

Sol

$$Q = \frac{\frac{1}{100} \times 6 \times 10^{4}}{24 \times 60 \times 60} \, m^3/s = \frac{1}{n} A R^{2/3} S^{1/2}$$

$$A = \frac{\pi}{4} D^2$$

$$R = D/4$$

$$6.94 \times 10^{-3} = \frac{1}{.011} A R^{2/3} S^{1/2}$$

$$6.94 \times 10^{-3} = \frac{1}{.011} \times \left(\frac{\pi}{4} D^2\right) \times \left(\frac{D}{4}\right)^{2/3} (3 \times 10^{-3})$$

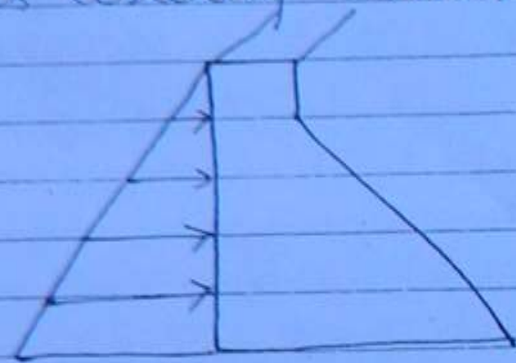
.0324

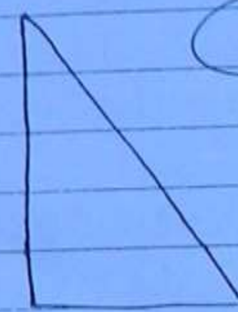
$$D^{2+2/3}$$

$D =$ .132m

The dam is a kind of obstruction or barrier which is constructed accross a river or stream in order to store water

The water which is stored on the upstream side is usually called the reservoir.





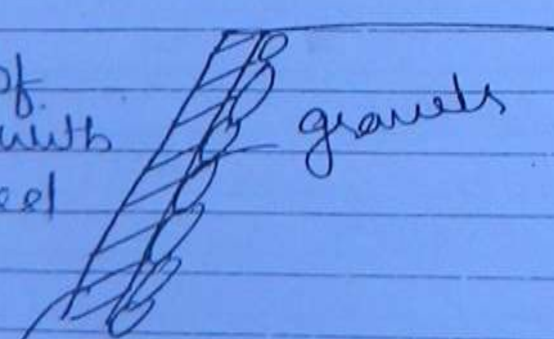
theoritical Shape

Types of Dams :-

Earthen / Earth dam :-
This dam is made up of soil which has been solidly compacted

Rockfull dams :-
These are made up of loose rocks and boulders which are piled up in the river bed.

The upstream face of the dam is provided with the slab of reinforced concrete



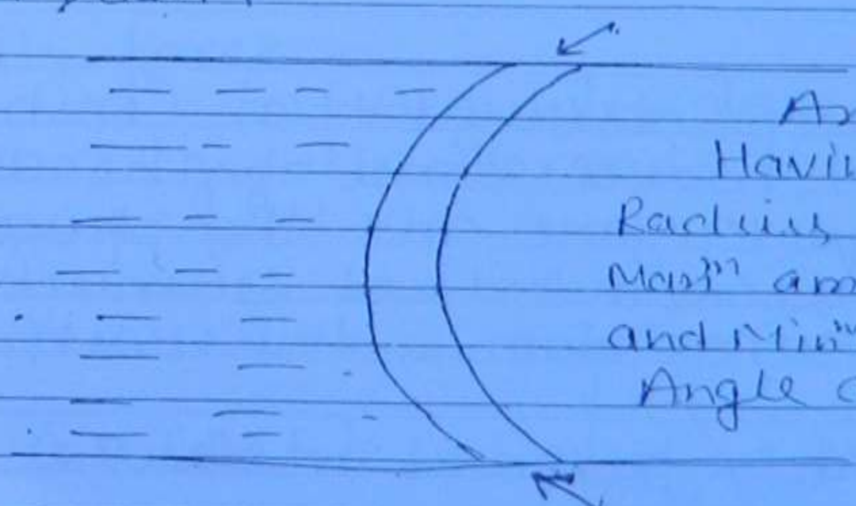
3 Solid masonary gravity dam.

4 Hollow masonary gravity dam :-
Its design is same as that of solid masonary gravity dam but concreter requirement is only above 40%.

5. STEEL dams
These are not used for major works and are primarly used ~~for~~ as an aid (help) in the construction of dam.
eg :- Coffer dam.

6 Timber dams :-
These are used for minor works and are extensively used by cultivators for meeting their water requirement.

7 Arch Dam :-



IES
Arch dam.
Having constant Radius and require Max^m amount of conc and Min^m in constant Angle arch ~~Angle~~ dam.

The design and construction of an arch dam is complex and complicated.
eg :- Idduki dam (in Kerela)

70-75% by thermal
1-2% Nuclear plant Wind solar

Rubber dam :-
This is the special type of dam which can be inflated or deflated according to the requirement.
eg:- Dam on Jhanjhavati in A.P (ht. 6m)

Gravity dam:- Grand Dixence → 284m ht. (in Switzerland)
Bhakra Nangal dam → 226m high

156

1) Topography of the region s/d be such that it s/d have deep valleys of low submergence area

2) Base of dam side where foundation is to be laid s/d be impervious.

3) Region s/d be free from Earth quake activity.

NOTE It has been observed that construction of dam makes some of the areas Earth quake zone and this phenomena is called Reservoir Induced Seismicity.
eg:- Koyna dam in Maharastra.

4) River s/d be perennial in nature.

5) Easy Availability of construction material & cheap labour.

