Civinnovate

Discover, Learn, and Innovate in Civil Engineering

https://civinnovate.com/civil-engineering-notes/

INDEX

NAME: ______ STD.: ______ SEC.: ______ ROLL NO.: ______ SUB.: ______

INTRODUCTION

Hydrology means the Science of water which deals with the occurrence, circulation and distribution of water on the earth surface and in atmosphere.

② The total Quantity of water is about 1.36×10^{18} m³.

③ About 97% of total water is saline water which is available in ocean and sea.

④ About 30% of total fresh water is in liquid form.

⑤ The surface area covered by the ocean and sea is about 71% of total earth surface.

⑥ In India the average depth of rainfall in one year is 119 cm.

⑦

*****
TECHNICAL TERMS

ARTICLE	DESCRIPTION
① Isohyet line	The line joining points of equal depth of rainfall.
② Isopluvial line	The line joining points of equal depths of rainfall based on the frequency of the rainfall (Return Period of the rainfall)
③ Isopleth	The line joining points of equal depth of Evapotranspiration (potential evapotranspiration) (PET)
④ Isochrone	The line joining points of equal time of travel of runoff water
⑤ Hyetograph	A graph in between the Intensity of rainfall (Y-axis) and duration of rainfall (X-axis)

	A graph between (Rate of flow) (Y-axis) and the time of flow of river (X axis)
⑦ Flow Mass curve (Ripple curve)	A graph b/w cumulative inflow ~~Int~~ volume of a river into reservoir (Y-axis) and duration of flow of river (X-axis). It is an integral curve of hydrograph
⑧ * Flow duration curve	A graph b/w the discharge of the river (Y-axis) and the percentage of time of flow ~~(x-a~~ (% of probability on x-axis)
⑨ Rainfall Mass curve	A graph b/w cumulative depth of rainfall (Y-axis) and the duration of rainfall (X-axis)
⑩ Double Mass curve	A graph b/w cumulative depth of rainfall at a certain rain gauge station (At missing rainfall station) (Y-axis) and the cumulative depth of rainfall of the surrounding rain gauge stations (X-axis). It is mainly used to check the consistency of the rainfall at a particular station

*****

WATER-BUDGET EQUATION (Continuity Equation)

It states that the total quantity of inflow of water into a reservoir (catchment area) is equal to quantity of outflow of water + change in storage of water, i.e

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

P → Total precipitation (rainfall)
R → Total runoff
G → Net ground flow
E → Total evaporation
T → Total transpiration

$$\bar{I} \times \Delta t = \bar{Q} \times \Delta t + \Delta S \leftarrow (\Delta H \times A_{Res.})$$

$$\frac{I_1+I_2}{2} \times \Delta t + \bar{P} \times A_{Res.}$$

$$= \frac{Q_1+Q_2}{2} \Delta t + E \times A_{Res.} + \underset{\substack{\downarrow \\ \text{Seepage}}}{G} \times A_{Res.} + \underset{\substack{\Downarrow \\ \Delta H \times A}}{(S_2 - S_1) \times A}$$

⇒ Continuity Eqn, Water-Budget Eqn; Hydrology Eqn.

① A Reservoir has initial water surface elevation

101 (RL). The average surface area of reservoir is 6000 Ha. The mean inflow rate in one month is 10 cumec. The avg. outflow rate in one month is 7 cumec. The avg. depth of rainfall is 150 mm. The avg. evaporation in one month is 100 mm. Neglect ground water flow. Determine elevation of water surface in the reservoir at the end of 30 days.

soln:

$$(10\times 30\times 24\times 60\times 60) + (0.15\times 6000\times 10^4)$$

$$= (7\times 30\times 24\times 60\times 60) + (0.1\times 6000\times 10^4) + (S_2-101)\times 6000\times 10^4$$

$$34.92\times 10^6 = 24.144\times 10^6 = (S_2-101)\times 6000\times 10^4$$

$$10.776\times 10^2 = (S_2-101)\times 6000$$

$$0.1796 = S_2 - 101$$

$$\underline{\underline{S_2 = 101.1796\ m}} = \text{final elevation (RL)}$$

$$\Delta H = 0.1796\ m$$

$$= \underline{\underline{179.6\ mm}}$$

PRECIPITATION & IT'S MEASUREMENT

① The term precipitation denotes all forms of ~~precip~~ water reaching to the earth Surface from the atmosphere.

