1 INTRODUCTION

Hydrology is an earth science involving study of water of earth.

Hydrological cycle :- It is a global sun driven process in which water is transported from oceans to the Atmosphere then to the Land and then back to the sea.
It is a contineous process with no definate starting point. A convineint starting point to describe the cycle is taken as oceans.

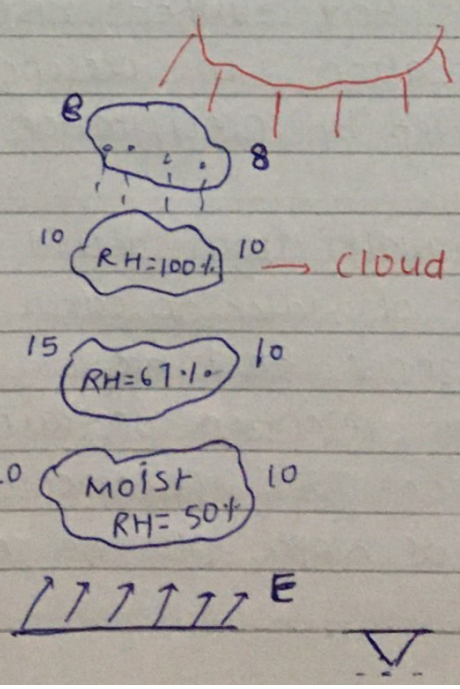
Extent :- 1 km below the earth surface to 50 km above the earth surface.

Relative Humadity :-

$$RH = \frac{\text{Actual vapor carried by Air mass}}{\text{vapor carrying capacity}} \times 100$$

at given temp.

With Temp ↓, RH ↑ (vise-versa)



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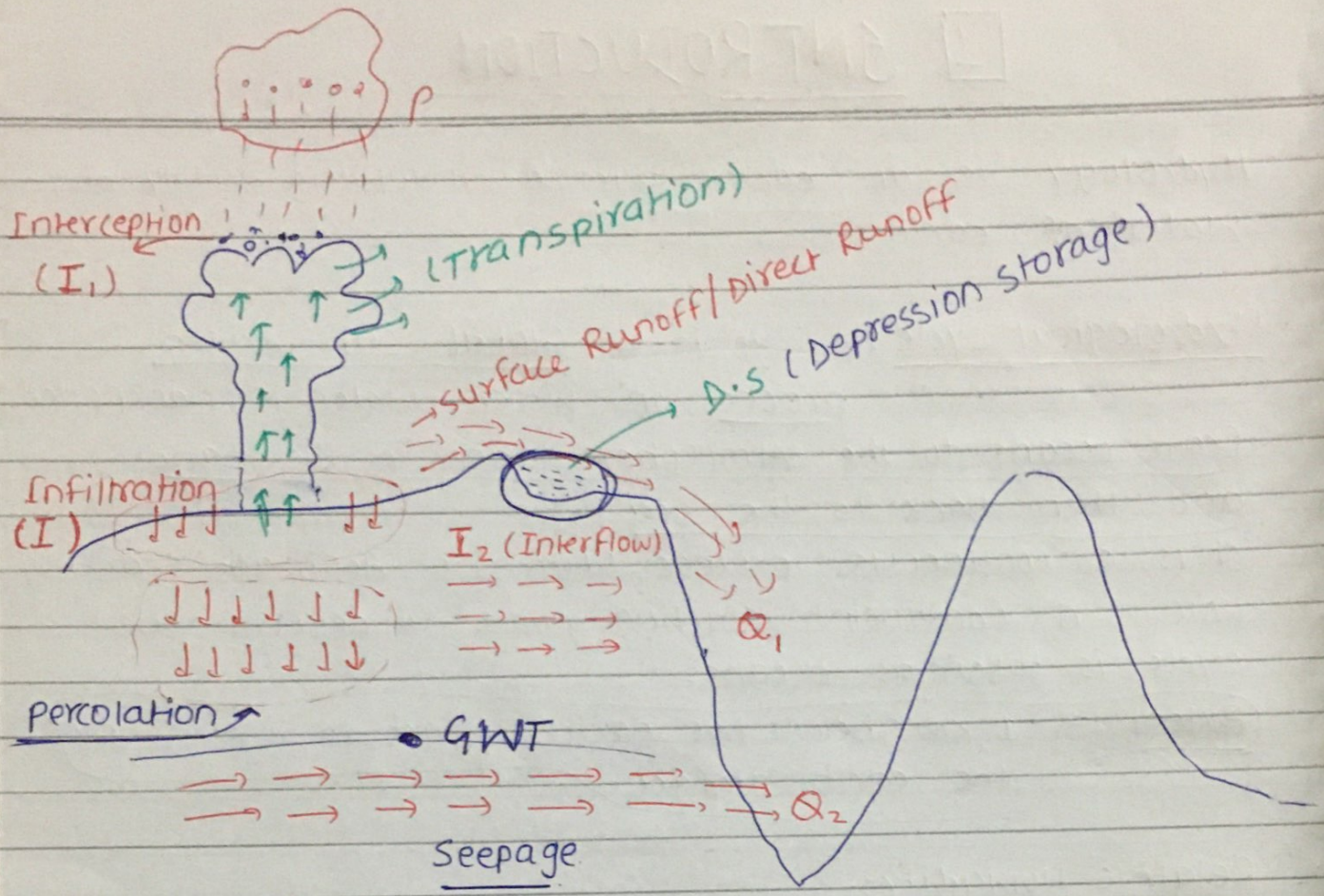

Due to solar radiation falling on ocean surface water evaporates and mixes with the dry air above making it moist. Moist air being Lighter than Dry air Rises and in the process cools down thereby increasing it's RH. RH subsequently reaches 100% (cloud formation). Any further rise in moist air causes condunsation (in presence of Condensation Nuci) followed by precipitation.

Evaporation (E) change of water from liquid to gas state

precipitation (P) Deposition of water on earth surface as Rain, snow, hail etc.

Interception (I_1) Short term retention of rain water by vegitation, roof Tops, pavements etc.

Infiltration (I) Movement of water into the soil and the surface

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percolation (P_1): Movement of water from one soil zone to the lower soil zone

Transpiration (T): Water absorved from Ground and evaporated into the Atm. through leaves

Interflow (I_2): It is the Ground water flowing Horizontally above the Water table (very less amount)

It is also known as sub surface flow.

Depression storage (D.S): Rain water accumulated in small depression or ditches above the surface

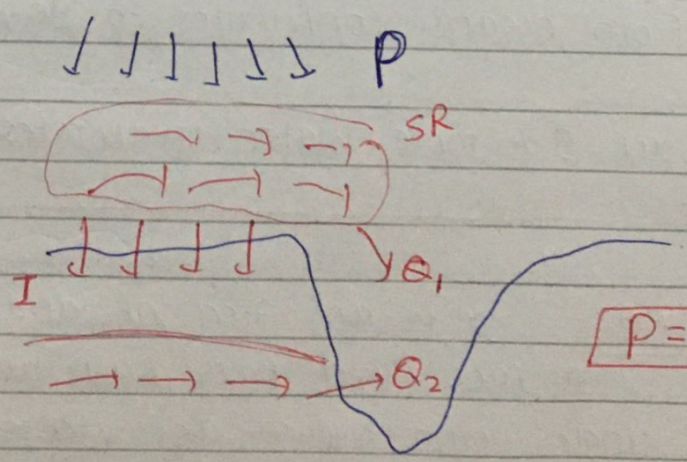
Surface Runoff :- (SR) It is the part of rain water which reaches the stream outlet soon after the rainfall flowing over the surface. Also sometimes called Direct Runoff (DR), Eff. Rainfall or Rainfall excess

NOTE: Actually DR is slightly more than SR But for all practical calculations they are treated as same

Q2: Discharge obtained in the stream due to GWT through seepage. It is also known as Base Flow, Dry Weather Flow or Effluent seepage

Q1: Discharge obtained in the stream due to SR/DR

Extra:-



$P = S.R + I$ *

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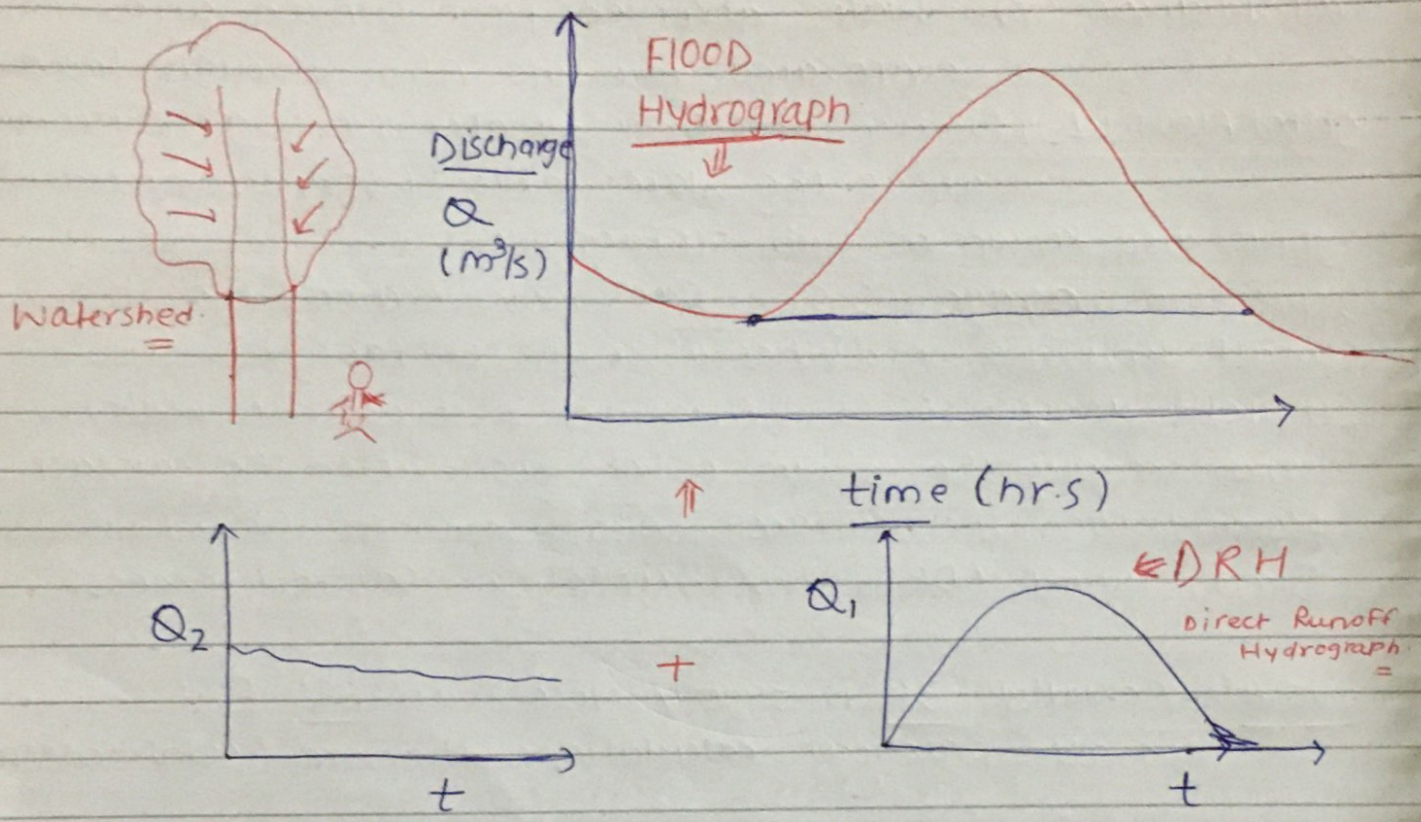
Hydrograph:-

It is a plot of runoff (Discharge) Vs Time.



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NOTE:- Area of DRH = Volume of SR / DR / Eff. Rainfall

* DRH will always start from 0 and will end at 0
  The only exception is S-curve (that is hypothetical)

> Evaporation from oceans contributes to 90% of Atmospheric Moisture
> In oceans about 9% more water evaporates then falls back as ppt

CATCHMENT AREA:- It is an Area of land where surface water from Rain and Melting snow converges to a single point known as catchment outlet where the water joins another water body like lake, river or ocean.

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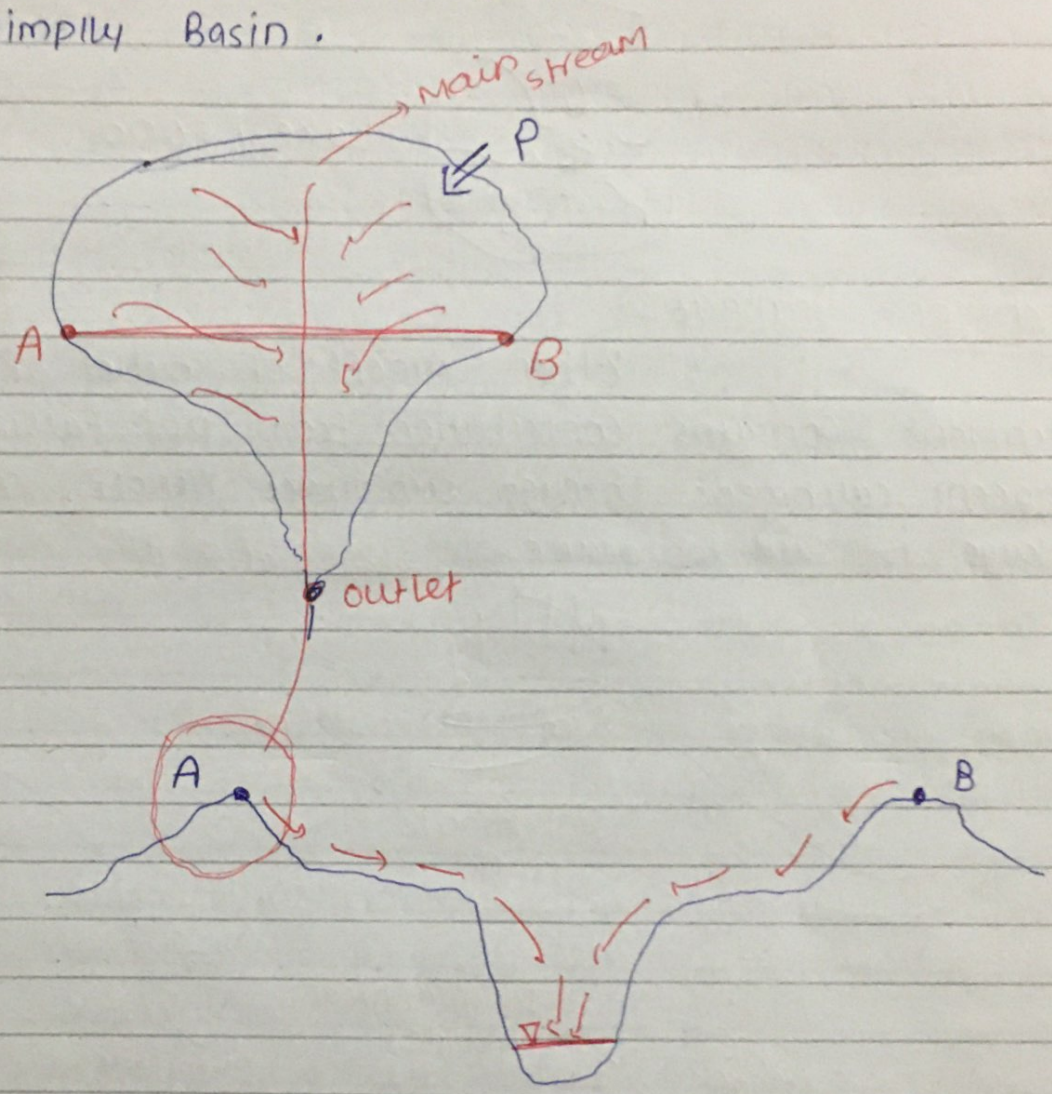
Catchment Area is also known as Watershed, River Basin or simplly Basin.



Each catchment is separated Topographically from adjacent catchment by Geogropical barrier such as Mountains or hill or ridge.

The line which devides the SR b/w two adjacent catchments is called Topographic devide, Watershed devide, Water devide or simply devide

The devide follows the ridge line crossing the Main stream only at the outlet.

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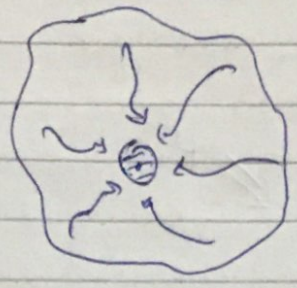
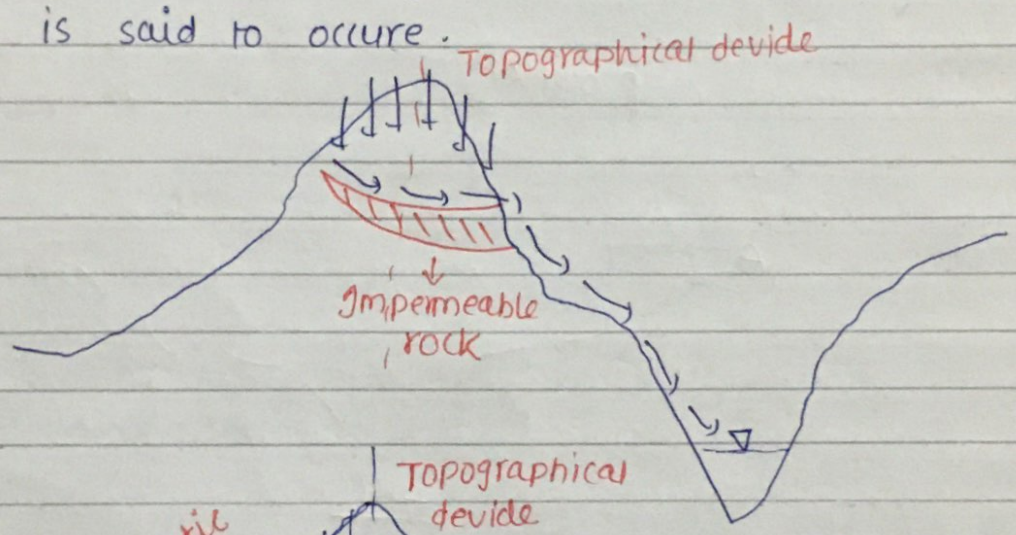
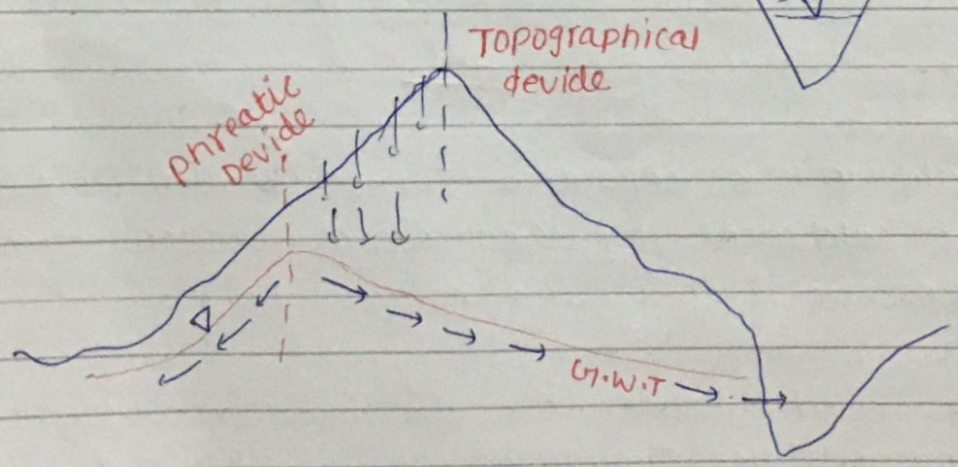
NOTE :- If all water converges to a single point inside the Basin it is known as closed catchment



Catchment Leakage :-

When runoff of outlet of one catchment contains contribution from ppt falling on adjacent catchment through subsurface runoff, catchment leakage is said to occure.





phreatic Devide is not coinsiding with Topographical devide

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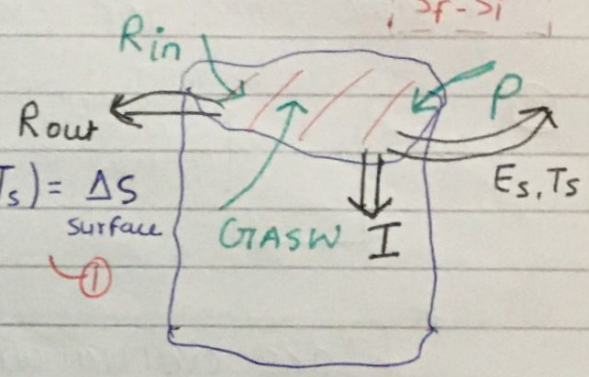
WATER Budget / Hydrological Budget :-

It is based on law of conservation of mass
It states that

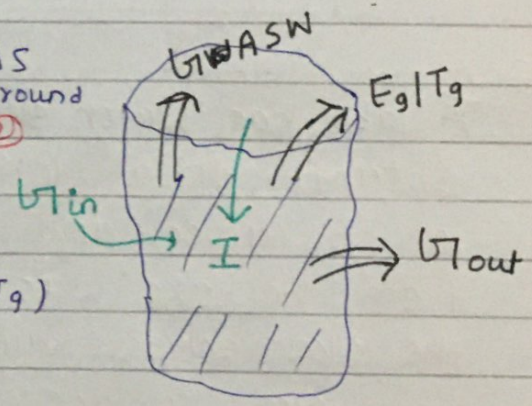
Mass inflow − Mass outflow = change in storage (ΔS)
$S_f - S_i$

for surface flow :-



$$(P + R_{in} + GASW) - (I + R_{out} + E_s + T_s) = \Delta S_{surface} \quad (1)$$

for ground flow :-



$$(I + G_{in}) - (GASW + G_{out} + E_g + T_g) = \Delta S_{ground} \quad (2)$$

(1) + (2)

$$(P + R_{in} + G_{in}) - (R_{out} + E_s + T_s + G_{out} + E_g + T_g) = \Delta S \text{ (Total)}$$

$$P - (R_{out} - R_{in}) - (E_s + E_g) - (T_s + T_g) - (G_{out} - G_{in}) = \Delta S$$

$$\boxed{P - R - E - T - G = S}$$

P → Total ppt
E → Total Evaporation
T → Total Transpiration
G → Net Ground Water outflow
R → Net runoff out
S → change in storage (Final − initial)

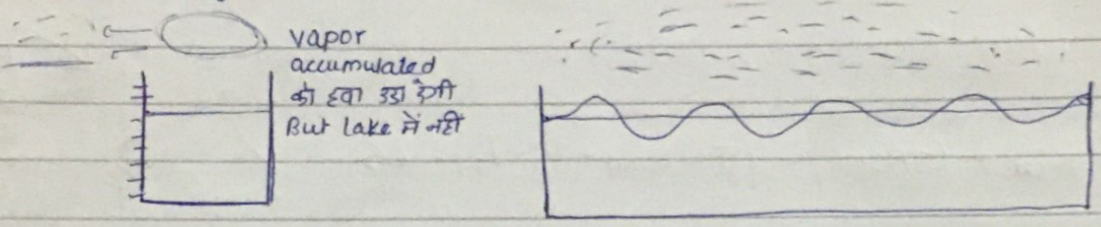
जो निकालना है वो unknown है बाकी जो नहीं दे रखा उसे Negligible मान लो !

for long period of time S is negligible

* ① precipitation as Rainfall is expressed in terms of depth over the horizontal projection of the Area

$$\text{Depth of Rainfall} = \frac{\text{Volume of Rainfall}}{\text{projected Area}}$$

→ Same goes for evaporation also

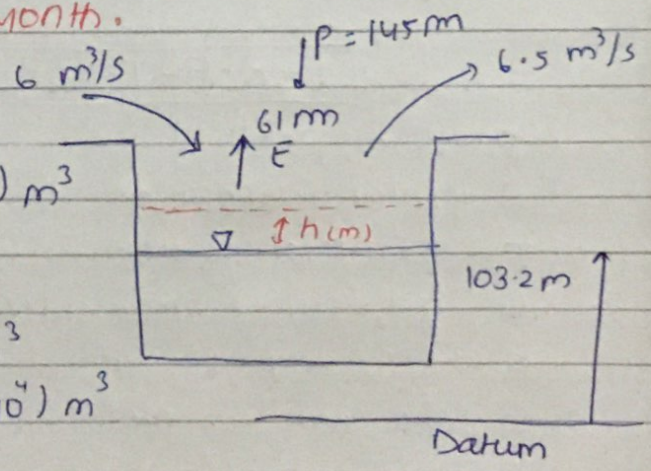
② 

*

$$\boxed{\text{Lake evaporation} = \text{Pan cofficient} \times \text{pan evaporation (rate of evapo}^n\text{)}}$$

Que.

A lake has water surface elevation of 103.2 m above Datum. In a month the lake recives an avg. inflow of 6 cumec and in the same period the outflow from the lake was 6.5 cumes. In the same month the lake recives Rainfall of 145 mm and the evaporation from lake surface was 6.1 cm. The surface Area of lake = 5000 ha. Calculate the surface elevation of lake at the end of this month.



$$\text{Inflow} = (6 \times 30 \times 24 \times 60 \times 60)\,m^3 + (145 \times 10^{-3} \times 5000 \times 10^4)\,m^3$$

$$\text{Inflow} = 22.802 \times 10^6\ m^3$$

$$\text{Outflow} = (6.5 \times 30 \times 24 \times 60 \times 60)\,m^3 + (61 \times 10^{-3} \times 5000 \times 10^4)\,m^3$$

$$\text{Outflow} = 19.898 \times 10^6\ m^3$$

$$\Delta S = (h \times 5000 \times 10^4)\,m^3$$

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Inflow - outflow = Δs

$h = 0.058\ m$.

Final elevation = 103.258 m.

Que. Following observations were made while conducting a water budget for a reservior over a period of 1 month. Mean surface inflow rate = 10 cumec, Mean surface outflow rate = 15 cumec, Avg. surface Area = 10 km^2, Rainfall = 10 cm, fall in Reservior = 1.5 m, pan evoporation = 20 cm, Assume pan cofficient = 0.7. Estimate the avg. seepage Discharge from reservior During this month.

$$\text{Inflow} = \underbrace{(10 \times 30 \times 24 \times 60 \times 60)}_{R_{in}} + \underbrace{(10 \times 10^{-2} \times 10 \times 10^{6})}_{P}\ m^3$$

$$= 26.92 \times 10^6\ m^3$$

$$\text{Outflow} = \underbrace{(15 \times 30 \times 24 \times 60 \times 60)}_{R_{out}} + \underbrace{(14 \times 10^{-2} \times 10 \times 10^{6})}_{E} + (q \times 30 \times 24 \times 60 \times 60)\ m^3$$

$$\Delta s = -1.5 \times 10 \times 10^6\ m^3$$

$$\Delta s = \text{Inflow} - \text{outflow}$$

$$\Rightarrow \boxed{q = 0.6327\ m^3/s}$$

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2 PRECIPITATION

Forms of precipitation:-

(i) **Rain:**

↳ principle Mode of precipitation in India = (120 cm Annually)

↳ Denotes water Droplets whose size ranges from 0.5 mm to 6 mm

↳ on the Basis of Intensity it is classified as

- Light (0 – 2.5 mm/h)
- Medium (2.5 – 7.5 mm/h)
- heavy (≥7.5 mm/h)

(ii) **Snow:**

They are Flaky ice crystels having a density of 0.1 g/cc

(iii) **Drizzle:** Are water Droplets less than 0.5 mm with intensity less than 1 mm/hr

(iv) **Sleet:-** frozen Raindrops

(v) **Hail:-** It consists of large spheres of ice having dia 5 mm to 50 mm

(vi) **Glaze:-** When water droplets cames in contact with cold surface it freezes to form an ice coating known as glaze.

* Isonif is a line joining points having equal snow Fall

Isohyet → Rainfall

Isochrone → Travel Time

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Intensity of Rainfall :-

If X depth of Rain falls over a catchment Area in Time t is the avg. intensity of Rainfall i is given as

$$* \quad \boxed{i = \frac{X}{t}}$$

It is Generally expressed in mm/hr or cm/hr

Que. The Intensity of Rainfall and time interval of a Typical storm is given below. The Max. Intensity of Rainfall for a 20 min Duration of Rainfall in mm/min is ?

time interval (Min)	intensity (mm/Min)	Depth (mm)
0 – 10	0.7	7
10 – 20	1.1	11
20 – 30	2.2	22
30 – 40	1.5	15
40 – 50	1.2	12
50 – 60	1.3	13
60 – 70	0.9	9
70 – 80	0.4	4

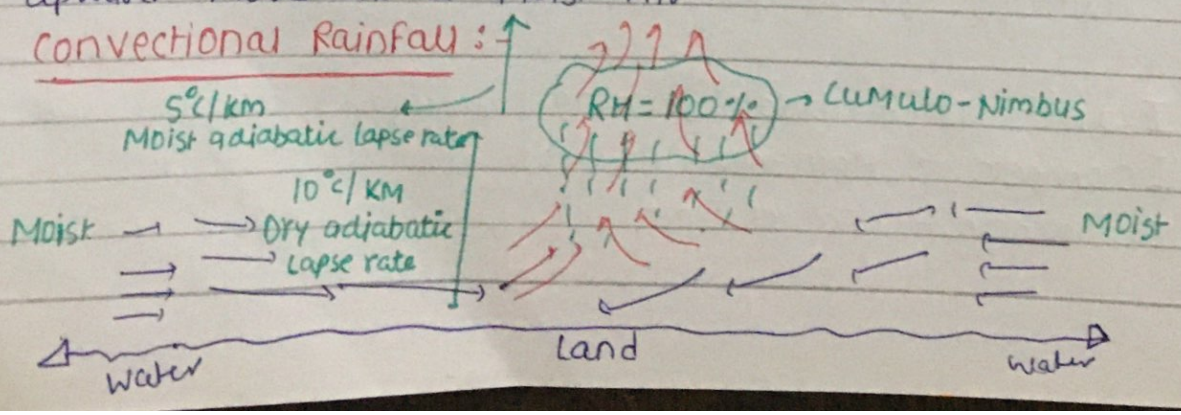
37 mm

$i_{max} = \frac{37}{20} = 1.85$ mm/min

Types of Rainfall :-

precipitation as rainfall is classified on the Basis of conditions and Mechanisms of upward movement of Moist Air.

1) Convectional Rainfall :-



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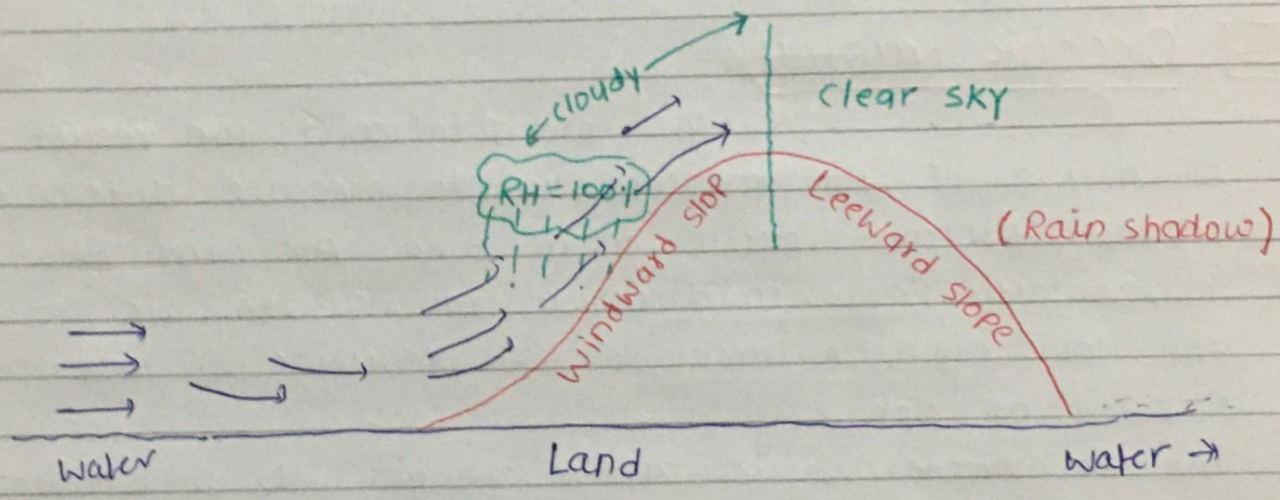
It occurs due to thermal convectional currents formed by heating of Ground surface. Moist adiabatic lapse rate is less than Dry adiabatic lapse rate due to release of latent heat of vaporization

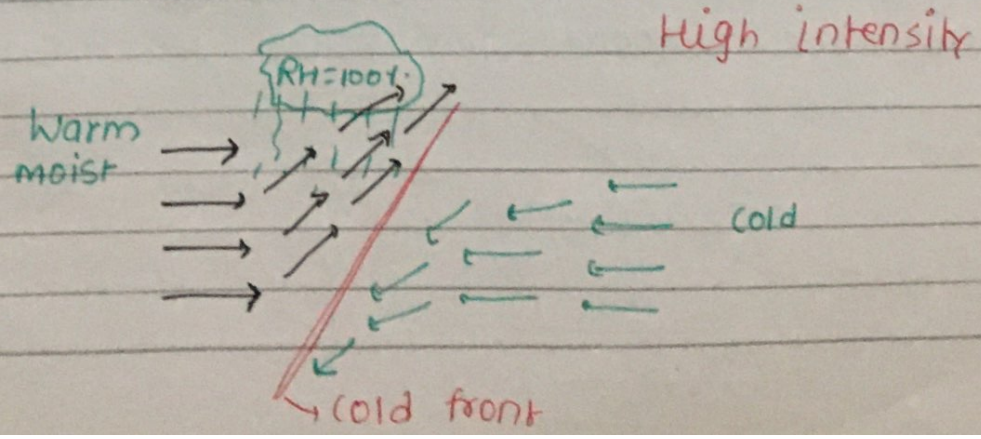
2 Orographic Rainfall:-



This precipitation is caused by ascent of air from large Water bodies forced by Mountain Barriers. Most of World's precipitation including India's occurs through Orographic Rainfall *

3 Frontal Rainfall :-

(i)



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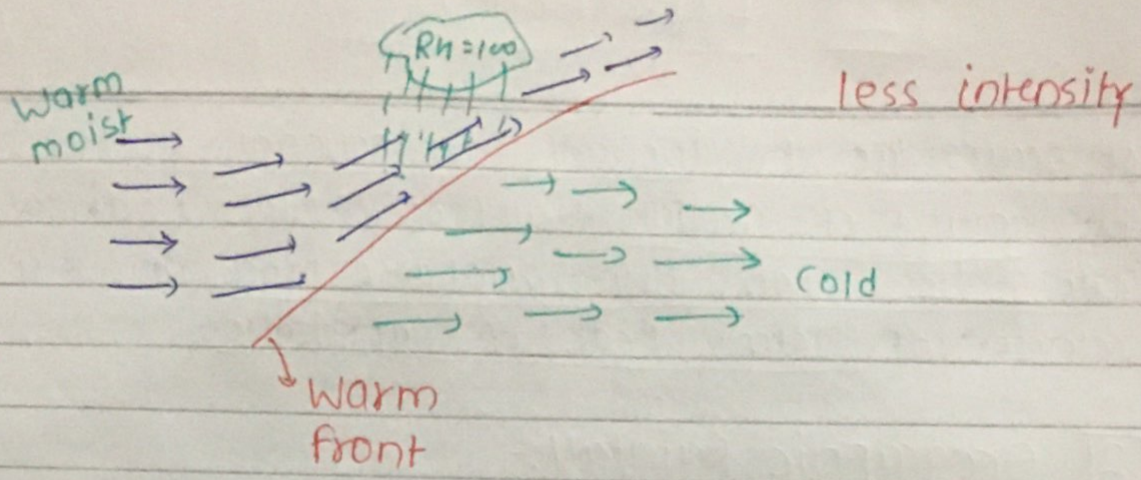
(ii)



This precipitation occurs due to conflict b/w Two Air masses of Different Temp. and densities.

4 Cyclonic Rainfall :-

This precipitation is the result of lifting of Air masses due to pressure differences

- If low pressure occurs in a Area Air will Flow in horizontally from all directions causing the air in the low pressure area to lift resulting in condensation followed by ppt.

The Air rushes horizontally changes into a whirling mass due to the rotation of the earth (~~for~~ coriolis effect) and is called cyclone.

Cyclone	Anti-cyclone
• low pressure @ center	• High pressure @ center
• counter clockwise in Northen Hemisphere and clockwise in southern.	• clockwise in Northen A.C.W in southern Hemisphere
• Indicator of Rain	• Indicator of Good weather
Ex: Hurricane, Typhoone	

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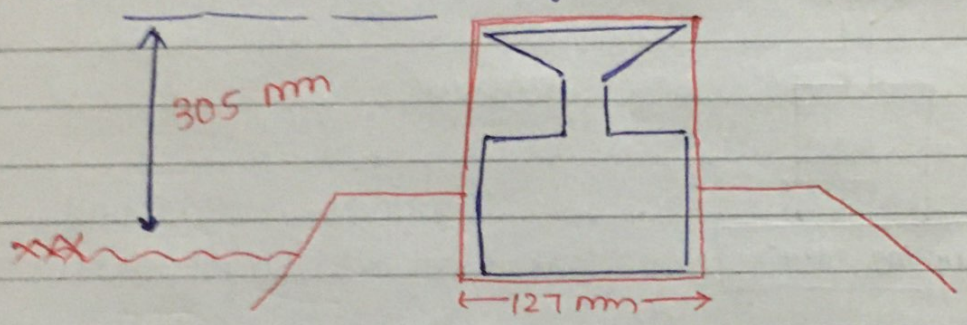
Measurement of precipitation :-

precipitation as rainfall is measured by a device known as Rain Gauge. It is also known as ombrometer, pluviometer or Hyetometer

Types of Rain Gauges :-

1) Non-recording Type :- They don't give a contineous plot of Rainfall against Time

Non-recording Gauge widely used in India is ~~sim~~ symon's rain Gauge.

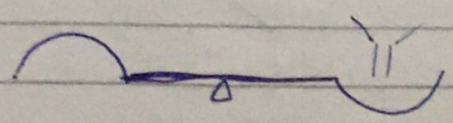


- It is installed in an open Area on a concrete foundation
- Distance of Rain Gauge from the nearest object should at least be twice the height of the objects
- Measurements are to be done @ 8:30 Ist and reported as rainfall of past 24 hours.

2) Recording Type rain Gauge

They give a contineous plot of rainfall against time

a) Tipping Bucket Type :-



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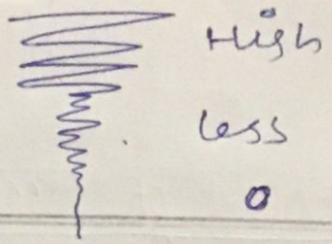
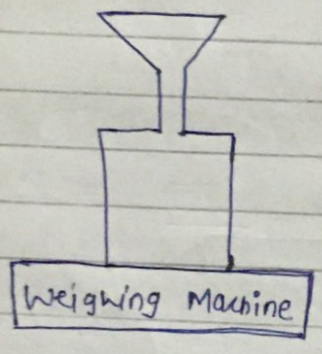
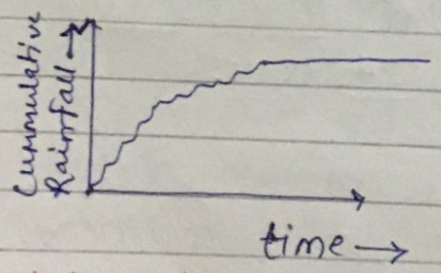


It consists a pair of Bucket so arranged that when 0.25 mm of rainfall collects in one Bucket it Tips and Brings the other one in position.
- This Tipping activates an electrical pen to trace the record on a clock work driven chart
(It gives intensity vs Time)

(b) Weighing Bucket type



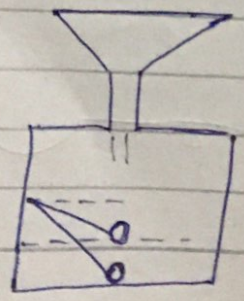
Rainfall is collected into a Bucket mounted on a weighing Machine which records the weight on a clockwork driven chart
- This gives a plot of cummulative Rainfall vs Time

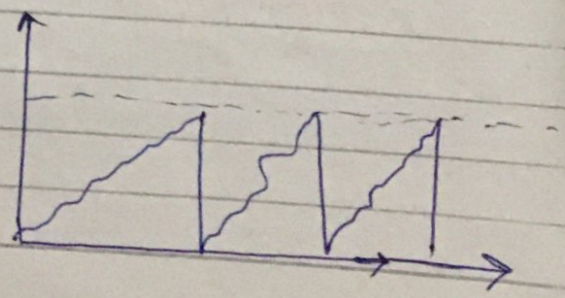


(MASS- CURVE)

Slop
Vol vs time → Discharge
depth vs time → intensity

(c) Natural syphon type or Float type Gauge :-





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Rain water is collected into a Bucket with a Float which rises with the Water level and upon reaching a pre set value opens a drain to empty the bucket

The elevation of the Float is contineously recorded on a clock work driven chart.
It is used as the standard type of recording rain Gauge in India

NOTE: Weighing bucket is best suited for SNOW, Hail etc.

Telemetering Rain Gauge:-

It is used for Mountanious and other inaccesible places. Rainfall Data is sent in the form of an electronic pulse.
Tipping Bucket is best suited

Rain Gauge Network:-

According to BIS, It should be as follows:-
1) For Plain Areas – 1 rain Gauge / 520 Km^2
2) For Moderatly elevated Areas - (upto 1000 m height from MSL) 1 Rain Gauge in 260 - 390 Km^2
3) In Hilly Areas - 1 Rain Gauge / 130 Km^2

NOTE: According to WMO at least 10% of Rain Gauges should be of Recording Type.

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Optimum No. of Rain Gauges :-

It is the no. of Rain Gauges required to calculate the Avg. Rainfall value with a certain acceptable % of error (e %) (e=10% if Not given)

For a catchment with $\underline{\underline{n}}$ no. of existing Rain Gauges having Rainfall values P_1, P_2 --- P_n it is estimated statically As below -

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

$C_v \rightarrow$ cofficient of variance

$$C_v = \frac{\sigma_{n-1}}{P_m} \times 100$$

$\sigma_{n-1} \rightarrow$ standard deviation

$$\sigma_{n-1} = \sqrt{\frac{(P_1-P_m)^2 + (P_2-P_m)^2 + \cdots + (P_n-P_m)^2}{n-1}}$$

$P_m \rightarrow$ mean ppt of existing rain Gauges

$$P_m = \frac{P_1 + P_2 + \cdots + P_n}{n}$$

Hence No. of additional Rain Gauges required = N-n

Que In a Watershed 4 Non. recording Rain Gauges have been installed. The annual ppt of one of the Year is given as below. Calculate the existing % of error in the estimation of Mean Rainfall.