Date:

In India there are 3 main regions of hydroelectric potential

1) Himalyan region

2) Central India (sathpura & Vindhya range)

3) Westren ghats (Sahyadri)

Forces acting on a gravity dam :-

1. Gravity force
2. Water pressure
3. uplift force
4. Earthquaake force
5. Silt pressure
6. Wave pressure
7. Ice pressure
8. Wind pr. or Wind force

Gravity force :-

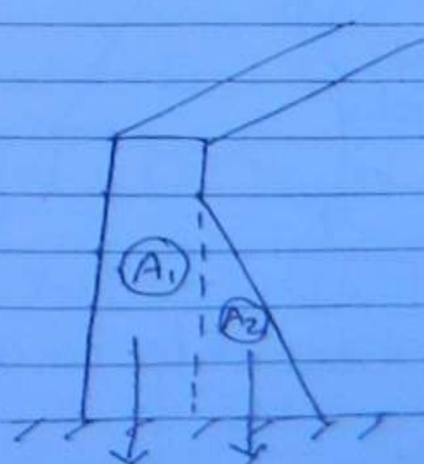
A gravity dam is called so bcoz gravity force is the major stablizing force.

$W = \gamma_{conc} \times Vol.$

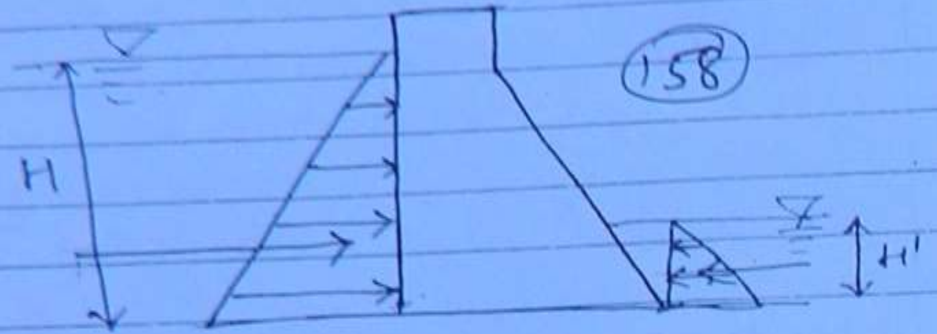
$= \gamma_c \times (A \times L)$

$= \gamma_c \times (A \times 1)$

$W = \gamma_c (A_1 + A_2 + --) \times 1$

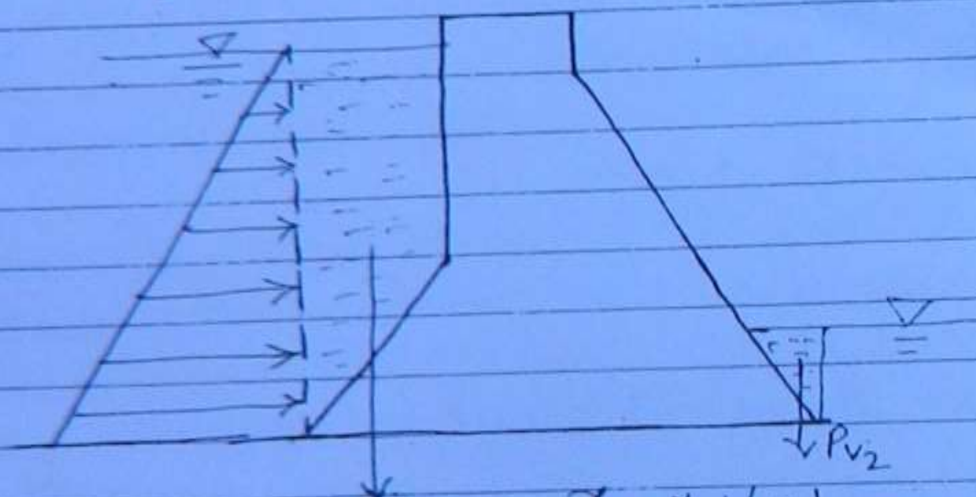


Water pressure :-

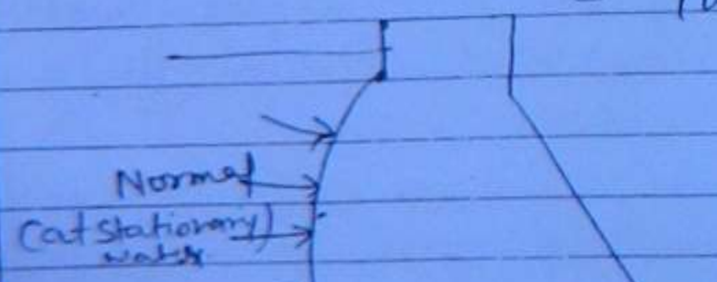


$P_{H_1} = \frac{1}{2} \gamma_w H^2$ @ $H/3$ from base

$P_{H_2} = \frac{1}{2} \gamma_w H'^2$ @ $H'/3$ from base



$$P_{V_1} = \gamma_w \times Vol$$
$$= \gamma_w \times (A \times 1)$$

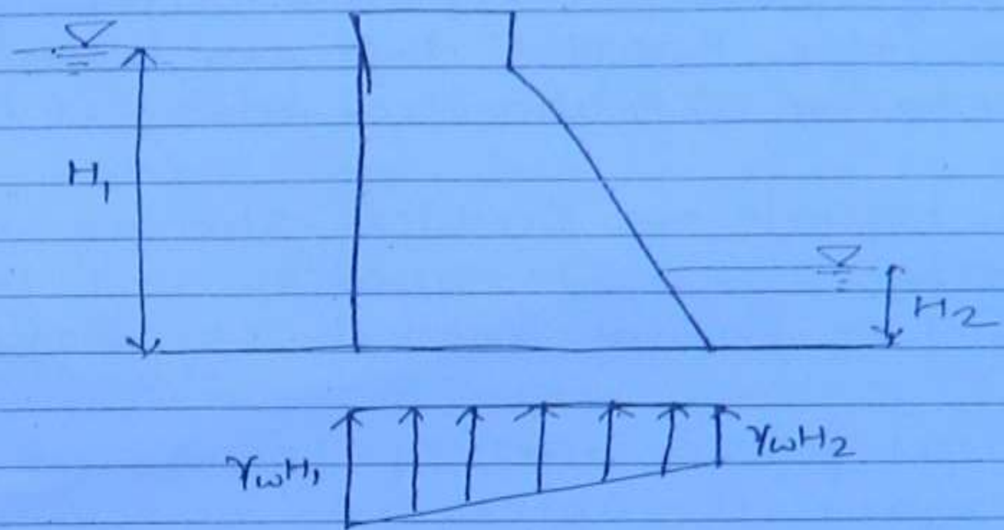


More desirable bcoz increasing the load.