② There are foll. forms of precipitation -

(i) Drizzle (ii) Rainfall (iii) Snow (iv) Glaze

(v) Sleet (vi) Hail

** ③ If the intensity of rainfall is less than 2.5 mm/hr then it is known as light rainfall.

* 2.5 mm/hr - 7.5 mm/hr → Moderate rainfall

More than 7.5 mm/hr → Heavy rainfall

* ④ The rainfall is measured by instrument called -

(i) Rain gauge (ii) pluviometer

(iii) Ombrometer (iv) Hyetometer

*** ⑤ The frequency of rainfall at a particular place, i.e the chance of occurrence (probability of occurrence) of a point rainfall is given by relation,

$$\boxed{R = p = {}^nC_r\, p^r q^{n-r} = \frac{\underline{n}}{\underline{r}\ \underline{n-r}} \times p^r q^{n-r}}$$

$${}^nC_r\, p^r q^{n-r} = \frac{\underline{n}}{\cancel{\underline{r}}\ \underline{n-r}} \times p^r q^{n-r}$$

where, n → no. of Years (life of the hydraulic structure)

r → no. of event of occurrence of rainfall

p → $\frac{1}{T}$ (occurrence in next year)

T → Return period

q → Non-occurrence in next year

Probability of non- occurrence = ...

→ The probability of a point rainfall atleast to occur once in the given period ~~of~~ (life of the structure) is given by relation,

$$R = 1 - \left[1 - \frac{1}{T}\right]^n \qquad (R \rightarrow \text{Risk or Chance})$$

→ Non-occurrence = 1 − R

$n = 50$ yrs.
$r = 1$
$T = 100$ yrs.

$nC_r p^r q^{n-r} = ~~0~~\ 0.305 = 30.5\%$

$\boxed{r=1}$

If $r = 2 \Rightarrow 0.0705 = ~~70.5~~\ 7.5\%$ //

Atleast once in a year

$$R = 1 - \left(1 - \frac{1}{T}\right)^n = 0.395 = 39.5\%$$

Next year, $P = \frac{1}{T} = \frac{1}{100} = 0.01 = 1\%$

**

⑥ The missing rainfall at a particular station is determined by relation,

Rain gauge station	A	B	C	D
Normal annual rainfall (Back 30 years mean)	N_A	N_B	N_C	N_D
Storm rainfall (Current years)	P_A	P_B	P_C	(Missing)

Case - I → Normal annual rainfall differ within 10%.

$$P_D = \frac{P_A + P_B + P_C}{3}$$

Case-II → If they differ by more than 10%, then

$$P_D = \frac{\left(P_A \times \frac{N_D}{N_A}\right) + \left(P_B \times \frac{N_D}{N_B}\right) + \left(P_C \times \frac{N_D}{N_C}\right)}{3}$$

$N_A = 80mm$ $N_B = 60mm$ $N_C =$ ~~110mm~~ 105mm $N_D = 100mm$

$P_A = 50mm$ $P_B = 40mm$ $P_C = 70mm$

$$P_D = \frac{\left(50 \times \frac{100}{80}\right) + \left(40 \times \frac{100}{60}\right) + \left(70 \times \frac{100}{105}\right)}{3}$$

$$= 65.27 mm$$

**

ADEQUACY OF RAIN GAUGE STATIONS (Optimal no. of Rain gauge stations)

① The optimal no. of rain gauge stations required in a particular zone is based on statistical formula.

② The optimal no. of rain gauge station is compared with the existing no. of rain gauge station to decide whether extra number of rain gauge station is required or not.