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Also calculate optimum No. of Rain Gauges required for an error of 10% in Mean ppt calculation.

Stn	rainfall (cm)
A	100
B	120
C	140
D	80

$$P_m = \frac{100+120+140+80}{4} = 110\,cm$$

$$\sigma_{n-1} = \sqrt{\frac{(100-110)^2 + (120-110)^2 + (140-110)^2 + (80-110)^2}{4-1}}$$

$$= 25.82$$

$$C_v = \frac{\sigma_{n-1}}{P_m} \times 100 = 23.46\,\%$$

<u>For existing % of error.</u>

$$4 = \left(\frac{C_v}{e}\right)^2 \quad , \quad e = 11.73\,\%$$

<u>for optimum No. of rain Gauges.</u>

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

$$N = \left(\frac{23.46}{10}\right)^2$$

$$N = 5.5$$

$$\boxed{N = 6}$$

Additional No. of Rain Gauges $= N - n = 6 - 4 = 2$

Out of which at least one should be of recording Type

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Normal precipitation :- It is the avg. of Rainfall for a perticular day, Month or Year over a specified 30 year period.

It is recalculated after every decade to account for climatic changes.

Estimation of Missing Data :- (Continuity check)

Let there be n rain Gauges with Rainfall Values $P_1, P_2, P_3 ---- P_n$ and Normal ppts $N_1, N_2, ----- N_n$ surrounding the rain Gauge station x with missing Data P_x and Normal ppt N_x

The Missing Data P_x is calculated As follows -

1) **Airthmetic Mean Method :-**

If Normal ppts of each of the surrounding n RainGauges are within 10 % variation (± 10 %) of the Normal ppt of station x then P_x is given as

$$P_x = \frac{P_1 + P_2 + P_3 + --- + P_n}{n}$$

2) **Normal Ratio Method :-**

If the Normal ppt of any of the surrounding n RainGauges is beyond 10 % variation of the Normal ppt of st^n x then P_x is calculated as -

$$\frac{P_x}{N_x} = \frac{1}{n}\left(\frac{P_1}{N_1} + \frac{P_2}{N_2} + ----- + \frac{P_n}{N_n}\right)$$

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Que: Find the Missing Rainfall @ station X

St^n	long yr Avg.	Annual ppt.
A	1125	875
B	910	1021
C	765	915
X	830	??

1125 > 830 ± 10%
use Normal Ratio Method

$$\frac{P_x}{830} = \frac{1}{3}\left(\frac{875}{1125} + \frac{1021}{910} + \frac{915}{765}\right)$$

$$P_x = 856.5 \text{ mm}$$

Inconsistency of Records :-

The rainfall values recorded by a rainGauge are said to be inconsistent if the values are not in accordence with present environmental and geographical conditions.

Causes of Inconsistency :-

(i) Shifting of Rain Gauge to a New location
(ii) Surrounding of Rain Gauge St^n undergoing significant change.
(iii) Replacement of old instruments by new one
(iv) change of observer or Method of observation

Inconsistency of records is checked and corrected by double Mass curve technique -

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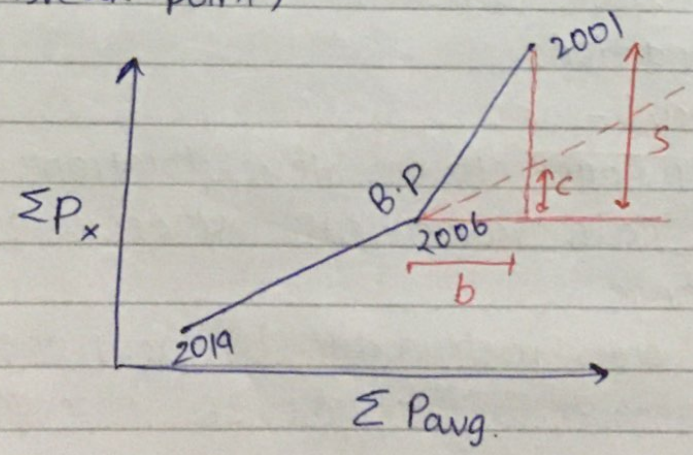
Double Mass curve :-

Let there be n rain Gauges surrounding the rain Gauge st^n x for which consistency is to be checked
The accumulated rainfall of st^n x (ΣP_x) and the accumulated Rainfall of avg. of surrounding Rain Gauges st^ns $\Sigma(P_{avg.})$ is calculated in reverse cronology that is starting from the latest record for permanent change.

The value ΣP_x as ordinate is plotted against $\Sigma(P_{avg.})$ as absisa to obtain a curve known as Double mass curve

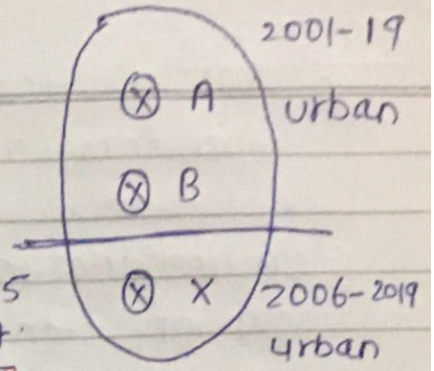
- If the curve obtained is a straight line it is said to be consistant and if a change of slope is observed then st^n x is said to be inconsistent beyond the point slop changes (Break point)



The values of ppt of st^n x for years beyond break point are corrected by multiplying them by a correction factor f

* if change is temporary then plotting shall be in chronological order.

$$\boxed{f = \frac{c/b}{s/b} = c/s}$$



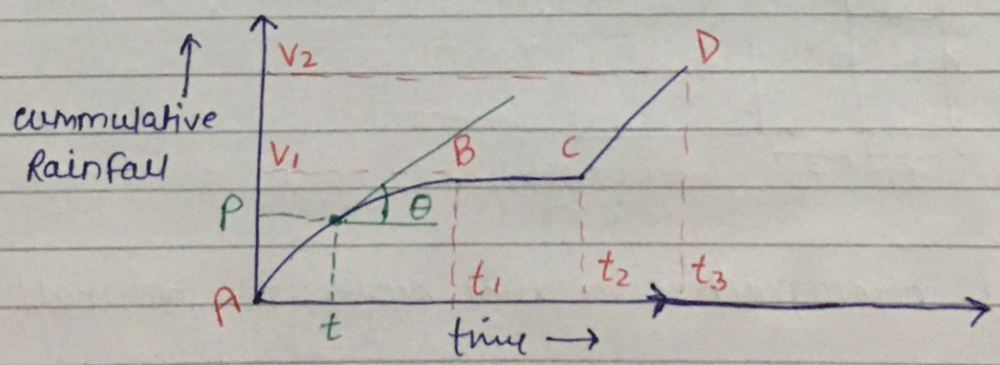
	X	A	B	ΣX	ΣP_{avg}
2019	X_{19}	A_{19}	B_{19}	X_{19}	$(A_{19}+B_{19})/2 = 1$
2018	X_{18}	A_{18}	B_{18}	$X_{19}+X_{18}$	$(A_{18}+B_{18})/2 + 1$
2017	X_{17}	A_{17}	B_{17}	$X_{19}+X_{18}+X_{17}$	$(A_{17}+B_{17})/2$
⋮	⋮	⋮	⋮	⋮	⋮
2001		A_1	B_1		

cummu-lative

Presentation of Rainfall Data :-

(i) Mass curve :- It is a plot of cummulative Rainfall against Time.

cummulative vol^m vs time Mass curve :-



Avg. Discharge from $A \rightarrow B = \frac{V_1}{t_1}$

$B \rightarrow C$ (period of No rainfall)

$C \rightarrow D = \frac{V_2 - V_1}{t_3 - t_2}$

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NOTE

① Straight line on a volume vs time mass curve represents a period of constant discharge

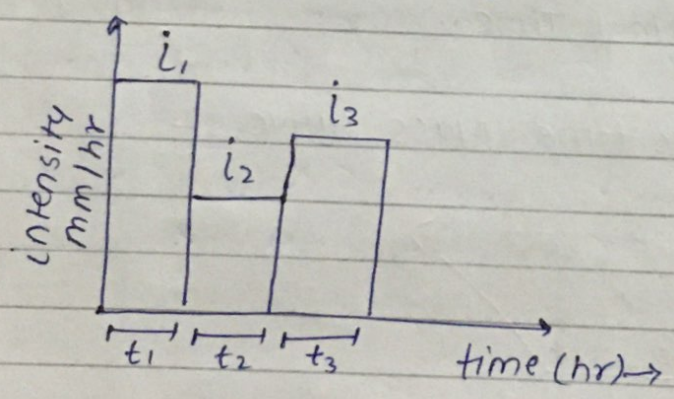
② Instantaneous Discharge at any point t is the slop of tangent at that point '$\tan\theta$' or $\frac{dv}{dt}$

Cumm. Vol vs time → slop → Discharge
Cumm. Depth vs time → slop → Intensity

$$\frac{\text{Volume of Rainfall}}{t} = \frac{\text{depth}}{t} \times \text{Area}$$

$$\boxed{Q = i \times A}$$

(ii) Hyetograph :- plot of avg. intensity of Rainfall against Time.



Area of hyetograph gives total depth of Rainfall.

Que plot the hyterograph for Data given below:-

time	Cummulative Rain (cm)
0	0
3	9
6	12
9	18
12	18

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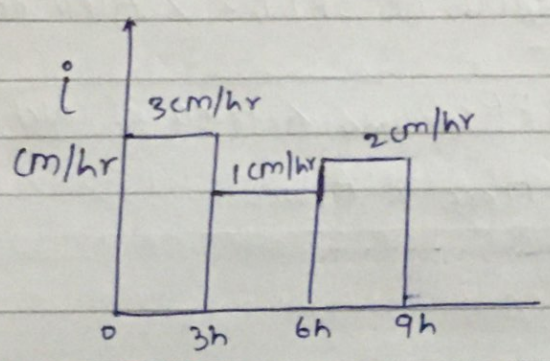
Sol^n

time interval	depth of Rainfall	$i_{avg.}$
0-3	9	3 cm/hr
3-6	3	1 cm/hr
6-9	6	2 cm/hr
9-12	0	0

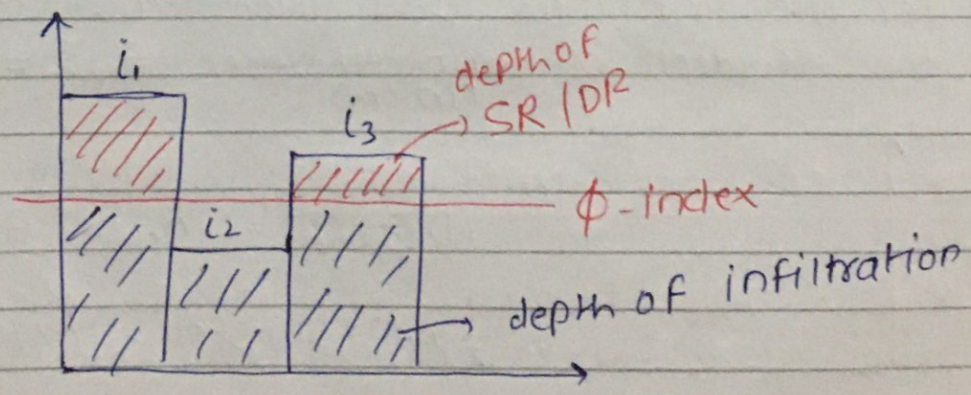


Extra

ϕ-Index: It is an approx infiltration capacity used to express total infiltration within the Duration of rainfall.

It is so defined that the Area of hyetograph above ϕ index = Surface Runoff's depth or effective Rainfall / depth of DR or Rainfall excess (h_{eff})

and the Area of hyetograph below ϕ-index will give us the depth of infiltration.



$\phi \rightarrow$ Not real, imaginary (capacity of infiltration)

if $i < \phi$, actual infiltration will be $= i$

if $i > \phi$, actual infiltration $= \phi$

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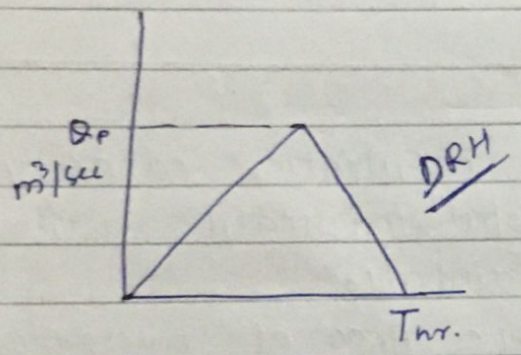
- The portion of hyetograph above ϕ is called eff. Rainfall hyetograph ERH

Hence Area of ERH = depth of SR / DR / $h_{eff.}$

Vol^m of SR / DR = depth of SR / DR × Area of catchment

* $(DRH)_{Area} = (ERH)_{Area} \times$ Area of catchment

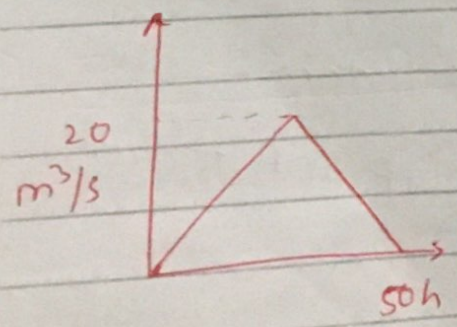
$(DRH)_{Area} = h_{eff.} \times A_{catch}$



$$\frac{1}{2} \times T_{hr} \times Q_P = h_{eff} \times Area_{catchment}$$

$$\frac{1}{2} \times \underset{\times 3600}{T_{hr}} \times Q_P \ m^3/sec = \underset{\times 10^{-2}}{h_{cm}} \times A_{catch.} \ km^2 \times 10^6$$

Que for the Tringular DRH given below calculate the eff. depth of Rainfall (in cm) when $A_{catch} = 150 \ km^2$



$(DRH)_{Area} = h_{eff} \times A_{catch}$

$\frac{1}{2} \times 20 \times 50 \times 3600 = h \times 10^{-2} \times 150 \times 10^6$

$h = 1.2$ cm

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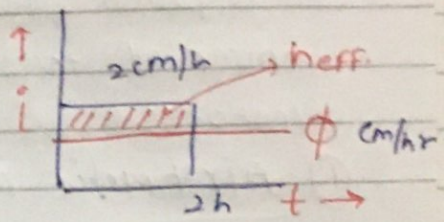
Que: for the previous Question calculate the ϕ-index if the Rainfall depth was 4 cm and the Duration of Rainfall was 2 hour

$$i = \frac{\text{depth}}{\text{time}} = \frac{4}{2} = 2 \text{ cm/hr}$$

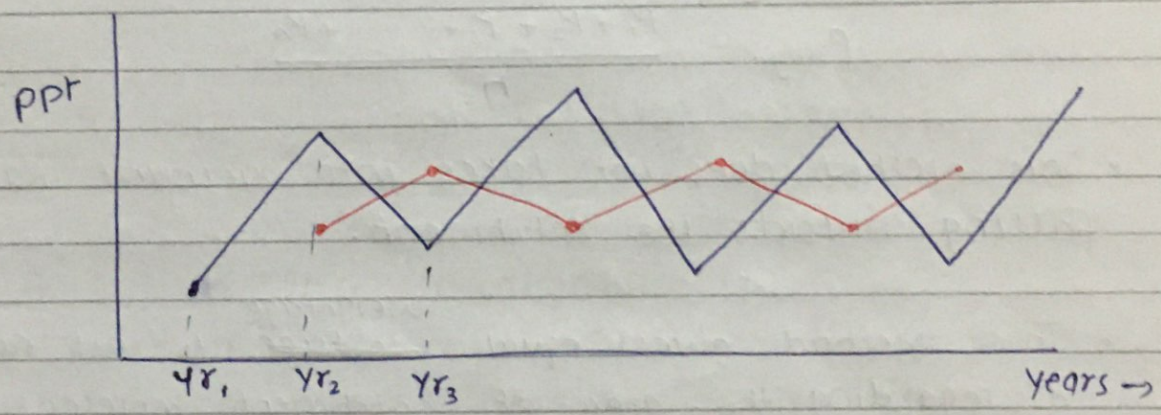


$$(2-\phi)\times 2 = h_{eff}$$

$$(2-\phi)\times 2 = 1.2$$

$$\phi = 1.4 \text{ cm/hr}$$

(iii) Moving Average Method :-



It is used to determine the trend of Rainfall

NOTE: 7 year moving avg. will be flatter than 5 year moving avg. which will be flatter than 3 year moving avg. and so on.

Yr	ppt	3 Y moving	5 Year moving
2001	P_1		
2002	P_2	$(P_1+P_2+P_3)/3$	
3	P_3	$(P_2+P_3+P_4)/3$	$(P_1+P_2+P_3+P_4+P_5)/5$
4	P_4	$(P_3+P_4+P_5)/3$	$(P_2+P_3+P_4+P_5+P_6)/5$
5	P_5	⋮	⋮
6	P_6	⋮	⋮
7	P_7	⋮	⋮
8	P_8	⋮	⋮
9	P_9		

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Calculation of avg. depth of Rainfall :-

The point Rainfall values of Diff. Raingauges are converted to avg. Rainfall depth using following Methods

① Airthmetic Mean Method :-

For a catchment with n Rain Gauges inside the catchment with Rainfall values $P_1, P_2, --- P_n$, the avg. Rainfall over the catchment is calculated as –

$$P_{avg.} = \frac{P_1 + P_2 + P_3 + --- + P_n}{n}$$

- This Method does not takes into account Rain Gauges falling outside the catchment.
- This Method gives equal ~~weitage~~ weightage to all rain gauges disregarding the area of catchment represented by each Rain Gauge
- Least Accurate, although quickest
- Yields Fairly correct results for plain Areas with uniform Rainfall distribution

② Thiessen - polygon Method :

This Method calculates weightage avg. Rainfall considering the Areas of catchment represented by each Rain Gauge both falling inside as well as outside the Catchment.

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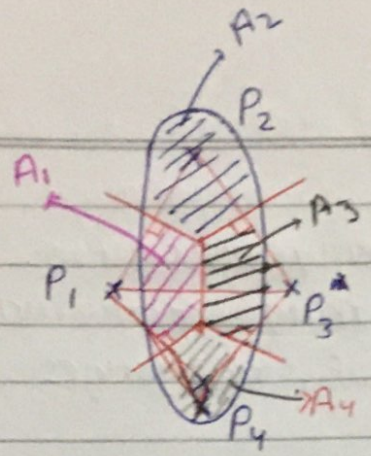
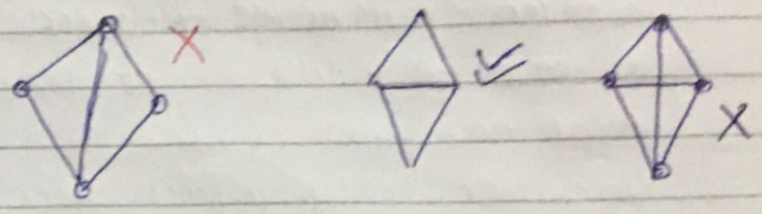


- We prefer connecting shorter side
- At vertex there should be a Rain Gauge



$$P_{avg.} = \frac{P_1A_1 + P_2A_2 + P_3A_3 + P_4A_4}{A_1 + A_2 + A_3 + A_4}$$

$$= \frac{\Sigma P_i A_i}{\Sigma A_i}$$

$$= \Sigma P_i W_i \quad ; \quad W_i = \frac{A_i}{\Sigma A_i}$$

$P_i \rightarrow$ ppt of ith rain Gauge

$A_i \rightarrow$ Area of catchment represented by i^{th} Rain Gauge (Also known as Area of influence)

$\Sigma A_i \rightarrow$ Total Area of catchment

$W_i \rightarrow$ weight of ith Rain Gauge

Thiessen polygon :-

It is constructed by perpendicularly bisected sides of Tringle Formed by joining the raingauge through straight lines

Area of Influence :-

It is the Area of thiessen polygon falling inside the catchment.

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- Weightage avg. is calculated
- polygon needs to be constructed only once for a given raingauge network requiring reconstruction in case of addition or removal of raingauges

Disadvantage

- variation of Rainfall due to elevation differences are not taken into account
  Hence reliable only for plain Areas

Que. For the Data given below calculate avg. Rainfall using
a) airthmetic Mean Method
b) Theissen polygon Method

St^n	Rainfall	Area of influence	I → inside, O → outside
1	45 ✓	8	I
2	25	10	O
3	40	50	O
4	50 ✓	35	I
5	35 ✓	20	I

a) $\frac{45+50+35}{3} = 43.33$ cm ($P_{avg.}$) { considering the raingauges inside the catchment only.

b) $$P_{avg.} = \frac{45\times8 + 25\times10 + 40\times50 + 50\times35 + 35\times20}{8+10+50+35+20} = 41.14 \text{ cm}$$

Que. For the catchment shown below calculate the avg depth of Rainfall by theissen polygon Method. the catchment consist of equilateral Tringle and square of side 4 km with 6 Raingauges (5 @ vertices and 6th @ center of square) as shown below:-

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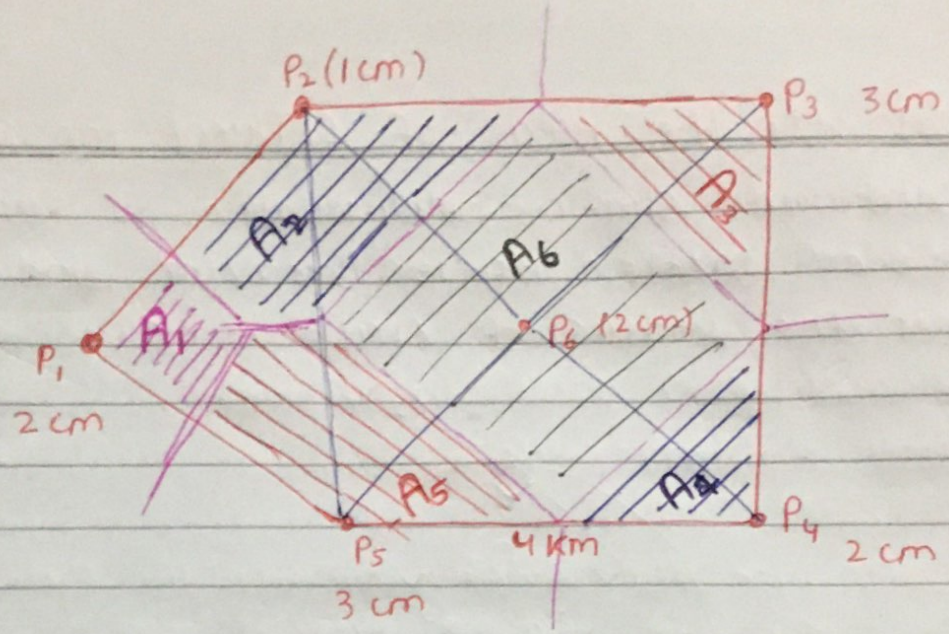

$$A_6 = \frac{a^2}{2}$$

$$A_3 = A_4 = \frac{1}{4}\left(a^2 - \frac{a^2}{2}\right) = \frac{a^2}{8}$$

$$A_1 = \frac{a^2}{4\sqrt{3}} \qquad \left\{ A = \frac{\frac{\sqrt{3}}{4}a^2}{3} \right.$$

$$A_2 = A_5 = \frac{a^2}{4\sqrt{3}} + \frac{a^2}{8}$$

$$P_{avg.} = \frac{P_1A_1 + P_2A_2 + P_3A_3 + P_4A_4 + P_5A_5 + P_6A_6}{A_1 + A_2 + A_3 + A_4 + A_5 + A_6}$$

$$P_{avg.} = \frac{2\times\frac{4^2}{4\sqrt{3}} + 1\times\left(\frac{4^2}{4\sqrt{3}} + \frac{4^2}{8}\right) + 3\times\frac{4^2}{8} + 2\times\frac{4^2}{8} + 3\times\left(\frac{4^2}{4\sqrt{3}} + \frac{4^2}{8}\right)\times 2\times\frac{4^2}{2}}{\frac{\sqrt{3}}{4}(4)^2 + 4^2}$$

$$P_{avg.} = 2.08 \text{ cm}.$$

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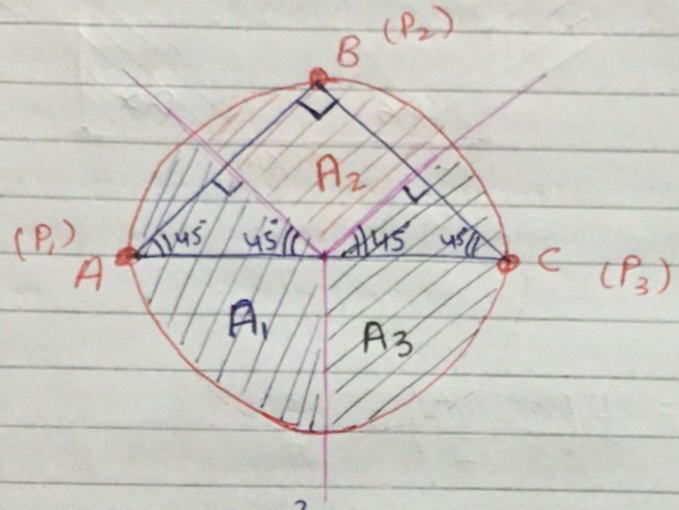
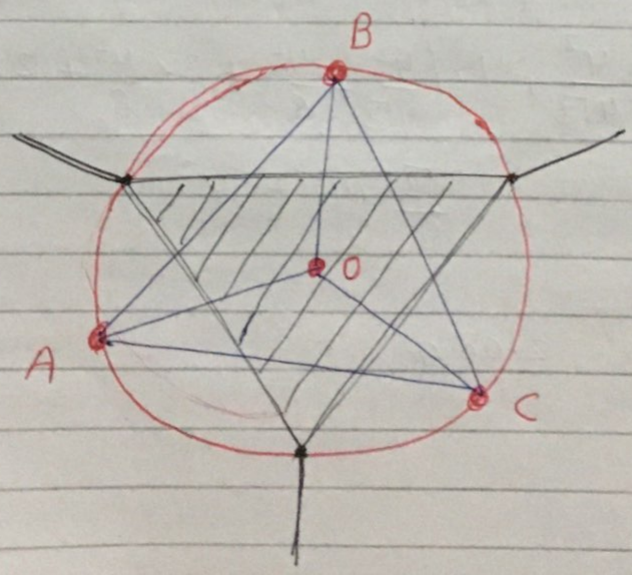
Que. Calculate the Areas of Influent for the circular catchment shown below with 3 Raingauges A, B, C, where AC is Diameter and AB = BC. Let the Radius of circle be r.

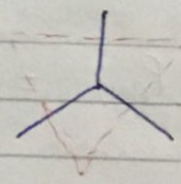


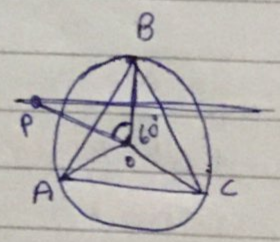
$$A_2 = \frac{\pi r^2}{4}$$

$$A_1 = A_3 = \frac{3\pi r^2}{8}$$

Que. Draw the thiessen polygon for circular catchment shown below with 4 raingauges 3 @ vertices of an equilateral Tringle and 4th @ center of circle







$$\cos 60^\circ = \frac{r/2}{OP}$$

$$OP = r$$

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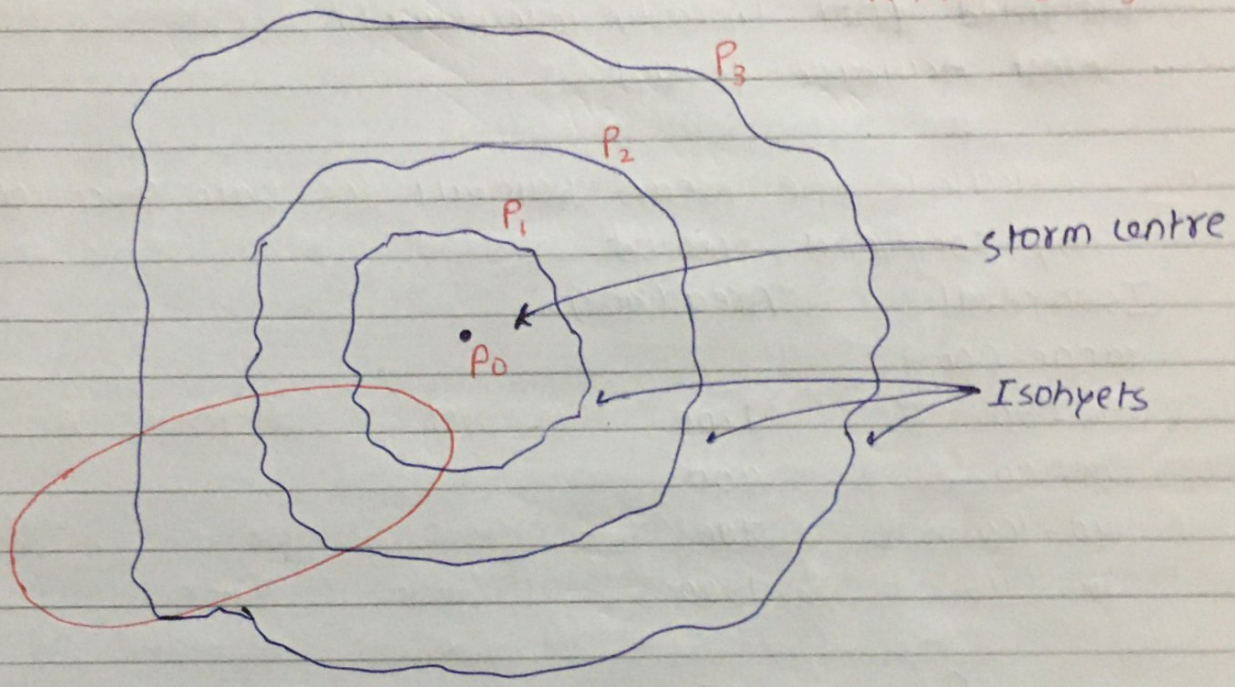
• Isohyet बनाते time उसे linear interpolation से नहीं pass करते उस point के पास से pass करवायेंगे जहां से point का elevation पास हो

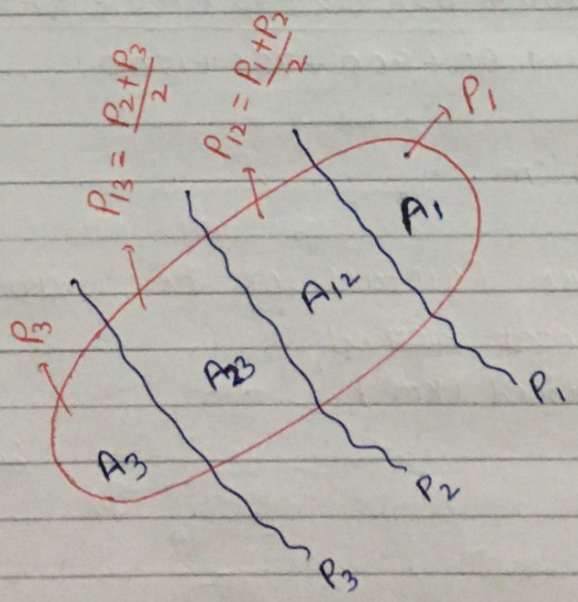
(conside elevation, not only distance)

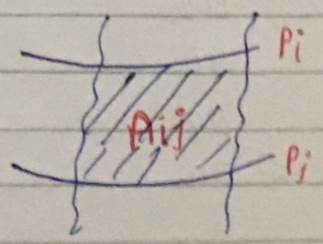
③ Isohyetal Method

Isohyets are imaginary lines joining points of equal rainfall depth

$P_0 > P_1 > P_2 > P_3$







$$P_{avg.} = \frac{P_1 A_1 + P_{12} A_{12} + P_{23} A_{23} + P_3 A_3}{A_1 + A_{12} + A_{23} + A_3}$$

$$P_{avg.} = \frac{\sum P_{ij} A_{ij}}{\sum A_{ij}} \qquad \text{where } P_{ij} = \frac{P_i + P_j}{2}$$

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- Weightage avg. is calculated
- Rainfall variation due to elevation differences is taken into account as the Isohyetals are orographically weighted (not linearly interpolated)
- Most Accurate Method

Que Calculate the Mean Rainfall for Data given below by Isohyetal Method.

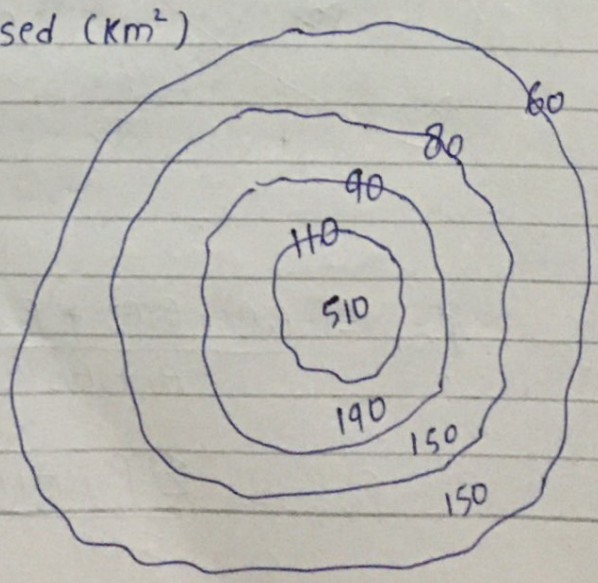
Isohyetal range (cm)	Area (km^2)
70 - 80	1200
80 - 90	900
90 - 100	700
100 - 115	1500

$$P_{avg.} = \frac{75 \times 1200 + 85 \times 900 + 95 \times 700 + 107.5 \times 1500}{1200 + 900 + 700 + 1500}$$

$$P_{avg.} = 91.686 \text{ cm}$$

Que Calculate the Mean Rainfall for the Data given below using Isohyetal Method

Isohyets (cm)	Area enclosed (km^2)
110	510
90	700
80	850
60	1000
	1200 (it given)



then Add 60 x 200 Also

$P_1 = 110$, $A_1 = 510$

$P_{12} = 100$ $A_{12} = 190$

$P_{23} = 85$, $A_{23} = 150$

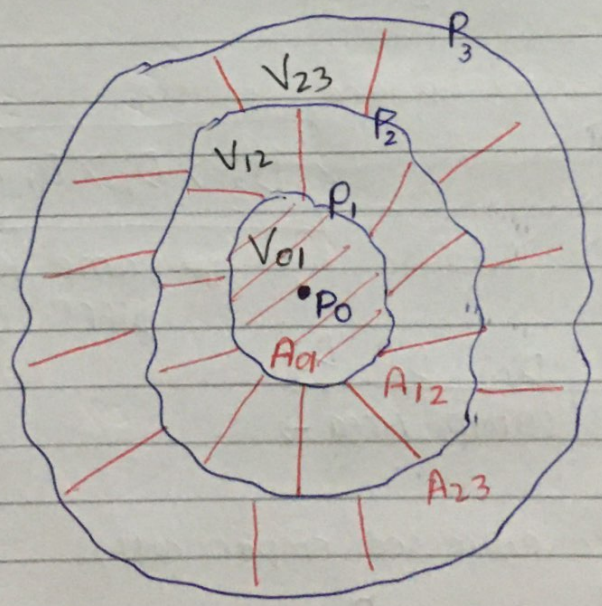
$P_3 = 70$ $A_3 = 150$

$$P_{avg.} = \frac{110 \times 510 + 100 \times 190 + 85 \times 150 + 70 \times 150}{1000}$$

$$P_{avg.} = 98.35 \text{ cm}$$

Depth - Area Duration Curve (DAD curve)

It is the plot of Max. avg. depth of Rainfall against cummulative Area for a perticular duration of Rainfall.



Steps

1. Isohyetal Map is prepared
2. Area b/w Isohyets is calculated using planimeter
3. Area b/w Isohyets is multiplied by the avg. of corresponding Isohyets to find the vol. of Rainfall in that Area

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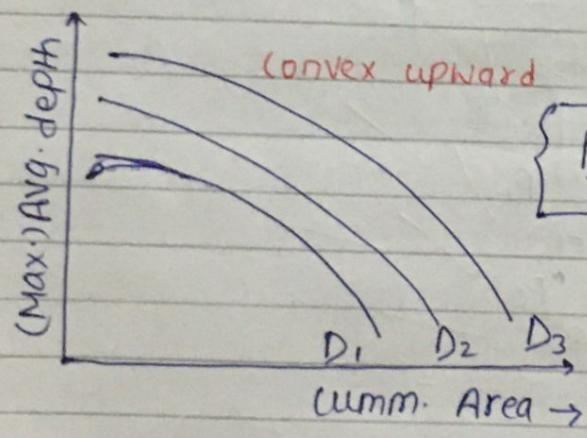
④ Cummulative volume and cummulative Area is calculated

⑤ cummulative volume is devided by cummulative Area to obtain the avg. depth of Rainfall over the cummulative Area

⑥ Avg. depth is plotted againt cummulative Area to obtain DAD curve.

Isohyetal Range (cm)	Area (km^2)	Vol^m	Cumm. Vol^m	Cum. Area	Avg depth
$P_0 - P_1$	A_{01}	$V_{01} = A_{01} \times \frac{P_0+P_1}{2}$	V_{01}	A_{01}	V_{01}/A_{01}
$P_1 - P_2$	A_{12}	$V_{12} = A_{12} \times \frac{P_1+P_2}{2}$	$V_{01}+V_{12}$	$A_{01}+A_{12}$	$V_{01}+V_{12}/A_{01}+A_{12}$
$P_2 - P_3$	A_{23}	$V_{23} = A_{23} \times \frac{P_2+P_3}{2}$	$V_{01}+V_{12}+V_{23}$	$A_{01}+A_{12}+A_{23}$	$\frac{V_{01}+V_{12}+V_{23}}{A_{01}+A_{12}+A_{23}}$

Decreasing



$\boxed{D_3 > D_2 > D_1}$

* same Rainfall Diff^n Duration

DAD curve is also expressed emperically

$$\bar{P} = P_0 e^{-KA^n}$$

$\bar{P}$ is avg. depth of Rainfall over an Area A km^2
P_0 is ppt @ storm centre
K and n are constants

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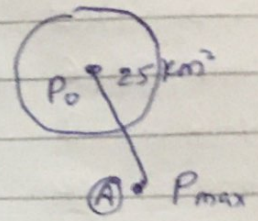
ppt @ storm centre P_0 is calculated from the approximation that the Max. Rainfall recorded by a RainGauge is the avg. Rainfall depth over an Area of 25 km^2

$$P_{max.} = P_0 e^{-k 25^n}$$



MAX. DAD CURVE

When DAD curve is constructed taking the Max. of various Rainfalls of a perticular Duration on Diff^n areas of the catchment, it is known as Max. DAD

(The term DAD and Max. DAD are interchangeably used)

Both implies max. Avg. depth over cumm. Area.

Que. A catchment is devided into three zones I, II, III with Area of each zone 100, 3000, 3000 km^2 respectively. The cumm. Avg. Rainfall in diff^n zones for a perticular Rainfall is given as below:- Draw Max. avg. Depth vs Area curve for Rainfall Durations of 2h & 4h, that is 2 hr Max DAD and 4 h max. DAD

	cum. Rain (mm)		
Time (hr.)	I	II	III
2	8	5	2
4	14	11	7
6	23	18	12
8	35	28	21
10	48	40	30

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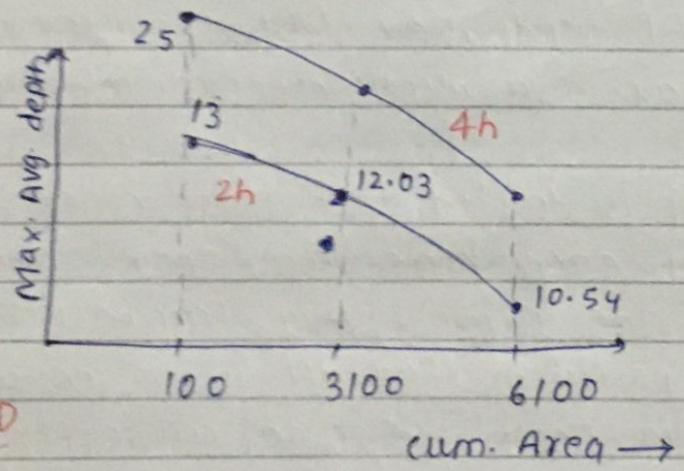
2h-Max DAD

Various 2h rainfall

Time	I	II	III
0-2	8	5	2
2-4	6	6	5
4-6	9	7	5
6-8	12	10	9
8-10	13	12	9

Zone	Area (km^2)	depth(mm)	Vol^m	Cum. Vol^m	Cum. Area	Avg. depth
I	100	13	1300	1300	100	13
II	3000	12	36000	37300	3100	12.03
III	3000	9	27000	64300	6100	10.54



4h-max DAD

Various 4h rainfall

Time	I	II	III
0-4	14	11	7
2-6	15	13	10
4-8	21	17	14
6-10	25	22	18

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Frequency of Rainfall :- In the design of hydraulic structure we need to know what is the chance of occurance or exceedence of a perticular Rainfall or what is the probability that a perticular rainfall will not be exceeded during the design life of the structure to analyse the safety of the structure against the Rainfall

The probability of occurance or exceedence (P) of a Rainfall in a year is calculated emperically as

Method	P
* 1) Weibull	$\frac{m}{N+1}$
2) California	$\frac{m}{N}$
3) Hazen	$\frac{m-0.5}{N}$

Where P is probability of occurance or exceedence of an event in an year having Rank m

N → Total no. of Datas available

m → Rank of an event obtained by arranging all the Datas in decreasing order and ranking them from 1 (To the Greatest event) to N (lowest event)

For example: Let the available datas be –

10, 6, 5, 7, 4, 5, 8, 9, 5 cm. N = 9

Decreasing ↓

Data	Rank (m)	P
10	1	10%
9	2	20%
8	3	30%
7	4	40%
6	5	50%
5	6	60%
5	7	70%
5	8	80%
4	9	90%

$P = \frac{m}{N+1} \times 100$

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60% When rainfall is advantageous as in hydroelectric power plant

80% When rainfall has adverse effect

Reoccurance interval or return period (frequency)

It is the avg. time after which a perticular Rainfall is expected to be equalled or exceeded

$$\boxed{T = \frac{1}{p}}$$

once the probability of occurance or exceedence has obtained other probabilities can also be found out such as

(i) probability of Non-occurance in a year

$$q = 1 - p$$

(ii) probability of occurance or exceedence of an event r times in n years $= {}^{n}C_{r}\, p^{r} q^{n-r}$

(iii) probability of non-occurance in n years $= q^{n}$

$\{r = 0$

for n = design life q^{n} gives the probability of the structure not falling in it's design life that is reliability

(iv) probability of occurance or exceedence at least once in n years $= 1 - q^{n}$

for n = design life $1 - q^{n}$ gives the probability of the structure falling in it's design life i.e Risk

Risk + Reliability = 1

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Que: At Kolkata Rainfall depth is 20 cm in 12 hr has a return period of 50 years. the probability of 12 hr Rainfall of Magnitude $\geq$ 20 cm occuring in the Next year is ?