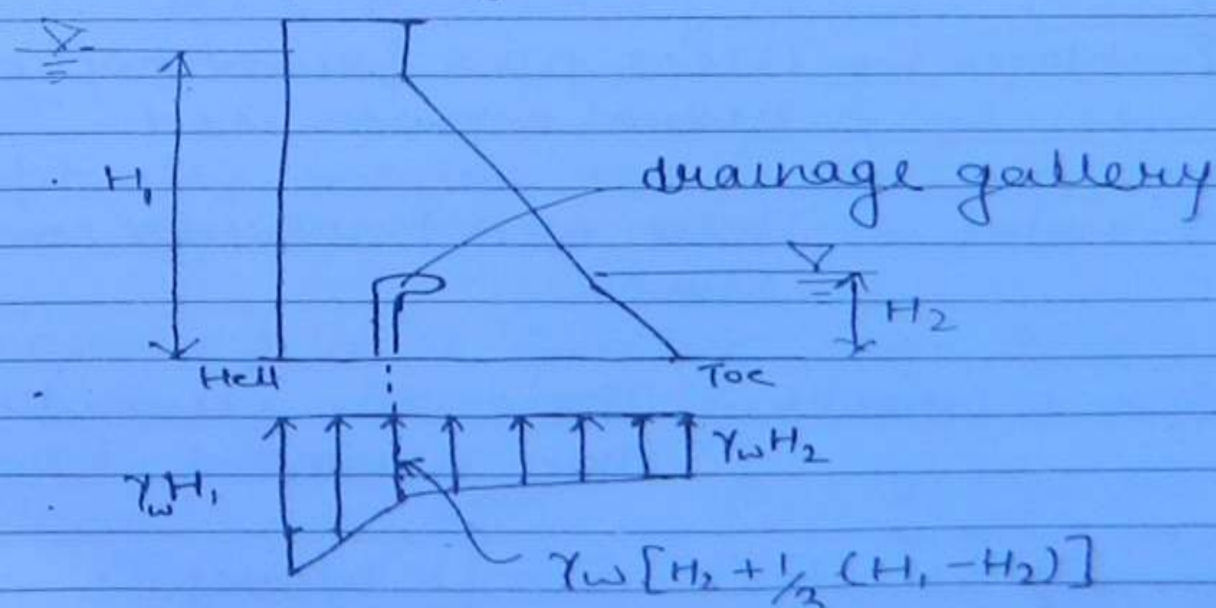
P_{v_1} and P_{v_2} are Equal to wt. of block of water and its act to the Centroid of the area.

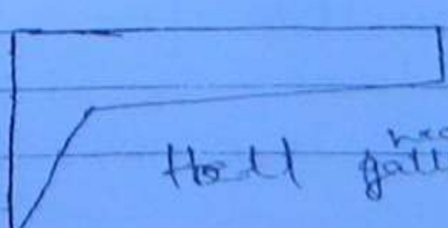
159

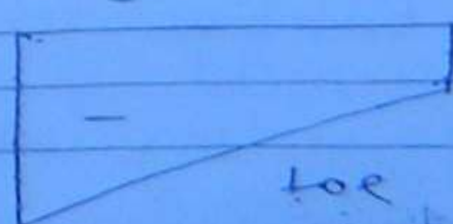
uplift force :



If there is a drainage gallery the uplift pressure diagram gets modified.







Imp
NOTE The pressure at the location of drainage gallary is Constant (depends only on H_1 & H_2)

Earthquake force :-
Earthquake takes place as a result of ~~certain~~ Sudden sliding of rocks in the Earths Interior due to plate tectonic activity.

The ~~plane~~ point of sudden sliding is Called focus of Earth quake and is generally lies in top 100 km of depth.

The vertical projection of focus on the Earth Surface is Epicentre.

From the focus 3 types of Waves are generated

P- Waves :- These are primary or longi-tudnal Waves.

Q- Waves :- These are transverse or Secondary Waves.

S or L Waves :- These are Shoked or love Wave.

Among all these S or L Waves is most destructive and is responsible for Vibration movement of Earth Surface. If it passes through big Water bodies it may produce Tsunami.

Date:

→ The Amplitude of P Waves is recorded in ~~log~~ log scale by a Seismograph & is commonly refered as magnitude of Earth quake in Richter Scale. (R.H)

161

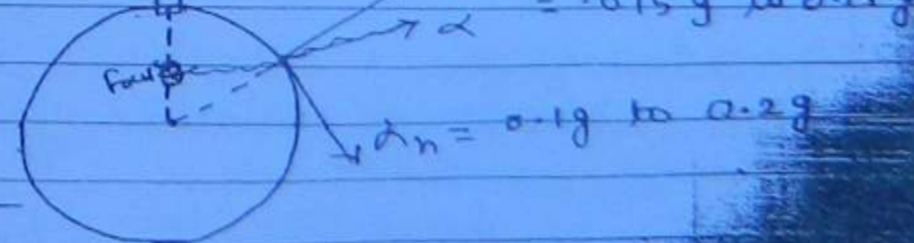
→ A New Scale has also been develop which is based on the degree of destruction and is called Mercalli's Scale.
in Sahra – 6-7 and in Japan – 2 (more population)

From geological point of view and occurance of Earthquake historically India has been divided into 5 zone. in which

Zone 1 → least serve
Zone 5 → Most destructive eg:- Himalya
7 Gujrat (Kacch), ~~North East~~
Zone 4 → Punjab, U.P, delhi

TE:- In 2001, BIS (Beauro of Indian standard) has revised the Codal provision in which zone 1st has merged in zone 2nd this Conceptually means that Earthquake proneness ~~of~~ of India has Increased.