③ The optimal no. of rain gauge station is dependent on foll. parameters –

(a) co-efficient of variation of rainfall (Cv)

(b) The permissible error in computation of mean rainfall (ϵ) (Generally it ~~is~~ may be taken as 10%)

(c) Mean rainfall value of existing rain gauge stations

(e) Existing no. of rain gauge stations (n)

④ According to Indian Standard –

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

$\epsilon \rightarrow$ permissible error (may be 10%)

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

$\downarrow$
(co-efficient of variation)

σ = standard deviation

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

$$\bar{P} = \frac{P_1 + P_2 + P_3 + \cdots P_n}{n}$$

Numericals:

① Determine the optimal no. of rain gauge stations if the permissible error is 10% and existing no. of rain gauge stations are A, B, C, D, E, F. The existing rainfall values are 85 cm, 110 cm, 135 cm, 95 cm, 115 cm, & 140 cm
$\quad$ (A, B, C, D, E, F)

sol: $\bar{P} = 113.33$

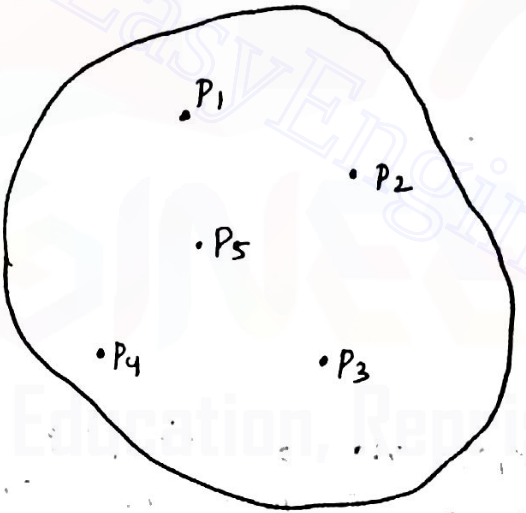
$$\sigma = \sqrt{\frac{\Sigma(P-\bar{P})^2}{n-1}} = 21.6 \text{ cm}$$

$$C_v = \frac{\sigma}{\bar{P}} = \frac{21.6}{113.33} \times 100 = 19.059\%$$

Computation of Mean Rainfall

There are 3 methods to determine mean rainfall over a catchment area -

(i) Arithmetical mean method - This method is applicable if the rainfall value differs little from each other (less than 10%)
The mean rainfall is the sum of rainfall values divided by number of rain gauge station
This method is not reliable and hence now-a-days it is not used.

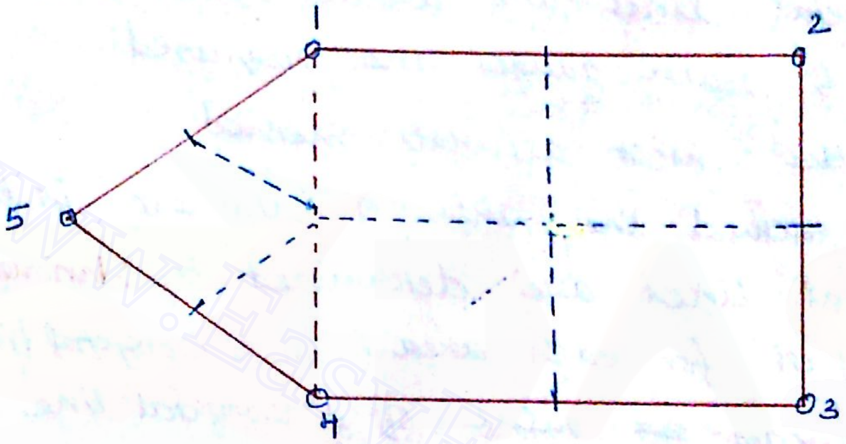


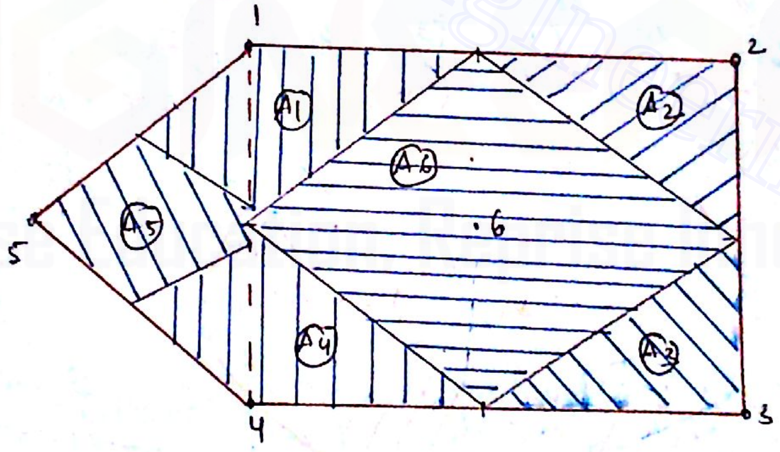
$$\bar{P} = P_{AV} = \frac{P_1 + P_2 + P_3 + \cdots}{n}$$