$T = 50$ Yrs.

$$P = \frac{1}{T} = \frac{1}{50} = 2\% = 0.02$$

Que: The probability of 10 cm Rain in 1 hr occuring in Bombay was found to be 1/60. What is the probability that a 1 hr Rain of Magnitude 10 cm or larger will occure once in 30 Years?

r times in n Years $= {}^{n}C_{r}\, p^{r} q^{n-r}$

$$\begin{cases} P = 1/60 \\ q = 59/60 = 1-P \end{cases}$$

$$= {}^{30}C_{1} (1/60)^{1} (59/60)^{30-1}$$

$$= 0.307$$

Que: A 6hr Rainfall of 6 cm occuring @ place X was found to have a return period of 40 years. The probability that a 6 hr Rainfall of Magnitude $\geq$ 6 cm occuring at least once in 20 Years at X is ?

$T = 40$

$P = 1/40$

$q = 39/40$

probability of occurance at least once in 20 Years $= 1 - q^{n}$

$$= 1 - \left(\frac{39}{40}\right)^{20}$$

$$= 0.397$$

Que: For a return period of 40 years calculate the probability of occurance or exceedence at least twice in 20 years.

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$T = 40$ yrs

$P = 1/40$, $q = 39/40$

$$P_{(0)} + P_{(1)} + \underbrace{P_{(2)} + \cdots + P_{(20)}}_{?} = 1$$

$$P_{\text{zero times in 20 years}} + P_{\text{once in 20 years}} + P_{\text{at least twice}} = 1$$

$$P_{\text{at lease twice}} = 1 - q^n - {}^{20}C_1\, p^1 q^{20-1}$$

$$= 1 - \left(\frac{39}{40}\right)^{20} - {}^{20}C_1 \left(\frac{1}{40}\right)\left(\frac{39}{40}\right)^{19}$$

$$= 0.088$$

Que. A hydraulic structure is to be designed for a 30 year return period storm under the Max. Risk of failure of 30%. The Max. design life of the structure = ? (integeral value)

$$1 - q^n \leq 0.3$$

$T = 30$

$P = 1/30$, $q = 29/30$

$$1 - \left(\frac{29}{30}\right)^n \leq 0.3$$

$$\left(\frac{29}{30}\right)^n \geq 0.7$$

$$n \ln\left(\frac{29}{30}\right) \geq \ln 0.7$$

$$n \leq \frac{\ln 0.7}{\ln(29/30)}$$

$$n \leq 10.52$$

(10) Ans

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Que. Annual ppt value at a place having 70 yrs of Record is given below. Calculate the probability of having a) an annual Rainfall ≥ 120 cm in a Year

b) Two successive Years in which annual Rainfall is ≥ 140 cm

(That is ≥ 140 Twice in two years)

c) An annual Rainfall < 60 cm in a Year

Range (cm)	No. of Years
<60	6
60 – 79.9	6
80 – 99.9	22
100 – 119.9	25
120 – 139.9	8
≥ 140	3

~~≥ 120~~

Data	Rank	p
≥ 140	3 (1-3)	
120-139.9	11 (4-11)	
100-119.9	36 (12-36)	
80-99.9	58 (37-58)	
60-79.9	64 (59-64)	
<60	70 (65-70)	

a) ≥ 120 is same as $\geq$ (120-139.9)

$$p = \frac{11}{70+1} = 0.1549$$

b) $$p_{\geq 140 \text{ in a year}} = \frac{3}{70+1} = 0.042$$

$$p_{(\text{Twice in 2 years})} = {}^2C_2\, p^2 q^{2-2} = p^2 = 1.78 \times 10^{-3}$$

c) $$p_{(\geq 60 \text{ in a year})} = \frac{64}{71}$$

$$P_{(<60)} + P_{(\geq 60)} = 1$$

$$P(<60) = 1 - \frac{64}{71} = \frac{7}{71} = 0.09859$$

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Dependability = 100P

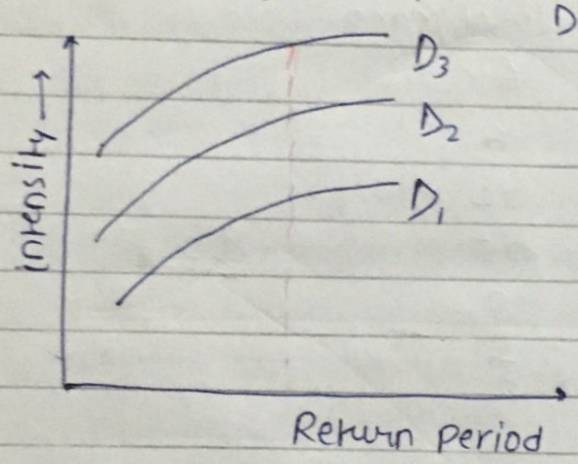
INTENSITY - Duration - Frequency curve (IDF)

Plot of intensity vs Return period for a perticular duration of Rainfall or intensity vs Duration for a perticular return period of Rainfall.

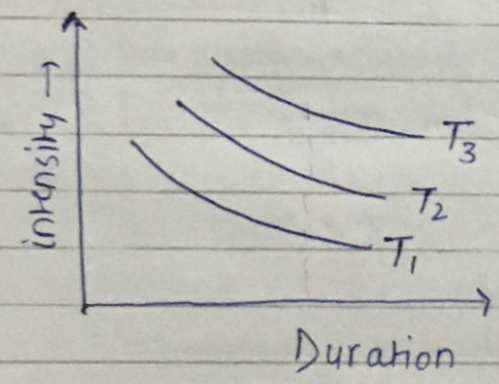
Observed fact:-

① Intensity $\propto$ Return period

② Intensity $\propto \dfrac{1}{\text{Duration of Rainfall}}$



$D_3 < D_2 < D_1$



$T_3 > T_2 > T_1$

IDF curve is also expressed as emperically

$$i = \frac{KT^x}{(D+a)^n}$$

Sherman

$$i = \frac{\alpha}{(D+a)^n}$$

IDF curve / Eq^n is used to calculate design discharge at the catchment outlet

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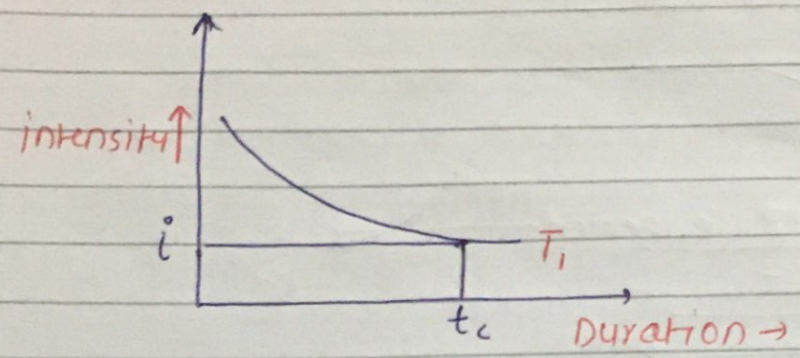
Estimation of Design Discharge :-



$$i = \frac{KT^x}{(t_c+a)^n}$$

$Q = CiA$

Design Discharge is calculated using Rational formula

$Q = CiA$

$Q \rightarrow$ Design Discharge

$A \rightarrow$ catchment Area

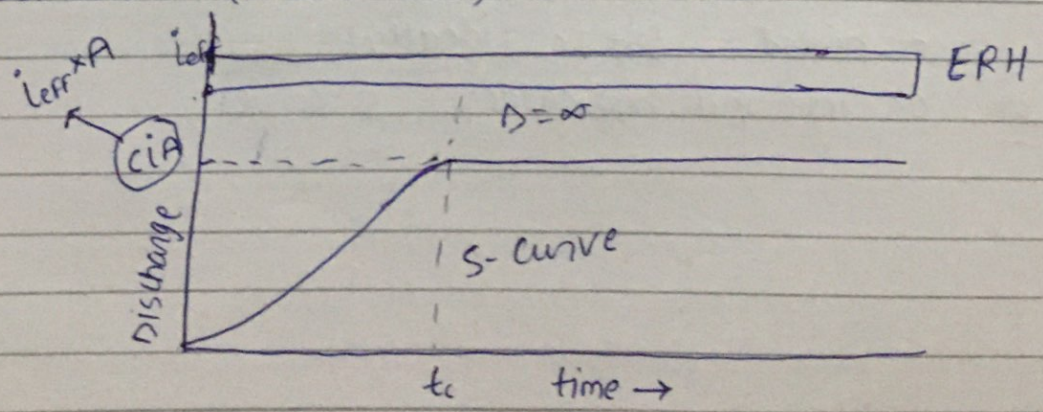
$C \rightarrow$ Runoff coefficient or impermeability factor

$i \rightarrow$ intensity of Rainfall calculated from IDF curve / EQ^n corresponding to concentration time of the catchment

$$C = \frac{\text{Runoff (SR)}}{\text{Rainfall}}$$

Consentration Time t_c :-

It is the time taken by water to travel from hydrolically remotest point (fartherest point with respect to time not distance) to the catchment outlet. When Rainfall duration equals or exceeds concentration time of the catchment, max. possible discharge is obtained ($Q = CiA$)



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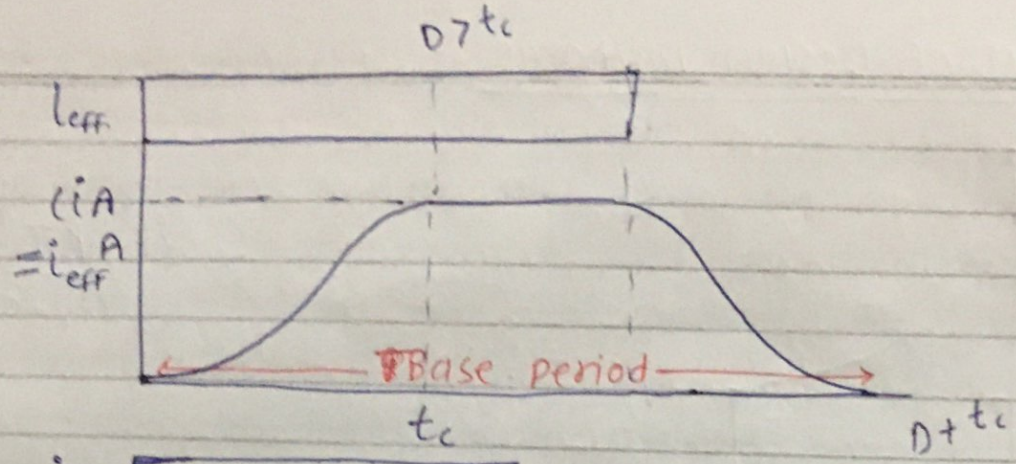

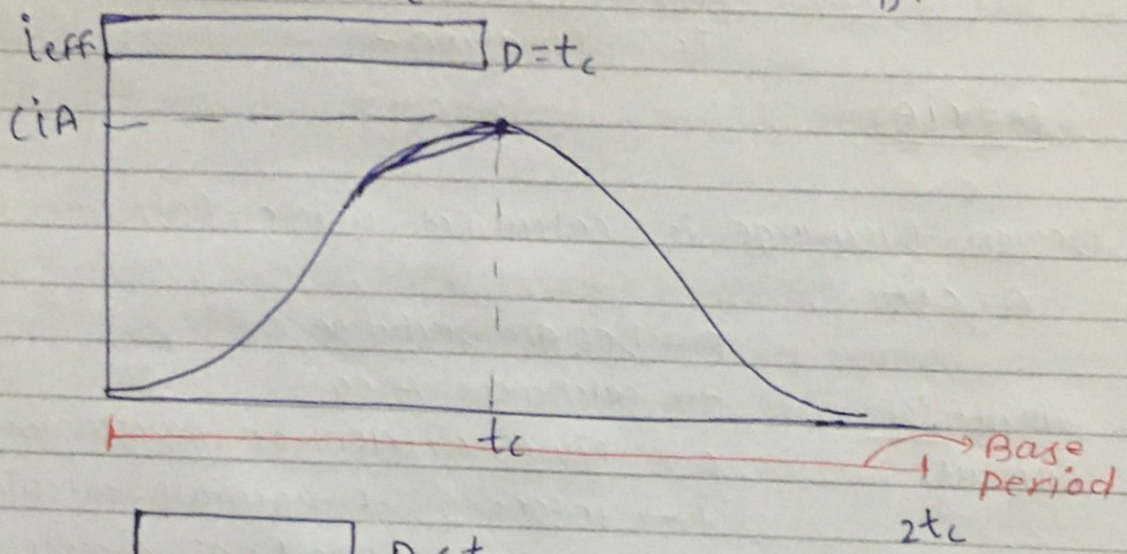

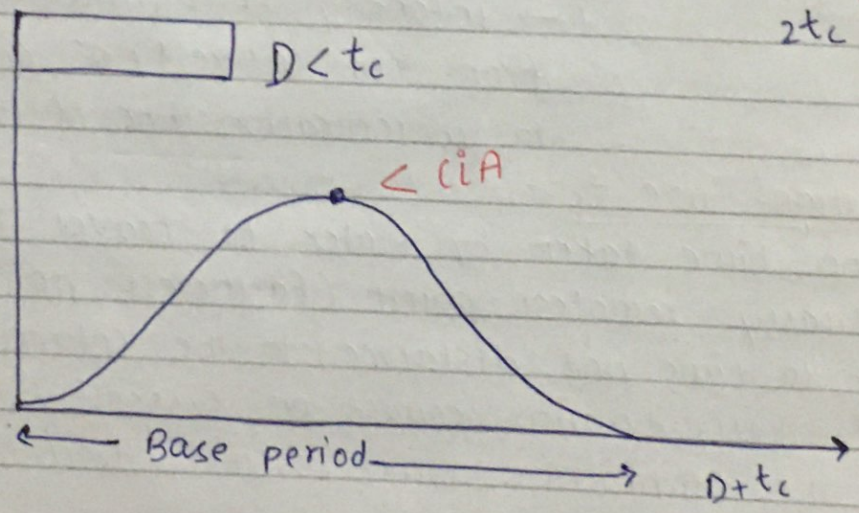

NOTE: Base period = D + tc (ideally)

Ex: as in unit hydrograph, S-curve.

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Probable Max. PPt :- (PMP)

- It is defined as the greatest or the extreme Rainfall of a given duration that is physically possible over a Basin or station
- PMP is estimated statistically as (By Gumbel's Method)

$$PMP = \bar{P} + K\sigma_{n-1}$$

Where $\bar{P}$ is mean of annual Data series
K is frequency factor
σ_{n-1} is standard deviation of annual Rainfall series

PMP is used for the design of large hydraulic structure for which there is virtually NO probability of failure

Standard project storm :- (SPS)

It is the greatest storm that is expected with out Modifying the rainfall Data for most favourable condition
For Design of Major and intermediate importance structures SPS is used.

NOTE

Max. Rainfall that is expected in any part of the World is emperically expressed as –

$$\boxed{P = 42.16\, D^{0.475}}$$

P is ppt in cm
D is Duration in hour

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Station Year Method :-

When long records are not available the records at two or more stations are combined to get one long record for the purpose of reoccurance interval calculations. For this we treat one year record of n rain gauge stations as n years record of one rain gauge station.
This Method is known as station year method

Que. The Number of times the storm of intensity 6 cm/hr was equalled or exceeded in 3 diff^n Rain Gauge st^n in a Region were 4, 2, 5 for periods of 36, 25 and 48 years. Find the reoccurance ~~of~~ interval of 6 cm/hr storm in that Area by station year method.

$$p = \frac{4+2+5}{36+25+48}$$

$$T = 1/p = 9.91 \text{ years}$$

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3. Abstraction From ppt

processes like evaporation, transpiration, interception, Depression storage and infiltration is collectively known as abstraction from ppt.

Evaporation

(1) Factors affecting Evaporation

1) Vapor pressure :- Rate of Evaporation is proportional to the diffⁿ b/w saturated vapor pressure e_s and actual Vapor pressure e_a

$$E_L = C(e_s - e_a) \quad \text{Dalton's law}$$

2) Wind :-
Rate of evaporation ↑ with wind speed upto critical velocity of wind (the velocity @ which wind removes all the vapor particles). Increasing wind speed above critical velocity does not increases rate of evaporation

3) Atmospheric pressure :-
Other factors remaining same rate of evaporation ↓ with increase in atmospheric pressure

4) Salidity :- Rate of Evaporation ↓ by about 1% for every 1‰ ↑ in salidity

Ex: River water will have a higher Rate of evaporation than saline sea water.

5) Temprature :- Other factors remaining same Rate of Evaporation ↑ with Temp ↑

6) Size of Water body :- (Shallow Vs Deep) : In case of shallow water bodies due to low temp gradient we get Max Rate of Evapo. during summer season

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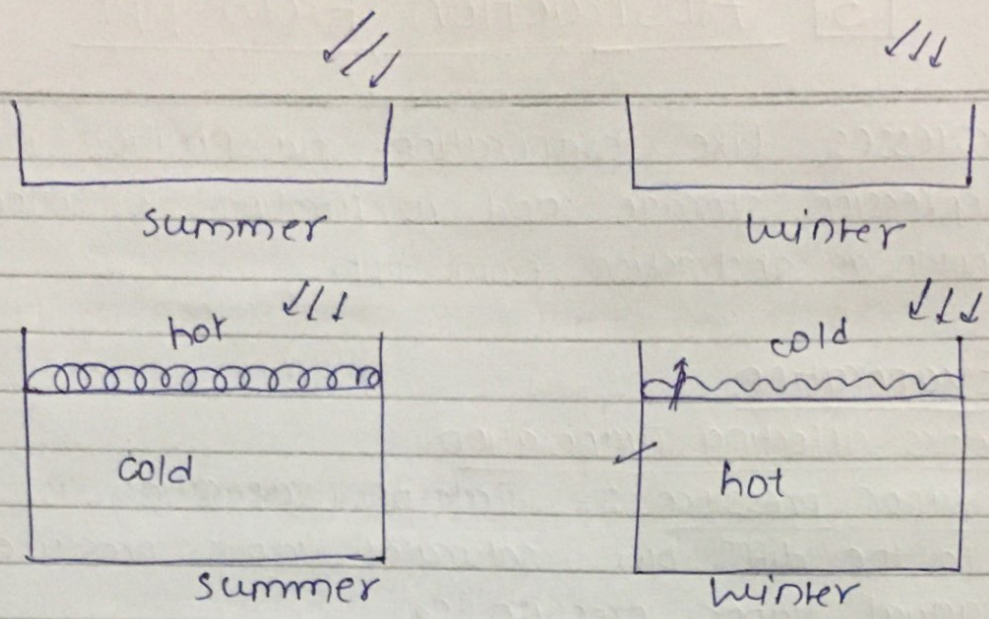

In case of Deep Water bodies due to temp. Gradient and release of stored energy in winter season we gets Max. Rate of evaporation during winter season

⑦ Shape of Water body :- (small vs large) :-

In large water bodies due to creation of vapor blanket over the water surface we get lower Rate of evaporation as compare to small water bodies

② Measurement of Evaporation :-

① using Evaporimeter :-

lake evapo. = pan cofficient x pan evaporation (Rate)

Type of Evaporimeter	pan cofficient
1) US class A pan	0.7
2) Colorado sunken pan	0.78
3) US Geological survay Floating pan	0.8
4) Indian standard pan	0.8

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2) Using emperical EQ^n :- (Mayer's EQ^n)

As per mayer's EQ^n we have

$$E_L = K_m (e_s - e_a)\left(1 + \frac{U_9}{16}\right)$$

Where, E_L → Rate of Evapo in mm/day

e_s, e_a → saturated vapor pressure and actual vapor pressure in (mm of Hg)

U_9 → wind velocity in kmph 9 m above Ground

K_m → Mayer's constant

0.36 → for large water body

0.5 → for small water body

Relation b/w elevation and wind velocity

$$U_h = C h^{1/7} \quad \rightarrow 1/7 \text{ power law}$$

U_h → wind speed h m above Ground (kmph)

3) Analytical Method

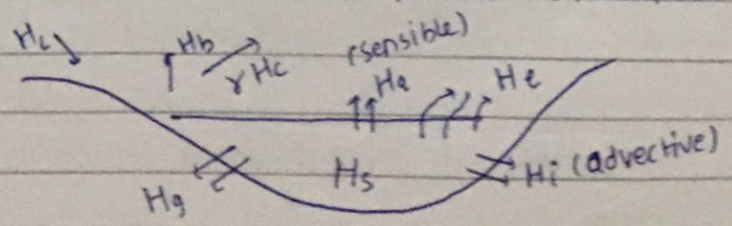
a) Water budget Method :- This Method is based on mass conservation law

$$P - R - E - T - G = S$$

$$E = P - R - T - G - S$$

b) Energy Balance Method :- Based on law of conservation of energy

In this Method energy used for evaporation is calculated and from that Rate of Evapo. is estimated.



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$$H_n = H_c - rH_c - H_b$$

$H_n \rightarrow$ net heat absorbed

$H_c \rightarrow$ Total heat incident

$rH_c \rightarrow$ Heat reflected ($r \rightarrow$ reflective constant)

$H_b \rightarrow$ back radiation

From energy balance we have

$$H_n = H_a + H_e + H_g + H_i + H_s$$

$H_a \rightarrow$ Sensible Heat transfer, it is calculated using Boween's Ratio β, where $\beta = \frac{H_a}{H_e}$ ← latent

or Heat of Conduction

$$H_a = \beta H_e$$

$H_e \rightarrow$ Heat used in evaporation

$$H_e = \rho L E_L$$

$\rho \rightarrow$ density of water

$L \rightarrow$ latent heat of vaporization

$E_L \rightarrow$ Rate of Evaporation in mm/day

$H_g \rightarrow$ Heat flux into the ground

$H_i \rightarrow$ Heat loss through seepage (advective)

$H_s \rightarrow$ Heat stored

$$H_n = \beta H_e + H_e + H_g + H_i + H_s$$

$$(1+\beta) H_e = H_n - H_g - H_i - H_s$$

$$E_L = \frac{H_n - H_g - H_i - H_s}{\rho L (1+\beta)}$$

NOTE: All energy terms are in calories / mm^2 / day

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c) Mass transfer Method :- This Method is based on theories of turbulent mass transfer in a boundary layer

③ Reduction of Evaporation :-

1) By providing a mechanical surface cover over the water surface.
2) By opting for Deep reservoirs against shallow reservoirs for storing same vol^m of water
(In case of Deep reservoir rate of Evapo. will be more But total evaporation will be less)
3) By adding chemicals like steryle alcohol (hexadeconol) or steryle alcohol (octadecanol) which forms a protective Film over the water surface thus preventing Evoporation.

Evapotranspiration :-

Evaporation when studied in conjugation with transpiration is called evapotranspiration (ET) or consumptive use (Cu)

Field capacity :- It is the Max. amount of water soil can retain against the Forces of Gravity.

@ Field Capacity soil contains capillary water and hygroscopic water

permanent wilting point :- It is the moisture content of soil @ which even though soil contains some moisture plants are unable to extract it

@ this moisture content of soil plants permanently wilts and does not recovers even when the moisture content of soil is raised

@ PWP soil contains Hygroscopic water

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Potential Evapotranspiration (PET) :-

When sufficient moisture is always available to completly meet the needs of plant the resulting Evapotranspiration is called PET

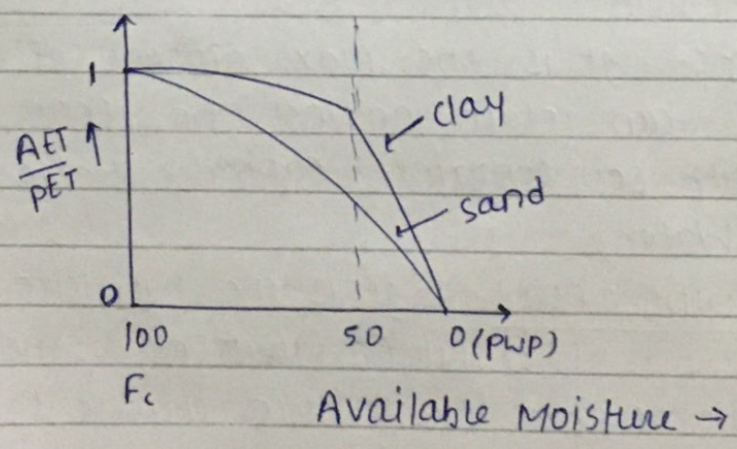
Actual Evapotranspiration (AET) :-

Evapotranspiration taking place @ existing or pre condition is called actual Evapotranspiration.

- AET varies from PET (@ F_c) to 0 (zero)(@ PWP)

$$(PWP)\ 0 \leq \frac{AET}{PET} \leq 1\ (F_c)$$

NOTE The Ratio $\frac{AET}{PET}$ remains approximatly = 1 upto 50% loss in available moisture for clay soils



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Measurement of AET :-

A) **Field plot Method :-** Water budget is used on a field plot to obtain rate of ET

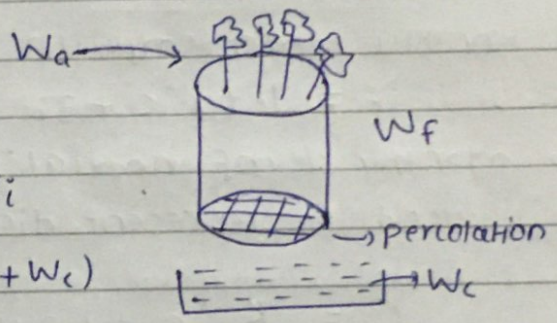
$$P - R - (ET) = S$$

$$ET = P - R - S$$

S → change in moisture content of soil

R → Runoff , P → ppt / irrigation (avoiding Deep percolation)

B) **using Lysimeter :-**



$$ET = (W_i + W_a) - (W_f + W_c)$$

W_i → initial wt of lysimeter + soil + crop

W_a → water added

W_f → final wt of lysimeter + soil + crop

W_c → wt of water collected @ Bottom.

ET → wt of water Evapotranspirated

Measurement of PET :-

PET is Measured using Peonman's Eq^n (Analytical Eq^n) and Blaney criddle Eq^n (emperical Eq^n)

① **penmann's Eq^n**

It is Based on combination of energy Balance Method and Mass transfer approach

$$\boxed{PET = \frac{AH_n + E_a\gamma}{A + \gamma}}$$

PET → Daily PET in mm/day , γ → const. = 0.49 mm of Hg /°c

A → Slop of saturation Vapor pressure Vs temp. curv @ Mean Air temp. in mm of Hg /°c

H_n → Net radiation in (mm) of evaporable water

E_a → parameter including Wind Speed and saturation deficit

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② **Blaney - Criddle Eqⁿ :-**

In this method ET is related to solar radiation which is represented by monthly mean temp. and day-time hour

PET for a cropping season is given as

$$\boxed{E_r = 2.54\, K \sum \frac{P_h T_F}{100}}$$

$E_r \rightarrow$ PET of a cropping season in cm

$K \rightarrow$ constant depending upon crop, locality etc.

$T_F \rightarrow$ Monthly mean temp. in °F

$$°F = 1.8 \times °C + 32$$

$P_h \rightarrow$ Monthy % of annual day time hour

Σ is used to represent diffⁿ months of cropping season

$$C_u = \frac{K}{40} P_h \left(1.8(°C) + 32\right)$$

NOTE: $\boxed{\text{phytometer}}$ is a device used to measure only transpiration

$\boxed{\text{Isopliths}}$ are imaginary lines joining points of equal ET

Interception :- It comprises of 3 component

a) Interception loss :- Actual loss through Evaporation

b) Stemp fall] → Finally reaches ground, Not considered as loss

c) Through fall] (पत्तियों पर जो पानी है। wt. ज्यादा होने से direct गिर जाएगा जमीन पर)

NOTE Coniferous trees (Has Needle shape leaves) show more interception loss than Deciduous trees (Has broad leaves)

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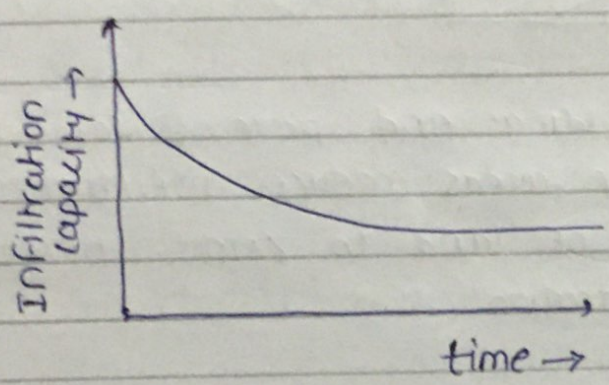
NOTE: Interception loss and depression storage is collectively known as initial loss

INFILTRATION :-

① **Infiltration capacity :-** It is defined as the max. rate @ which soil can absorb water at a perticular instance of time

It decays exponentially with time



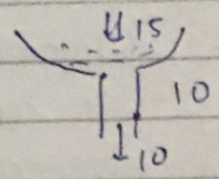
Factors affecting Infiltration :-

① **Rainfall :-** When the intensity of Rainfall i is less than infiltration capacity f_c then the actual Rate of infiltration $(f) = i$

if the $i > f_c$, then actual Rate of infiltration $= f_c$

$i < f_c$; $f = i$

$i > f_c$; $f = f_c$ → ponding condition



② **soil charecteristic :-**

- Infiltration Rate Dry soil > Moist soil
- permeable sandy soil > clayey soil

clayey soils are rendered virtually impermeable due to Rain drop compaction effect

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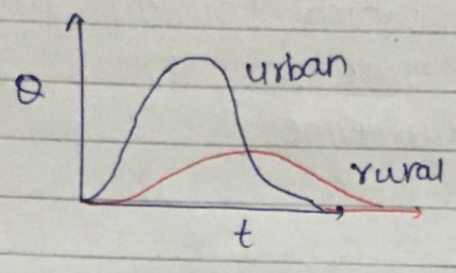
③ Surface cover / Rural Area / Vegetation :-

Vegetation ↑ infiltration by
a) retarding surface flow velocity (more time to infiltrate)
b) shielding the soil from direct Rain drop compaction effect



NOTE: spread of buildings and pave surfaces (Highways) in case of urban areas reduces infiltration capacity of various patches of land to zero, thereby reducing total infiltration

Measurement of Infiltration :-

1) Emperical EQⁿ
2) Infiltration indices
3) Infiltrometer

① **Emperical EQⁿ** various emperical eqⁿ to estimate infiltration are as follows:-

a) Green amp
b) Huggin's Monka
c) soil conservation practice
d) Horton's curve

Horton's curve :- It is an approximate (idealised) infiltration capacity of soil used to express infiltration within the duration of Rainfall.

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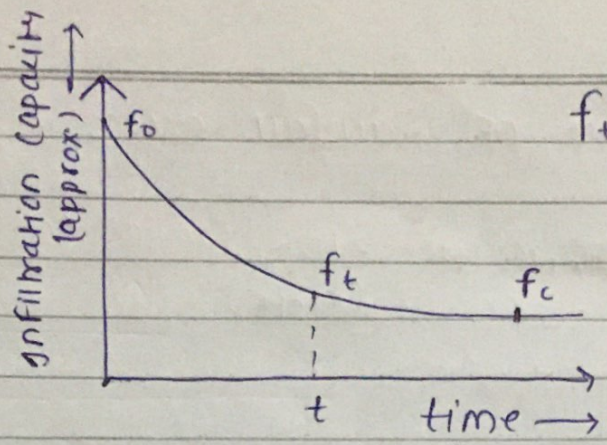


$$f_t = f_c + (f_0 - f_c)e^{-\alpha t}$$

$f_t \rightarrow$ cm/hr or mm/hr

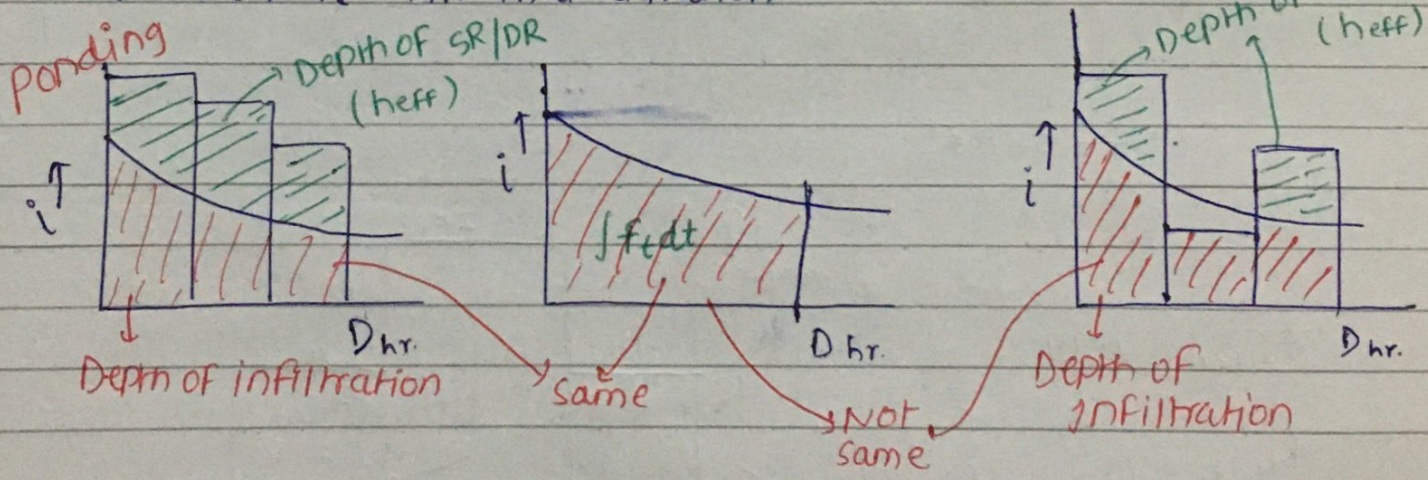
Where f_0 is infiltration capacity @ $t=0$
f_c is infiltration capacity @ full saturation
f_t is infiltration capacity @ any time t
$\alpha \rightarrow$ constant, unit $\rightarrow$ 1/time

Imp. Statements

① Area of hyetograph above horton's curve gives depth of SR / DR (h_{eff})

② Area of hyetograph below horton's curve gives depth of infiltration

③ Area of horton's curve (Integration of $\int f_t dt$) gives depth of infiltration only when ponding condition exists i.e $i > f_c$ in that duration



Que During a 3h storm event it was observed that all abstractions other than infiltration were negligible. The Rainfall was idealised as three 1h-Rainfall of intensity 10, 20, 10 mm/hr respectively. The infiltration was idealised as Horton's curve $f = 6.8 + 8.7e^{-t}$ where f is in mm/hr and t is in hour.

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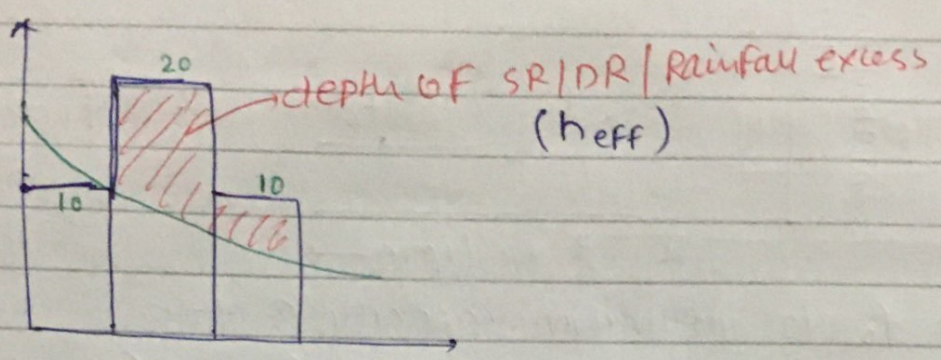
Calculate the depth of Rainfall excess in (mm).

Sol^n



$f|_{t=0} = 15.5$

$f|_{t=1} = 10$

$f|_{t=2} = 7.97$

$f|_{t=3} = 7.25$

$h_{eff.} = (20\times1 + 10\times1) - \int_1^3 6.8\times8.7e^{-t}dt$

$h_{eff.} = 13.63$ mm.

Directly.

$10 + \int_1^3 6.8 + 8.7e^{-t}$

Que. For the rainfall as in previous question calculate the depth of infiltration when the horton curve is idealised as $f = 5 + 8e^{-t}$

$f = 5 + 8e^{-t}$

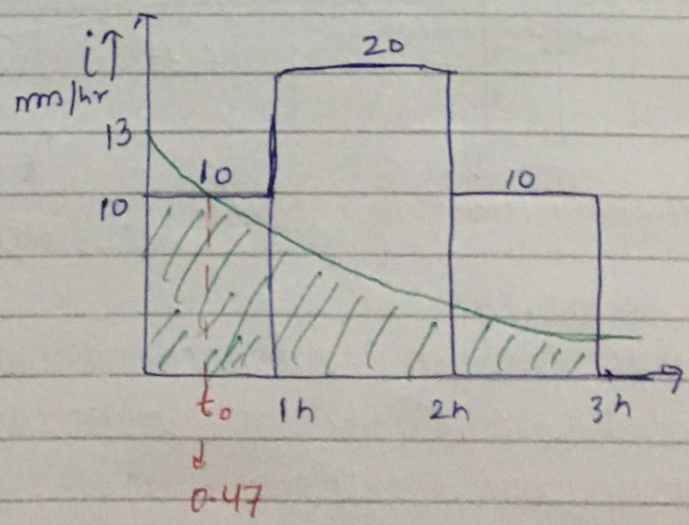
$f|_{t=0} = 13$

$f|_{t=1} = 7.94$

$f|_{t=t_0} = 10$

$\Rightarrow 5 + 8e^{-t_0} = 10$

$\Rightarrow t_0 = 0.47$ hr



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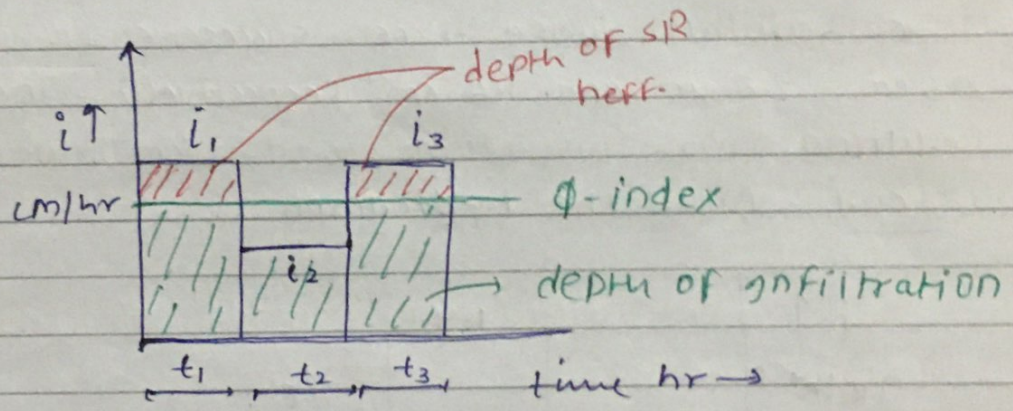
SR के साथ eff. आता है।

$$\text{Depth of infiltration} = 0.47 \times 10 + 3\int_{0.47}^{3} (5 + 8e^{-t})\,dt$$

$$= 21.955 \text{ mm}$$

② INFiltration indices :-

① **ϕ-index :-** It is an approximate infiltration capacity so defined that the Area of hyetograph above ϕ-index given depth of SR / DR and the Area of hyetograph below ϕ-index approx. gives depth of Infiltration.



$t_1 + t_3$ is effective duration of Rainfall.

in interval t_1, eff. intensity = $i_1 - \phi$

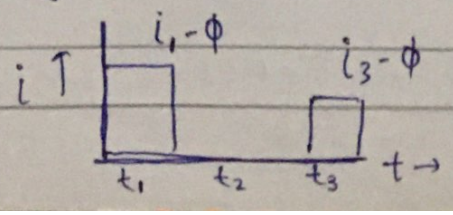
——— t_2 –, eff. intensity = 0

——— t_3, eff. intensity, $i_3 - \phi$

portion of hyetograph above ϕ index is called ERH effective Rainfall hyetograph.

Area of ERH = Depth of SR/DR or h_{eff}

NOTE. $\phi \times t$ will give the depth of Infiltration only When ponding cond. exists. i.e $(i > \phi)$ in that duration



$t_1 + t_3 \Rightarrow$ eff. duration

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$$\phi = \left(\frac{P-R}{t}\right)$$

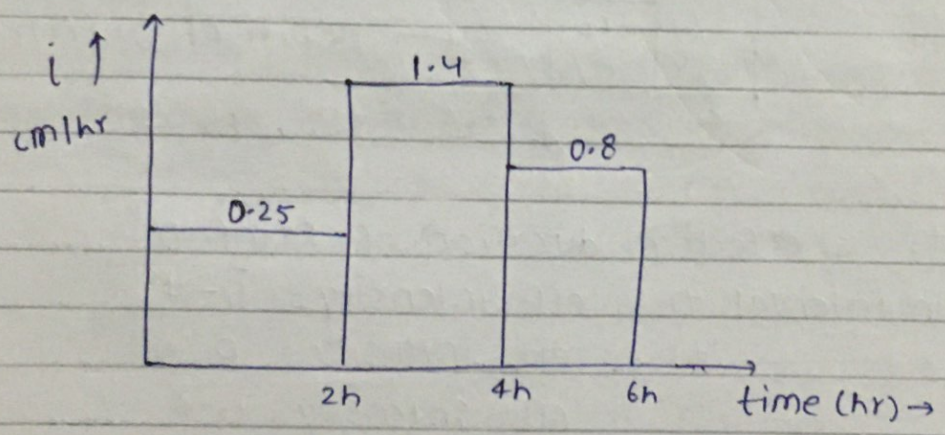
P → ppt in eff. duration
R → Runoff
t → effective duration

This statement is correct but we can not calculate ϕ from this because we need ϕ to get t.