Earthquake Waves can be taken into account by Considering the acceleration Imparted by Shock Waves.



α_h can be Expressed as

$$\alpha_h = Kg$$

Where,

$$K = (\beta I \alpha_0) g$$
$$= Kg$$

Where, K = Seismic coefficient
β = Soil foundation system factor and its value is 1 for gravity dam
I = Importance factor value is 2 for a gravity dam
α_0 = basic Seismic coefficient which depends on the zone in which the area is located

1893

α_0	Zone
0.01	I
0.02	II
0.04	III
0.05	IV
0.08	V

Seismic Coeff. = .15 (bhakra nangal)

1 Effect of horizontal Acceleration :- The horizontal Inertia force is Introduce in the body of dam whose magnitude given as

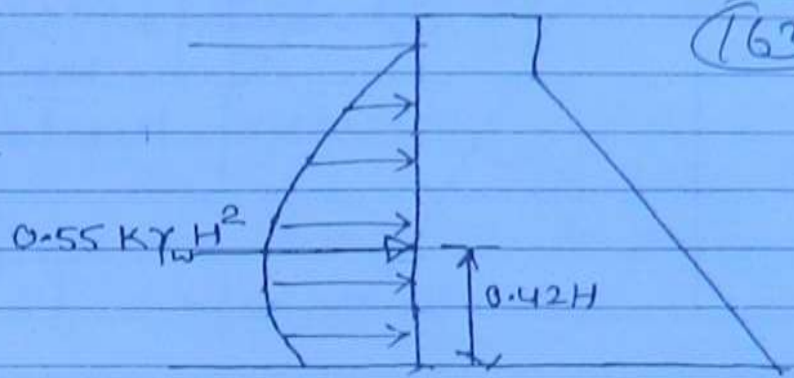
$$F_h = M \alpha_h$$

$$F_h = \frac{W}{g} \alpha_h$$

2 Hydrodynamic force Effect :-

It is the pressure force exerted by water on the dam.

$K \simeq \alpha_h$



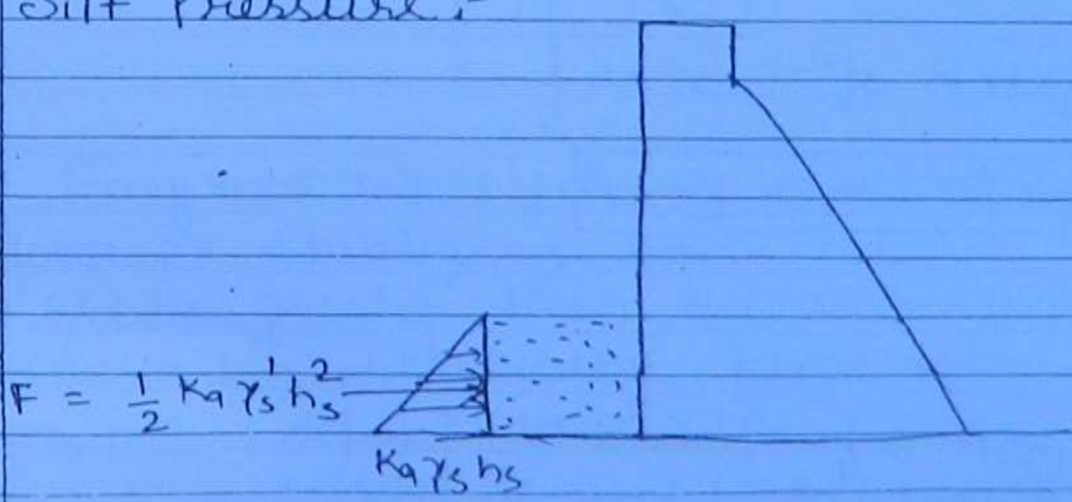
Effect of Vertical Acceleration :-

This Increase or decrease the Inertia force and the resultant gravity force under the action of g & α_v

$$F_R = \frac{W}{g}(g \pm \alpha_v)$$

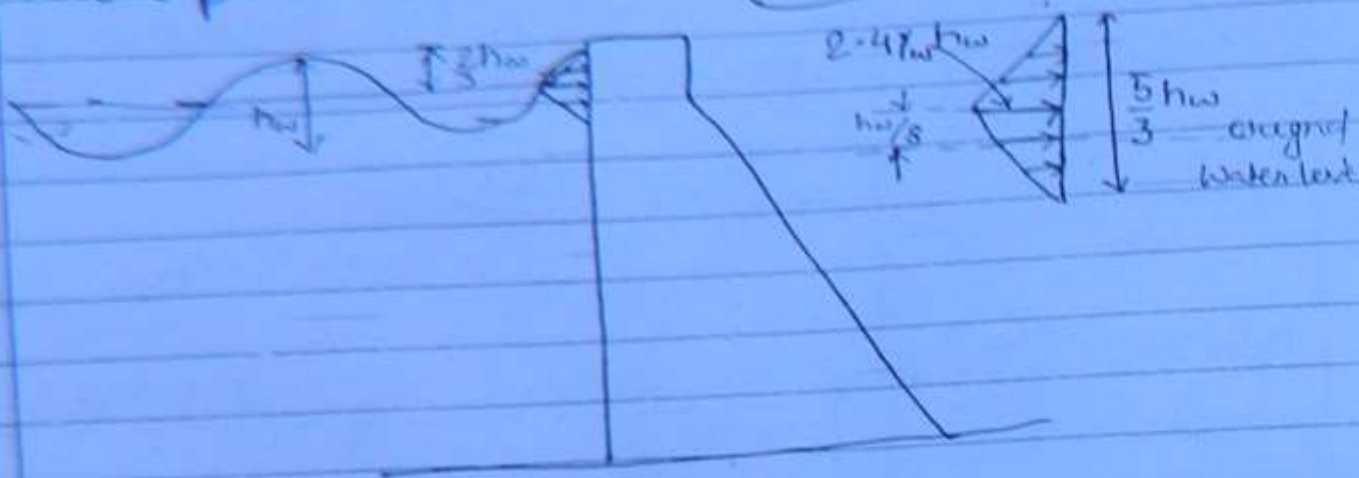
'+' → When the movement is upward (eg lift)

5. Silt Pressure :-



$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi}$$

6. Wave pressure :-



$$\text{Force} = \frac{1}{2} \times \frac{5}{3} h_w \times 2.4 \gamma_w h_w$$

$$\boxed{F = 2 \gamma_w h_w^2}$$

This force acts at a distance of $\frac{3}{8} h_w$ from still water level.