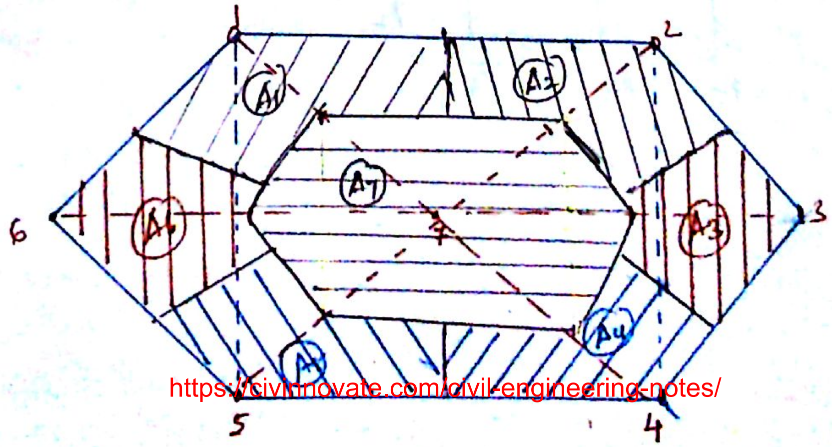
$$\boxed{P_{AV} = \bar{P} = \frac{\Sigma P}{n}}$$

(ii) Thiessen Polygon Method - In this method the rainfall is weighed according to the area enclosing the rain gauge station. The Area enclosing the rain gauge station to weigh the rainfall value is known as Thiessen polygon.

thiessen polygon is not the polygon joining the rain gauge stations directly. This method is better than the previous one (arithmetical mean method) The thiessen polygon is determined with the help of ~~dire~~ erection of perpendicular lines on the line joining the rain gauge stations as shown in fig. below -







$$\bar{P} = P_{AV} = \frac{P_1A_1 + P_2A_2 + P_3A_3 + \cdots}{A_1 + A_2 + A_3 + A_4 \cdots}$$