Calculation of ϕ:-

Que. The Rainfall during three successive 2h period are 0.5 cm, 2.8 cm and 1.6 cm Respectively. The SR Resulting from this storm is 3.2 cm. Calculate ϕ index.

Sol^n Step-1 Draw the hyetograph



Step-2 Assume ϕ lower than the lowest intensity of rainfall

$$\phi^{ist} \leq i_{lowest}$$ ***

Calculate ϕ from it's defination

$$\phi = \left(\frac{P-R}{t}\right)_{eff.\ duration}$$

$$\phi = \frac{(0.5+2.8+1.6) - 3.2}{6} = 0.283 \text{ cm/h}$$

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If the calculated ϕ supports our assumption, it is correct o/w move to Next step

$0.283 \not< 0.25$, Hence Not correct

Step-3:

Assume ϕ, Greater than the one calculated above and less than equal to the Next higher intensity of Rainfall

$0.283 < \phi^{2nd} \leq 0.8$ *** ध्यान रखें!

again calculate ϕ from it's defination and keep repeating this step until the calculated ϕ supports our assumption.

$$\phi = \frac{(2.8 + 1.6) - 3.2}{4} = 0.3$$

Que. A 6h storm with hourly intensities of 7, 18, 25, 17, 11, and 3 mm/hr produces a SR of 39 mm. ϕ index = ? (mm/hr)

Assume $\phi \leq 3$

$$\phi = \frac{(7+18+25+17+11+3) - 39}{6} = 7 \times$$

Assume $7 < \phi \leq 11$ $\quad \phi = \frac{(18+25+17+11) - 39}{4} = 8$ mm/hr ✓

② W-index :- It is a refined version of ϕ-index. It calculates Infiltration by excluding minor losses from Total loss.

$$W\text{-index} = \left(\frac{P - R - \text{minor losses}}{t} \right)_{\text{eff. duration}}$$

$$\simeq \left(\frac{P - R - \text{Minor losses}}{t} \right)_{\text{Total duration}}$$

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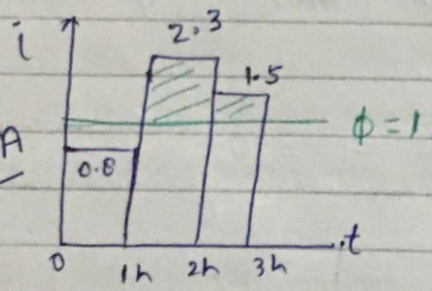
$\boxed{W\text{-index} \leq \phi\text{-index}}$

Que. An Isolated 3h storm over a Basin occurs as given below. Estimate the Runoff from the catchment due to the storm.

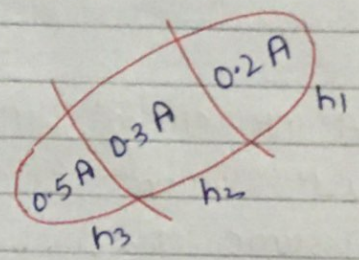
% of Catchment	ϕ-index cm/hr	Rainfall (cm) 1st hr	2nd hr	3rd hr
20	1	0.8	2.3	1.5
30	0.75	0.7	2.1	1
50	0.5	1	2.5	0.8

Soln.

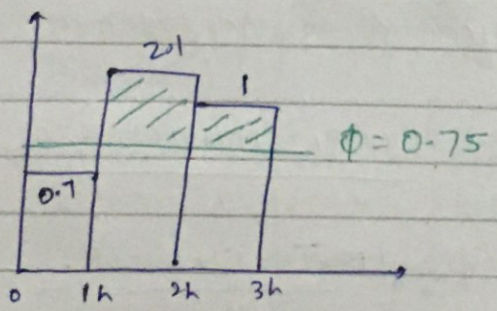
for 0.2A



$h_1 = (2.3-1) + (1.5-1) = 1.8$ cm

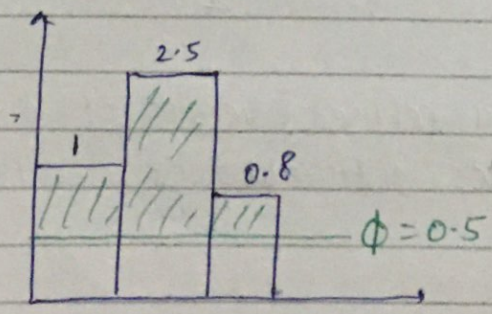


for 0.3A



$h_2 = 1.6$ cm

for 0.5A



$h_3 = 2.8$ cm

Avg. eff. depth $= 0.2 \times 1.8 + 0.3 \times 1.6 + 0.5 \times 2.8 = 2.24$ cm

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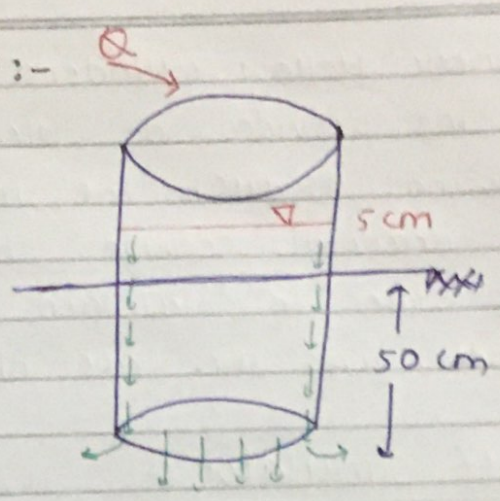
③ Infiltrometer :-

Types of Infiltrometer :-

① Flooding Type :-

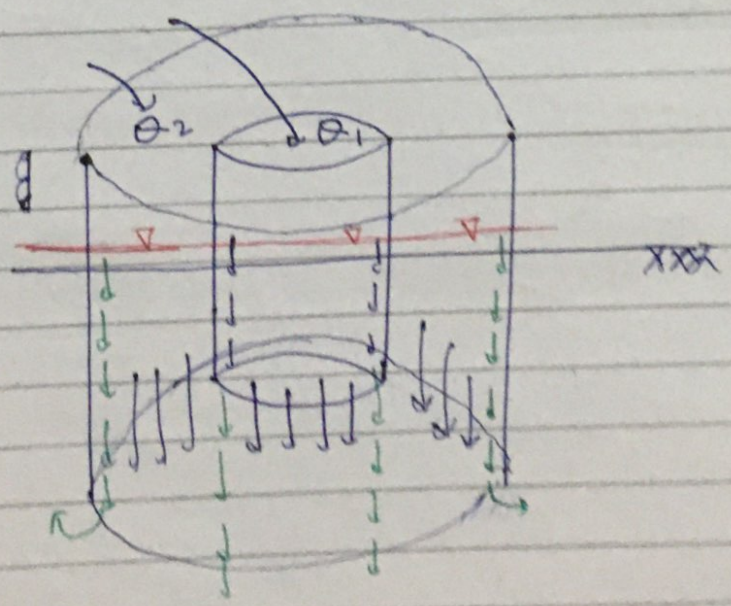


- It consist of a hollow metallic cylinder driven 50 cm into the Ground.

- The required rate of Addition of Water to Maintain a 5cm Water level inside the cylinder approx. gives Rate of Infiltration

The Rate of Infiltration reported by this infiltrometer is more than actual Infiltration due to horizontal seepage of unfiltered water through the bottom circumference of the cylinder

- This abnormality is rectified by using double ring infiltrometer



Infiltration is only due to verticle permeability of soil

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Two concentric Hollow cylinders are used.
The outer ring provides a water jacket around the water flowing through the inner cylinder thus preventing horizontal seepage. The required rate of addition of water to maintain a water level in the inner cylinder gives the rate of infiltration.

② Rainfall simulator :-

In this Method a 2m x 4m land plot with nozzles along the length is selected and arrangement is done to measure the surface Runoff from the plot. A known quantity of water is passed through Nozzles which rises upto a height of 2m and falls on plot, using water budget Rate of infiltration is estimated, this gives the Most Accurate (Also lowest in value) Measurement of infiltration due to simulation of Raindrop compaction effect.

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4 SURFACE - RUNOFF

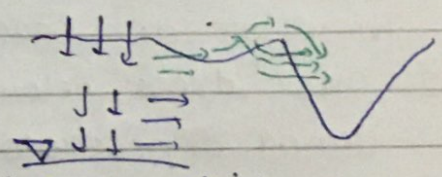
Based upon time delay b/w ppt and RUNOFF (total volume of water exiting the outlet) RUNOFF is classified as — 1) Direct Runoff
2) Base Flow

① **Direct RUNOFF :-** It is the part of ppt which reaches the stream outlet soon after the Rainfall.

- It comprises of 1) Surface Runoff → It is the part of ppt which reaches the stream outlet flowing over the surface
  It has two components
  a) over land flow — low velocity, laminar flow
  b) channel flow — high velocity, Turbulent flow

  2) Direct ppt → It is the part of Rainfall falling directly onto river surface.

  3) prompt interflow →



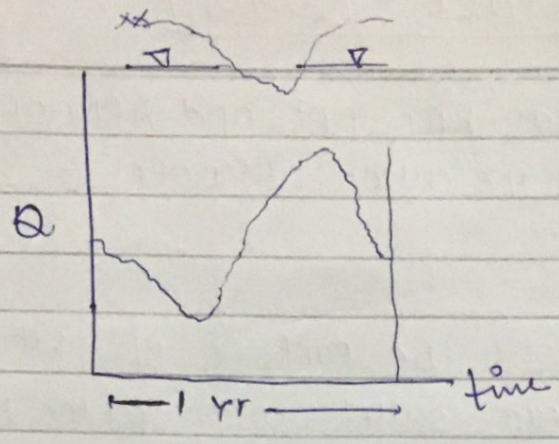
② **Base Flow :-** It is the part of Rainfall which reaches the stream outlet long ~~after~~ time after the Rainfall.

It comprises of — 1) Ground water table contribution through seepage
2) Delayed interflow

Basically Base Flow is Delayed Ground water Flow {Combination of 1) and 2)}

Runoff charecteristic of Streams :-

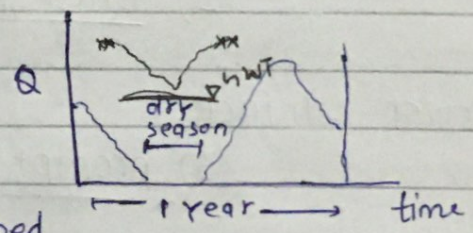
① **perinial streams:-** Such streams contains discharge (some) throughout the year



② Ground water Table remains ~~the~~ above the river bed throughout the year

③ Such streams contains significant contribution from Base flow

② Intermittent streams :-

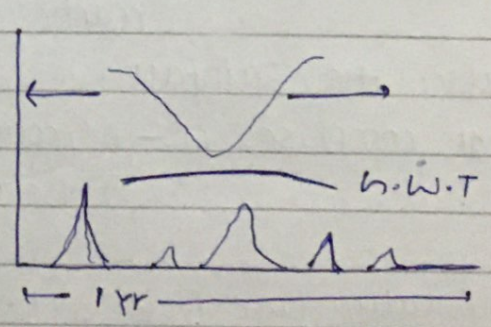


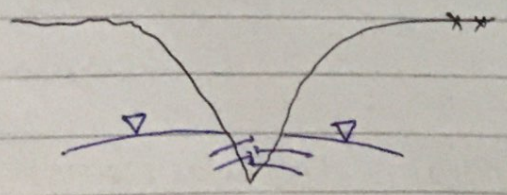
① Such river rise up for some time in dry season

② GWT drops below the river bed during dry season

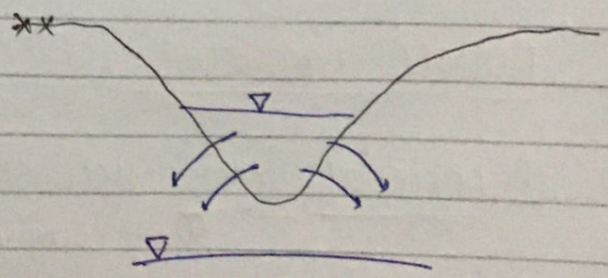
③ such stream contains less contribution from Base flow.

③ Ephemeral streams :-





Effluent streams



Influent streams.

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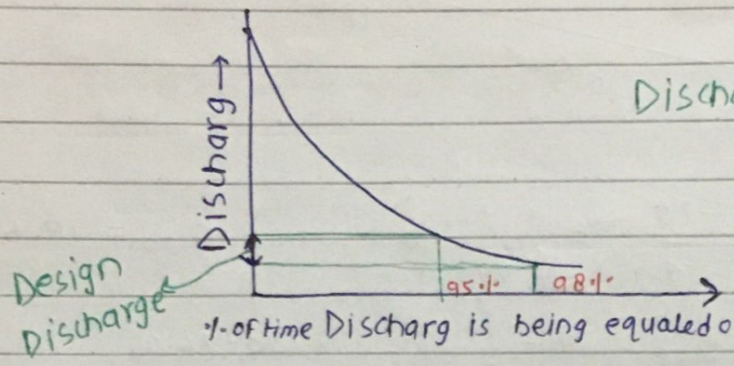
Flow Duration Curve:- It is a plot of discharge against % of time and discharge is being equaled or exceeded that is dependability.



Discharge $\propto \frac{1}{\text{Dependability}}$

Dependability $= \frac{m}{N+1}$

जितने छोटे period के लिए curve बनायेंगे उतना ज्यादा Fluctuations.

Dependability is % of time discharge is being equaled or exceeded and it is calculated by using Wiebull's eqn $\left(\frac{m}{N+1}\right)$

Que. The observed mean monthly flow of a stream for 1 year period from June to May is given below. Find the flow which can be expected 80% of time or 80% dependable flow. Calculate the dependability of discharge 20 m^3/sec and 40 m^3/sec. If runoff river plant is proposed @ the catchment outlet what power can be produced using an available head of 5m, Efficiency of production 80% for 90% dependable flow.

Month	Q (m^3/s)		Rank (m)	Dependability $\left(\frac{m}{N+1}\right)$
June	18	46	1	
July	20	42	2	40 → 15.38%
Aug	46	37	3	23.08%
Sep	42	33	4	
Oct	37	30	5	
Nov.	30	23	6	
Dec.	33	21	7	
Jan	23	20	8	
Feb	20	20	9	
March	21	19	10	76.9% — 80%
April	19	18	11	84.61% — $Q_{90} = 11$
May	8	8	12	92.3%

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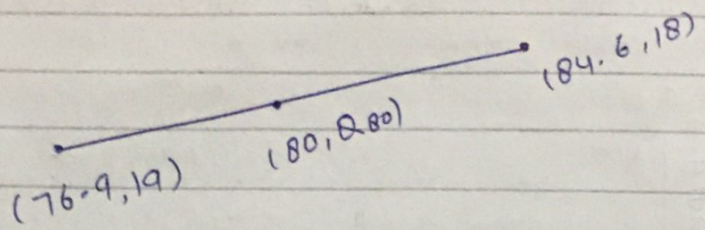
$$0.8 = \frac{m}{N+1} = \frac{m}{12+1} \quad , m = 10.4$$



$$\frac{Q_{80} - 19}{80 - 76.9} = \frac{18-19}{84.6 - 76.9} \Rightarrow Q_{80} = 18.6$$

$$Q_{80} = 19 - \frac{19-18}{84.61 - 76.9} \times (80 - 76.9)$$

$$= 18.6 \ m^3/sec$$

as this is advantageous so we'll take lower m for 20 m^3/s , $m = 8$

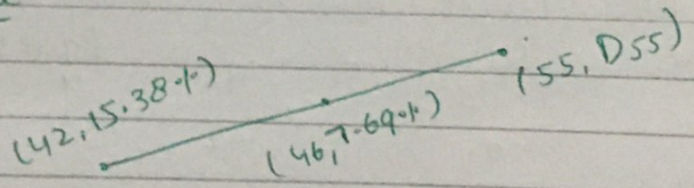
$$\text{Dependability} = \frac{8}{12+1} \times 100 = 61.5\ \%$$

$$D_{40} = 15.38 + \frac{23.08 - 15.38}{42 - 37} \times (42 - 40)$$

$$D_{40} = 18.46\ \%$$

if we want to calculate D for 55 m^3/s

extra polate



$$D_{55} = 7.69 - \frac{15.38 - 7.69}{46 - 42} \times (55 - 46)$$

$$D_{55} = -9.7 \quad \text{(Not dependent)}$$

$$P = \frac{E}{t} = \frac{mgh}{t} = \frac{V \times \rho g H}{t} = Q \gamma H$$

$$\boxed{P = \eta \gamma Q_{90\%} H}$$

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$$Q_{90} = 18 - \frac{18-8}{92.3-84.61} \times (90-84.61)$$

$$Q_{90} = 10.99 \simeq 11 \ m^3/sec.$$

$\eta = 80\%$

$$P = 0.8 \times 9810 \times 11 \times 5$$

$$P = 431.69 \ KN$$

* When no. of Data is large instead of working on Discreate value classify the Data into class interval and the depandability is assumed for the mid-point of the class interval taking the Higher Rank

Ex:

Range	No. of times	mid	Q	Rank
0-10	10	25	20-30	22 (1-22)
10-20	25	15	10-20	47 (23-47)
20-30	22	5	0-10	57 (48-57)

$$D_{25} = \frac{22}{57+1} \times 100$$

$$D_{15} = \frac{47}{57+1} \times 100$$

$$D_5 = \frac{57}{57+1} \times 100$$

To calculate D_{20} interpolate b/w 25 m^3/s – 15 m^3/sec.

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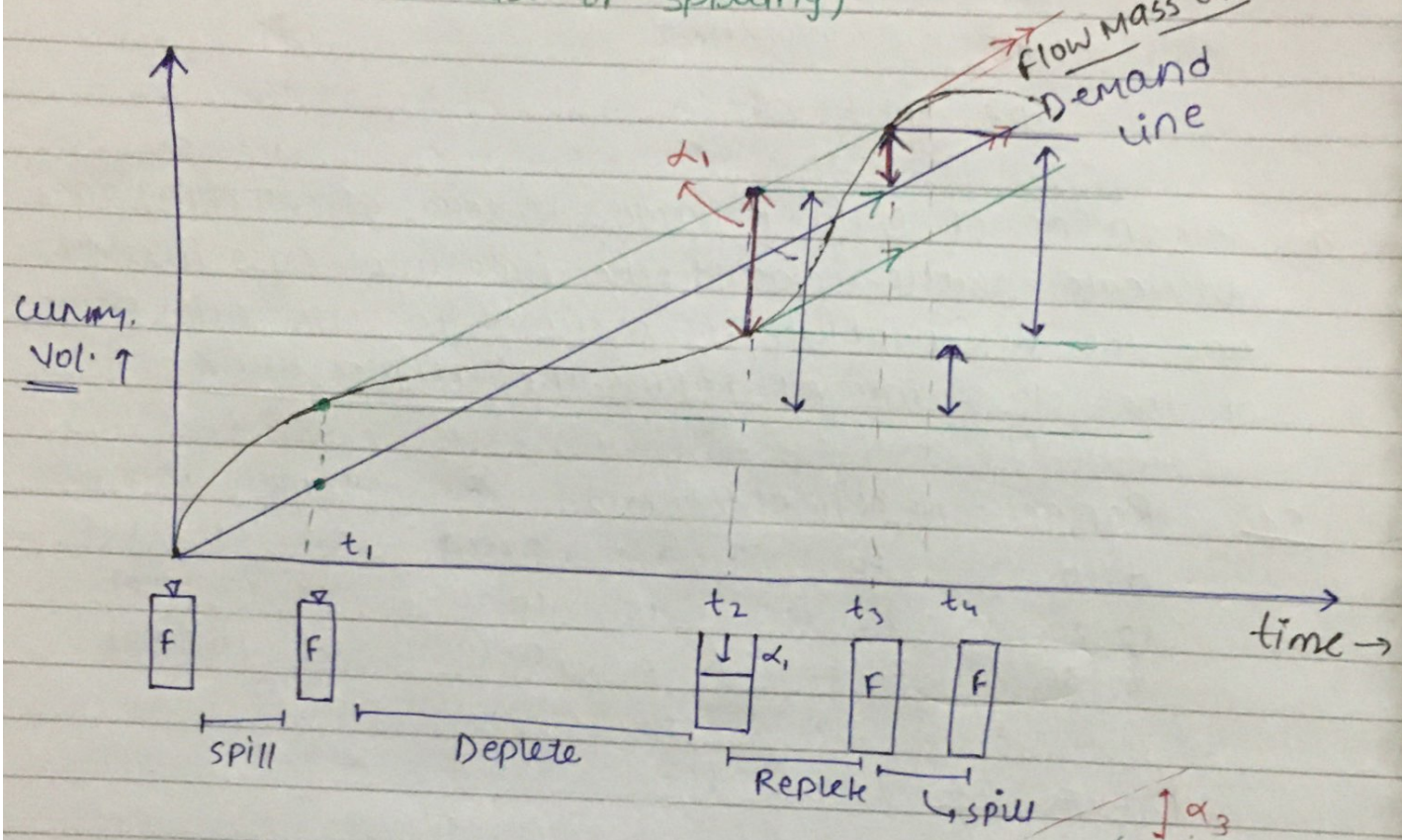
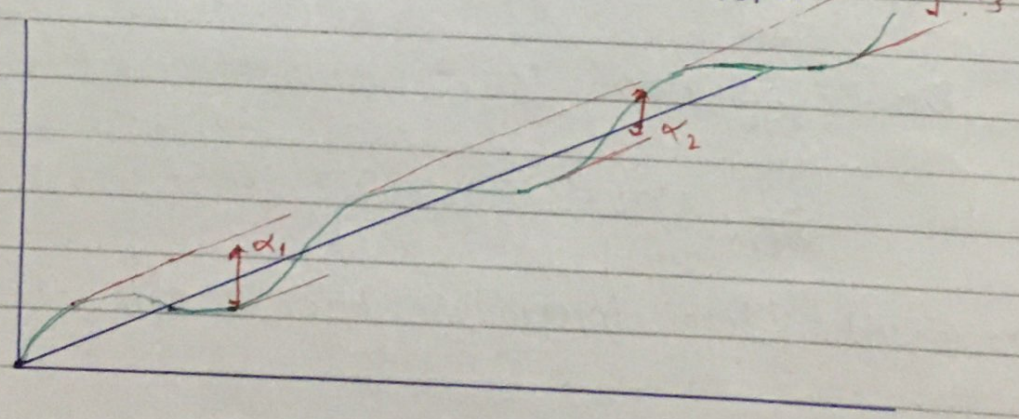
FLOW MASS CURVE :- It is a graphical plot of cummulative volume of Runoff against time

- It is used to determine storage capacity of Reservoir

Case-I. When total supply is more than total demand (that is case of spilling)





0 - t_1 → Inflow Rate > Demand Rate and since the reservoir is assumed full @ t = 0, spilling will occure from 0 - t_1

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$t_1 \rightarrow t_2$ Inflow Rate < Demand Rate Hence Reservoir will be depleted by amount α_1

α_1 = Volume of Water consumed from $t_1 \rightarrow t_2$ − Volume of water supplied from $t_1 \rightarrow t_2$

$t_2 \rightarrow t_4$ Inflow Rate > Demand Rate. Hence Reservoir will start filling and it will again become full @ time t_3.

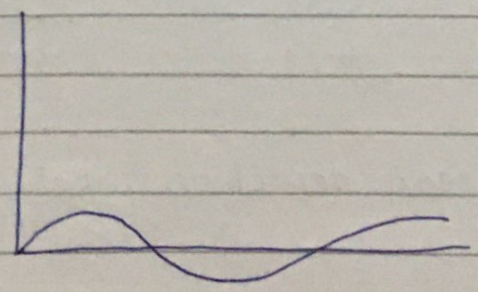
spilling will occure from $t_3 \rightarrow t_4$

In Graphical Method we Draw tangent (parallel to demand line) from peak and valley point of Flow - mass curve and calculate the vertical ordinates b/w then. The Max. vertical ordinate (Max^m of $\alpha_1, \alpha_2, \alpha_3 \cdots$) is taken as the reservoir capacity

** If the tangent from peak point on Flow mass curve never cuts the Flow mass curve again, it means that the reservoir will never become full and such demand can not be fullfill by the given supply.

Sequence peak algorithem is also used to calculate Storage capacity. in which residual mass curve is plotted

residual mass curve = Flow mass curve − Demand curve



Residual curve

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Que: obtain storage requirement for a reservoir from the Data given below.

Month	Inflow Mm^3 (I)	Demand Mm^3 (D)	Deficit (D-I)	Cum. Deficit	Description
June	3	8.33	5.33	5.33	
July	6	8.33	2.33	7.66	
Aug.	16	8.33	-7.67	~~0.01~~ →	fill + 0.01 spill
Sep.	30	8.33	-21.67		spill
Oct	18	8.33	-9.67		spill
Nov.	15	8.33	-6.67		spill
Dec.	10	8.33	-1.67		spill
Jan.	8	8.33	0.33	0.33	
Feb	6	8.33	2.33	2.66	
March	4	8.33	4.33	6.99	
April	3	8.33	5.33	12.32	
— May	1	8.33	7.33	19.65	
June	2	8.33	6.33	25.98	
July	5	8.33	3.33	29.31 *	
Aug.	17	8.33	-8.67		— filling starts (20.64)
Sep.	28	8.33	-19.67		filling 0.97
Oct.	20	8.33	-11.67		spill
Nov.	15	8.33	-6.67		spill
Dec.	12	8.33	-3.67		spill
Jan.	7	8.33	1.33	1.33	
Feb	5	8.33	3.33	4.66	
March	4	8.33	4.33	8.99	
April	3	8.33	5.33	14.32	
— May	2	8.33	6.33	20.65	

Storage capacity = Max. depletion that is 29.31

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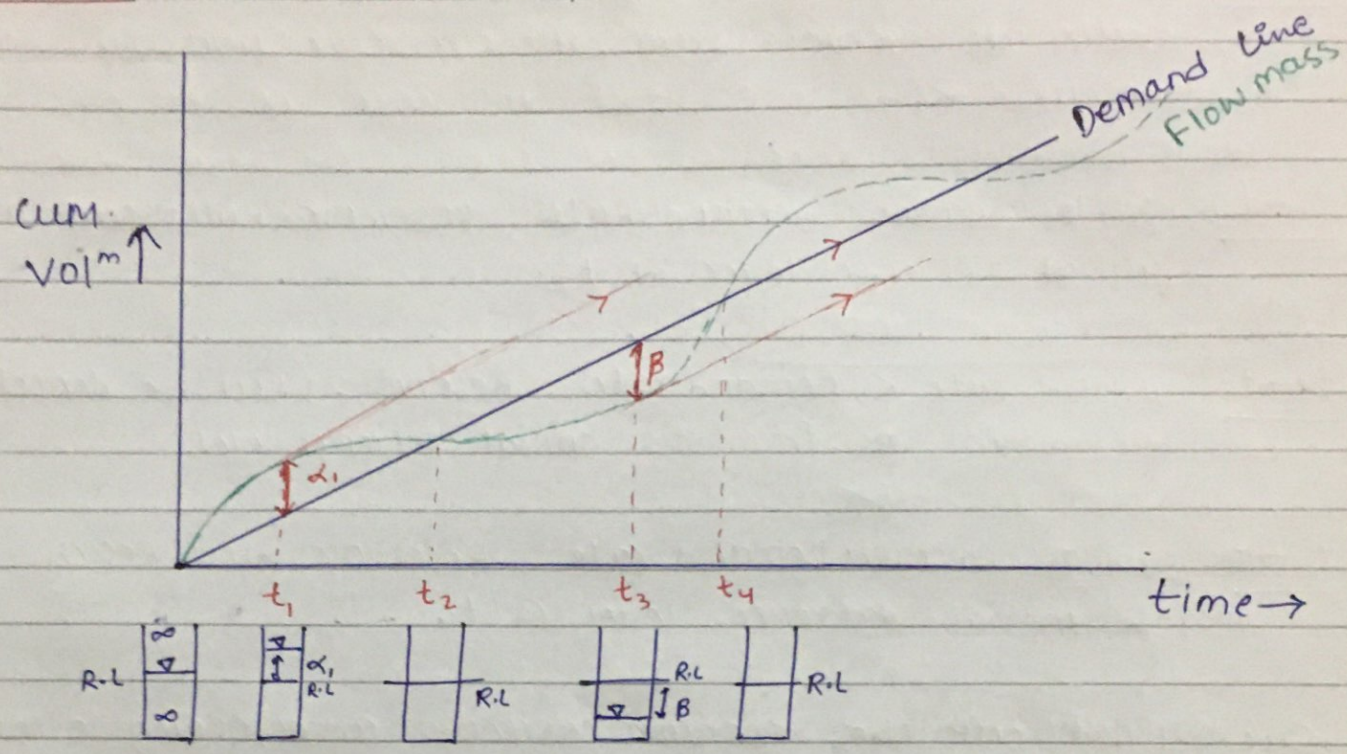
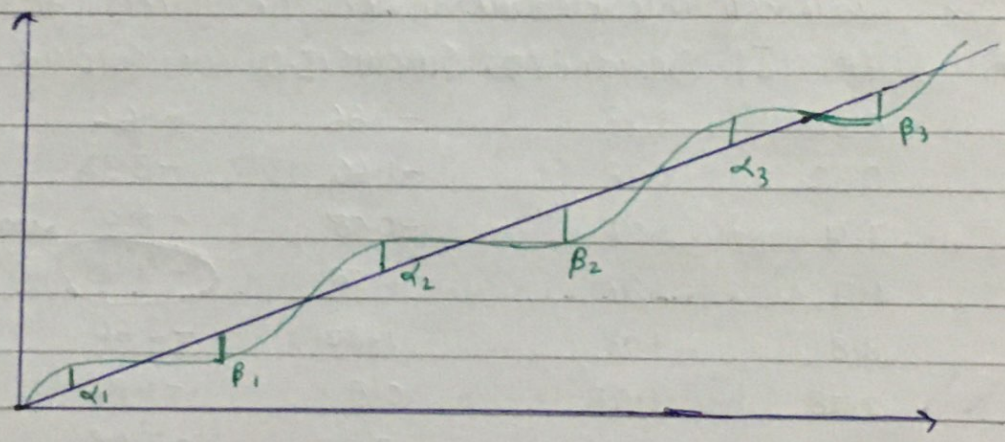
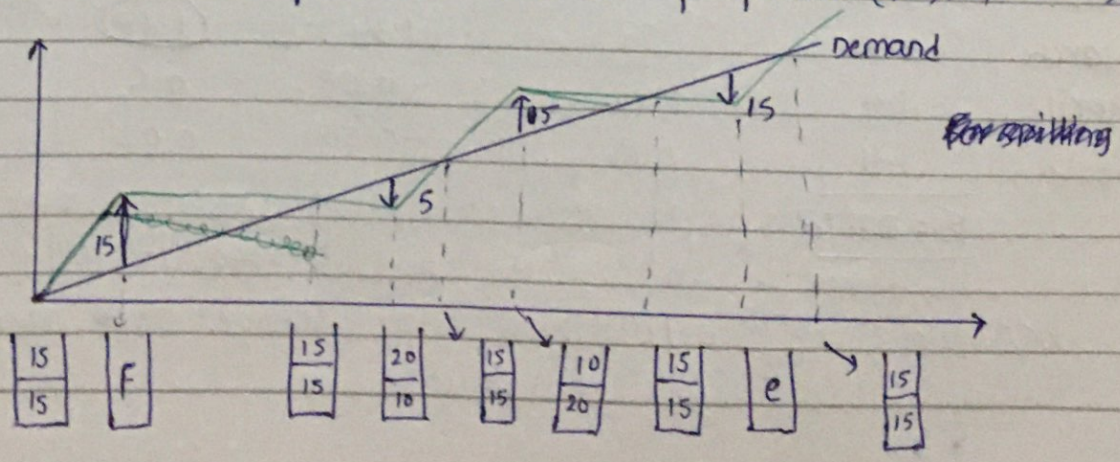
Case-II When Total supply = Total Demand





Storage = |max. surplus| + |max. depletion|

= |Max. ($\alpha_1, \alpha_2, \alpha_3$ ----)| + |Max. ($\beta_1, \beta_2, \beta_3$ ----)|



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$0 \rightarrow t_1$ Inflow Rate > Demand Rate, Hence accumulation will occur in reservoir from the refrence level by amount α_1

$t_1 \rightarrow t_2$ Inflow Rate < Demand Rate, Reservoir will Reach it's initial Refrence lever at t_2

$t_2 \rightarrow t_3$ Inflow rate < Demand rate, Reservoir will be depleted by amount β_1 from it's initial Refrence level.

$t_3 \rightarrow t_4$ Inflow Rate > Demand rate, Reservoir will Reach it's initial Refrence level @ t_4

Que. Find out the reservoir capacity corresponding to the Max. Maintainable demand for the given inflow.

Month	Inflow (I)	Demand (D)	Surplus (I-D)	Cum. Surplus
June	0.32	1.98	-1.66	-1.66
July	0.32	1.98	-1.66	-3.32
Aug	1.4	1.98	-0.58	-3.9
Sep	1.4	1.98	-0.58	-4.48 Max. Depletion
Oct	3.8	1.98	1.82	-2.66
Nov	2.78	1.98	0.8	-1.86
Dec.	1.76	1.98	-0.22	-2.08
Jan.	2.56	1.98	0.58	-1.5
Feb	3.32	1.98	1.34	-0.16
March	3.32	1.98	1.34	1.18 Max. Surplus
April	1.4	1.98	-0.58	0.6
May	1.4	1.98	-0.58	0.02
	$\Sigma = 23.78$			

$$\text{Demand} = \frac{23.78}{12} = 1.98 \ Mm^3$$

$$\text{Storage} = |-4.48| + |1.18|$$

$$\text{Storage} = 5.66 \ Mm^3$$

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RUNOFF - RAINFALL RELATION :-

① **Using Regration Analysis :-** From records of Rainfall (P) and Runoff (R) we try to find a relation b/w them using regration. Generally linear regration is used. The assumed relation b/w P and R given as:-

$$R = aP + b$$

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

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

Where N is no. of Data sets of P & R available

The assumed relation is said to give good results if the co relation cofficient (γ) lies b/w 0.6 – 1

$$\gamma = \frac{N\Sigma PR - \Sigma P \Sigma R}{\sqrt{[N\Sigma P^2 - (\Sigma P)^2][N\Sigma R^2 - (\Sigma R)^2]}}$$

For Large catchment exonential relation gives better result

$$R = cP^d$$

$$\ln R = \ln c + \ln P \times d$$

$$d = \frac{N\Sigma \ln P \ln R - \Sigma \ln P \Sigma \ln R}{N\Sigma (\ln P)^2 - (\Sigma \ln P)^2}$$

$$\ln c = \frac{\Sigma \ln R - d\Sigma \ln P}{N}$$

$$r = \frac{N\Sigma \ln P \ln R - \Sigma \ln P \Sigma \ln R}{\sqrt{[N\Sigma (\ln P)^2 - (\Sigma \ln P)^2][N\Sigma (\ln R)^2 - (\Sigma \ln R)^2]}}$$

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Normally for any two variable corelation cofficient varies from -1 to 1, But in case of Runoff and Rainfall, since Runoff can only have +ve corelation with Rainfall, R varies from 0 to 1

② Using Emperical Eq^n :-

① Khosala's theory
② Strange's table
③ Bar low's table
④ Binny's percentage

Khosala's Eq^n :- It gives monthy Runoff from monthly ppt.

$$R_m = P_m - L_m \quad ; \quad (L_m < P_m)$$
$$= 0 \quad ; \quad o/w$$

R_m is monthly Runoff in cm
P_m is Monthly ppt in cm
L_m is Monthly losses in cm

Losses are calculated emperically as -

$$L_m = 0.48 T_m \quad ; \quad (T_m > 4.5^\circ c)$$

$T_m \rightarrow$ Monthly Mean temp in °c
for $T_m = 4.5\,^\circ c$, $L_m = 2.17$ cm.

Khosala's eq^n is indirectly Based on Water budget and Monthy Mean temp. In this Method losses are primarely assumed to be comprised off Evapotranspiration. Which are calculated using Monthly Mean temp.

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Draught :- It is a climatic anomaly characterised by deficit supply of moisture. Unlike Flood it is of creeping kind i.e it develops in a region over a period of time

Types :-

1) **Meteorological Draught :-** It is a condition where is more than 25% decrease in ppt from Normal.
Draught prone area → probability of Draught 20-40%
cronically Draught prone Area → > 40 %
Draught year → year in which more than 20% of the country is affected from Meteorological Draught is called Draught year.

% decrease from Normal	class
0-25	No- Draught
25-50	Moderate
>50	severe

2) **Hydrological Draught :-** Metreological Draught when prolonged results in hydrological Draugh with significant deplition of surface and ground waters

3) **Agricultural Draught :-** When soil moisture and Rainfall are insufficient to support healthy crop during rainy season. called Agricultural Draught. it is classified on the Basis of Airidity index.

$$\text{Aridity index (AI)} = \frac{PET - AET}{PET} \times 100\%$$

AI (%) →	-ve or 0	1 → 25	25 → 50	> 50
class →	No Drought	Mid- Arid	Moderate Arid	Severe Arid

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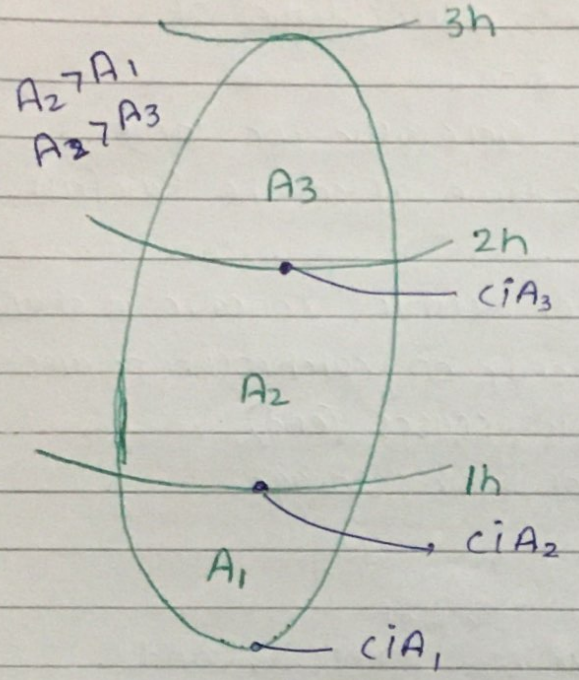
5 Hydrograph

It is a plot of Runoff Discharge against time.