From base it act at distance $= \left(H + \frac{3}{8} h_w\right)$

h_w = height of wave and is given as

$h_w = 0.032 \sqrt{VF}$ if $F > 32$ km

F = fetch of water

$\boxed{h_w = 0.032 \sqrt{VF} + 0.763 - 0.271 F^{1/4}}$ if $F < 32$ Kg.

V = vel. of wind in kmph

h_w = height of wave in meter.

7. Ice pressure :-

The magnitude of this force is about 50 t/m². However the exact value depends on temperature.

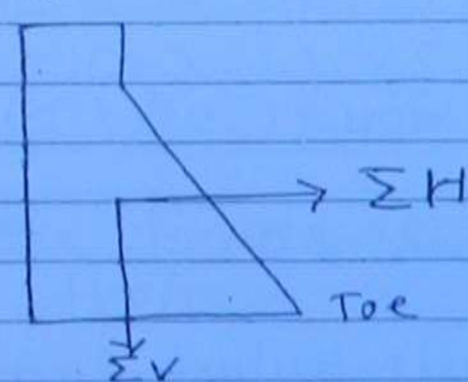
8. Wind pressure

The magnitude of wind pressure may be taken as 1 - 1.5 KN/m² of exposed area.

Modes of failure of dams :-

1. Overturning failure
2. Crushing or compression failure
3. Crack due to tension or tension failure
4. Sliding or shear failure :

Overturning failure :-



For no overturning moment of restoring force or stablizing force about the toe s/d be greater than the overturning moment caused by disturbing force about the toe

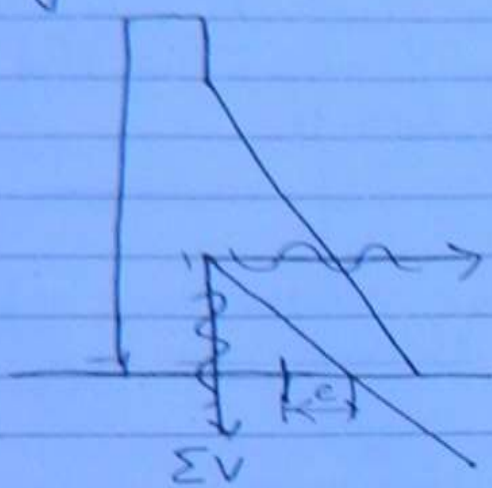
$$\Sigma M_R > \Sigma M_o$$

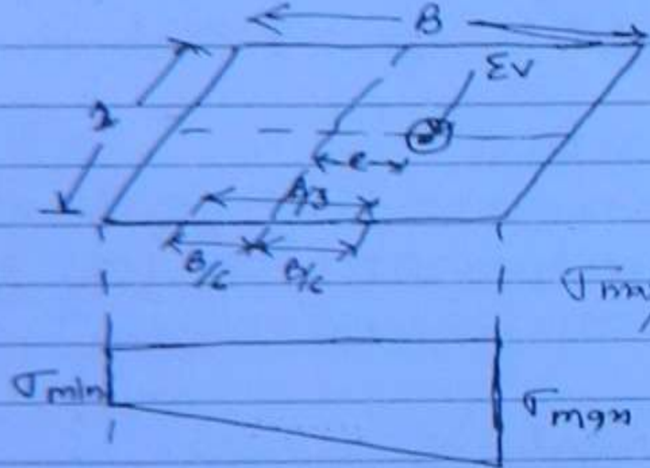
$$F.O.S = \frac{\Sigma M_R}{\Sigma M_o}$$

Min^m F.O.S = 1
Usually [illegible]

M

Crushing or Compression failure :-





$$\sigma = \frac{P}{A} \pm \frac{M}{Z}$$

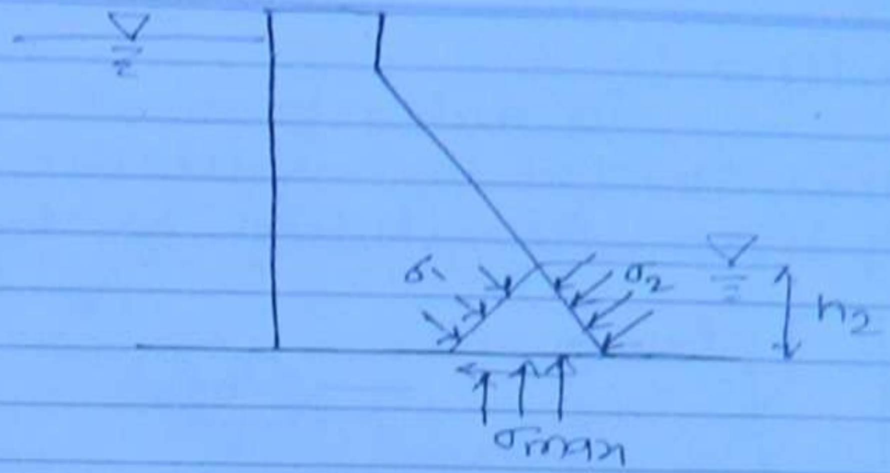
$$\sigma_{max/min} = \frac{\Sigma V}{B \times 1} \pm \frac{(\Sigma V e)6}{B^2}$$