$$\bar{P} = P_{AV} = \frac{\Sigma PA}{\Sigma A}$$

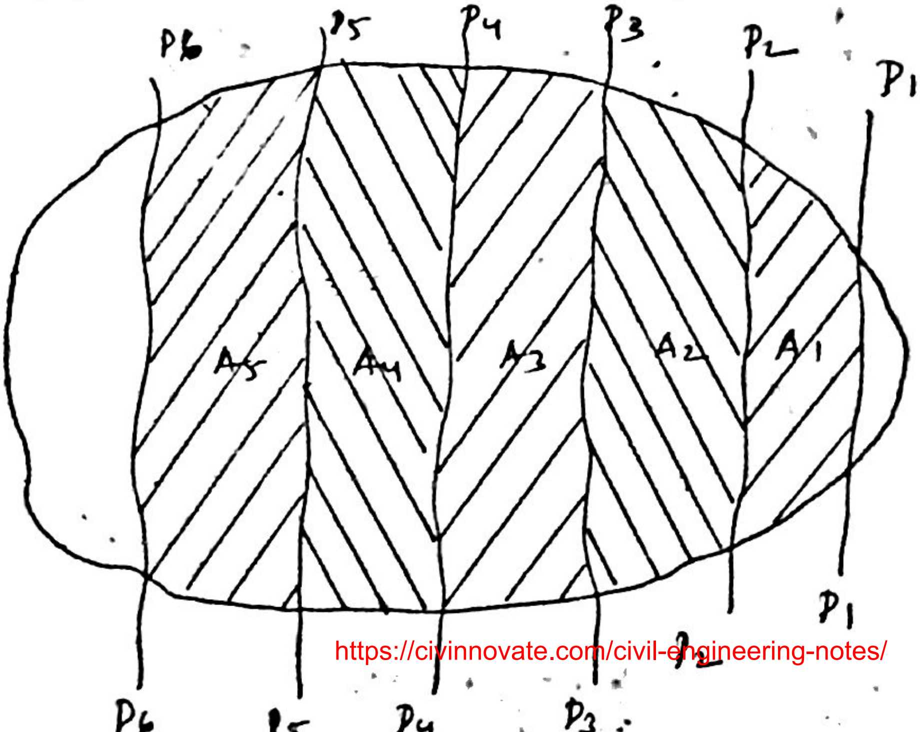
**

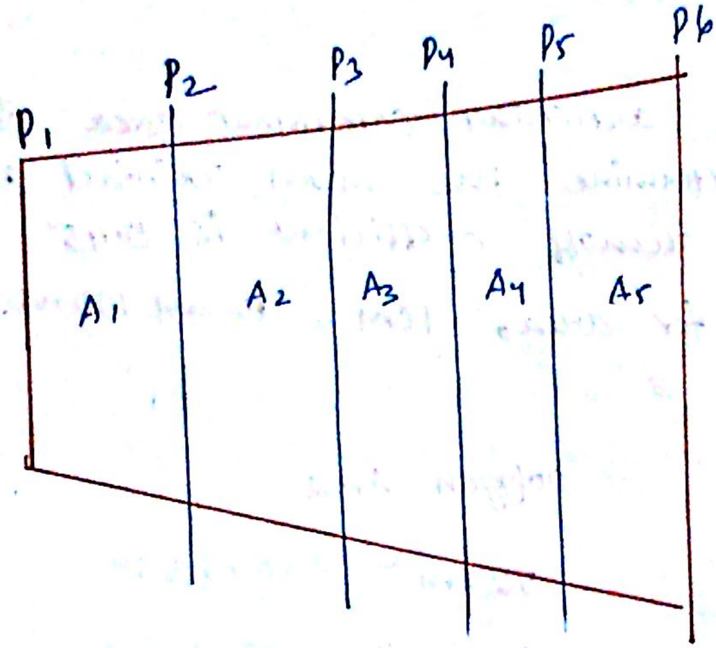
(iii) **Isohyetal method** - Isohyetal line is a line joining points of equal depth of rainfall, ie contour of rainfall. In this method a number of Isohyetal lines are drawn and hence a number of rain gauges are required.

→ This is the most accurate method

→ In this method the area of catchment in b/w 2 isohyetal lines are determined by Planimeter

→ The rainfall for each area (b/w 2 isohyetal line) is the mean value of 2 isohyetal line as shown in fig. below -





$$\bar{P} = P_{AV} = \frac{\frac{P_1+P_2}{2} \times A_1 + \frac{P_2+P_3}{2} \times A_2 + \frac{P_3+P_4}{2} \times A_3 + \cdots}{A_1 + A_2 + A_3 + \cdots}$$

Numericals

① The rainfall on a particular catchment area is given as below. Determine the mean rainfall and runoff volume if runoff co-efficient is 0.45. Take scale adopted for area, 1 cm = ~~10 cm~~ 100 m.

Rain gauge Station	Rainfall	polygon Area
1.	30 cm	50 cm² $= 50 \times (100)^2$
2.	55 cm	55 cm² $= 55 \times (100)^2$
3.	80 cm	100 cm² $= (100 \times 100^2)$
4.	100 cm	80 cm² $= 80 \times 100^2$
5.	90 cm	60 cm² $= 60 \times 100^2$

soln:

$$P_{AV} = \frac{P_1A_1 + P_2A_2 + \cdots}{A_1 + A_2 \cdots}$$

$$= \frac{\Sigma PA}{\Sigma A}$$

$= (74.35 \text{ cm})$ ~~100%~~

$= 74.35$ cm

(ii) $C = \frac{R}{P}$

$R = 74.35 \times 0.45 = 33.4575$ cm

Run off volume = Runoff depth (R) × Total Surface area

$\frac{R}{100} \times A_{total}$

$=$ ~~115.43×~~ $= 1.154 \times 10^6$ m³

$= 1.15$ MCM

Runoff discharge, $Q = \frac{Vol}{}$

① The change in weather [climate] takes place in Troposphere (upto 11 km from the earth surface)