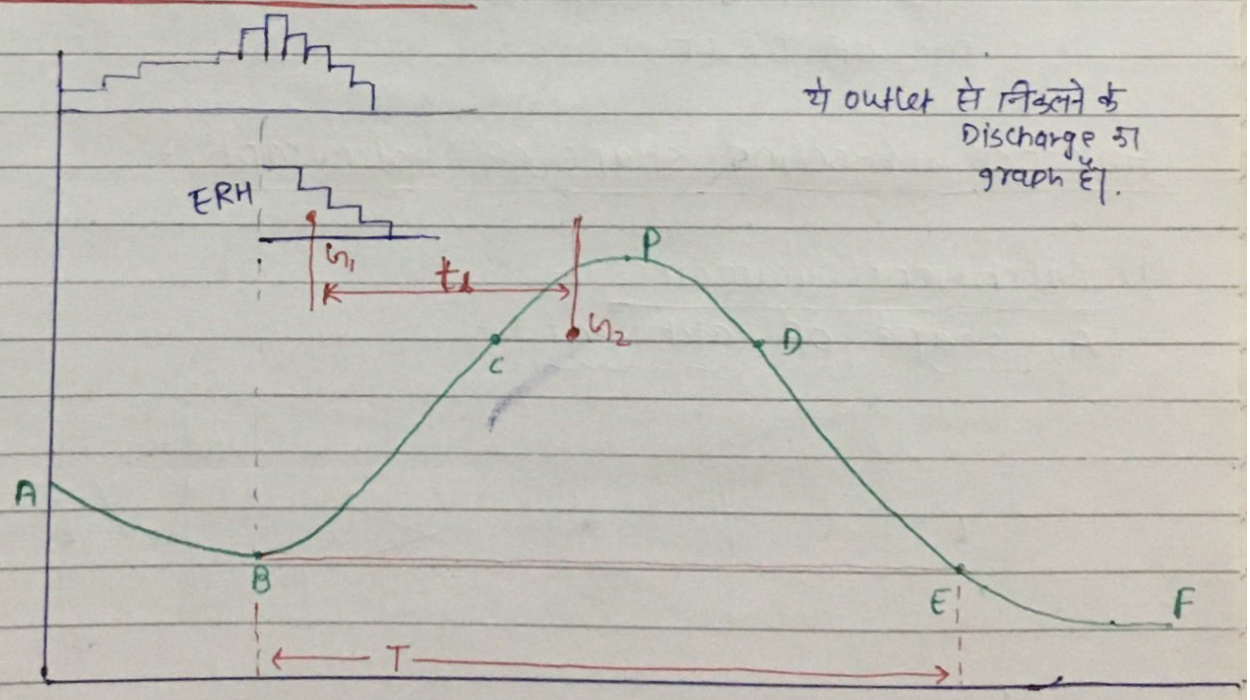
$t_c > D$

↓ 3 hr ↓ 1 hr

t	Q
0	0
1	ciA_1
2	ciA_2 ← Qmax.
3	ciA_3
4	0

$D + t_c$

Components of Hydrograph :-



ये outlet से निकलने के Discharge का graph है।

A → Start of Rainfall
ABEF → Base Flow
B → Start of DR/SR

BC is called rising limb or concentration curve. It is concave in shape and it's shape depends both on catchment as well as Rainfall parameters

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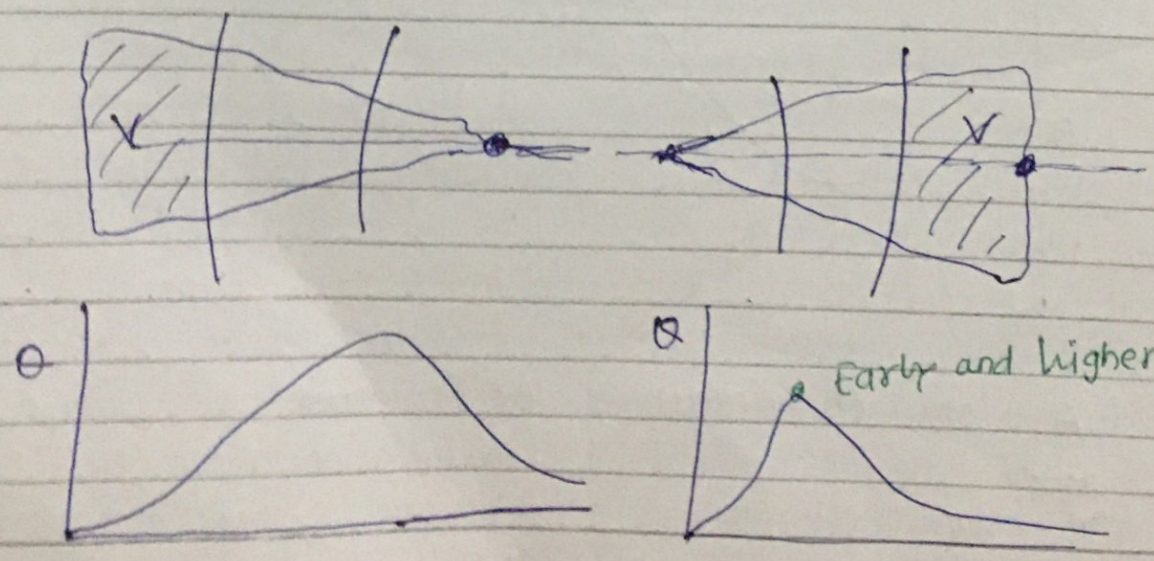
CPD is called crash segment, it is convex in shape
C, D are called inflection points, curvature of Hydrograph changes @ these points
P → peak of hydrograph and generally for large catchments peak occurs some time after the Rainfall has ended
DE → Falling limb or recession curve, concave in shape and it's shape depends only on catchment parameters.
D → point where overland flow ceases (end)
It is also taken as point of max. storage
E → End of DR/SR
BCPDE → DRH
t_1 → lag time i.e distance b/w c.g of ERH G_1 and c.g of DRH G_2
T → Base period, start from start of DR (B), end at end of DR (E)

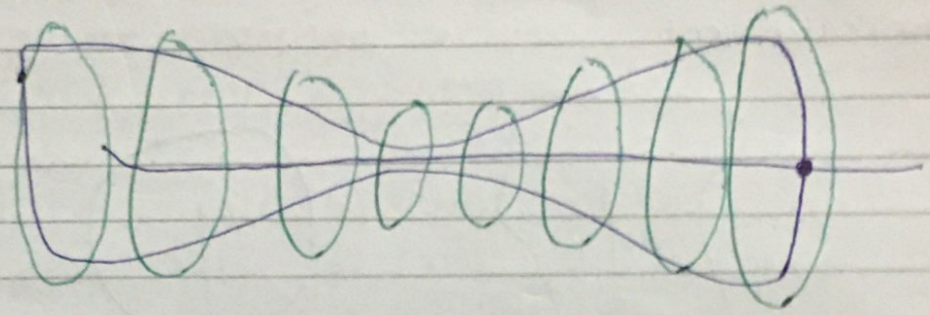
Factors affecting shape of Hydrograph:-

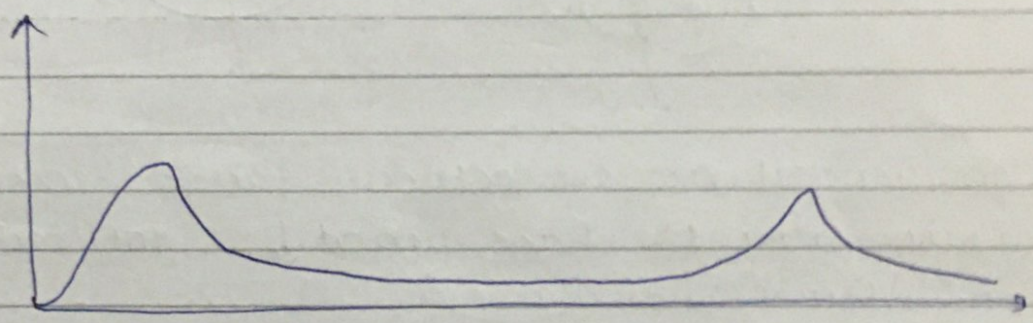
(i) Catchment parameters :-

A) shape of catchment :-



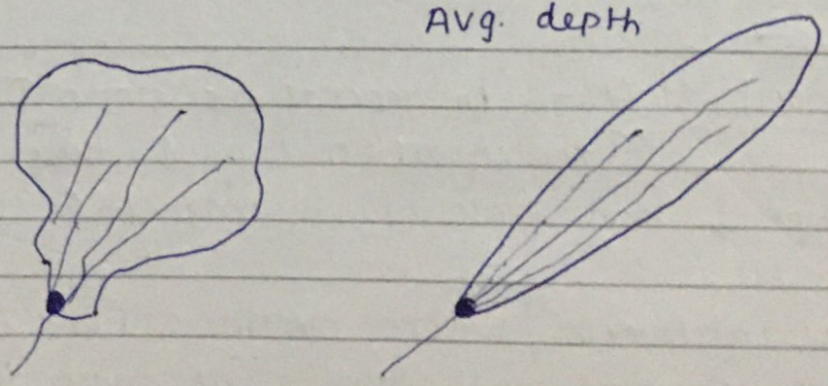
Hydrograph ~~from~~ For a catchment with broad and near the outlet will have higher & early peak as compared to a catchment with narrow end towards the outlet





B) Form Factor:- It is defined as the ratio of the avg. ~~length~~ width of the catchment to avg ~~width~~ length of the catchment.

$$\text{Form factor} = \frac{\text{Avg. width}}{\text{Avg. depth}}$$



In general higher is the form factor lower is the base period, lower is the infiltration and higher is the runoff discharge

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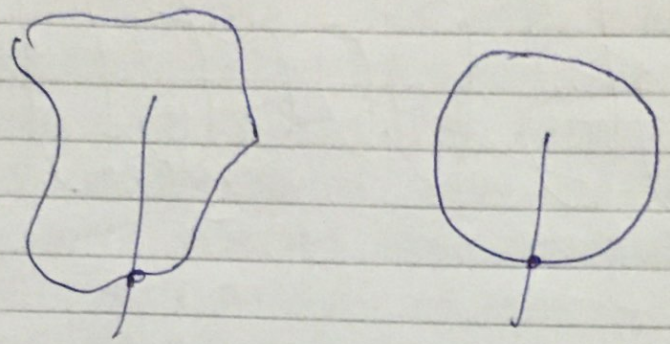
c) Compactness factor :-

It is defined as the ratio of the parimeter of the catchment to the parimeter of form circle.
Form circle is a circle having same Area as that of the catchment



In General as compactness factor decreases (Min value 1) Base period ↓, Infiltration ↓ and Runoff discharge ↑

$$\text{compactness factor} = \frac{\text{Perimeter of catchment}}{\text{perimeter of form circle}}$$

NOTE. Fan shaped catchment Nearly semi-circular will have a higher and early peak than a fern shape (elongated) catchment

D) Catchment slope:- In general as slope ↑ flow velocity ↑ concentration time ↓. Base period ↓, Infiltration ↓ and peak of hydrograph ↑

For small catchments where overland flow is more dominant, general land slop is of more importance and for large catchment where channel flow is more dominant, Main stream flow is more important.

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e) Drainage Density :- defined as Ratio of sum of lengths of all streams to the catchment Area.

In Genral as Drainage density ↑ . Flow velocity ↑ infiltration ↓ . peak of hydrograph ↑

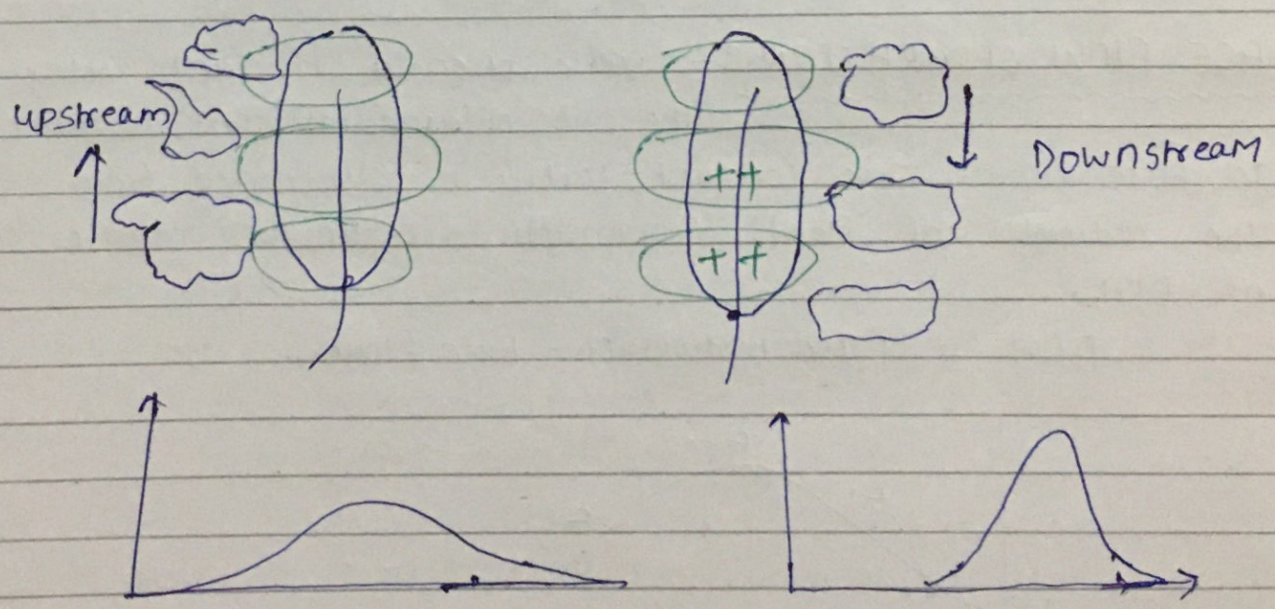
f) Land use :- Urbanigation reduces infiltration and increases flow velocity . thereby decreasing Base period and increasing the peak of hydrograph.

ii) Rainfall parameters :-

A) Intensity of Rainfall :- As Intensity ↑ , peak ↑

B) Antecedent precipitation :- It redudes Infiltration and increases peak of hydrograph.

c) Direction of Storm movement :-



When storm movement is downstream peak of hydrograph is higher as compare to storm movement is upstream.

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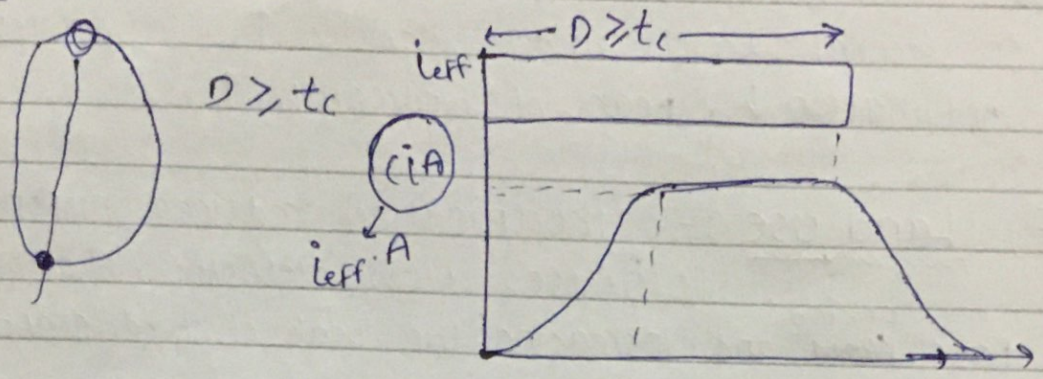
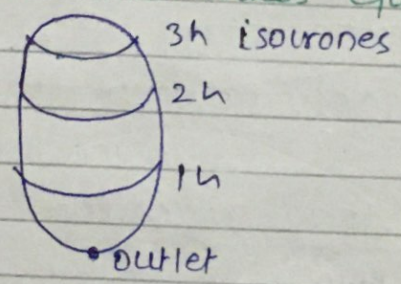
D) Duration of Rainfall :- when Rainfall duration equals or exceeds concentration time of catchment, we get Max. possible theortical discharge. $Q = CiA$

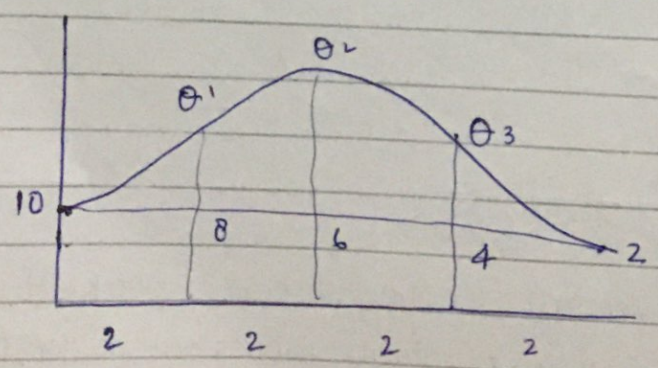


Isocrones :- Are imaginary lines joining points on catchment from where water takes equal time to reach the outlet.



BASE FLOW SEPARATION :- Very often a constant value (the one obtained at start) is taken as Base Flow. This constant value is subtracted from the ordinates of flood hydrograph to obtain the ordinates of DRH.

DRH = Flood hydrograph - Base flow



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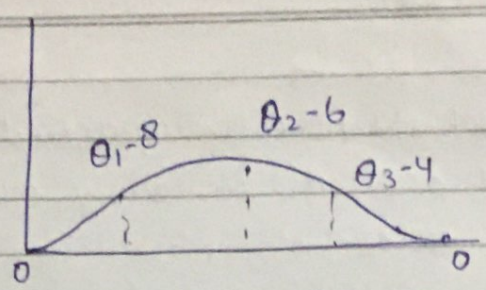


SOME IMPORTANT POINTS :-

① Area of ERH = Depth of SR/DR/Eff. Rainfall / h_{eff}

② Area of DRH = Volume of SR/DR/Eff. Rainfall

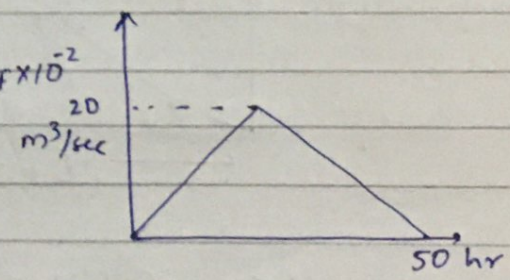
$$\text{Vol}^{m} \text{ of SR/DR} = \text{Depth of SR/DR} \times \text{Area}_{catchment}$$

$$\text{Area of DRH} = \text{Area of ERH} \times \text{Area}_{catchment}$$
$$= h_{eff.} \times A_{catchment}$$

Que. A triangular DRH is given below calculate the Eff. Rainfall depth when the catchment Area is 180 km²

$$\frac{1}{2} \times 20 \times 50 \times \cancel{86400}\ 3600 = 180 \times 10^{6} \times h_{eff} \times 10^{-2}$$

$$h_{eff.} = 1 \text{ cm}.$$



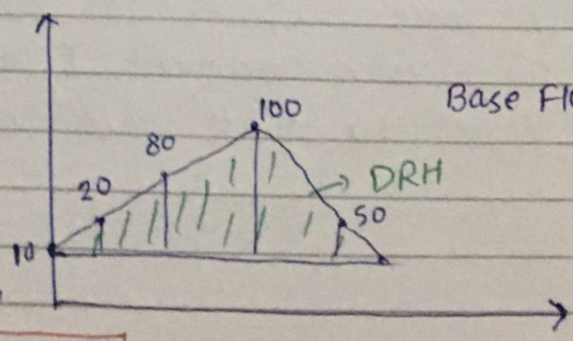
Que. A Flood hydrograph with a Base time of 90 hours has ordinates 10, 20, 80, 100, 50 and 10 m³/s at elapsed time 0, 10, 20, 30, 50 and 90 hrs. Area of catchment = 300 km². Calculate the Rainfall excess in the storm.

DRH time	0	10	20	30	50	90
Q	0	10	70	90	40	0

$$\text{Area} = \left(\frac{10 \times 10}{2} + \frac{10+70}{2} \times 10 + \frac{70+90}{2} \times 10 + \frac{90+40}{2} \times 20 + \frac{40+0}{2} \times 40\right) \times 3600$$

$$= h_{cm} \times 300 \text{ km}^2$$

$$\boxed{h = 4.02 \text{ cm}}$$



Base Flow = 10

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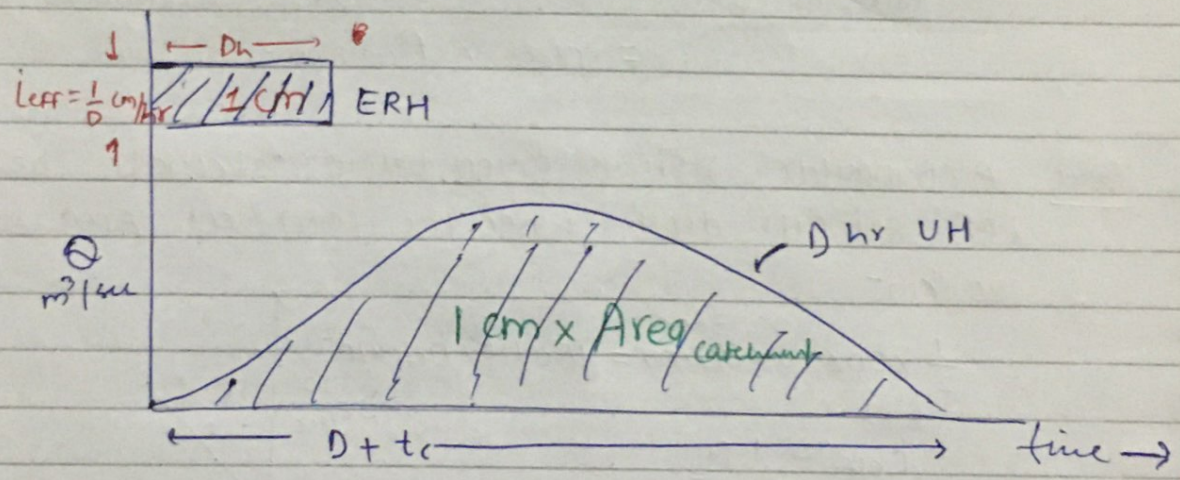
UNIT - Hydrograph :- (Sherman)

It is defined as a direct Runoff Hydrograph due to 1 cm of eff. Rainfall applied uniformly over the catchment at a uniform Rate.

It is expressed as D hr UH, where D represents Duration of Rainfall

$$i_{eff.} = \frac{h_{eff}}{D}$$

Eff. intensity in D hr UH = $\frac{1}{D}$ cm/hr



Area of UH = 1 cm x Area of Catchment

NOTE: A-4h-UH will have a higher peak than 8hr UH for the same catchment because Areas of Both UH have to be same But Base period of 8-hr UH will be more.

For a given catchment I-UH (Instantaneous UH) will have the lowest Base period (t_c) and highest peak.

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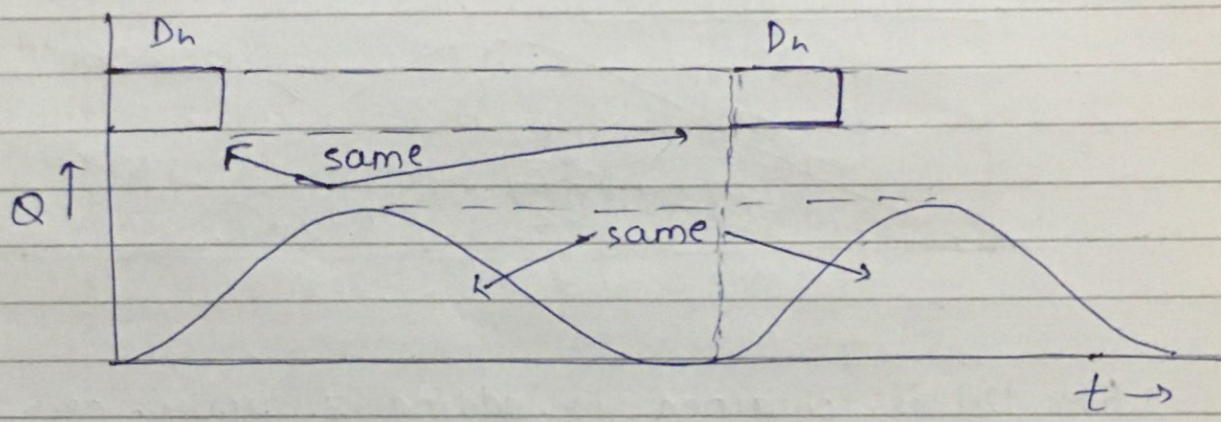
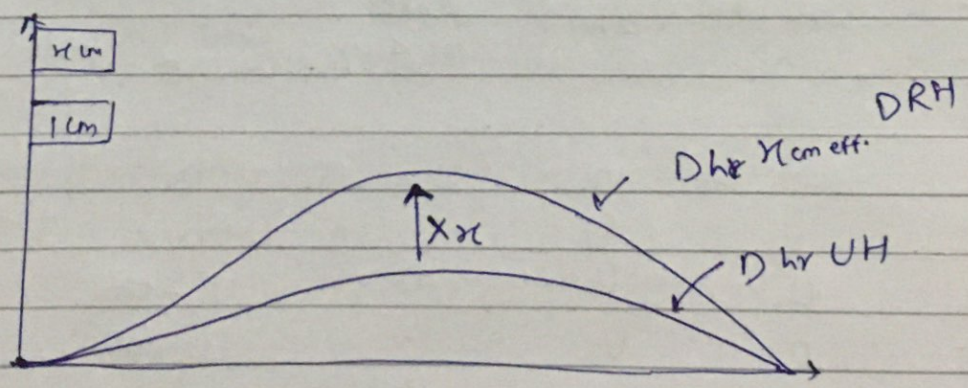
Assumptions :-

① Intensity of Rainfall is constant
② Rainfall is uniformly distributed over the catchment
③ Time Invarience :-
The DRH due to given eff. Rainfall will always be same irrespective of when the Rainfall occurs.



④ Linear response :- A DRH due to D hr. Rainfall of eff. depth x cm is obtained by multiplying the ordinates of D hr UH by x and vice-versa.



Linear response is applicable on eff. Rainfall depth for same Duration of Rainfall.

Que. If the peak of D hr x cm eff. DRH is Q_p then the peak of D hr Y cm DRH is ?

$$\frac{Y}{x} \times Q_p$$

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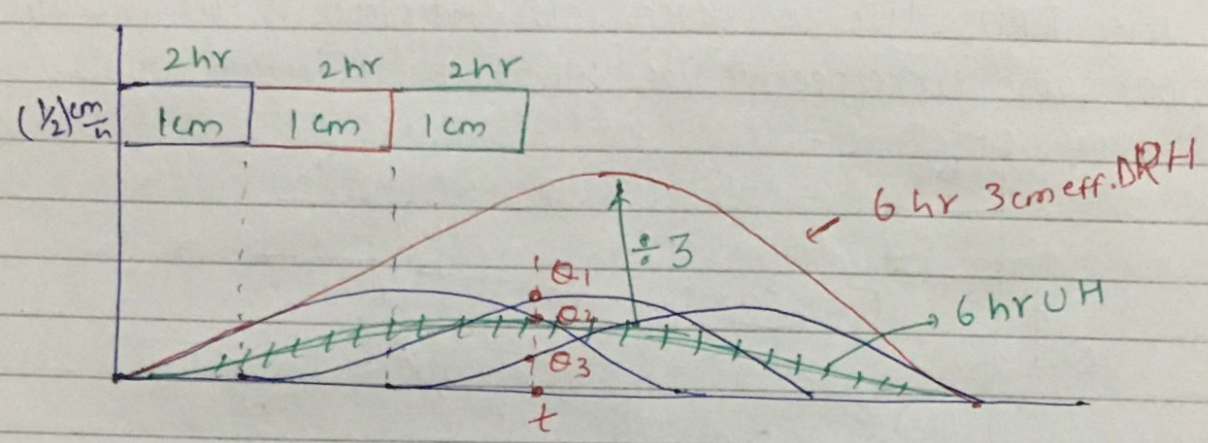
Conversion of Unit Hydrograph :-

Case-I Converting a D Hr UH → m × D hr. UH
where m is a +ve integer.

for ex: let us convert a 2 hr UH into 6 Hr UH



6 hr UH is obtained by adding 3, 2H UHs each lagged by 2 Hr. and then deviding the ordinate of the resultant DRH (6 hr 3cm eff DRH) by 3.

t	2h UH	2h UH lagged by 2H	2H UH lagged by 4 Hr	Resultant DRH 6hr, 3cm DRH	6Hr UH = 6hr 3cm DRH / 3
0	0			0	0
1	U_1			U_1	$U_1/3$
2	U_2	0		U_2	$U_2/3$
3	U_3	U_1		U_1+U_3	⋮
4	U_4	U_2	0	U_4+U_2	⋮
5	0	U_3	U_1	U_3+U_1	⋮
6		U_4	U_2	U_2+U_4	⋮
7		0	U_3	U_3	$U_3/3$
8			U_4	U_4	$U_4/3$
9			0	0	0

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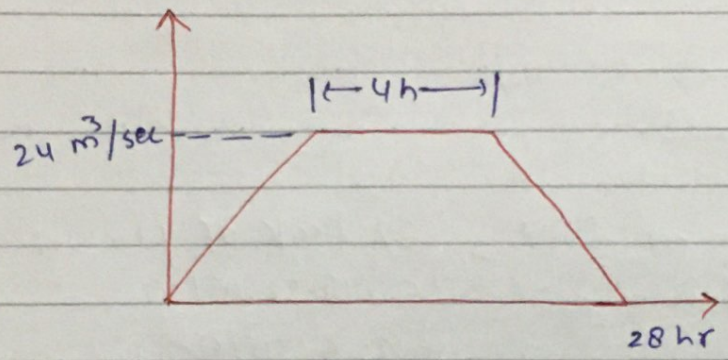
Que. A 2h-UH can be approximated as A Trapezoid as shown below. The UH refers to a catchment Area of (in km^2) ?



Area of (UH) = 1 cm x A_{catch}

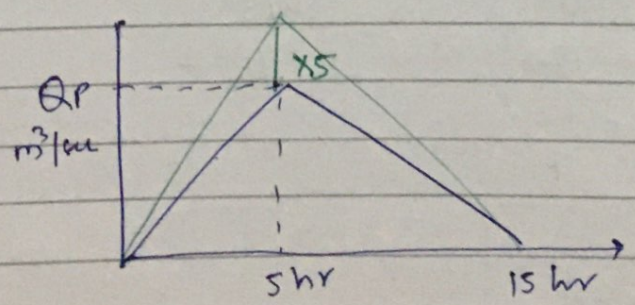
$\frac{1}{2} \times (4+28) \times 24 \times 3600 = 1 \times 10^{-2} \times A \times 10^{6}$

$A = 138.24 \ km^2$

Que. The Drainage Area of a Watershed is 50 km^2. The ϕ index is 0.5 cm/hr and the Base flow @ outlet is 10 m^3/sec. 1 hr-UH of the catchment is triangular in shape with a Base period of 15 hr. the peak occurs at 5 hrs.

(i) the peak ordinate of UH in m^3/s ?

(ii) For a storm of depth 5.5 cm and duration 1-hr the peak ordinate of hydrograph ?



i) Area of UH = 1 cm x A_{catch}

$\frac{1}{2} \times 15 \times Q_p \times 3600 = 1 \times 10^{-2} \times 50 \times 10^{6}$

$Q_p = 18.52 \ m^3/sec.$

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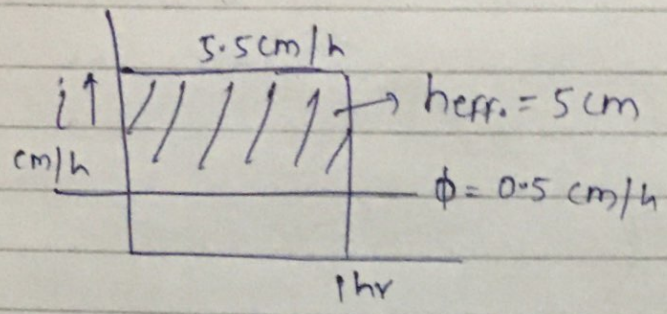
ii) Eff. Rainfall depth in Given rainfall.



peak of 1 hr. 5 cm eff. DRH (given Rain) = 5 × peak of 1 hr UH
= 5 × 18.52 m³/sec
= 92.6 m³/sec.

peak of hydrograph = Peak of DRH + Base flow
= 92.6 + 10
= 102.6 m³/sec.

Que ordinates of 1 hr. UH at 1 hr. interval starting from t = 0 are 0, 2, 6, 4, 2, 1, 0 m³/sec.
i) the catchment Area represented by this UH in km²?
ii) ordinate of 3h-UH for the catchment @ t = 3hr. in m³/s?

t	1H-UH			Resultant 3hr 3 eff DRH	3hUH = Resultant/3
0	0			0	0
1	2	0		2	2/3
2	6	2	0	8	8/3
3	4	6	2	12	12/3 = 4
4	2	4	6	12	
5	1	2	4	7	
6	0	1	2	3	
		0	1	1	

when time interval is same.

Area = $t_{hr} \times (Q_1 + Q_2 + Q_3 + Q_4 + Q_5)$ m³/sec
= 1 × (15) × 3600
=

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Area of UH = 1 cm × A_{catch}

$15 \times 3600 = 1 \times 10^{-2} \times A \times 10^{6}$

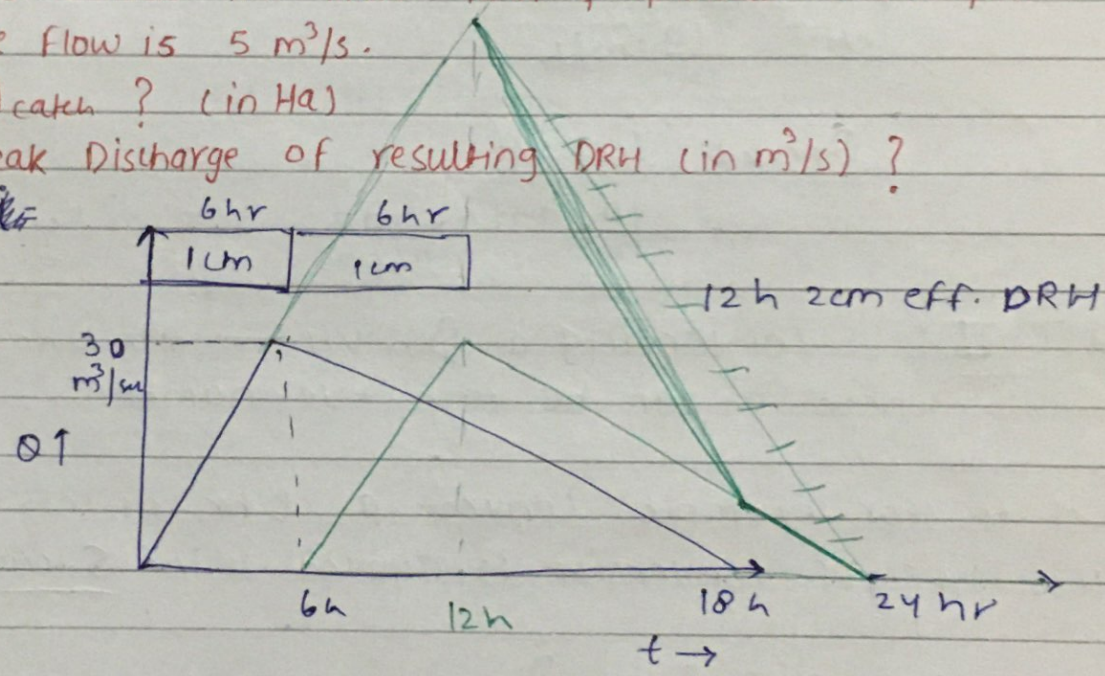
$A = 5.4\ km^2$

Que. An avg. Rainfall of 16 cm occurs over a catchment in 12 hr. with uniform intensity. A 6h-UH for catchment rises linearly from 0-30 cumec in 6h and then falls linearly from 30-0 cumec in next 12 h., ϕ index = 0.5 cm/hr. Base flow is 5 m^3/s.

i) A_{catch} ? (in Ha)

ii) peak Discharge of resulting DRH (in m^3/s) ?

(i)

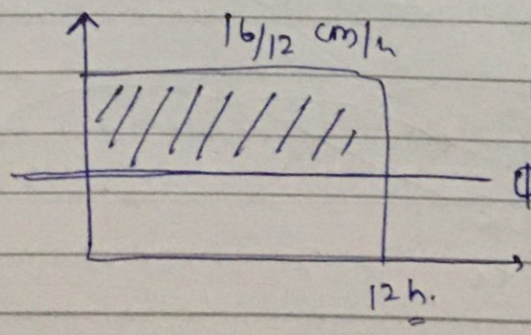


Area = $\frac{1}{2} \times 30 \times 18 = 1\ cm \times A\ hac.$

$\Rightarrow 15 \times 18 \times 3600 = 1 \times 10^{-2} \times A \times 10^{4}$

$\boxed{A = 9720\ ha}$

(ii)



$h_{eff.} = (1.33 - 0.5) \times 12 = 10\ cm$

$\phi = 0.5\ cm/h$

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Equilibrium Discharge for S-curve → $2.778 \frac{A}{D}$ m³/sec.

peak of Resultant DRH (12 hr 10 cm eff. DRH)

= peak of 12 hr 2cm eff. DRH × 5

= 5×45

= 225 m³/sec.

alternative.

$$D + t_c = 18$$

$$6 + t_c = 18$$

$$t_c = \underline{\underline{12h}}.$$

$$i_{eff} = \frac{10}{12} \text{ cm/h}$$

$$Q_{peak} = i_{eff} \cdot A$$

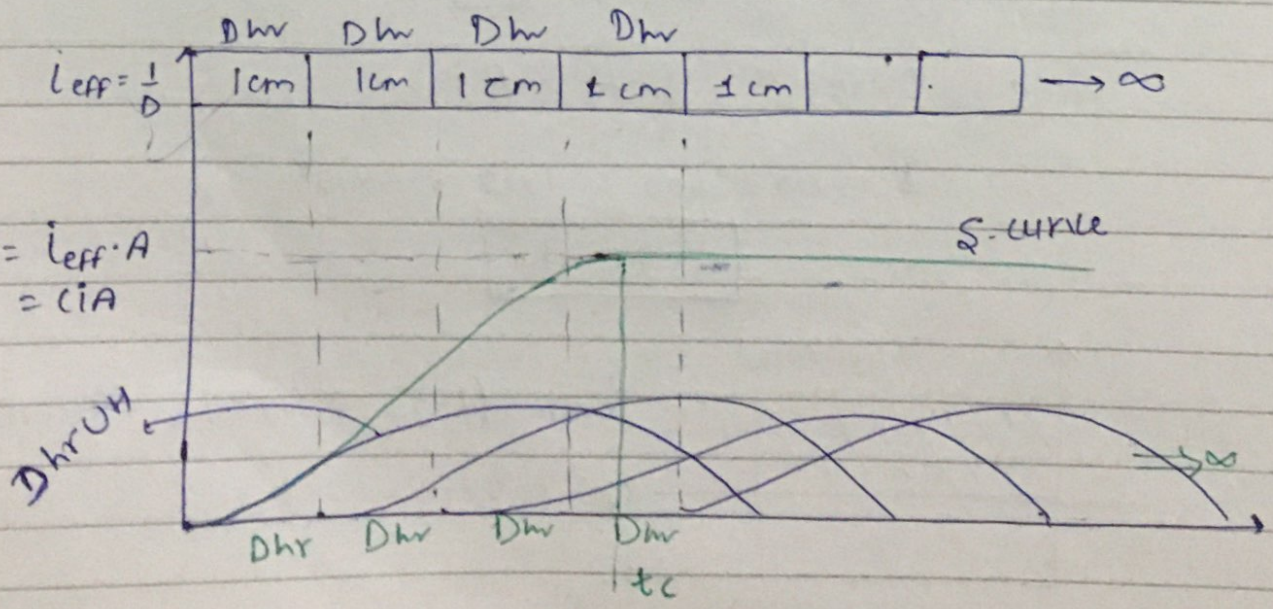
$$= \frac{10}{12} \times 9720 \times \frac{100}{3600} = 225 \text{ m}^3/\text{sec}.$$

Case-II :- converting a Dh UH ⟶ m× Dhr UH

where m can be any +ve number.

Let us for example convert a D hr UH into T hr UH

This type of conversion is achived using S-curve



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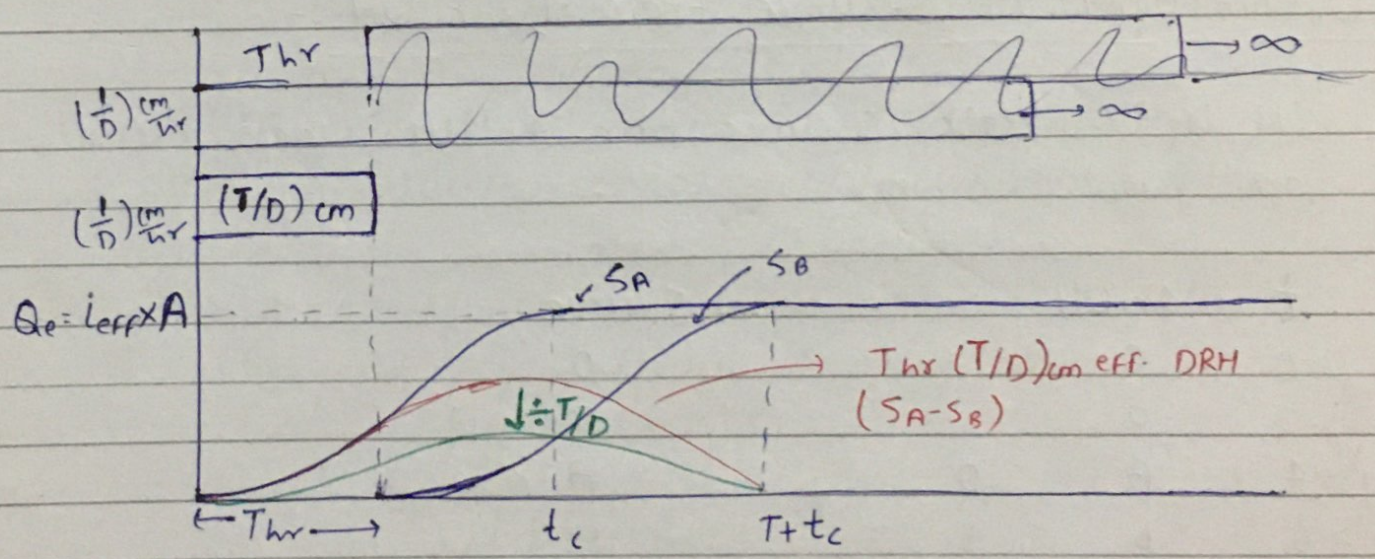
S- curve Hydrograph May be defined as a direct Runoff Hydrograph due to a contineous Rainfall (∞ - Duration) of Eff. intensity $1/D$ cm/hr.
S curve is constructed by adding an ∞- series of D-hr UH each lagged by D-hr from the previous one.

The Max. discharge obtained in S- curve corresponding to t_c of catchment (approximatly near the end of 1st UH) is called equilibirium discharge Q_e

$$Q_e = i_{eff} \cdot A_{catchment}$$

$$Q_e = \left(\frac{1}{D}\right)_{cm/hr} \times A$$



Thr UH is obtained by deviding the ordinates of $S_A - S_B$ (i.e Thr $(T/D)_{cm}$ eff. DRH) by T/D

$$T_h \text{ UH} = \frac{S_A - S_B}{(T/D)}$$

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Where S_A is S-curve prepared from D hr UH and S_B is S_A legged by T hr.

Instanteneous UH (I-UH) (Clark) :-

I-UH is a direct Runoff Hydrograph due to an ∞-decimaly small duration (i.e dt hour) of effective depth 1 cm.

$$\boxed{I_{UH} = \frac{ds/dt}{i_{eff}}}$$

Shape of I-UH depends only on catchment parameters.

Generation of S-curve from D-hr UH :-

Let us generate S-curve from 4h UH whose intervals are given @ 2 hr.

t	4h UH				S-curve
0	0				0
2	a				a
4	A	0			A
6	b	a			b+a
8	B	A	0		A+B
10	τ	b	a		a+b+τ
12	C	B	A		A+B+C
14	d	τ	b		a+b+τ+d
16	D	C	B	$\Rightarrow \infty$	A+B+C+D
18	e	d	τ		⋮
20	E	D	C		⋮
⋮	⋮	e	d		⋮
		E	D		

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t	4h-UH
0	0
1	1
2	2
3	3
4	0+4
5	1+5
6	2+6
7	3+7
8	0+4+8
9	1+5+9
10	2+6+10
11	3+7+11
12	0+4+8+12

If ordinates of 4h UH given @ 1 hr intervals then S-curve is Drawn by taking cummulations @ 4-hrs.

Que. For a catchment S. curve due to a rainfall of eff. intensity 1 cm/hr is given as —

$$Q = 1 - (1+t)e^{-t}$$

Where t is in hr. and $Q \rightarrow m^3/sec$.

i) What is the Area of catchment in km^2?

ii) What will be the ordinate of 2 hr UH @ t = 3 hrs.

i) $Q_e = Q|_{t=t_c} = Q|_{t=\infty}$

$1 - \frac{1+t}{e^t} \rightarrow 0$

$t=0,\ Q=0$

$t=1,\ Q=.264$

$t=2,\ Q=0.59$

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$i_{eff} \cdot A = 1\ m^3/sec$

$1\ cm/hr \times A\ km^2 = 1\ m^3/sec$

$\frac{10^{-2}}{3600} \times A \times 10^6 = 1$ $\boxed{A = 0.36\ km^2}$

ii)

t	S_A	S_B	2h-UH : $\frac{S_A - S_B}{2/1}$
0	0		
1	0.264		
2	0.59	0	
3	0.800	0.264	$\frac{0.800-0.264}{2} = 0.268$
4	0.908	0.59	
5		0.800	

Que: The ordinates of 2hr UH @ 1 hr interval starting from time t=0 are 0, 3, 8, 6, 3, 2, 0 m^3/s. Use Trapezoidal Rule for numerical integration.