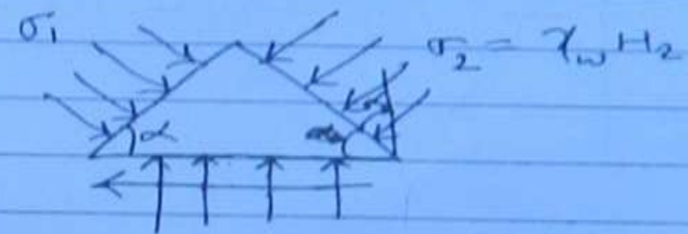
$$\sigma_{min} = \frac{\Sigma V}{B}\left(1 - \frac{6e}{B}\right) = 0 \Rightarrow e = \frac{B}{6}$$

$$\sigma_{min} \geq 0 \; ; \; e \leq B/6$$

For No tensile failure Eccentricity s/d ~~be~~ be ~~be~~ Equal to B/6

and Considering Empty reservoir condn in Which resultant will pass from left hand side of Center of gravity of base, it is seen that eccentricity s/d be with in the middle 3rd portion.
This is called the Middle 3rd Rule.





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

$$\sigma_v = \sigma_1 \cos^2\alpha + \sigma_2 \sin^2\alpha$$

$$\boxed{\sigma_1 = \sigma_v \sec^2\alpha - \sigma_2 \tan^2\alpha}$$

α = Angle of d/s stream face from verti-cal

$$\boxed{\tau = (\sigma_v - \sigma_2)\tan\alpha}$$

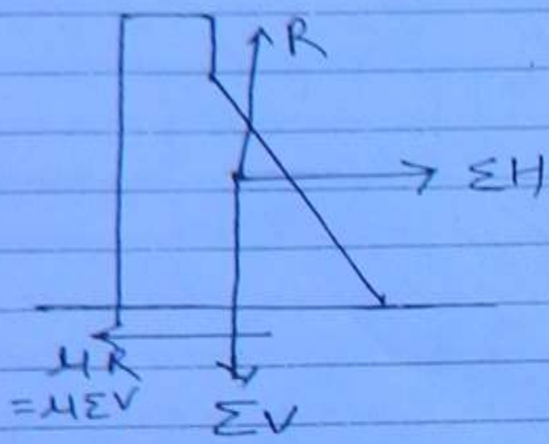
If Earth quake forces are also taken into account then

$$\sigma_1 = \sigma_v \sec^2\alpha - (\sigma_2 - p_e)\tan^2\alpha$$

$$\tau = [\sigma_v - (\sigma_2 - p_e)]\tan\alpha$$

for no compression failure $\sigma_1 <$ strength of conc.

Sliding failure :- In order to avoid Sliding failure the friction resistance S/d be more than the Sliding force



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

$$\boxed{F.O.S = \frac{\mu \Sigma V}{\Sigma H}}$$

The value of μ is generally b/w 0.6 & .75

NOTE:-

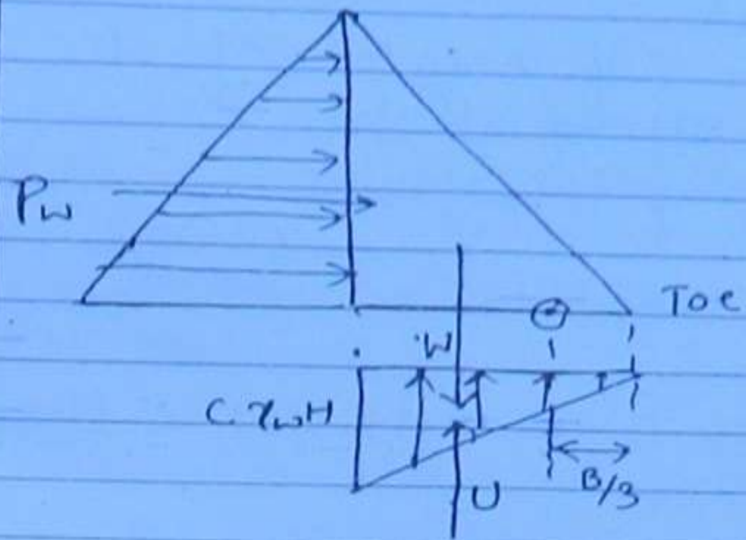
If bond strength of Conc. or Shear strength of Conc. is also taken into account then for economic design resisting force will be Equal to

$$\mu \Sigma V + (B \times 1) q$$

q = bond or Shear strength

$$\boxed{FOS = \frac{\mu \Sigma V + (B \times 1) q}{\Sigma H} = SFF}$$

Shear friction factor.



Minimum base width of dam can be obtained using the following two criteria.

Stability criteria against sliding:-

$$\mu \Sigma V \geq H$$

$$\because \Sigma V = W - H$$

$$B \geq \frac{H}{\mu(S_c - C)}$$

$\gamma_c = S_c \gamma_w$ $\quad S_c = 2.4$

$\mu = 0.7$

$$\boxed{B \geq \frac{H}{1.68}}$$ if $C = 0$

Stability criteria against tensile failure:-

$$B \geq \frac{H}{\sqrt{S_c - C}}$$

Date:

Low & High dams :- If height of the dam is less than or equal to

$$H_{dam} \leq \frac{f}{\gamma_w (S_c + 1)}$$ (120)