② The temperature in troposphere decreases along t height (altitude) called lapse rate, having value 6.56 °c / Km

③ The forms of precipitation –

	Forms of precipitation	Description
①	Drizzle	Dia of water droplet < 0.5mm, intensity < 1mm/hr
②	Rain	dia of rain drop > 0.5mm, less than 6mm. intensity > 1mm/hr
③	Snow	In Himalayan zone, density of ice = $\frac{1}{10}$ 0.1gm/cc, $\frac{1}{10}$th of density of water ($10\rho_w$)
④	Glaze	The rain drop becomes ice at 0°c near the earth surface called glaze
⑤	Sleet	The rain drop may become ice while falling from the cloud before reaching the earth surface called sleet, the combination of rain & snowfall is called sleet in U.K
⑥	Hail	The snowfall having ice crystal more than 8mm in size with heavy vertical current

④ The earth Surface covered by the ocean is about 71% and the saline water in the ocean & sea is about 97% of total water on earth surface.

⑤ The Tropical Cyclone is also called ~~Tropical~~ cyclone in India, Hurricane in U.S.A and Typhoon in South East Asia.

* ⑥ In the case of cyclone low pressure develops having anti-clockwise wind at Northern hemisphere. The pressure in cyclone is 915 millibar (less than atmospheric pressure → 1.1 bar)
The aerial extent of cylone is about 100 - 200 km and the eye (centre of cyclone) is 10 km - 20 km.

⑦ In case of anti-cyclone the pressure is higher than atmospheric and the rotation of wind is clockwise at Northern hemisphere.

⑧ The interface of hot and cold wind (2 distinct air mass) is called Front

⑨ The localized hitting and pulling of air mass producing short duration heavy rain for small area (about 10 km²) (10x10) is called convective precipitation.

⑩ The precipitation from the cloud occurs only when the Cloud -

(a) must contain nucleus (nuclei)

(b) must contain Solid Salt

(c) must contain colloidal particles

(d) Independent of wind velocity

⑪ In India the monsoon (south-west monsoon) onset (starts) in Early June (June 1st) in Kerala and Assam. and Ends in September (June to September) and the 75% of total rainfall in India occurs during monsoon only, and the value of rainfall during monsoon is -

(a) In Kerala and Assam → more than 200 mm (upto 400 mm
(North East states)

(b) Western ghat (Bombay) → About 200 cm

(c) West Bengal → less than 200 cm (150 to 200 cm)

(d) Uttar pradesh → 120 to 140 cm

(e) Average rainfall during monsoon → About 95 cm

(f) Jammu & Kashmir → less than 200 cm

⑫ The rainfall taking place in Tamil Nadu is due to Post monsoon (october, November) where low pressure in Bay of Bengal takes place called cyclone (in southern peninsula → southern coast).

⑬ In India the winter rain occurs due to western disturbances in Afganistan.

⑭ In India the least variation of rainfall occurs in Heavy rainfall Zone

⑮ According to IS 5225-1969, there are foll. types of rain gauges -

(i) Non-Recording type - According to Mr. Symon the dia of non-recording rain gauge is 127 mm.
(12.7 cm)

→ Recording time → 8:30 A.M. IST

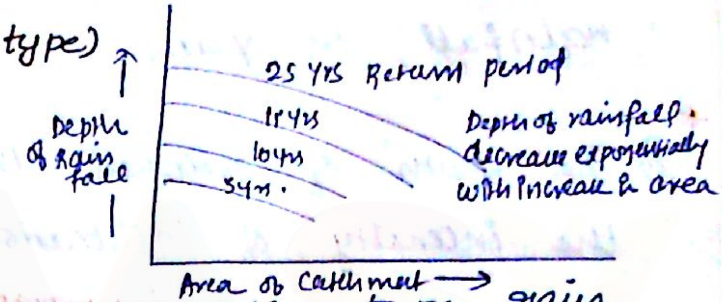
(b) According to IMD (Indian meteorological department)
Area of c/s of non-recording rain gauge collector is 100 cm² and 200 cm² :

(ii) Recording Rain gauge - (Automatic rain gauge)

(a) In this type of rain gauge the intensity of rainfall is recorded automatically.