(i) Catchment Area represented by this UH in km^2

(ii) A storm of 6.6 cm occurs uniformly over the catchment in 3 hr, ϕ index = 0.2 mm/hr. Base flow = 5 m^3/sec. What is the peak flow due to the storm?

i) Area of UH = 1 cm × $A_{catchment}$

$1h\ (0+3+8+6+3+2+0)\ m^3/sec = 1\ cm \times A_{catch}$

$22 \times 3600 = 10^{-2} \times 10^6 \times A_{catch}$

$A_{catch} = 7.92\ km^2$

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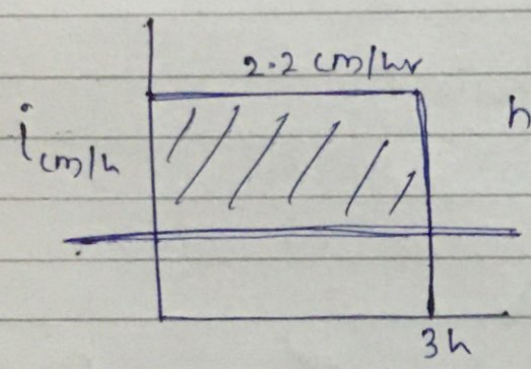
ii)

t	2H-UH	S_A	S_B	3H-UH $= (S_A - S_B)/3/2$
0	0	0		0
1	3	3		2
2	8	8		5.33
3	6	9	0	6 ⇒ Max.
4	3	11	3	5.33
5	2	11	8	2
6	0	11	9	1.33
			11	0



$h_{eff} = 2 \times 3 = 6\,cm$

$\phi = 0.2$ cm/hr

peak of resultant DRH = 6 × Peak of 3H-UH

3h (6cm) eff DRH

$= 6 \times 6$

$= 36\ m^3/sec$

peak of flood hydrograph = peak of DRH + Base Flow

$= 36 + 5$

$= 41\ m^3/sec.$

Que. Ordinates of 1hr UH of a catchment is given below $\phi = 0.5$ cm/hr, Base flow can be assumed to be 5 m^3/sec. at the begining and increasing by 2 m^3/sec. for every 2 Hrs. find the flood hydrograph for catchment for rainfall given below.

t	0	1	2	4	5	6
1hr-UH	0	2	6	2	1	0

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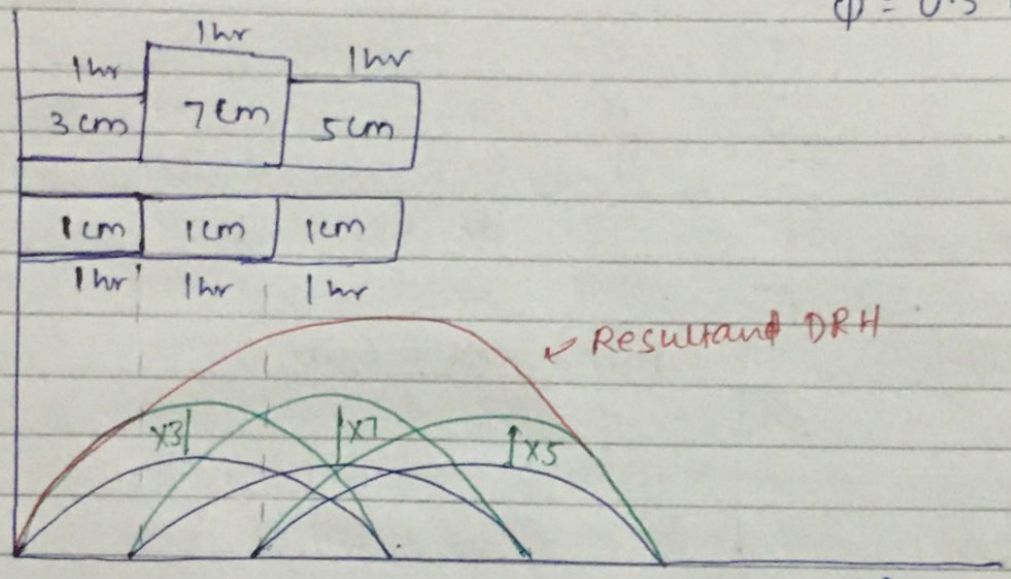
time	cum. Rain (cm)
0	0
1	3.5
2	11
3	16.5

time interval	rain (cm)	Eff. Rainfall cm.
0-1	3.5	3
1-2	7.5	7
2-3	5.5	5

$\phi = 0.5$ cm/hr.



t	1hr UH	DRH due to 1st 1hr Rain	DRH due to 2nd 1hr rain	DRH due to 3rd 1hr Rain	Resultant	Base flow	Flood Hydro graph
0	0	0x3			0	5	5
1	2	2x3	0x7		6	5	11
2	6	6x3	2x7	0x5	32	7	39
3	4	4x3	6x7	2x5	64	7	71
4	2	2x3	4x7	6x5	64	9	73
5	1	1x3	2x7	4x5	37	9	46
6	0	0x3	1x7	2x5	17	11	28
			0x7	1x5	5	11	16
				0x5	0	13	13

Que. The flood hydrograph resulting due to two successive 2h storms with Rainfall depth 2.5 cm & 3.5 cm respectively is given below, $\phi = 0.25$ cm/hr. Find the ordinates of 2h-UH

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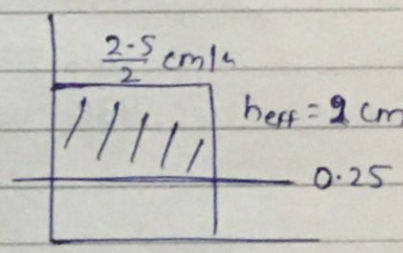
Sol^n *

t	Flood hydrograph	Resultant DRH	Assumed 2h-UH	DRH due to 1st 2h rain	DRH due to 2nd 2h rain	Resultant DRH
0	10	0	0	0×2		0
1	16	6	U_1	$U_1 \times 2$		$2U_1$
2	26	16	U_2	$U_2 \times 2$	0×3	$2U_2$
3	31	21	U_3	$U_3 \times 2$	$U_1 \times 3$	$2U_3 + 3U_1$
4	40	30	U_4	$U_4 \times 2$	$U_2 \times 3$	$2U_4 + 3U_2$
5	32	22	U_5	$U_5 \times 2$	$U_3 \times 3$	$2U_5 + 3U_3$
6	19	9	0	0×2	$U_4 \times 3$	$3U_4$
7	16	6			$U_5 \times 3$	$3U_5$
8	10	0			0×3	0

t	Rainfall	eff. Rainfall
0-2	2.5	2
2-4	3.5	3



$2U_1 = 6 \Rightarrow U_1 = 3\ m^3/sec$

$2U_2 = 16 \Rightarrow U_2 = 8\ m^3/sec$

$2U_4 + 3U_2 = 30 \Rightarrow U_4 = 3\ m^3/sec$

$2U_3 + 3U_1 = 21 \Rightarrow U_3 = 6\ m^3/sec.$

$3U_5 = 6 \Rightarrow U_5 = 2\ m^3/sec.$

Que: The ordinates of 4-H UH for a perticular catchment is given below. Derive S-curve and 2-h UH also calculate the Max. ordinate of S-curve for the catchment area of 630 km^2.

t	0	2	4	6	8	10	12	14	16	18	20	22	24
4hr UH	0	25	100	160	190	170	110	70	30	20	6	1.5	0

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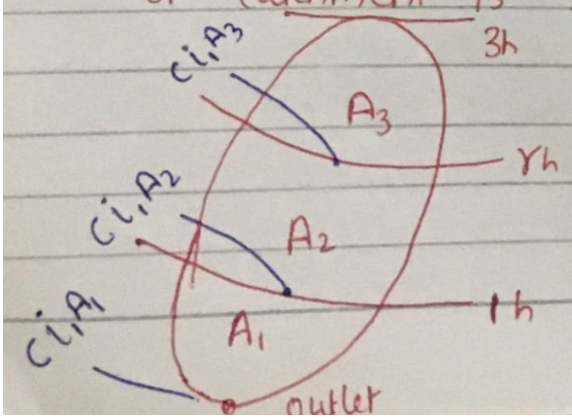
t	4hr-UH	S_A	S_B	$S_A - S_B$		2H-UH
0	0	0		0		0
2	25	25	0	25		50
4	100	100	25	75		150
6	160	185	100	85		170
8	190	290	185	105		210
10	170	355	290	65		130
12	110	400	355	45		90
14	70	425	400	25		50
16	30	430	425	~~5~~	18.75	37.5
18	20	445	430	~~15~~	12.5	25
20	6	436	445	~~-9~~	6.25	12.5
22	1.5	446.5	436	~~10.5~~	0	0
24	0	436	446.5	-10.5	0	0
			436		0	
					0	
					0	

$$Q_e = i_{eff} \times A_{catch}$$

$$= \frac{1}{4}\ cm/h \times 630\ km^2$$

$$= \frac{1}{4} \times \frac{10^{-2}}{3600} \times 630 \times 10^6 = 437.5\ m^3/sec.$$

Que: For the catchment shown below calculate the hourly discharge due to two successive 2 h storm of intensity i_1 and i_2 respectively. Runoff cofficient is c and the concentration time of catchment is 3 hr.



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t	Discharge due to 1st 1 hr. Rain	Discharge Due to 2nd 1 hr. Rain	Discharge due to 3rd 1 hr rain	Discharge due to 4th 1 hr Rain	Resultant
0	0				0
1	ci_1A_1	0			ci_1A_1
2	ci_1A_2	ci_1A_1	0		$ci_1A_2+ci_1A_1$
3	ci_1A_3	ci_1A_2	ci_2A_1	0	$ci_1A_3+ci_1A_2+ci_2A_1$
4	0	ci_1A_3	ci_2A_2	ci_2A_1	$ci_1A_3+ci_2A_2+ci_2A_1$
5		0	ci_2A_3	ci_2A_2	$ci_2A_3+ci_2A_2$
6			0	ci_2A_3	ci_2A_3
7				0	0

SYNTHETIC UNIT HYdrograph:- SUH

It is an artificial UH for an ungauged catchment.
It is obtained by using pre-etablished relation b/w catchment parameters and shape of unit hydrograph for a gauged catchment.

Snyder used 3 parameters for development of SUH

1) Lag time:- (t_l)
   Mid of rainfall to peak of UH
2) Base period (T):
3) peak Discharge Q_p

$$t_l = C_t (L L_{CA})^{0.3}$$

$$\boxed{T = 72 + 3t_l}$$

$$Q_p = \frac{2.78 C_p A}{t_l}$$

$t_l \rightarrow$ lag time in Hrs.

$C_t \rightarrow (1.35-1.65)$. $C_p \rightarrow (0.56-0.69)$

$L \rightarrow$ length of Basin measured along Main stream

$L_{CA} \rightarrow$ Distance b/w centroid of catchment to outlet in km along Main stream.

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$T \rightarrow$ Base period in Hrs.
$Q_p \rightarrow$ peak discharge in m^3/s
$A \rightarrow$ catchment Area in km^2

The standard duration of SUH that is D hour is given as $\frac{t_l}{5.5}$

$$\boxed{D = \frac{t_l}{5.5}}$$

In order to generate D' hr SUH the lag time is modified as below

$$\boxed{t_l' = t_l + \frac{D' - D}{4}}$$

$$T = 72 + 3t_l'$$

$$Q_p = \frac{2.78 C_p A}{t_l'}$$

for smooth plotting of SUH, US Army corporation has given the width of SUH as below

$$W_{50} = \frac{5.87}{(Q_p/A)^{1.08}}$$

$$\boxed{W_{75} = \frac{W_{50}}{1.75}}$$

$W_{50} \rightarrow$ width of SUH @ 50% Q_p
$W_{75} \rightarrow$ width of SUH @ 75% Q_p

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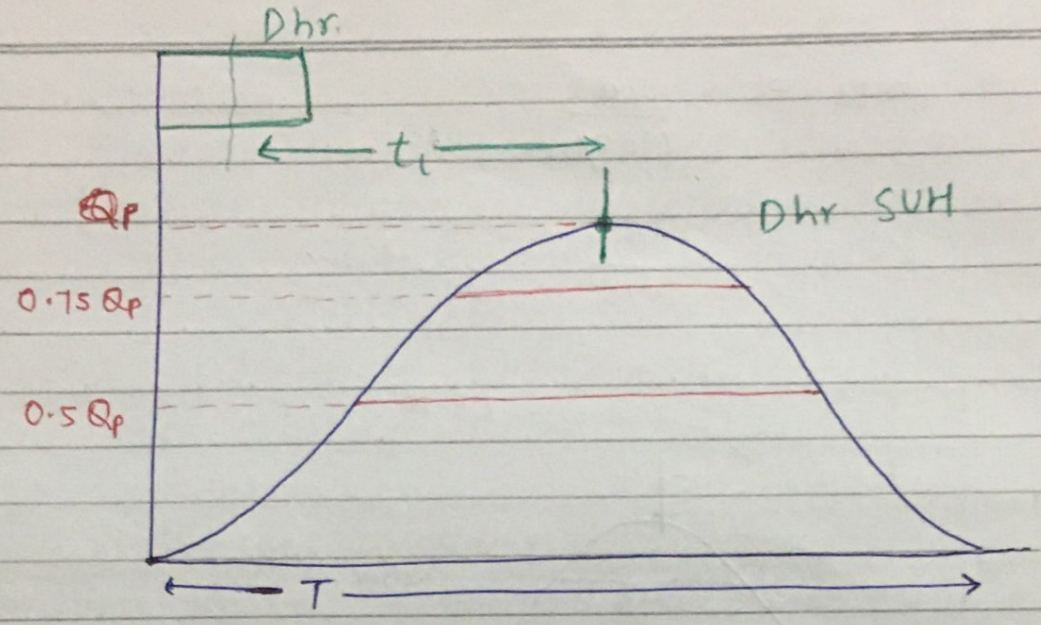

Que. Derive a 3H SUH Based on the Following Data

catchment Area = 2500 km^2

Length of Main stream = 120 km

Distance of centroid from outlet = 80 km

C_t & C_p are 1.5 & 0.6

$$t_l = C_t (L L_{CA})^{0.3}$$

$$= 1.5 (120 \times 80)^{0.3} = 23.48 \text{ hr.}$$

$$D = \frac{t_l}{5.5} = \frac{23.48}{5.5} = 4.27 \text{ hr}$$

$$t_l' = t_l + \frac{D' - D}{4}$$

$$t_l' = 23.48 + \frac{-4.27+3}{4}$$

$$t_l' = 23.16 \text{ hr}$$

$$T = 72 + 3t_l'$$

$$T = 141.5 \text{ hr}$$

$$Q_p = \frac{2.78 C_p A}{t_l'} = 180 \text{ m}^3/\text{sec}$$

3Hr UH

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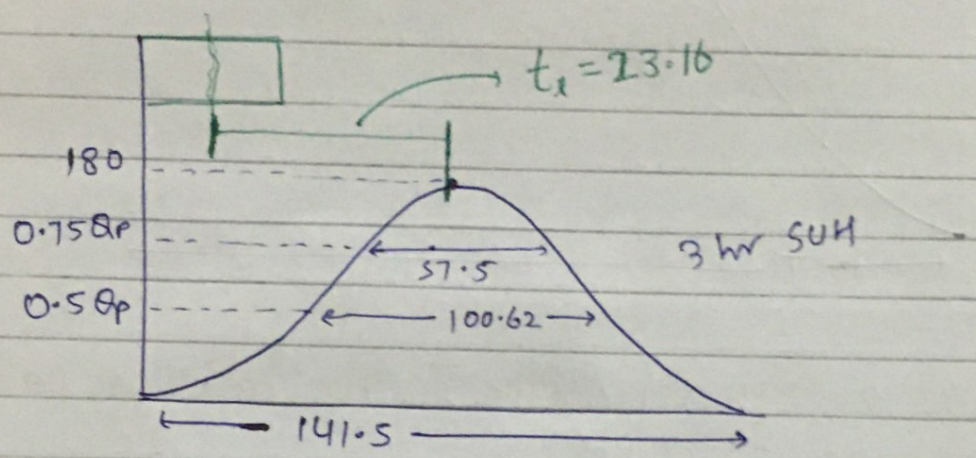
$$W_{50} = \frac{5.87}{(180/2500)^{1.08}} = 100.62 \text{ hr}$$

$$W_{75} = \frac{100.62}{1.75} = 57.5 \text{ hr.}$$



Limitations of UH :-

1) UH can't be used for a catchment with Area < 2 km² and Area > 5000 km²

2) ppt should be in the form of Rainfall only

3) Catchment should not have unusally large storage.

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6 Stream Flow Measurement

Water flowing in stream is known as stream flow and if this flow is unaffected by artificial divergence or storage it is called virgin stream flow or runoff.

Stage :- Height of water level in stream from an arbitory datum

Measurement of stage :-

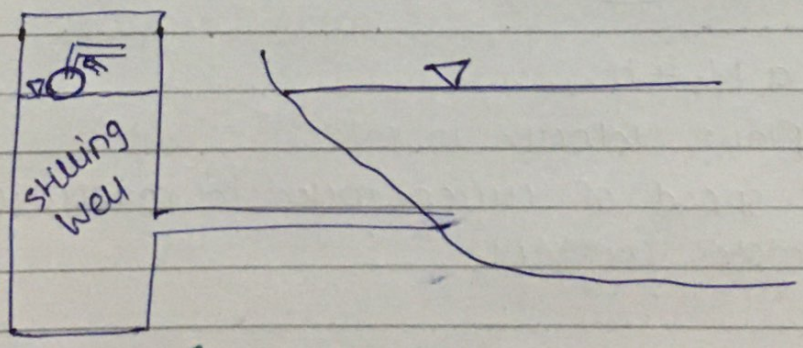
1) Manual stage measurement :-

A) Staff Gauge :- Datum below water surface

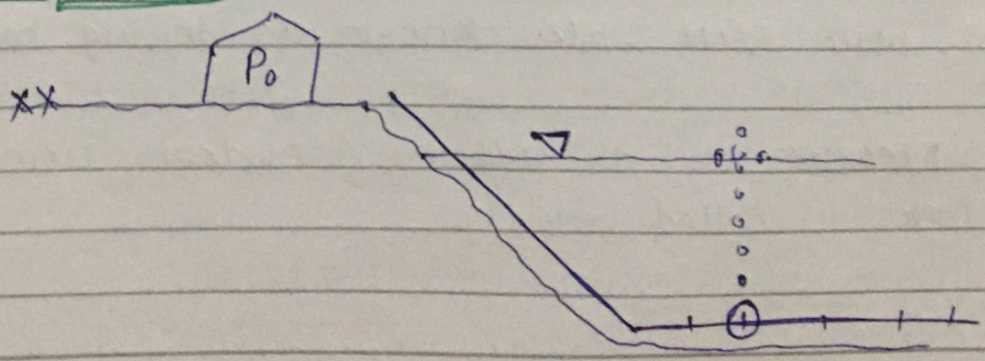
B) Wire Gauge :- Datum above water surface

2) Automatic stage measurement :-

A) Float Gauge :- The elevation of the float in the stilling well is contineously recorded which gives the variation of the stage in the river



B) Bubble Gauge :-



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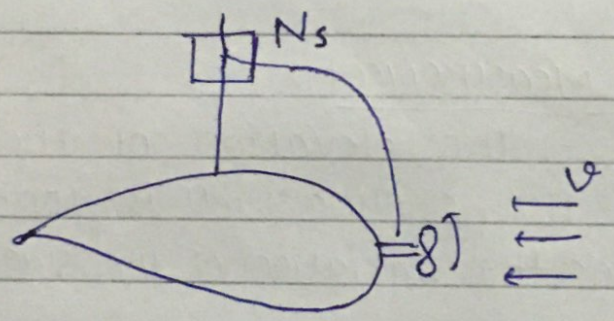
The pressure of compressed air P_o required to create Air Bubbles on the surface is used to calculate stage @ the corresponding station

$$P_o = \rho g h$$

$$h = \frac{P_o}{\rho g}$$

Measurement of velocity :- it is measured using a device known as current meter. It consists of a rotating element (propaler) which rotates with an angular speed proportional to the velocity of the stream due to the reaction (thrust of the stream)



$$v = a N_s + b$$

$v \rightarrow$ stream flow velocity in m/s

$N_s \rightarrow$ Angular speed of current meter in revolution/sec

$a, b \rightarrow$ current meter constants

Current meter callibration is done in a long narrow tank with still water known as towing tank

NOTE :- Measurement of velocity @ a depth using current meter is called Wading.

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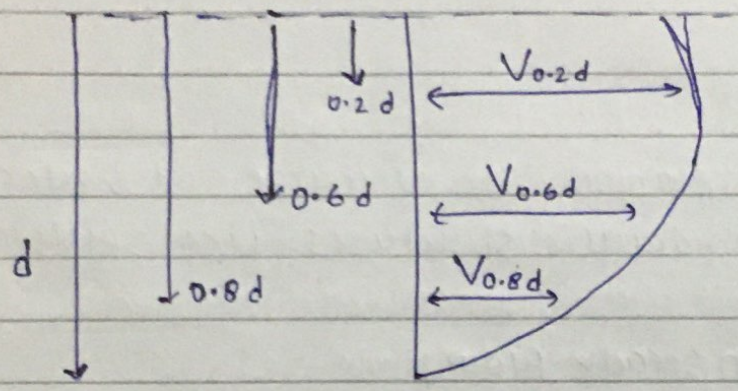
The velocity distribution in a stream accross a vertical sec^n is logrithmic in nature



In case of shallow streams (upto 3 m. deep) the avg. sectional velocity is given as

$$V_{avg.} = V_{0.6d}$$ (1 point Method)

$V_{0.6d}$ is velocity @ 0.6 times depth from surface

In deep streams (> 3m Deep) the avg. sectional velocity is given as

$$V_{avg.} = \frac{V_{0.2d} + V_{0.8d}}{2}$$

$V_{0.2d}$, $V_{0.8d}$ → velocities @ 0.2 and 0.8 times depth from Surface respectively.

2 point method

Avg. velocity is also calculated using surface velocity

$$V_{avg.} = K\, V_{surface}$$

$$K = 0.85 \rightarrow 0.95$$

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Measurement of Discharge :-

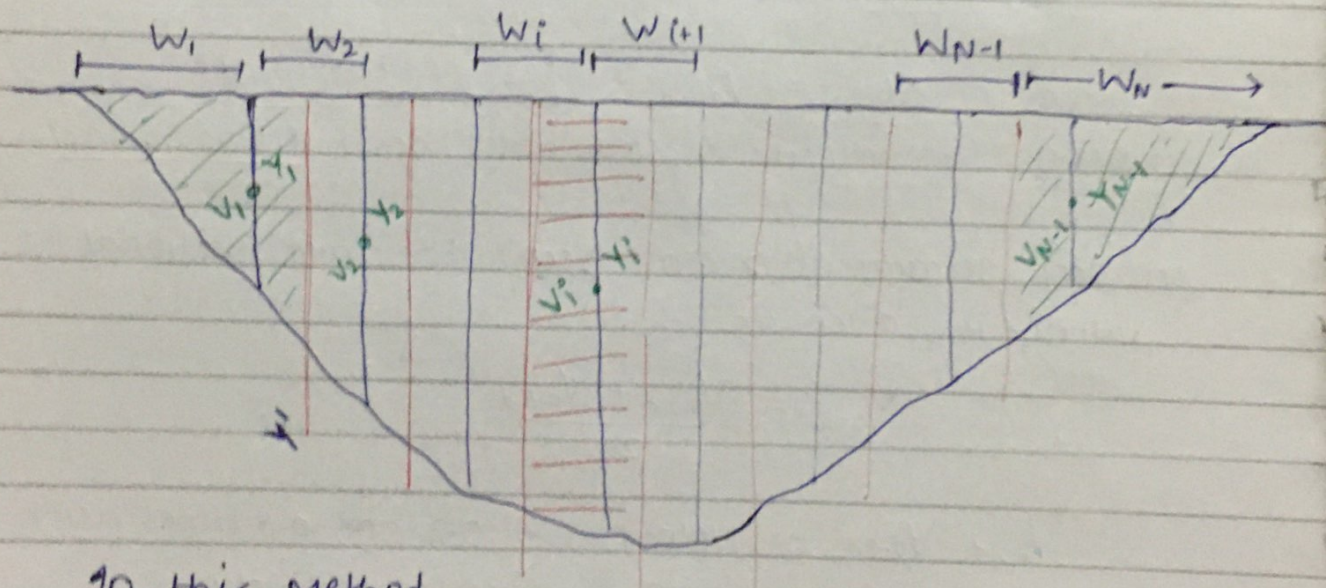
I) Direct Methods

1) Area velocity Method (moving boat Method)
2) Dilution technique (steady Flow)
3) ultrasonic Method
4) electromagnetic Method

Indirect Methods

1) using pre-established relation b/w discharge and water surface elevation for hydraulic structures like spillways wiers, knotches (etc)
2) Slop Area Method (Un steady Flow)

① Area velocity Method :-



In this Method
The whole secⁿ of river is devided into a No. of secⁿs and then sectional discharge is calculated assuming 1st and last secⁿ to be tringles and intermediate secⁿ to be rectangles.

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Summation of sectional discharges gives total discharge

$$Q_{Total} = \underset{\text{Tringle}}{Q_{1st}} + \underset{\text{Rectangle}}{Q_{intermediate}} + \underset{\text{Tringle}}{Q_{last}}$$

$$\underset{\text{Tringle}}{Q_{1st}} = \left[\frac{1}{2} \times \left(W_1 + \frac{W_2}{2}\right) \frac{Y_1\left(W_1 + \frac{W_2}{2}\right)}{W_1}\right] V_1 = \frac{\left(W_1 + \frac{W_2}{2}\right)^2 Y_1 V_1}{2 W_1}$$

$$\left\{ \frac{Y'}{W_1 + \frac{W_2}{2}} = \frac{Y_1}{W_1} \right.$$

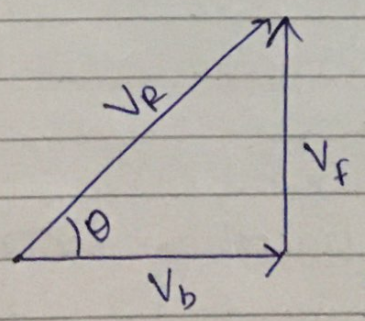
$$Y' = \frac{Y_1 \left(W_1 + \frac{W_2}{2}\right)}{W_1}$$

$$\underset{\text{Tringle}}{Q_{last}} = \frac{\left(W_n + \frac{W_{n-1}}{2}\right)^2 Y_{N-1} V_{N-1}}{2 W_n}$$

$$\underset{\text{Rectangles}}{Q_{intermediate}} = \sum_{i=2}^{N-2} \left(\frac{W_i + W_{i+1}}{2}\right) Y_i V_i$$

Another alternative is moving boat Method in which contrary to the Area velocity method the Observer moves in a boat ~~that~~ is Not stationary.

The moving observer collect Data in real time.



$$V_f = V_R \sin\theta$$

$$V_b = V_R \cos\theta$$

$V_R \rightarrow$ resultant velocity measured by current meter attached to the boat.

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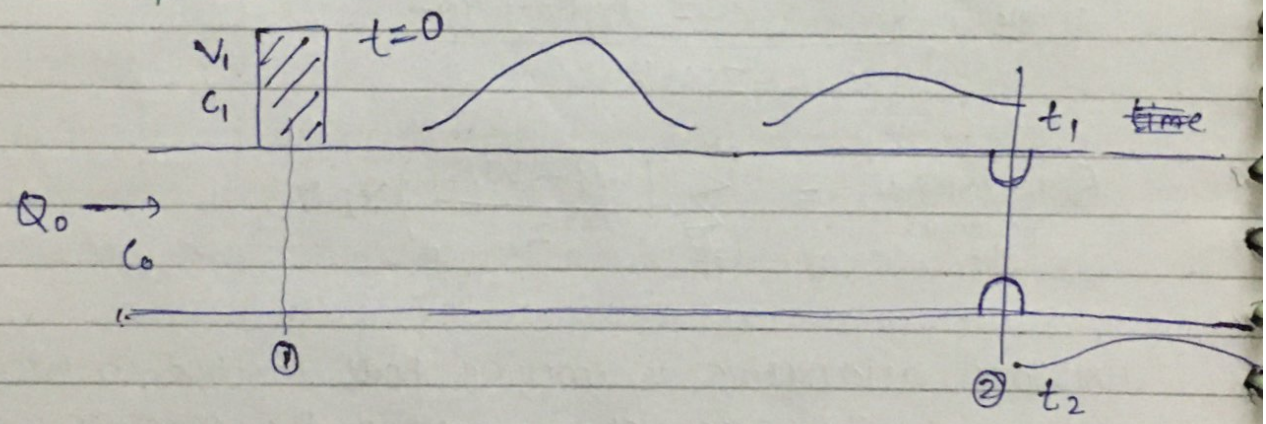
$V_f \rightarrow$ flow velocity
$V_b \rightarrow$ velocity of boat $\perp$ to the flow

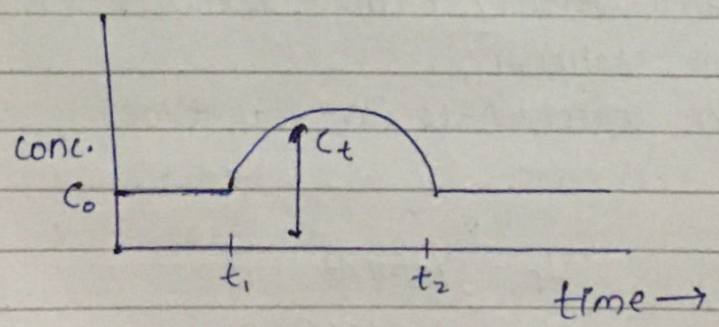
width = $\perp$ velocity $\times$ time

Obj

2) **Dilution Technique :-** In this technique a stable solution like commen salt or sodium dicromate (known as tracer) is injected into the stream either all at once ~~or~~ (Gulp Method) or at a constant rate (Plateau Method)
Mass conservation for tracer is used to calculate Stream flow discharge.

1) **Gulp Method :-**





Applying mass conservation for the tracer b/w sec^n ① & ②
We have

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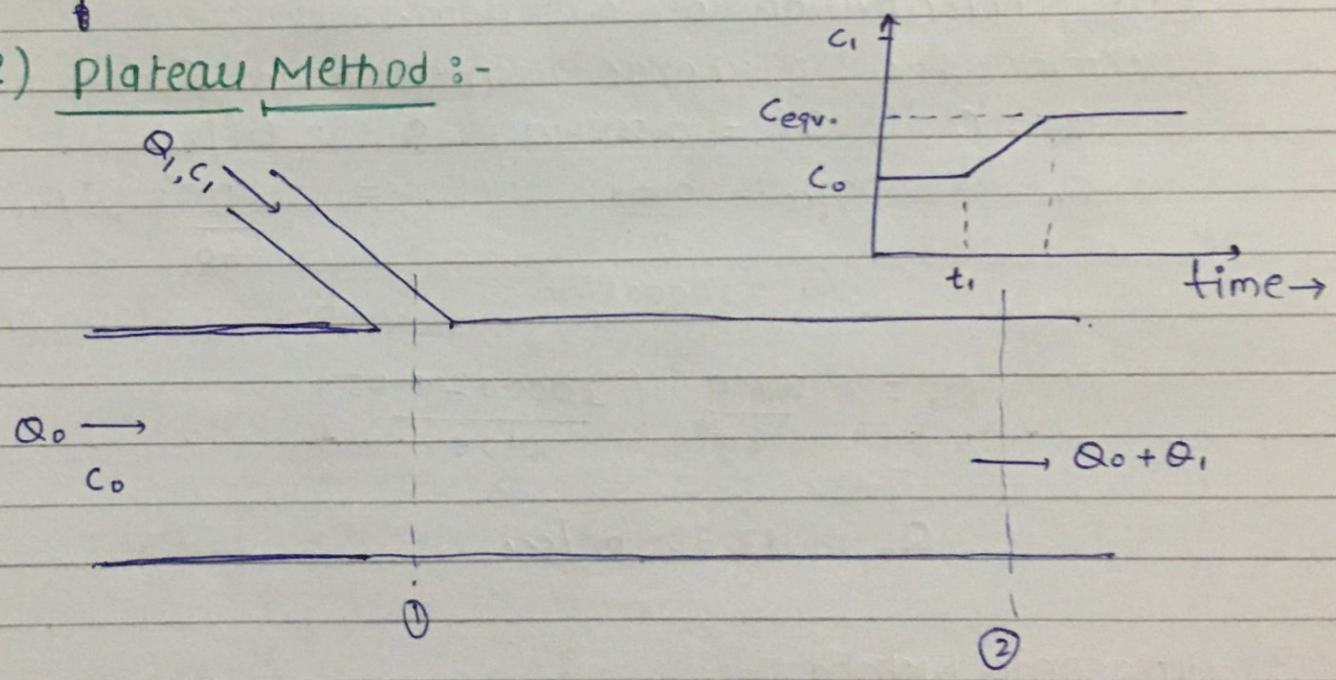
$$V_1 C_1 = \int_{t_1}^{t_2} \left(Q_0 + \cancel{\frac{V_1}{t_2 - t_1}} \right) dt \, (C_t - C_0)$$

Negligible

$$Q_0 = \frac{V_1 C_1}{\int_{t_1}^{t_2} (C_t - C_0)\, dt}$$

Q_0 is initial discharge in the river
C_0 is initial concentration of the tracer in river
V_1 is the volume of high concentration C_1 added
C_t is concentration of tracer @ secⁿ ② @ any time t

2) Plateau Method :-



applying mass conservation for tracer b/w secⁿ ① & ② after equilibirium has reached @ secⁿ ② we've

$$(Q_0 dt) C_0 + (Q_1 dt) C_1 = (Q_0 + Q_1) dt \, C_{eq.}$$

$$Q_0 (C_0 - C_{eq}) = Q_1 (C_{eq.} - C_1)$$

$$Q_0 = \frac{Q_1 (C_{eq.} - C_1)}{(C_0 - C_{eq.})} = \frac{Q_1 (C_1 - C_{eq})}{(C_{eq} - C_0)}$$

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$$\left\{ \frac{mg}{lit} = ppm \right.$$

Q_0 is initial discharge in the stream
C_0 is initial concentration of tracer in the stream
Q_1 is constant rate of addition of high concentration C_1 @ sec^n ①

C_{eq} is equilibirium concentration @ sec^n ②

Que. Into a stream with no trace of salt initially a salt solution with concentration of 20 mg/cc is introduced at a constant rate of 2 lit/min. The samples collected at a downstream sec^n sufficiently far away indicated and equilibirium salt concentration of 0.05 ppm.
Determine the Discharge in the stream from this Data.

$C_0 = 0$, $C_{eq.} = 0.05 ppm$, $Q_1 = 2$ lit/min

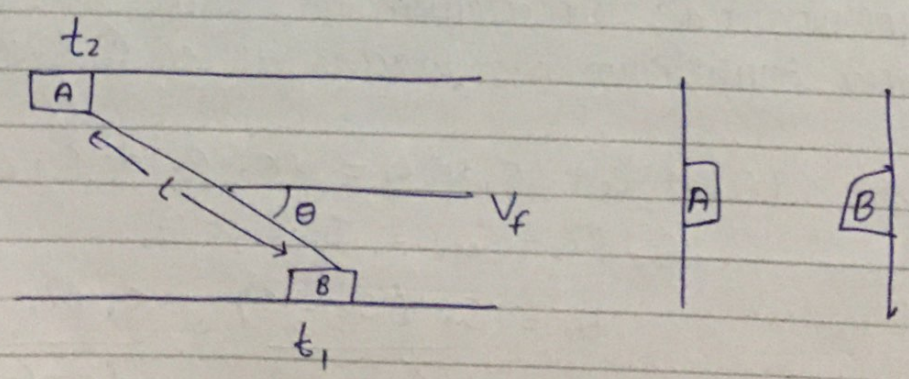
$C_1 = 20$ mg/cc $= \frac{20 mg}{ml}$ $= \frac{2 \times 10^{-3}}{60}$ m^3/sec.

$C_1 = 20000 ppm$

$$Q_0 = \frac{2 \times 10^{-3}}{60} \left(\frac{20000 - 0.05}{0.05} \right)$$

$$Q_0 = 13.33 \ m^3/sec.$$

3) Ultrasonic Method :-



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$$U + V_f \cos\theta = \frac{L}{t_1}$$

$$U - V_f \cos\theta = \frac{L}{t_2}$$

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

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

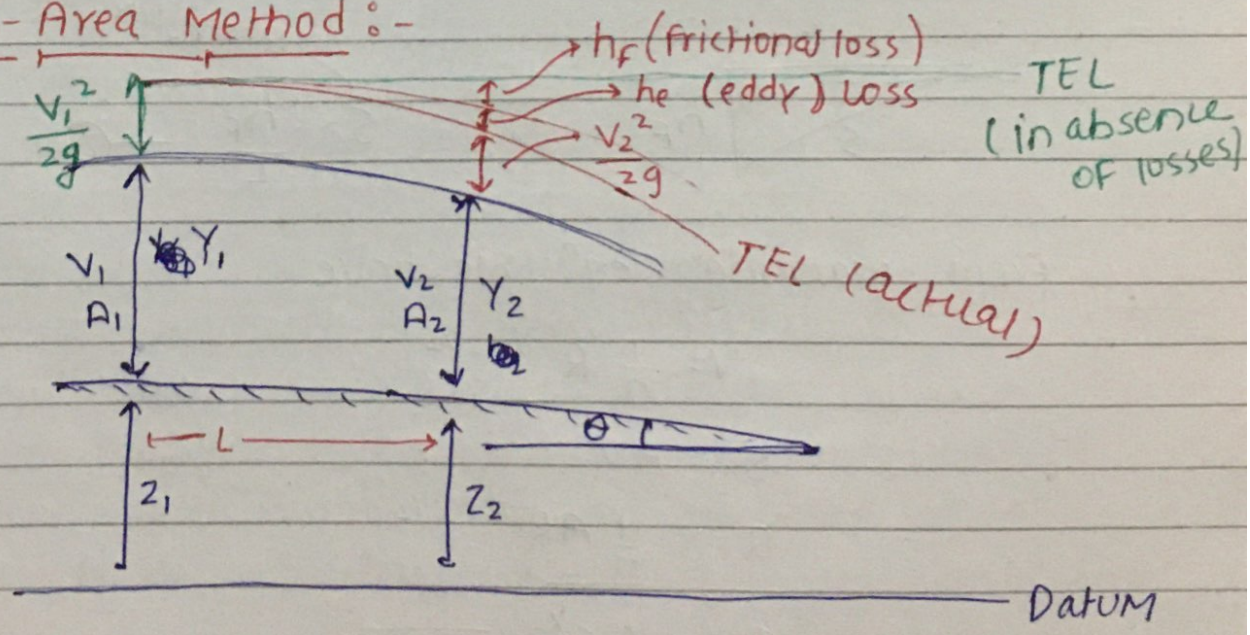
$$Q = V_f A$$

$U \rightarrow$ speed of ultrasonic pulse in still water
$V_f \rightarrow$ velocity of flow
$t_1 \rightarrow$ time taken by pulse to travel from A to B
$t_2 \rightarrow$ time taken by pulse to travel from B to A
$L \rightarrow$ distance b/w st^n A & B

Indirect Method

UN-steady flow (Trial & error)

Slop-Area Method :-



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Applying Bernoulli's eq^n b/w sec^n ① & ②

We have $Z_1 + Y_1 + \frac{V_1^2}{2g} = Z_2 + Y_2 + \frac{V_2^2}{2g} + h_f + h_e$

$$h_f = \{(Z_1+Y_1)-(Z_2+Y_2)\} + \frac{V_1^2-V_2^2}{2g} - h_e$$

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

where, $Z_1+Y_1 = h_1$, $Z_2+Y_2 = h_2$

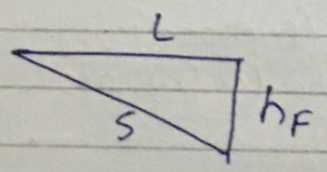
$h_e = K_e \left|\frac{V_1^2-V_2^2}{2g}\right|$

$K_e \rightarrow$ eddy constant

NOTE: Eddy constant K_e depends on change in c/s profile like sudden contraction, expansion etc.

$h_1 - h_2$ is drop in water surface elevation

Neglecting eddy losses and assuming the total energy line to be linear



Enery line slop

$$S = \frac{h_F}{L} \quad \text{—— ②}$$

From Mannings eq^n we have

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

$$Q = AV$$

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

$$\boxed{Q = K S^{1/2}} \quad \text{—— ③}$$

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

K is called Coverence of a channel

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from ② & ③ we have

$$\boxed{S = \frac{Q^2}{K^2} = \frac{h_F}{L}} \quad \text{— ④}$$

obj. ***

if convergence at 2 sec^n are given as K_1 and K_2 we ~~work~~ work in terms of equivelent convergence

$$K_{equ.} = \sqrt{K_1 K_2}$$

steps involved

Step 1) Assume $V_1 = V_2$

calculate h_f from eq^n ①

$$h_f = h_1 - h_2$$

Step 2) from eq^n ④ and the above calculated H_f calculate discharge

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

Step 3) for above calculated discharge calculate velocities V_1 & V_2

$$V_1 = \frac{Q}{A_1}, \quad V_2 = \frac{Q}{A_2}$$

Using above velocities again calculate h_f then the corresponding discharge.

keep repeating these steps until two conjucutive same value of Discharge is obtained which is the correct Discharge.

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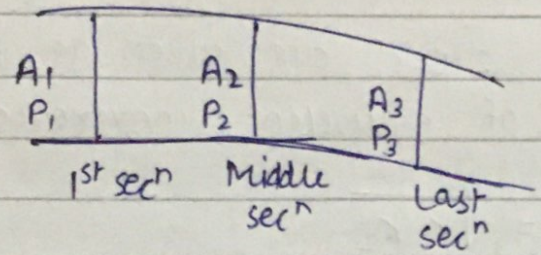
NOTE

When Areas and Wetted perimeter of 3 secⁿ are given equivilent conveyence is calculated as follows :-



$$A_{eq.} = \frac{A_1 + 2A_2 + A_3}{4}$$

$$P_{eq.} = \frac{P_1 + 2P_2 + P_3}{4}$$

$$R_{eq.} = A_{eq} / P_{eq.}$$

$$\boxed{K_{eq.} = \frac{1}{n} A_{eq.} R_{eq}^{2/3}}$$

Direct

But Do not follow this in exam!

$$\boxed{L\frac{Q^2}{K^2} = (h_1 - h_2) + \frac{Q^2}{2g}\left(\frac{1}{A_1^2} - \frac{1}{A_2^2}\right) - \frac{K_e Q^2}{2g}\left|\frac{1}{A_1^2} - \frac{1}{A_2^2}\right|}$$

Que: During a flood flow the depth of water in 10 m wide Rectangular channel was found to be 3 m & 2.9 m at secⁿ 200 m apart. A drop in water surface elevation was found to be 0.12 m. Assume mannings coffi = 0.025 and eddy const. = 0, Estimate the flood discharge through channel.