If height is low than this value than it is a low dam.
other wise high

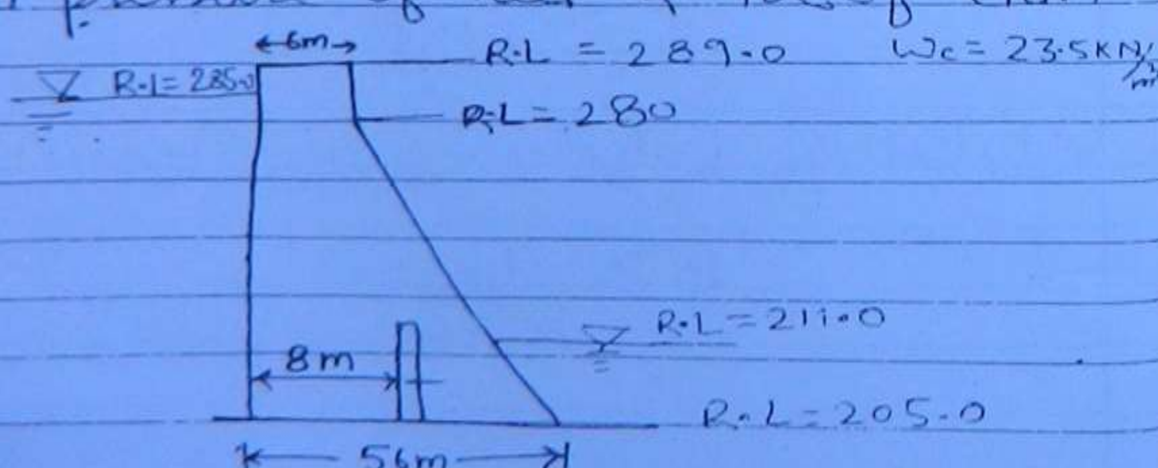
Where,
f = permissible Comp. strength of Conc.
S_c = Sp. gr. of Conc.

The approximate vertical normal stresses at the toe of the reservoir may be given as

$$\sigma_V = \gamma_w H (S_c - C)$$

$$\sigma_1 = \gamma_w H (S_c - C + 1)$$

Q Figure Shows the Section of a gravity dam made of Conc. Calculate Max^m Vertical pressure of heel & toe of dam



Sol. Design steps :-

1. Consider unit length of dam and find the value of ΣV & ΣH

2. Find the lever arm of all the forces about the toe

3. Find the moment of horizontal & vertical forces about toe & find the summition of restoring moment & overturning moment

$$\Sigma M_R , \Sigma M_o$$

4. Find the resultant algebric moment

$$\Sigma M = \Sigma M_R - \Sigma M_o$$

5. Find the location of resultant force from the toe ~~line~~

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

6. Find the Eccentricity from the centre of base of dam.

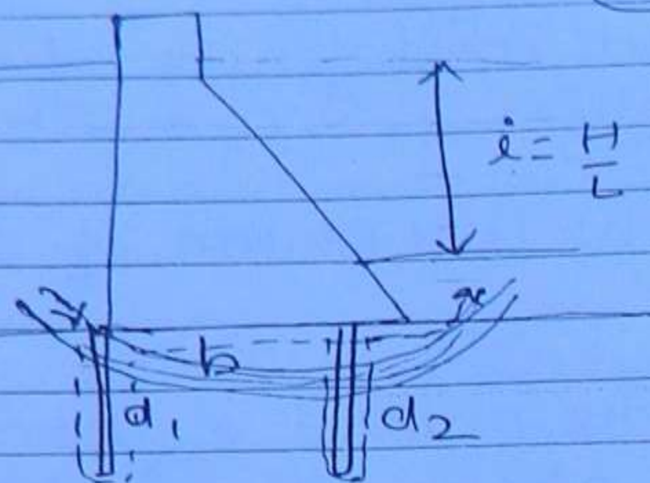
$$e = \frac{B}{2} - \bar{x}$$

If the reservoir is empty then $\bar{x} > B/2$ and

$$e = \bar{x} - B/2$$

Seepage theory :-

(172)



$i = \frac{H}{L}$ $i \downarrow$ by $L\uparrow$

- Bligh's
- Lane's
- Khosla's

$i = \frac{H}{L}$

$i \uparrow$ force $\uparrow$ grad $\uparrow$ (bcoz)

Bligh $L = 2d_1 + b + 2d_2$

Lane $L = 2d_1 + \frac{b}{3} + 2d_2$

Horizontal creep contribute $\frac{1}{3}$rd of vertical creep

t = Thickness of floor to prevent the water coming out from d/w str

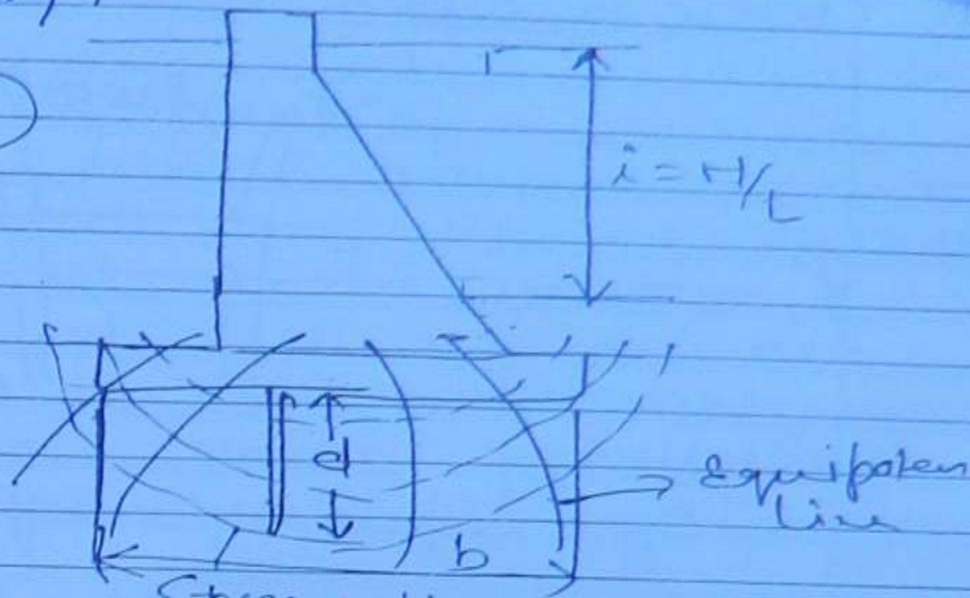
$$\boxed{t = \frac{h}{G-1}}$$ from Bligh's Lane

h = head of water above the floor
G = sp. gr. of conc.

Page No. ______
Date: ______

Khoslas theory :-

173



$$\boxed{\frac{\partial^2 \phi}{\partial x^2} + \frac{d^2 \phi}{d y^2} = 0} \rightarrow \text{this is laplacian eq}^n$$

However an approximate solution is given by Khosla which is called the method of Independent Variables.

Critical Exit gradient :-

. Critical Exit gradient $= (1-n)(G-1)$

n = porosity, G = Sp. gr.

~ s/d be less than

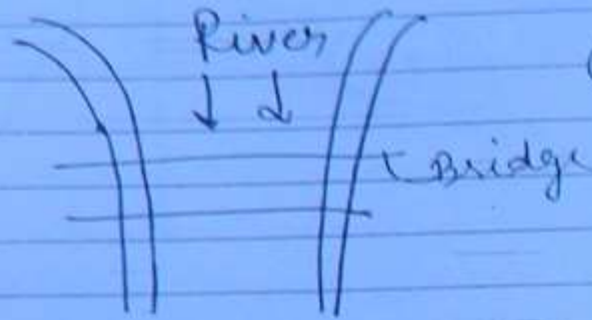
Exit gradient can be calculated as

$$G_e(\text{Exit gradient}) = \frac{H}{d} \frac{1}{\pi\sqrt{\lambda}}$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} \quad \& \quad \alpha = \frac{b}{d}$$

b = floor dimension, d = length of cut

→ River training



Spillway:

Ogee Spillway:- Eqn.

Or

→ Canal above drainage Via-duct → Valley without river

aquaduct

Canal base touching the drain.

Syphon aquaduct / Canal syphon

→ Canal below the drain

→ Super passage.

→ both are at the same level

Level Crossing

C.D Works ↓ by Ridge canal.

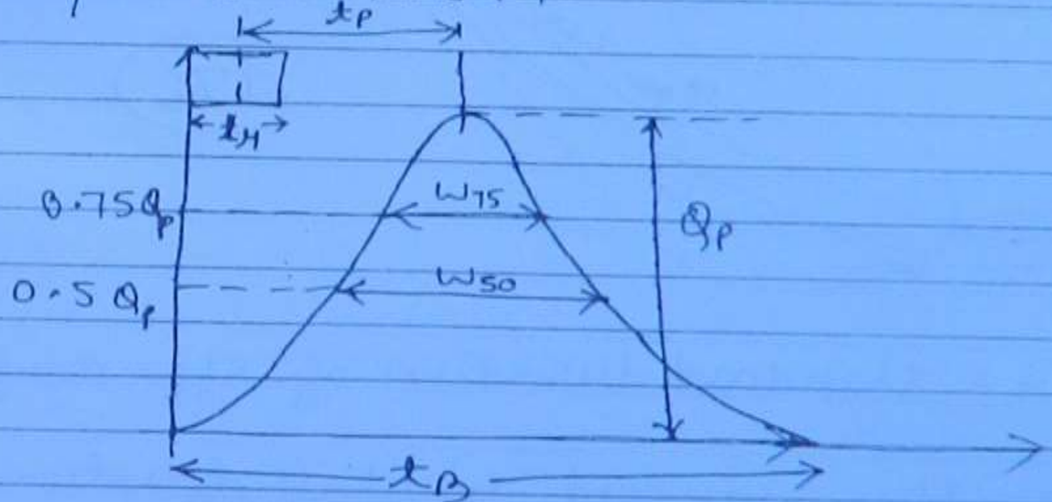
Page No.
Date:

Synthetic Unit hydrograph :- 175

In order to develop unit hydrograph of a catchment the Storm and areal characteristic are required. however such detailed information is not available at all the location & (Specially remote locations) and hence for developing the unit hydrograph of Such catchments certain Emperical eq^n. has been developed.

One such set of Emperical eq^n has been developed by Snyder in U.S.A.

Snyder's method :-



$$t_P = C_t (L L_{ca})^{0.3}$$

$$t_R = \frac{t_P}{5.5}$$

$$Q_P = \frac{2.78 C_P A}{t'_P} \quad ; \quad t'_P = \frac{21}{22} t_P + \frac{t_R}{4}$$

HYDROLOGY

$$t_B = (72 + 3t'_p)\ \text{hrs} \quad \text{or} \quad 5\left(t'_p + \frac{t_R}{2}\right)$$

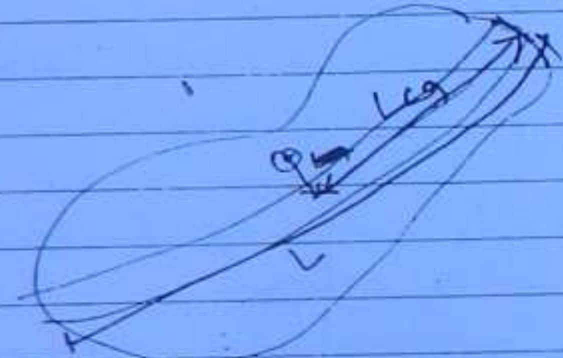
$$W_{50} = \frac{5.87}{q^{1.08}} \quad \& \quad W_{75} = \frac{W_{50}}{1.75} \quad \text{(in hours)}$$

$$q = \frac{Q_P}{A} \qquad Q_P = \text{Peak discharge } m^3/sec$$
$$A = \text{Area } (km^2)$$

Where,

L = Basin length measured along the water course (Km)

L_{ca} = distance along main water course from the gauging station to a point opposite the water shed centroid. (Km)



176

t_r = standard duration of eff. rainfall

t_R = Non standard rainfall duration
t_R = duration of given hydrograph.
eg. 2 hours unit hydrogr $t_R = 2$ (for numerical

C_p & C_t are regional constants

NOTE:- In ~~relation~~ ~~to~~ order to find the value of C_p & C_t another catchment area which is metrologically similar is selected