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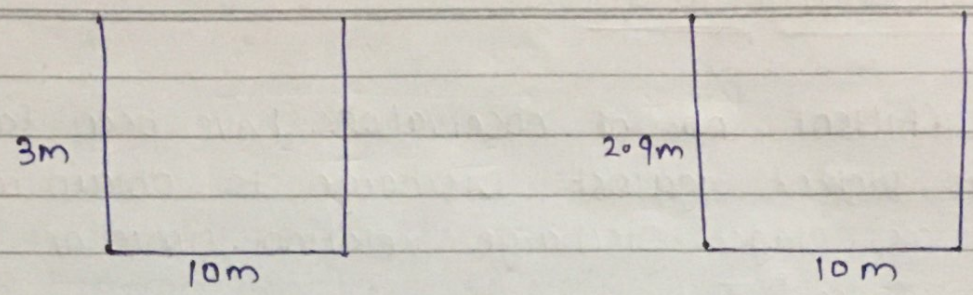
Sol^n.



$A_1 = 30\ m^2$ $\qquad$ $A_2 = 29\ m^2$

$P_1 = 16\ m$ $\qquad$ $P_2 = 15.8\ m$

$R_1 = \frac{30}{16} = 1.875\ m$ $\qquad$ $R_2 = \frac{29}{15.8} = 1.835\ m$

$K_1 = \frac{1}{n} A_1 R_1^{2/3} = 1824.7$ $\qquad$ $K_2 = \frac{1}{n} A_2 R_2^{2/3} = 1738.7$

$$K_{eq.} = \sqrt{K_1 K_2} = 1781.2$$

Step-1 $\quad V_1 = V_2$

$h_f = 0.12$

$$Q_1 = K_{eq.}\sqrt{\frac{h_f}{L}} = 1781.2\sqrt{\frac{0.12}{200}}$$

$$= 43.63\ m^3/sec.$$

$V_1 = \frac{Q_1}{A_1} = 1.454\ m/sec.$, $V_2 = \frac{Q_1}{A_2} = 1.504\ m/sec$

putting in eq^n ①

$$h_{f_2} = (h_1 - h_2) + \frac{V_1^2 - V_2^2}{2g}$$

$h_{f_2} = 0.1124$

$$Q_2 = K_{eq}\sqrt{\frac{h_{f_2}}{L}} = 1781.2\sqrt{\frac{0.1124}{200}} = 42.22\ m^3/sec.$$

$Q_1 \neq Q_2$

$V_1 = \frac{Q_2}{A_1} = \frac{42.22}{30} = 1.407\ m/sec$ $\qquad$ $V_2 = \frac{Q_2}{A_2} = \frac{42.22}{29} = 1.4558$

By iterration (Q_4 stage)

$$Q_3 = Q_4 = 42.32\ m^3/sec.$$

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Stage- Discharge Relation:-

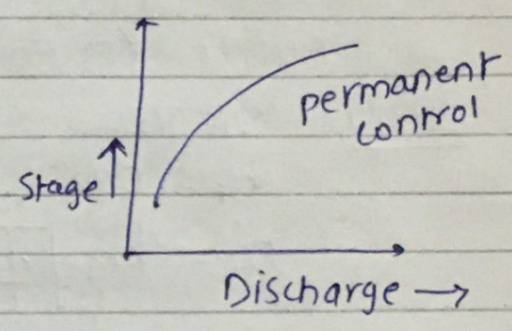
After sufficient no. of observations have been taken stage is plotted against Discharge to obtain a curve known as stage Discharge relation curve or "Rating curve."

If this relation does not changes with time it is called permanent control and if this relation changes with time it is called shifting control

$$Q = \alpha (G - \beta)^{\gamma}$$

Q → Discharge

G → Stage



Causes of shifting control:-

1) Weed Growth
2) Agradation or degradation of channel
3) Variable backwater effect
4) Unsteady flow

Correction for shifting control Due to

1) Variable back water effect It is corrected by using a secondary Gauge (Auxilary Gauge) installed at some location downstream of Main Gauging Station

The difference of stages b/w Main Gauging station and secondary Gauging station known as Fall (F) is calculated. The actual discharge at the Main Gauging station is given as

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$$\frac{Q}{Q_0} = \left(\frac{F}{F_0}\right)^m$$

$Q \rightarrow$ Actual Discharge at main Gauging station

$F \rightarrow$ Difference b/w stage at Main Gauging station and Auxilary Gauging station

$Q_0 \rightarrow$ Normalised discharge when the fall is f_0 at the same stage as that of Main Gauging station

Que. An Auxilary Gauge was used downstream of a Main Gauge in a river to provide corrections due to back water effect the following Data were Noted. If the Main Gauge reading is still 86 m. Estimate the Discharge in the river. Auxilary Gauge reads 85.3 m.

Main Gauge above Datum	Aux. Gauge above Datum	Q (m^3/sec)	
86 m	85.5 m	275	Q_1
86 m	84.8 m	600	Q_2
86 m	85.3 m	371	Q_3

$$\frac{Q_1}{Q_0} = \left(\frac{F_1}{F_0}\right)^m \quad \text{—(1)}$$

$$\frac{Q_2}{Q_0} = \left(\frac{F_2}{F_0}\right)^m \quad \text{—(2)}$$

$$\frac{Q_1}{Q_2} = \left(\frac{F_1}{F_2}\right)^m$$

$Q_1 = 275$

$Q_2 = 600$

$F_1 = 0.5$

$F_2 = 1.2$

$m = 0.89$

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$$\frac{Q_3}{Q_0} = \left(\frac{f_3}{f_0}\right)^m \quad —③$$

① % ③

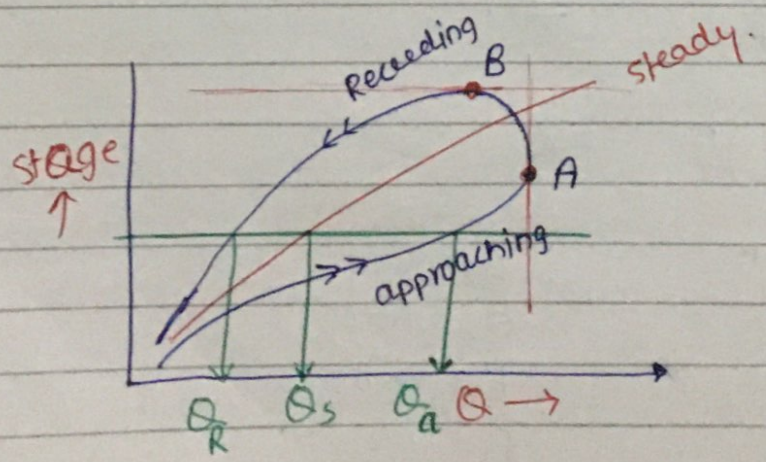
$$\frac{Q_1}{Q_3} = \left(\frac{f_1}{f_3}\right)^m$$

$$\frac{275}{Q_3} = \left(\frac{0.5}{0.7}\right)^{0.89}$$

$$Q_3 = 371 \ m^3/sec.$$

2) <u>UNsteady Flow</u> :- In advancing portion of flood wave the approching velocity is Greater than the corresponding steady state condition hence for same stage the Discharge is Greater in the advancing portion of wave as compare to steady state condition.

In the receeding portion of the wave the velocity is lower than the corresponding steady state cond. Hence for same stage the Discharge is lower than the steady state condition.



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A → represents Max. Discharge
B → represents Max. stage

NOTE Max. Discharge is obtained before Max stage in unsteady flow

The actual Discharge for a given stage is calculated as

$$\boxed{Q_{actual} = Q_{steady}\sqrt{1 + \frac{dh/dt}{V_w S_o}}}$$

Q_{steady} is steady state discharge at same stage
dh/dt → rate of changing of stage

V_w → wave velocity
S_o → Bed slop

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7 FLOOD

IMP.

Any flow which is relatively high and which crosses it's natural or artificial embankment may be called flood

Types of Flood :-

1) **Max. probable Flood** :- It is the estimate of flood occuring from most severe combination of Metreological and hydrological condition including extreamly rare combinations

- use for structures with storage > 60 Mm^3

Ex: spillways for large Dams

if MPF is not available 1000 return year period flood can be used instead.

2) **Standard project Flood** :- It differs from MPF in the way that it excludes extreamly rare combinations.

→ Used for permanent barrage with storage < 60 Mm^3

In absence of Data 100 year Return period flood can be used

NOTE: MPF is derived from PMP and SPF is derived from SPS.

SPF is approx 60 - 80 % of MPF

3) **Design Flood** :- It is the flood adopted for the Design of hydraulic structures. it can either be SPF or MPF or a flood of any desired return period depending upon economy and the degree of protection to be provided.

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Estimation of Design Flood :-

1) Envelop curve :-
2) Using emperical eqⁿ

a) dicken's Formula $\boxed{Q_p = C_D A^{3/4}}$

$Q_p \rightarrow$ peak flood discharge (m^3/s)

$C_D \rightarrow$ dicken's constant (6 — 30)

$A \rightarrow$ Area in km^2

Used for northen and central parts of the country

b) Rybes formula

$$\boxed{Q_p = C_R A^{2/3}}$$

Applicable for Andha, karnatake and Tamilnadu.

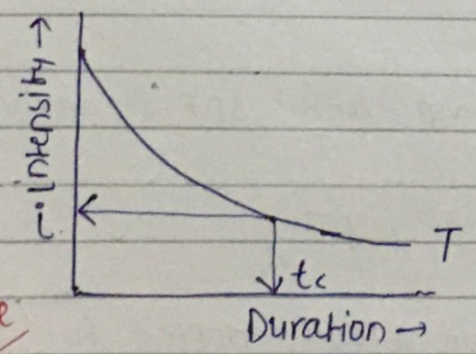
c) English formula

$$\boxed{Q_p = \frac{124 A}{\sqrt{10.4 + A}}}$$

Used in western ghats in Maharastra

d) Rational Formula for Areas < 50 km^2

$$\boxed{Q = CiA}$$



IDF curve

$$i = \frac{kT^x}{(D+a)^n}$$

$$i = \frac{kT^x}{(t_c+a)^n}$$

$$Q = CiA$$

$Q \rightarrow$ Design Discharge

$C \rightarrow$ Runoff cofficient or impermeability factor (Runoff / Rainfall)

$$C_{eq} = \frac{C_1A_1 + C_2A_2 + C_3A_3}{A_1 + A_2 + A_3}$$

$i \rightarrow$ intensity of Rainfall corresponding to t_c

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Estimation of concentration time (t_c) :-

1) US practice

For small catchment lag time (Mid of Rainfall to peak of DRH) is taken = t_c

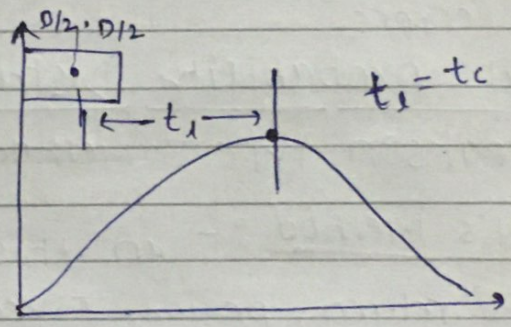
$$t_c = t_l = c\left(\frac{L L_{CA}}{\sqrt{S}}\right)^n$$

$t_c \rightarrow$ concentration time in Hr.

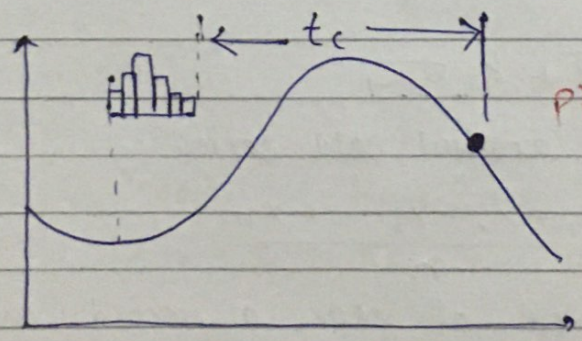
$t_l \rightarrow$ lag time in Hr.

$L_{CA}, L \rightarrow$ ~~length~~ same as in SUH

$S \rightarrow$ catchment slop



$T = D + t_c$ is not applicable.



② Kirpich EQⁿ :-

$$\boxed{t_c = 0.01947 \times L^{0.77} S^{-0.385}}$$

t_c in Minutes

$L \rightarrow$ Max. length of travel of water in (m)

$S \rightarrow$ Basin slop

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Estimation of Flood for a perticular return period :-

1) Emperical probability Distribution :-

Return period is calculated using Wiebull eq^n

$$T = \frac{N+1}{m}$$

A plot is prepared b/w discharge and their corresponding Return periods and then interpolating or extrapolating the curve Discharge corresponding to a perticular return period is calculated.

When return period is large, extrapolation introduces errors.

2) Theortical probability Distribution :-

a) Log pearson type 3 Method

b) Gumbel's Method :- In this Method Flood discharge (X_t) of return period T years is calculated statistically as below:-

$$X_T = \bar{X} + K_T \sigma_{n-1}$$

$\bar{X}$ → Mean of annual Data series

$$\bar{X} = \frac{X_1 + X_2 + X_3 \cdots + X_n}{n}$$

Where n is no. of years of record available.

σ_{n-1} is standard deviation of annual Data series

$$\sigma_{n-1} = \sqrt{\frac{(\bar{X}-X_1)^2 + (\bar{X}-X_2)^2 + \cdots (\bar{X}-X_n)^2}{n-1}}$$

K_T → frequency factor, it is calculated as

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

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Where $Y_T \rightarrow$ reduced variate and it is calculated as

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

$\bar{Y}_n$ is mean of reduced variate and
S_n is standard deviation of mean variate
the values of $\bar{Y}_n$ and S_n is calculated from a table corresponding to no. of years of record available

For $n > 200$ $\left.\begin{array}{l}\bar{Y}_n = 0.577 \\ S_n = 1.2825\end{array}\right\}$ can be used even for $n > 50$, without much error

Que. Flood frequency computations using Gumble's Method gave following results.
Estimate the flood magnitude with return period 500 years.

Return period (yr)	Discharge (m^3/sr)
50	40809
100	46300

$$X_{100} = \bar{X} + K_{100}\,\sigma_{n-1}$$

$$X_{50} = \bar{X} + K_{50}\,\sigma_{n-1}$$

$$X_{100} - X_{50} = (K_{100} - K_{50})\,\sigma_{n-1}$$

$$= \left(\frac{Y_{100} - \bar{Y}_n}{S_n} - \frac{Y_{50} - \bar{Y}_n}{S_n}\right)\sigma_{n-1}$$

$$= (Y_{100} - Y_{50})\,\frac{\sigma_{n-1}}{S_n}$$

$$X_{100} - X_{50} = \left(\ln \ln \frac{50}{49} - \ln \ln \frac{100}{99}\right)\frac{\sigma_{n-1}}{S_n} \quad \text{—(1)}$$

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Similarly $X_{500} - X_{100} = \left(\ln\ln\frac{100}{99} - \ln\ln\frac{500}{499}\right)\frac{\sigma_{n-1}}{S_n}$ — ②

② ÷ ①

$$\frac{X_{500} - X_{100}}{X_{100} - X_{50}} = \frac{\ln\ln\frac{100}{99} - \ln\ln\frac{500}{499}}{\ln\ln\frac{50}{49} - \ln\ln\frac{100}{99}}$$

$X_{500} = 58988.86\ m^3/hr.$

$X_{500} = 60{,}000\ m^3/hr$

For the Mean of annual Data series (that is $\bar{X}$) Gumble's Method Yields a return period of 2.33 years. Hence Flood with Return period 2.33 years is called Mean Annual Flood

$X_T = \bar{X} + K_T\,\sigma_{n-1}$

$\bar{X} = \bar{X} + K_T\,\sigma_{n-1}$

$K_T = 0$

$\frac{Y_T - \bar{Y}_n}{S_n} = 0$

$Y_T = \bar{Y}_n$

$-\ln\ln\left(\frac{T}{T-1}\right) = 0.577$

$T = 2.33$ years.

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Confidence Limit :- Due to limited Data values Flood discharge X_T determined by Gumble's Method can have errors.

Hence we define a confidence limit around the calculated X_T

$$X_{1,2} = X_T \pm f(\alpha) S_e$$

Where $f(\alpha)$ is a F^n of confidence probability α

α	50 %	68 %	95 %	98 %
$f(\alpha)$	0.675	1	1.98	2.58

$S_e \rightarrow$ probable error

$$S_e = \frac{b\,\sigma_{n-1}}{\sqrt{n}}$$

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

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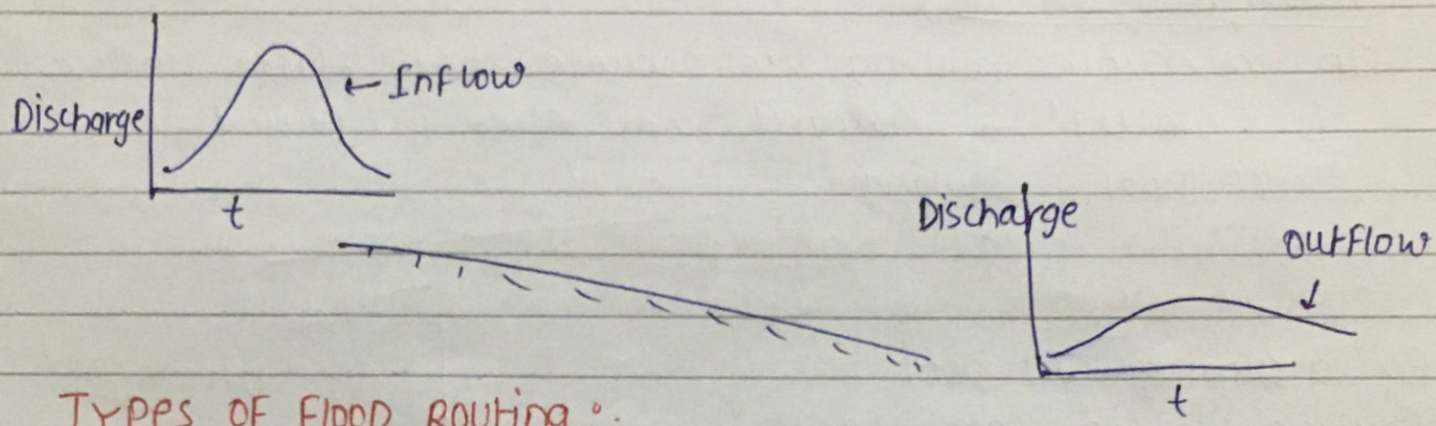
Flood Routing.

It is a procedure in which shape of flood hydrograph at a perticular location on the stream is calculated from a known or assumed flood hydrograph at some location up stream



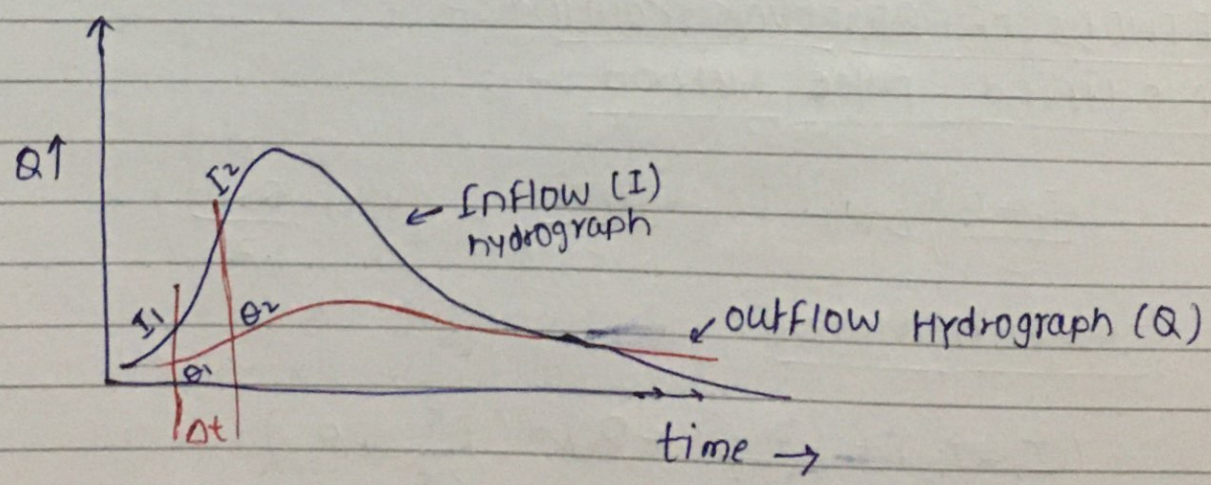
Types of Flood Routing :-

1) Hydrologic Routing or lumped Routing :-

1) Discharge is taken as a function of time
2) continuity eqⁿ is used

2) Hydralic Routing or Distributed Routing :-

1) Discharge is taken as a function of time and space
2) continuity and momentum eqⁿ are used.



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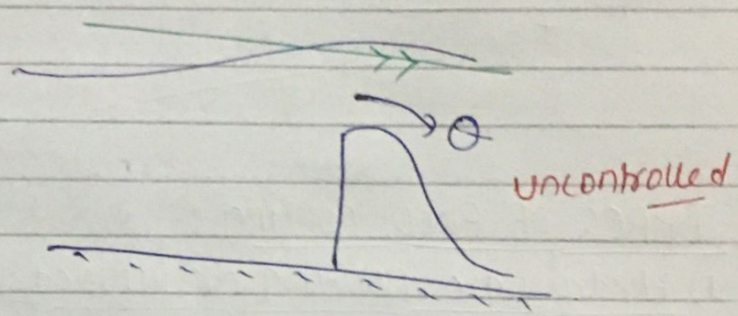
In time Δt

$$\left\{ I - Q = \frac{ds}{dt} \right\}$$

$$\boxed{\left(\frac{I_1 + I_2}{2}\right)\Delta t - \left(\frac{Q_1 + Q_2}{2}\right)\Delta t = S_2 - S_1}$$

Reservoir routing :-

In reservoir routing we assume that water surface is parallel to channel bed. This is called level pool Routing.



By assuming level pool Routing we can say storage is a function of water surface elevation

$$S = f(h)$$

outflow discharge is a function of water surface elevation

$$Q = f(h)$$

Thus we can storage is a function of outflow discharge only.

$$\boxed{S = f(Q)}$$

METHODS OF RESERVOIR routing :-

1) Modified puls Method :-

$$\left(\frac{I_1 + I_2}{2}\right)\Delta t + \left(S_1 - \frac{Q_1 \Delta t}{2}\right) = \left(S_2 + \frac{Q_2 \Delta t}{2}\right)$$

2) Goodrich Method :-

$$(I_1 + I_2) + \left(\frac{2 S_1}{\Delta t} - Q_1\right) = \left(\frac{2 S_2}{\Delta t} + Q_2\right)$$

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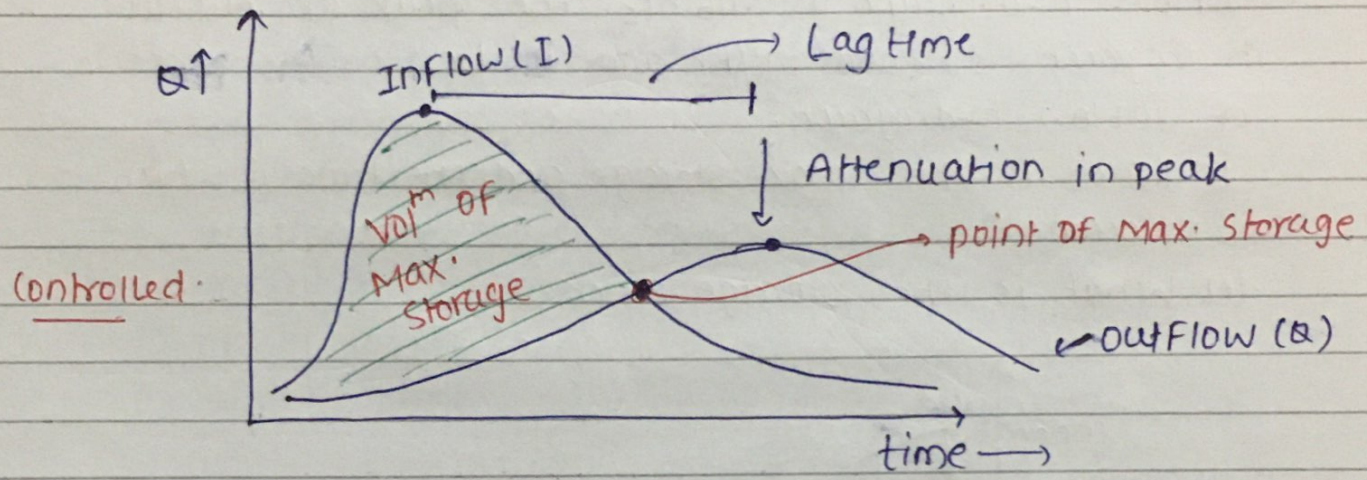
③ Standard 4th order Runge-Kutta Method :-

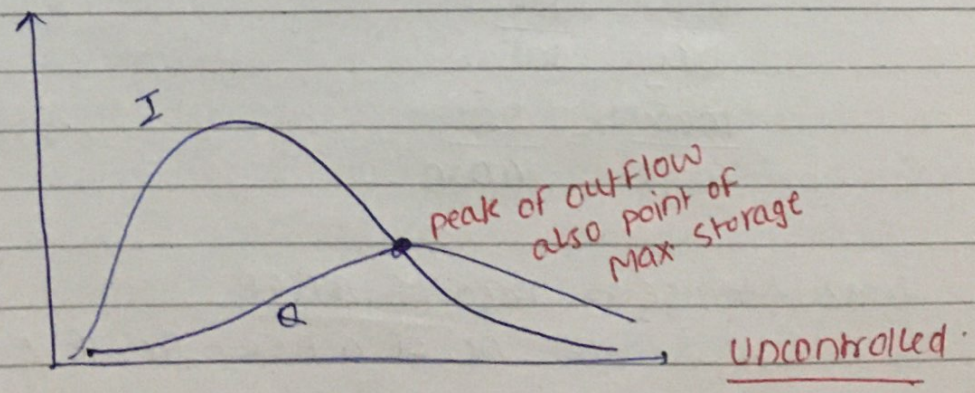
It is a Numerical Method

Attenuation, Lag & Storage Charecteristics :-



* the Max. Storage occurs at the point of intersection of inflow and outflow hydrograph.

* for uncontrolled reservoirs peak of outflow hydrograph consides with the point of intersection of inflow and outflow hydrograph



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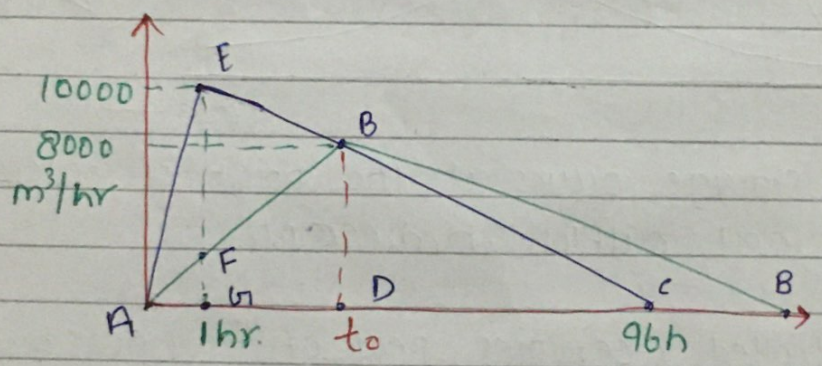
Que: Inflow & outflow hydrograph of a channel reach are triangular in shape and are plotted simultaneously as shown below:-

peak of inflow hydrograph is 10000 m^3/hr and occurs @ 1 Hr from start. Base ~~flow~~ period of inflow hydrograph is 96 hrs. The peak of outflow hydrograph is 8000 m^3/hr and it lies on the recession of inflow hydrograph.

i) What is the Max. storage and the time @ which it occurs

ii) What is the storage after 1 hr.



Δ DBC & Δ GEC are similar

$$\frac{GC}{DC} = \frac{DB}{GE}$$

$$\frac{95}{96-t_0} = \frac{8000}{10000} \Rightarrow t_0 = 20 \text{ hrs.}$$

Max. storage = Area of ΔAEB

= Ar. of Δ AEC − Ar. of Δ ABC

$= \frac{1}{2} \times 96 \times 1000 - \frac{1}{2} \times 96 \times 8000$

$\boxed{\text{Max. storage} = 96{,}000 \ m^3}$ *

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Storage after 1 hr. = Area of ΔAEh − Ar. of ΔAFh

Triangles AFh & ΔABD are similar

$$\frac{Fh}{Ah} = \frac{BD}{AD} \Rightarrow Fh = 400 \ m^3/hr$$

$$\text{Storage after 1 hr.} = \frac{1}{2} \times 1 \times 10{,}000 - \frac{1}{2} \times 1 \times 400$$

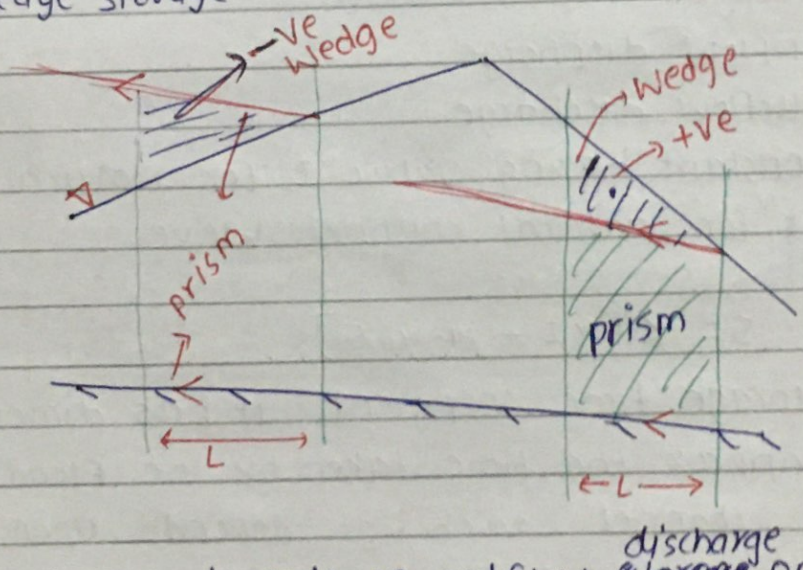
$$= 4800 \ m^3$$

Channel Routing :-

The volume of water stored in a channel reach can be expressed in two parts

i) prism storage

ii) wedge storage



* prism storage depends on outflow ~~storage~~ discharge only
* wedge storage depends on both inflow as well as outflow discharge

* +ve wedge storage ⟶ inflow > outflow (vice-versa)

* In case of level pool routing wedge storage = 0

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If the flow were uniform throughout the water surface would have been parellel to the channel bed However because of Non-uniform condition due to the flood wave water surface is not surface to the channel bed.

Muskingum Method:- A commen Method of channel routing is Muskingum Method in which the channel storage in a reach is expressed as a function of both inflow and outflow discharge.

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

$S \rightarrow$ storage
$I \rightarrow$ inflow discharge
$Q \rightarrow$ outflow discharge
$m \rightarrow$ constant having value 1 for Natural channels hence for natural channel we've

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

$K \rightarrow$ storage time cofficient, it has dimensions of time it is approx the time taken by the flood wave to cross the channel reach. It depends upon the length of channel reach and channel roughness charecteristics.

$x \rightarrow$ weightage factor

for $x = 0$, $S = KQ$

that is storage is a f^n of outflow discharge only such type of storage called linear storage or linear reservoir.

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NOTE: Linear channel is a channel in which only translation takes place without any attenuation (Not to be confused with linear storage)

For sufficiently small time Δt we have

$$S_2 - S_1 = K\left[x(I_2 - I_1) + (1-x)(Q_2 - Q_1)\right]$$

$$S_2 - S_1 = \frac{I_1 + I_2}{2}\Delta t - \frac{Q_1 + Q_2}{2}\Delta t \qquad \text{continuity eq}^{n}.$$

solving above two eq^n, eliminating S_1 & S_2

We've

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

$$C_0 = \frac{-Kx + 0.5\Delta t}{K - Kx + 0.5\Delta t}$$

$$C_1 = \frac{Kx + 0.5\Delta t}{K - Kx + 0.5\Delta t}$$

$$C_2 = \frac{K - Kx - 0.5\Delta t}{K - Kx + 0.5\Delta t}$$

$$C_0 + C_1 + C_2 = 1$$

It is seen that +ve values of C_1, C_2 & C_3 we get better routing results.

$$\boxed{2Kx < \Delta t < K}$$ We get $C_1, C_2, C_3 \rightarrow$ +ve.

Que The inflow hydrograph reading for a stream reach are given below for which Muskingum coffi. $K = 36$ hr $x = 0.15$ applies. Route the flood through the reach and determine the outflow hydrograph.

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Assume $\Delta t = 12\,hr$

t	Inflow (I) m^3/s	$C_0 I_2$	$C_1 I_1$	$C_2 Q_1$	Outflow (Q)
Δt [0	42				42 ($Q_1 = I_1$ assume)
12	45	0.0164×45	$+ 0.3115 \times 42$	$+ 0.6721 \times 42$	42.05
24	88	0.0164×88	$+ 0.3115 \times 45$	$+ 0.6721 \times 42.05$	43.72
36	⋮				
⋮	⋮				

$\Delta t = 12\,hr$

$x = 0.15$

$K = 36\,hr.$

$C_0 = 0.0164 \quad C_1 = 0.3115, \quad C_2 = 0.6721$

Reservoir sedimentation :-

Deposition of sediments in the reservoir is called reservoir sedimentation. The rate of sedimentation depends on Trap efficiency which is expressed as

$$\eta_{trap} = \frac{\text{Sediment retained}}{\text{Total sediments}} \times 100$$

Trap efficiency is also expressed as a function of capacity to inflow ratio

Que: The inflow rate in a reservoir of capacity $V\,(m^3)$ is $Q\ m^3/sec$, concentration of sediment in the inflow is C (ppm) and the density of sediments is $\rho_s\ \frac{kg}{m^3}$. The trap efficiency is η_{trap}. Calculate the No. of years in which 40% capacity remains.

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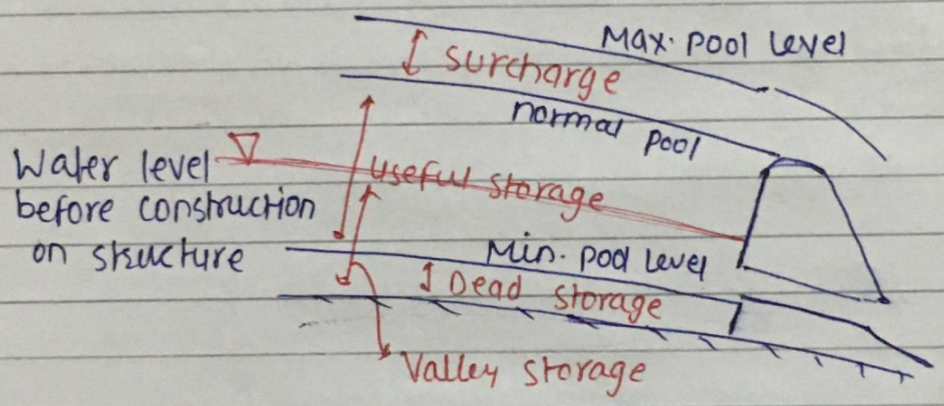
Let no. of years be n.

$$\eta_{trap} = \frac{0.6V}{\frac{(Q \times n_{yrs}) \times C_{mg/lt}}{\rho_s}} \times 100$$

check units.

Effective storage :-



Eff. storage = Surcharge + useful storage − Valley storage

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Ground Water Hydrology

Water below earth surface is called Ground water. Various water bearing stratums below earth surface is classified on the basis of ease with which water can be pumped out from them

Various water bearing stratums are :-

1) **Aquifer :-**

Water bearing stratum which can store as well as yield water relatively easily due to it's high permeability Ex: layers of sand, gravel etc.

2) **Aquitard :-**

only seepage is possible through this layer and yield is insignificant.

Ex: layers due to rock sedimentation

3) **Aquiclude :-**

Highly porus but not permeable

Ex: layers of clay

4) **Aquifuge :-**

Neither porus nor permeable

Ex: layers of Granite

Types of Aquifer :-

1) **Unconfined Aquifer :-** such aquifers which have no confinement from top are called unconfined aquifer or water table aquifer or phreatic aquifer

Water at the top is in direct contact with air hence the pressure is atmospheric

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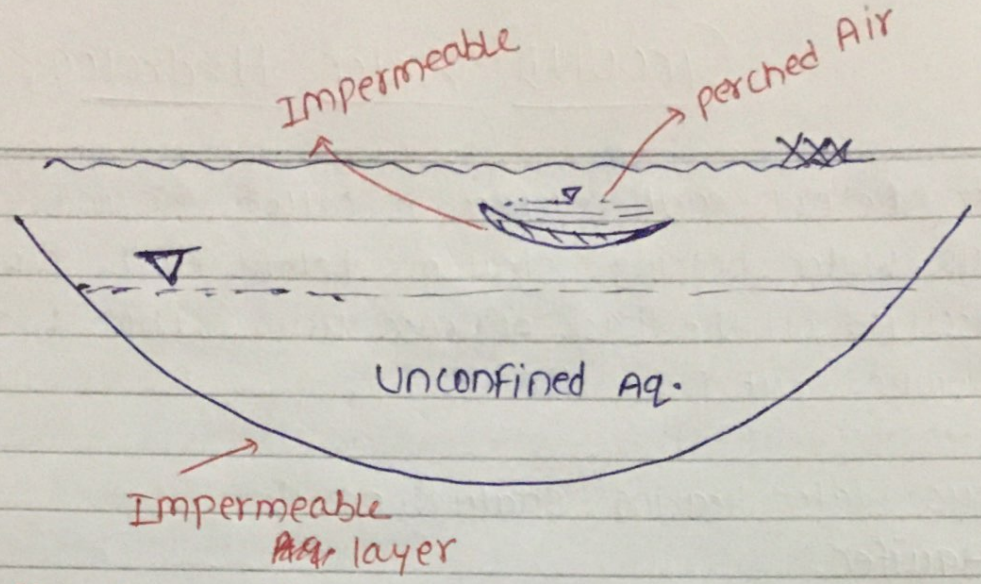
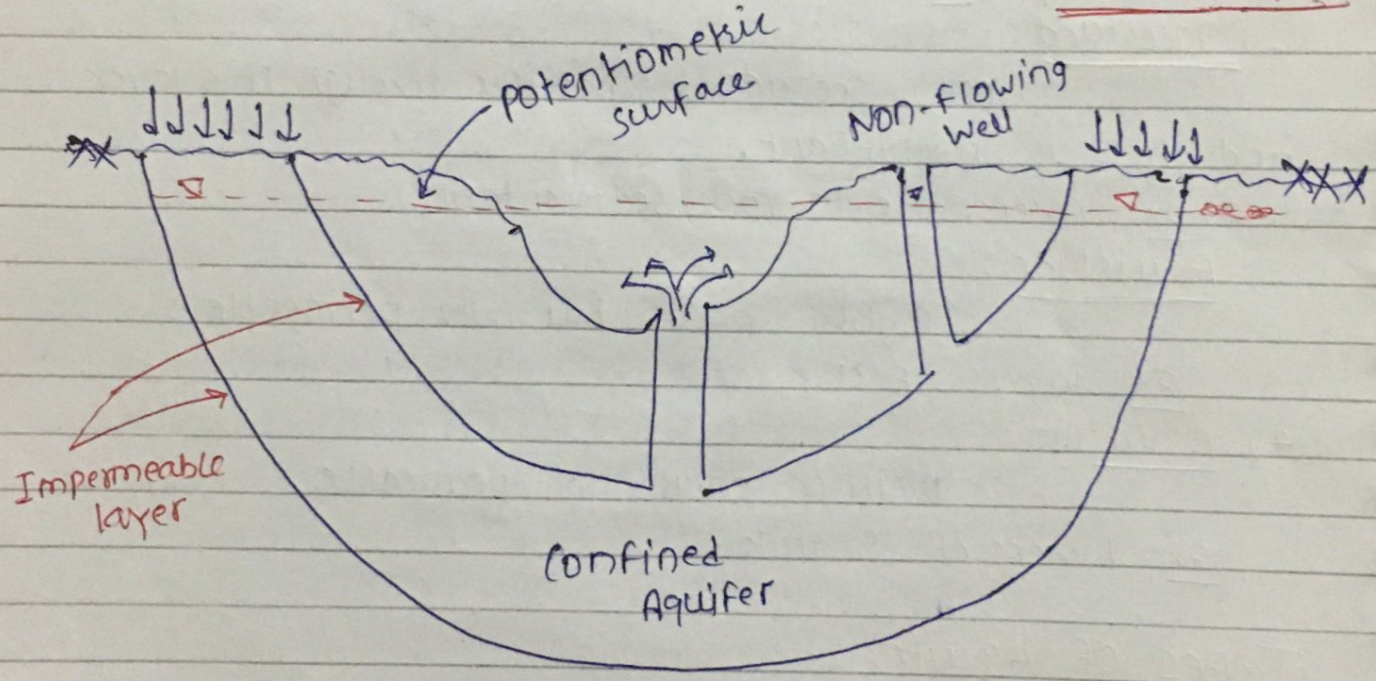


2) <u>confined aquifer</u> :- Aquifers with confinement from both top and bottom are called confined aquifer. Also known as <u>Artesian Aquifer</u>



In case of confined aquifers water is under very high pressure.

Aquifer properties :-

① <u>porosity</u> :→ it is the ratio of volume of voids to the total volume

$$\eta = \frac{V_v}{V_t} \times 100$$

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It represents the amount of water which can be stored by the aquifer

② **specific yield :-(S_Y) :-** The volume of water that can be drained under gravity per unit volume of aquifer is called specific yield.

$$S_Y = \frac{\text{Vol}^m \text{ drained}}{\text{Total Vol}^m} \times 100$$

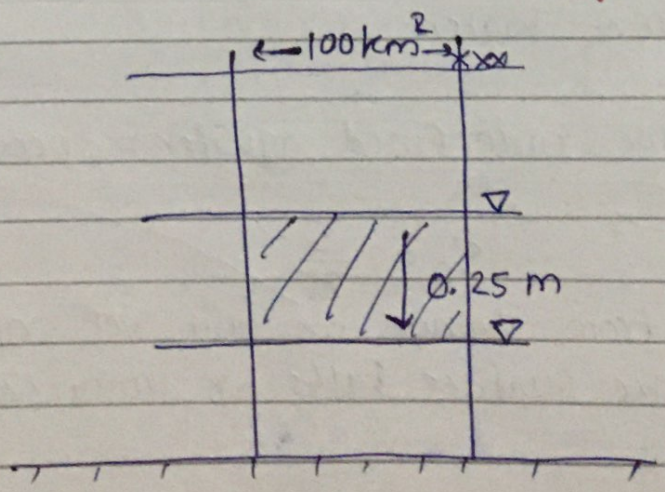
③ **specific Retention (S_R) :-** Vol^m of water that can not be drained under gravity per unit Vol^m of aquifer is called specific retention.

$$S_R = \frac{\text{Vol}^m \text{ Retained}}{\text{Total volume}} \times 100$$

$$\boxed{\text{porosity} = S_Y + S_R}$$

NOTE: As soil particle size decreases specific retention ↑

Que An uncontrolled Aquifer of porosity = 30 %, permeability 35 m/day and specific yield of 0.2 has an Area of 100 Km^2. If the water table falls by 0.25 m during drought the volume of water lost from storage in Mm^3 = ?



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$$S_Y = \frac{\text{Vol}^m \text{ drained (lost)}}{\text{Total vol}^m}$$

$$0.2 = \frac{\text{Vol}^m \text{ lost}}{100 \times 0.25}$$

$$\text{Vol}^m \text{ lost} = 5 \text{ Mm}^3$$

Storage coefficient or storativity :- In confined aquifer Vol^m of water given by unit plan Area of Aquefer wher potentiometric surface falls by unity is called storage cofficient it is expressed as –

$$S = \gamma n b \left(\frac{1}{K_w} + \frac{1}{n E_s} \right)$$

S → storage cofficient (dimension less)
γ → weight density of water
n → porosity
b → thickness of confined aquifer
K_w → Bulk modulus of compressibility
E_s → Modulus of Elasticity of soil

Storage cofficient is thought due to compression of soil and expansion of water

Storage cofficient for unconfined aquifer = specific yield

Specific storage :- (S_s)

$$S_s = \frac{S}{b}$$

Vol^m of water released from storage by unit Vol^m of confined Aquifer when potentiometric surface falls by unity called specific storage.

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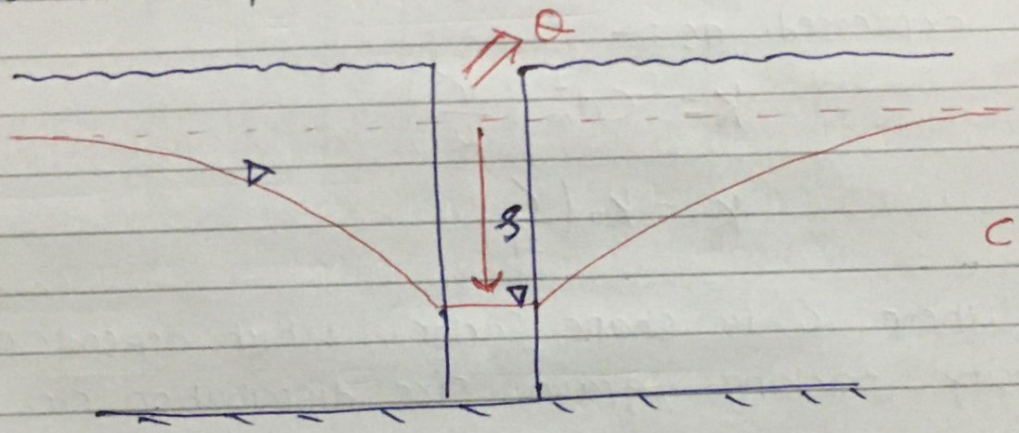
Simply put specific storage is storage cofficient per unit thickness (depth) of water.

Dimension → m^{-1}

Specific capacity:-

Discharge per unit drawdown in a well is called specific capacity.

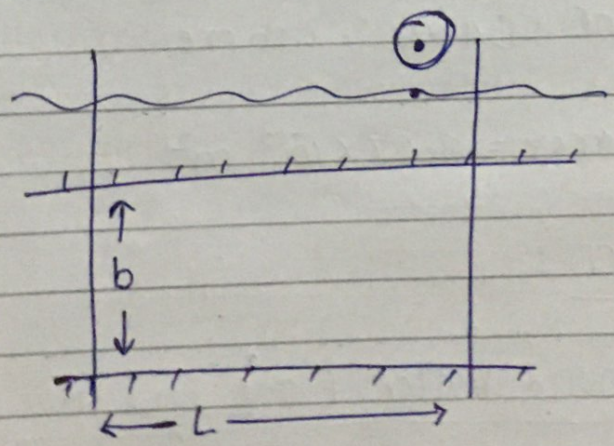


$$C = \frac{Q}{s}$$

It is a measure of performance of well

Cofficient of Transmissibility or Transmissivity:-

It is the product of ~~permee~~ cofficient of permeability and thickness of aquifer.



$V = ki$

$Q = VA$

$Q = kiA$

$Q = kiLb$

$Q = (kb)iL$

$\boxed{Q = TiL}$

T → Transmissibility =

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Transmissibility for confined aquifer is constant as b is constant

Cofficient of permeability (K):- It is the Rate of Flow per unit Area of aquifer under ~~unit~~ unit hydraulic Gradient (m/day) → unit

It is expressed as :-

$$K = C d_m^2 \left(\frac{\gamma}{\mu}\right)$$

$$K = K_0 \left(\frac{\gamma}{\mu}\right)$$

Where C is shape factor, which depends on porosity, packing, grain size distribution etc.

d_m → is Mean dia of soil particle
γ → Weight density of fluid (ρg)
μ → dynamic viscosity of Fluid

Where K_0 is known as intrinsic permeability which depends only on medium property.
it has units of Darcy, where

$$1 \text{ darcy} = 9.87 \times 10^{-13} \text{ m}^2$$

Equation of Motion :-

(i) **1-D confined Ground water Flow b/w two water bodies:-**

Surprise !

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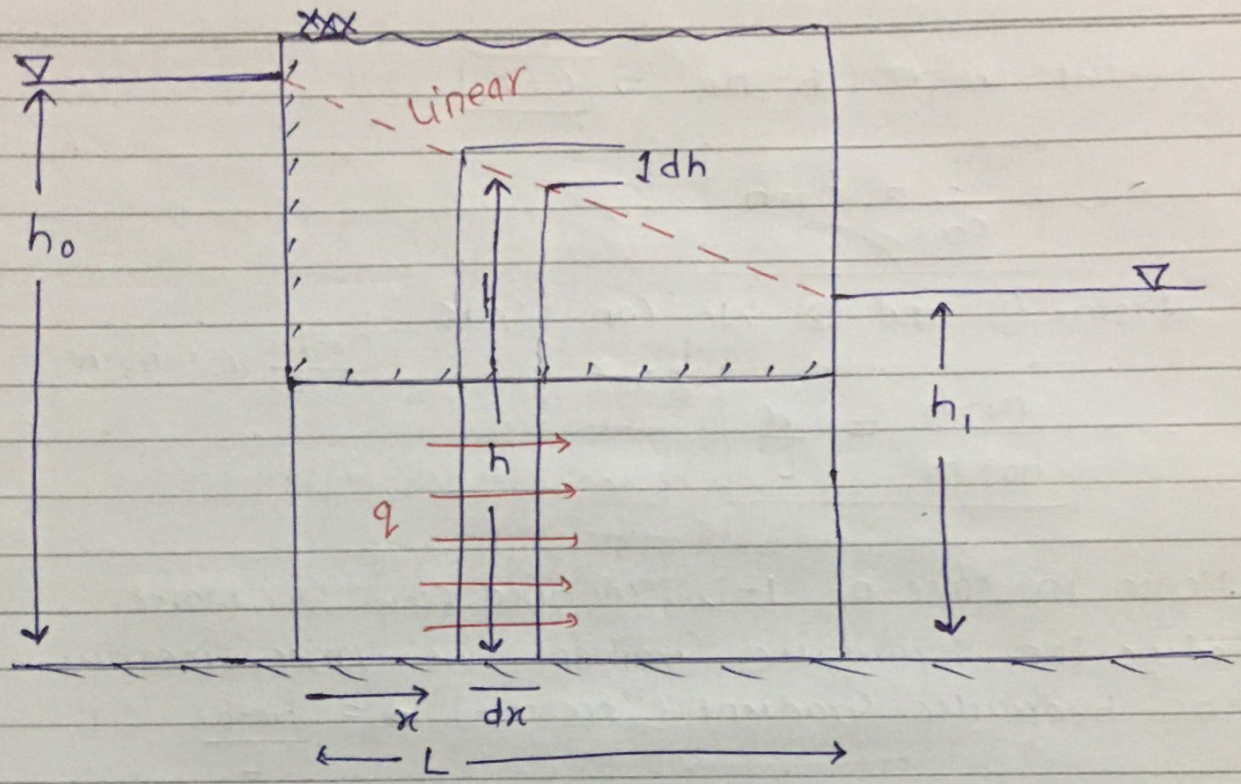

$$q = kiA$$

Area

$$= k\left(-\frac{dh}{dx}\right)(b \times 1)$$

$$\boxed{\frac{q}{kb}\,dx = -dh} \quad (1)$$

Definate integration of eq^n (1) gives

$$\frac{q}{kb}\int_0^L dx = -\int_{h_0}^{h_1} dh$$

$$\boxed{\frac{q}{kb} = \frac{h_0 - h_1}{L}}$$

Indefinate integration of eq^n (1) gives

$$\frac{q}{kb}\int dx = -\int dh$$

$$\frac{q}{kb}x = -h + c$$

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At $x=0$, $h=h_0 \Rightarrow \boxed{C=h_0}$

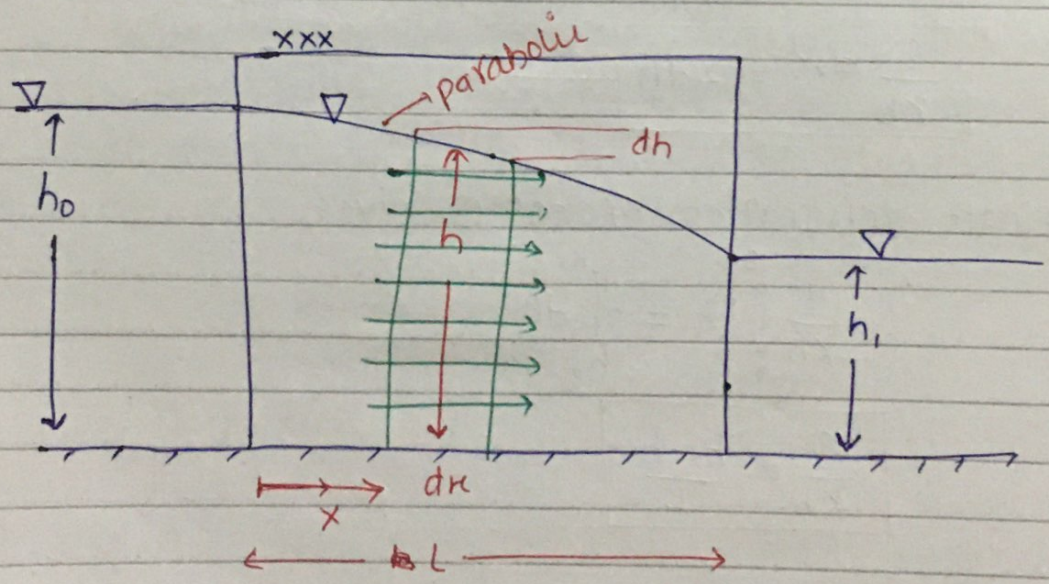
$$\boxed{\frac{q}{kb} = h_0 - h} \quad \text{—} (2)$$

from (1) and (2) we can write $\frac{h_0-h_1}{L}x = h_0-h$

$$\boxed{\frac{h_1-h}{h_0-h_1} = \frac{x}{L}} \quad ***$$

Hence in case of 1-D, confined flow b/w water bodies the hydraulic Gradient line varies linearly. The hydraulic Gradient (Constant) $= \frac{h_0-h_1}{L}$

(ii) 1-D unconfined Ground water flow b/w two water bodies :-



$q = kiA$

$q = k\left(-\frac{dh}{dx}\right)(h \times 1)$

$$\boxed{\frac{q}{k}dx = -h\,dh} \quad \text{—} (1)$$

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Definate integration of eq^n ① gives

$$\frac{q}{K}\int_0^L dx = -\int_{h_0}^{h_1} h\,dh$$

$$\boxed{\frac{q}{K} = \frac{h_0^2 - h_1^2}{2L}} \quad \text{—②}$$

Indefinate integration of eq^n ① gives

$$\frac{q}{K}\int dx = -\int h\,dh$$

$$\frac{q}{K}x = \frac{-h^2}{2} + C$$

At $x = 0$, $h = h_0$ $\Rightarrow$ $\boxed{C = \frac{h_0^2}{2}}$

$$\frac{q}{K}x = \frac{-h^2}{2} + \frac{h_0^2}{2}$$

$$\boxed{\frac{q}{K}x = \frac{h_0^2 - h^2}{2}} \quad \text{—③}$$

from ③ and ②

$$\boxed{\frac{h_0^2 - h^2}{h_0^2 - h_1^2} = \frac{x}{L}} \quad ***$$

Hence in case of 1-D unconfined Ground water flow b/w 2 water bodies the water surface profile is parabolic.

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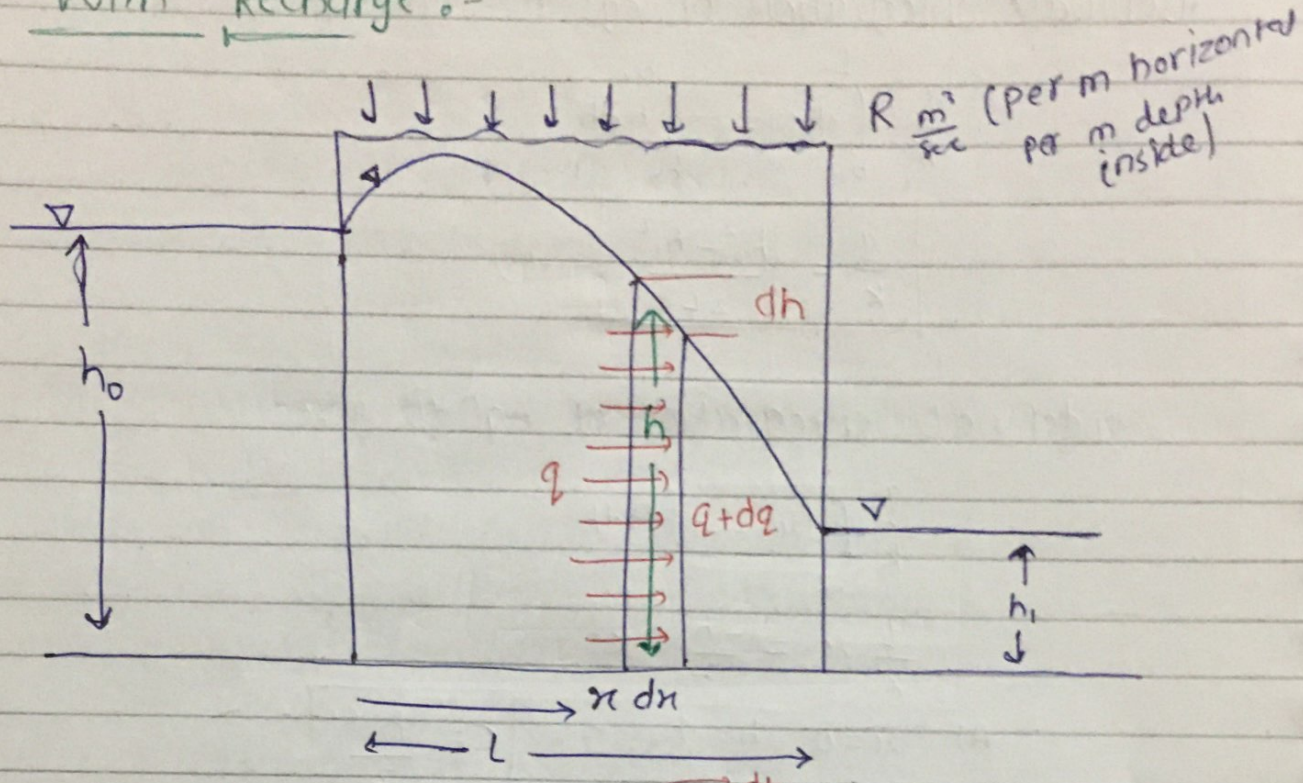
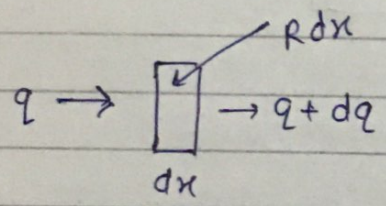
(iii) 1-D unconfined flow b/w two water bodies with Recharge :-





$$q + R\,dx = q + dq$$

$$R = dq/dx$$

$$q = kiA$$

$$q = K\left(-\frac{dh}{dx}\right)(h \times 1)$$

$$\frac{q}{K} = \frac{-h\,dh}{dx}$$

$$q = -\frac{K}{2}\frac{dh^2}{dx} \qquad \frac{dh^2}{dx} = 2h\frac{dh}{dx}$$

$$\frac{dq}{dx} = -\frac{K}{2}\frac{d^2h^2}{dx^2}$$

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$$\boxed{\frac{d^2h^2}{dx^2} = \frac{-2R}{K}} \quad \text{—— (1)}$$

Indefinate integration (Double integration) of eq^n (1) gives:-

$$h^2 = -\frac{R}{K}x^2 + C_1 x + C_2$$

at $x=0$, $h=h_0 \Rightarrow \boxed{C_2 = h_0^2}$

at $x=L$, $h=h_1 \Rightarrow h_1^2 = \frac{-R}{K}L^2 + C_1 L + h_0^2$

$$C_1 = \frac{RL}{K} - \frac{h_0^2 - h_1^2}{L}$$

$$\boxed{h^2 = -\frac{R}{K}x^2 + \left(\frac{RL}{K} - \frac{h_0^2 - h_1^2}{L}\right)x + h_0^2} \rightarrow \text{(A)}$$

Hence in case of 1-D unconfined Ground water flow b/w two water bodies with recharge the water surface profile is Eliptical

at the location of Max. water table $\frac{dh}{dx} = 0$

Differentiating eq^n (A) w.r.to x and substituting $\frac{dh}{dx} = 0$, we have

$$\boxed{x_0 = \frac{L}{2} - \left(\frac{h_0^2 - h_1^2}{2RL}\right)K} \quad \text{—— (B)}$$

* obj.

This location of Max. water table is called Ground water devide or phenetic devide.

At this location $\boxed{q=0}$ since $i = -\frac{dh}{dx} = 0$

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Calculation of Discharge

$$q = kiA$$

$$q = k\left(-\frac{dh}{dx}\right)(h \times 1)$$

$$\frac{q}{k} = h\frac{dh}{dx}$$

Differentiating eq^n A and substituting the value of $h\frac{dh}{dx}$ in above eq^n

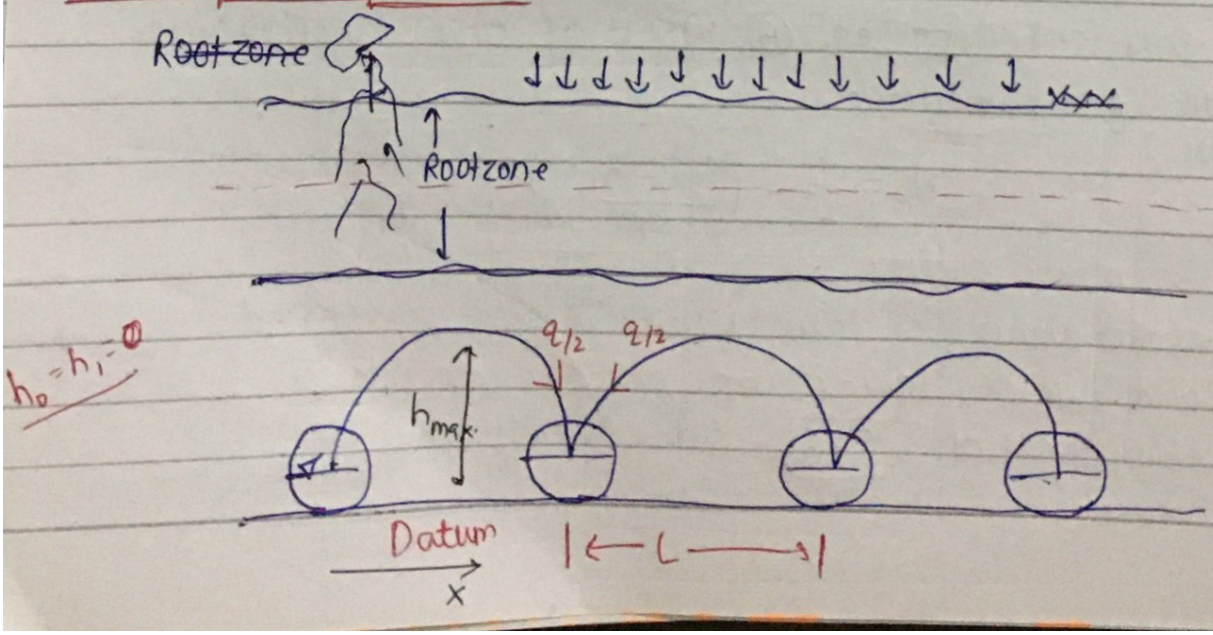
$$\boxed{q = k\left(\frac{h_0^2 - h_1^2}{2L}\right) - \frac{R}{2}(L - 2x)}$$

$$q\Big|_{x=0} = k\left(\frac{h_0^2 - h_1^2}{2L}\right) - \frac{RL}{2}$$

$$q\Big|_{x=L} = k\left(\frac{h_0^2 - h_1^2}{2L}\right) + \frac{RL}{2}$$

$$\boxed{q\Big|_{x=L} = q\Big|_{x=0} + RL}$$ *** obj

Depth of Tile Drains :-



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Assuming Datum as shown and Neglecting depth of Tile drains. comparing in two previous case we've $h_0 = h_1 = 0$

Substituting these values in eq^n (B) of previous case location of Max. water table $x_0 = L/2$

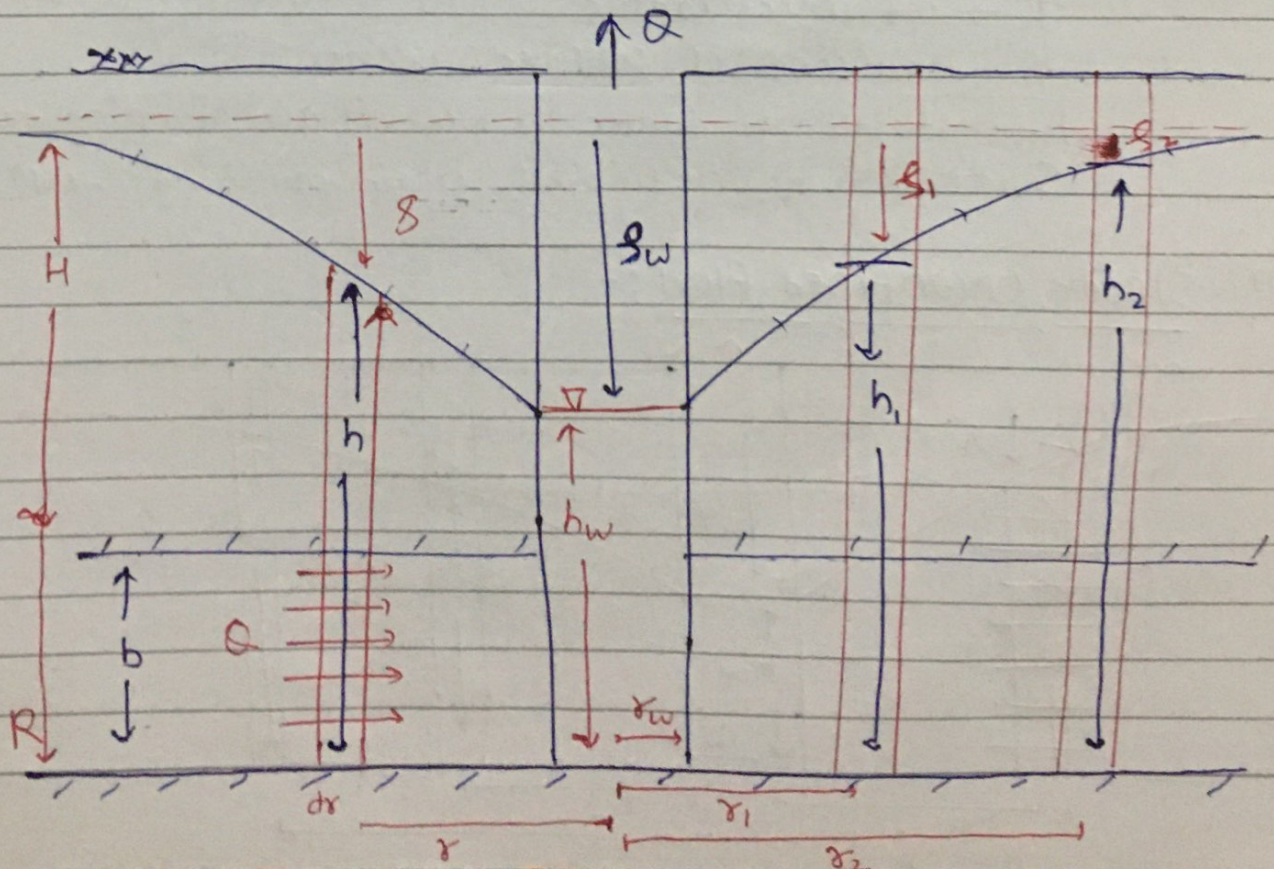
Substituting $x_0 = L/2$ and $h_0 = h_1 = 0$ in eq^n (A) of previous case we've

$$h_{max.} = \sqrt{\frac{RL^2}{4K}}$$

Hence Tile Drains should be installed $h_{max.}$ depth below Rootzone depth.

Flow in Wells :-

(i) Steady confined Flow (Fully penetrating) :-



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$$Q = KiA$$

$$Q = K\left(\frac{dh}{dr}\right) 2\pi r \times b$$

$$Q\frac{dr}{r} = (Kb\,2\pi)\,dh$$

$$Q\int_{r_1}^{r_2}\frac{dr}{r} = 2\pi kb\int_{h_1}^{h_2} dh$$

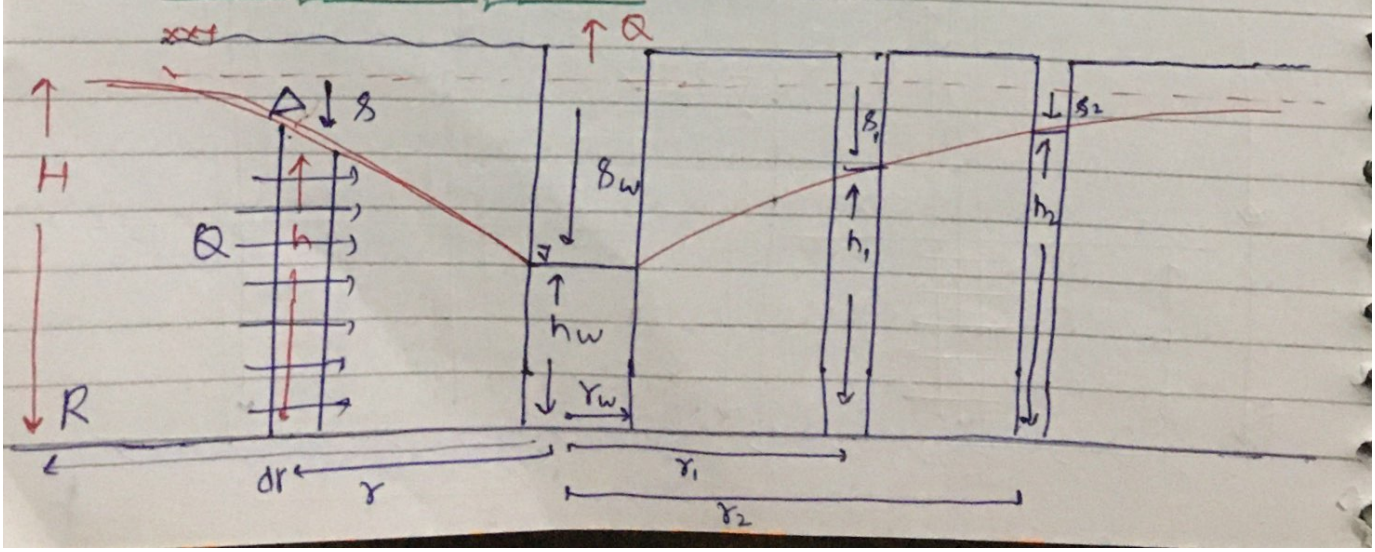
$$Q = \frac{2\pi kb\,(h_2-h_1)}{\ln(r_2/r_1)} \rightarrow s_1 - s_2$$

$$Q\int_{R_w}^{R}\frac{dr}{r} = 2\pi kb\int_{h_w}^{H} dh$$

$$Q = 2\pi kb\left(\frac{H-h_w}{\ln(R/r_w)}\right) \rightarrow s_w$$

$r_w \rightarrow$ Well Radius
$R \rightarrow$ influence radius
$b \rightarrow$ thickness of confined aquifer
$h_w \rightarrow$ height of water in well (Main pumping well)
$s_w \rightarrow$ Drawdown in well (Main pumping well)

(ii) Steady unconfined flow :-



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$$Q = kiA$$

$$Q = k\left(\frac{dh}{dr}\right) 2\pi r h$$

$$Q\frac{dr}{r} = (2\pi k)\, h\, dh \quad \text{—(1)}$$

$$Q\int_{r_1}^{r_2} \frac{dr}{r} = 2\pi k \int_{h_1}^{h_2} h\, dh$$

$$Q = \pi k \frac{(h_2^2 - h_1^2)}{\ln(r_2/r_1)}$$

$$Q\int_{r_w}^{R} \frac{dr}{r} = 2\pi k \int_{h_w}^{H} h\, dh$$

$$Q = \pi k \left(\frac{H^2 - h_w^2}{\ln(R/r_w)}\right)$$

NOTE: (1) gnitially Dupit gave eq^n for unconfined flow which were later on Modified by Thiem for confined flow

(2) Emperical eq^n for influence Radius

$$R = 3000\, S_w \sqrt{K}$$ Sichandt Eq^n

Non-Equilibirium Formula for Wells in Confined Aquifer:-

Steady state is established only after a sufficiently long time Hence we generally use unsteady state Formula for Analysis purpose

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$$s = \frac{Q}{4\pi T}\left(-0.5772 - \ln u + u - \frac{u^2}{2\cdot 2!} + \frac{u^3}{3\cdot 3!} \cdots\right)$$

$u \rightarrow$ dimension less term

$$u = \frac{r^2 S}{4tT}$$

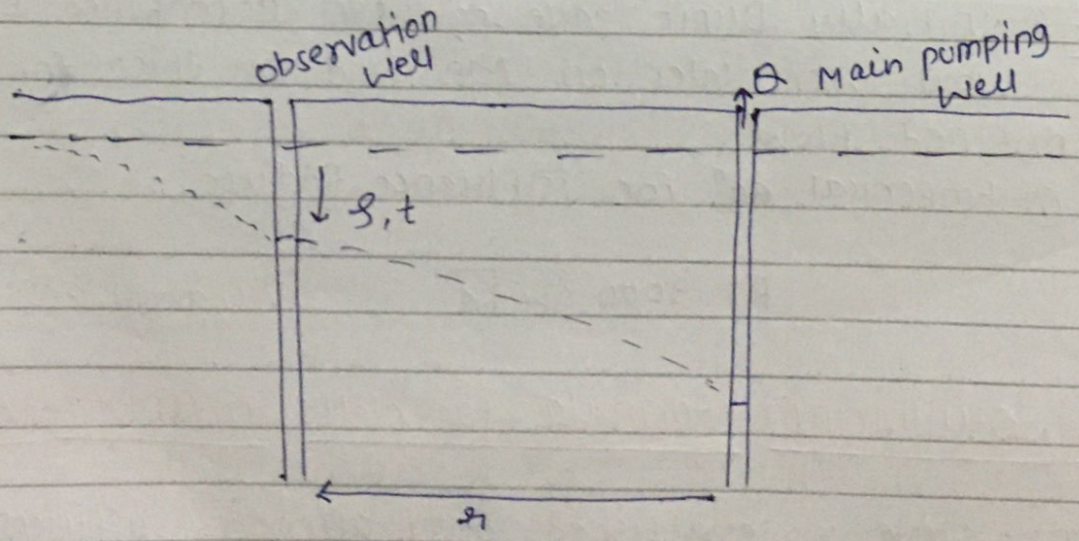
Jacob- opper approximation (for small radius and large time)

$$s = \frac{Q}{4\pi T}\left(-0.5772 - \ln u\right)$$

$$s = \frac{Q}{4\pi T}\left(\ln \frac{4tT}{r^2 S} - 0.5772\right)$$

$$\boxed{s = \frac{Q}{4\pi T} \ln\left(\frac{2.25\, tT}{r^2 S}\right)}$$

$s \rightarrow$ Drawdown in observation well @ radial distance r from main pumping well @ any time t



$Q \rightarrow$ discharge from main pumping well

$T \rightarrow$ Transmissibility

$S \rightarrow$ storativity

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*** obj statments:

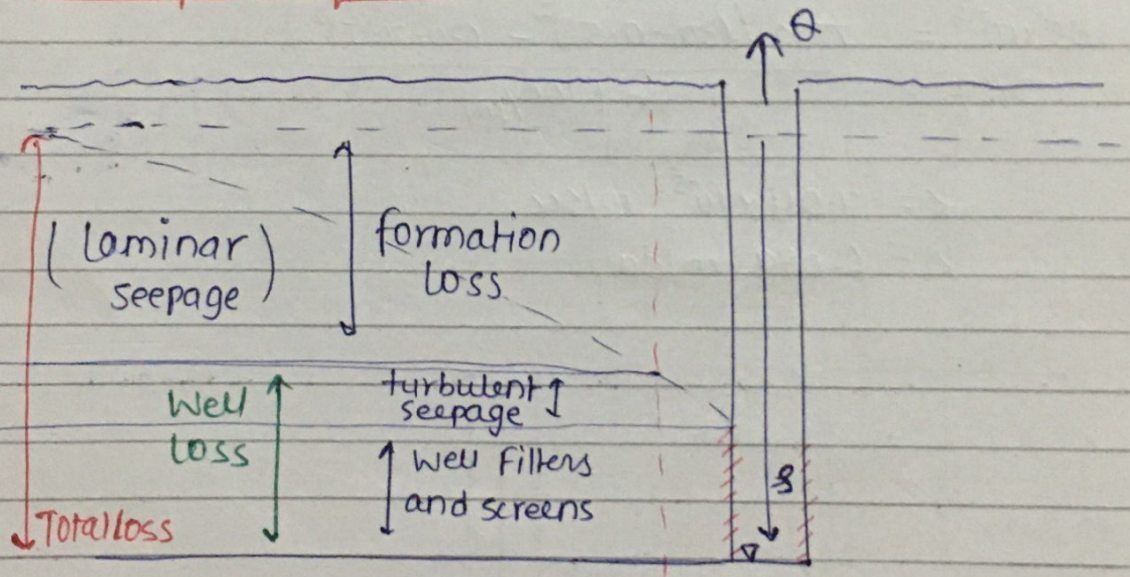
① if s_1 and s_2 are drawdowns in observation well @ times t_1 and t_2

$$s_2 - s_1 = \frac{Q}{4\pi T} \ln\left(\frac{t_2}{t_1}\right)$$

② if in two observation wells located at radial distances r_1 & r_2, drawdowns are same @ times t_1 and t_2

$$\boxed{\frac{t_1}{r_1^2} = \frac{t_2}{r_2^2}}$$

LOSSES IN WELL:-



Total loss (i.e Drawdown) = formation loss $C_1 Q$ (laminar seepage) + well loss ($C_2 Q^2$) (Turbulent seepage + loss in well screens & filters)

$$s = C_1 Q + C_2 Q^2$$

$$\text{Sp. capacity} = \frac{\text{Discharge}}{\text{Drawdown}}$$

$$C = \frac{Q}{C_1 Q + C_2 Q^2}$$

$$\boxed{C = \frac{1}{C_1 + C_2 Q}}$$

as Q↑ specific capacity ↓

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Lit $\rightarrow 10^{-3}\ m^3$

Que An unconfined Aquifer has a thickness of 30 m A fully penetrating 20 cm Dia well in this aquifer is pumped at a Rate of 35 Lit/sec. The Drawdown Measured in two wells at a distance of 10 m and 100 m from the well are 7.5 m & 0.5 m Respectively. Determine the avg. hydraulic conductivity of the aquifer and At what distance from the well Drawdown is significant

$$Q = \frac{\pi k (h_2^2 - h_1^2)}{\ln(r_2/r_1)}$$

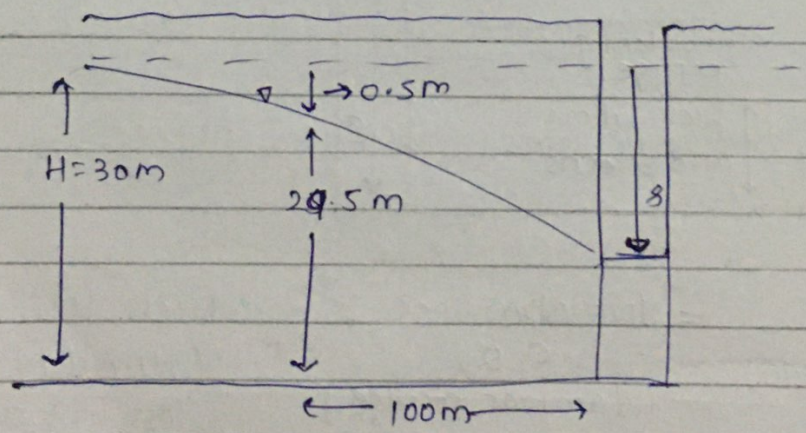
$h_2 = H - S_2$

$h_1 = H - S_1$

$$\underset{m^3/sec}{35 \times 10^{-3}} = \frac{\pi k \left((30-0.5)^2 - (30-7.5)^2\right)}{\ln(100/10)}$$

$K = 7.04 \times 10^{-5}$ m/sec

$K = 6.089$ m/day



$$Q = \frac{\pi k (30^2 - 29.5^2)}{\ln(R/100)} \Rightarrow \underset{m^3/sec}{35 \times 10^{-3}} = 3.14 \times 7.04 \times 10^{-5} \frac{(30^2 - 29.5^2)}{\ln(R/100)}$$

$R = 120.7$ m.

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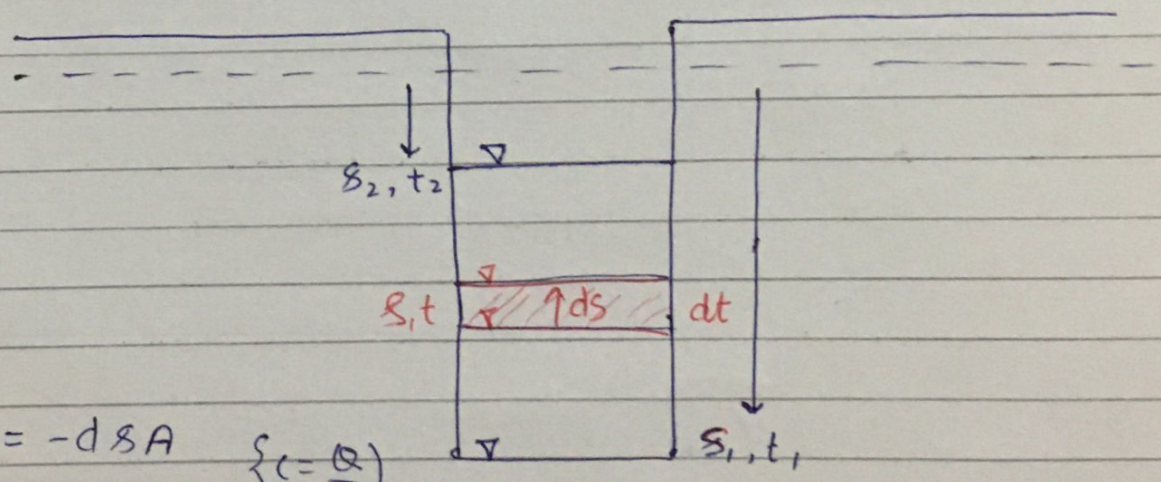
SAFE Yield:- Max. Discharge at which ground water can be with drawn without causing undesirable effects like permanent lowering of water table, Dislodging of soil particles, salt water imthrochment in costal Areas etc. is called safe yield.

Recuperation test:- This is done to calculate specific capacity of well so that we can calculate Discharge corresponding to a perticular Drawdown.

By pumping at a heavy rate a Drawdown s_1 is created at time t_1 and then the pumping is stopped the water level in the well recuperates and the drawdown at time t_2 becomes s_2. Let at any time t when the drawdown was s, the water level recuperates by ds in time dt



$$Q\,dt = -ds\,A \qquad \left\{ c = \frac{Q}{s} \right\}$$

$$c\,s\,dt = -ds\,A$$

$$c\,(dt) = \frac{-ds}{s} A$$

$$\boxed{\frac{c}{A} = \frac{\ln(s_1/s_2)}{t_2 - t_1}}$$

$c \rightarrow$ specific capacity m^2/s

$A \rightarrow$ open Area of well (m^2)

$\frac{c}{A} \rightarrow$ specific capacity per unit open Area of well ($1/s$) (s^{-1})

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Que. During a Recuperation test conducted on an open well in a region the water level in the well was depressed by 3 m and it was observed to rise by 1.75 m in 75 minutes. What is the specific capacity in that region.

$$\frac{C}{A} = \frac{\ln\left(\frac{3}{1.25}\right)}{75 \text{ min.}}$$

$$C = 0.7 \text{ hr}^{-1}$$

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