(b) There are 4 types of recording rain gauge -

① Natural Syphon type (Float type)
② weighing Bucket type
③ Tipping bucket type
④ RADAR type



→ In India the most popular recording type rain gauge adopted is Natural Syphon type (Float type), which helps to plot mass curve of rainfall.

The tipping bucket type is not used in India and it cannot help to produce mass curve of rainfall.

* ⑯ The ~~direct~~ ~~dad~~ DAD curve (Depth Area Duration curve) is a graph between area of catchment (km²) on x-axis, depth of rainfall on y-axis and duration (Frequency of rainfall) is variable curve.

The DAD curve is always of falling nature, ie

(a) If area of catchment increases then the depth of rainfall decrease for a given duration

(b) If the duration (frequency) of rainfall increases ~~for~~ then the depth of rainfall also increases for a given catchment area

⑰ To check the consistency (inconsistency) of rainfall a particular rain gauge station due to landslide, fire (or) earthquake, the *double* mass curve* is adopted.

Double mass curve is a plot b/w accumulated rainfall of surrounding rain ~~ao~~ gauge station on X-axis and the cumulative rainfall at the missing station (inconsistency rainfall) on Y-axis

* ⑱ The depth of rainfall, the depth of runoff and the intensity of infiltration (ϕ-index) are determined with the help of hyerograph.

Hyerograph is a plot b/w the intensity of rainfall on Y-axis (dependent) and the duration of rainfall on X-axis having bar chart.

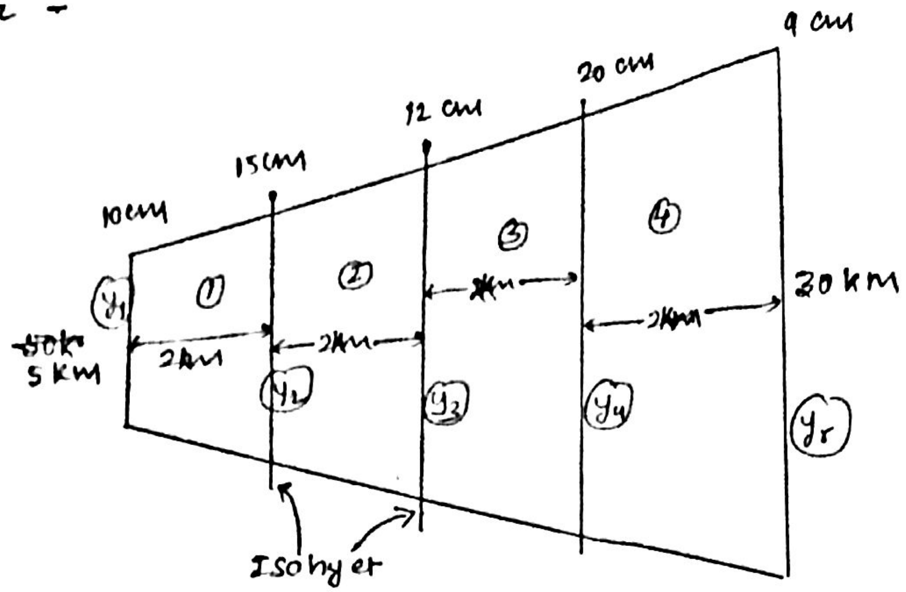
* ⑲ The rainfall due to presence of a barrier like mountain is known as Orographic precipitation.

⑳ According to WMO (World Meteorological organisation) atleast 10% of total rain gauge must be of recording type and the number of raingauges should be higher in hilly zone (mountaineous zone) and lowest in arid zone (desert zone)

㉑ The line joining points of equal snowfall is called isonif

Isohaline → Salinity
Isohel → Sunshine
Isobront → Thunder
Isobath → depth of water

① Determine the mean rainfall with the help of given data -



Sdn:

~~y_1~~ $y_2 = y_1 + \frac{y_5 - y_1}{8} \times 2$

$$\boxed{y_2 = 11.25 \text{ km}}$$

$y_3 = y_1 + \frac{y_5 - y_1}{8} \times 4$ $\qquad 5 + \frac{25}{8} \times 4$

$$\boxed{y_3 = \cancel{20 \text{ km}}}$$ 17.5 km

$y_4 = y_1 + \frac{y_5 - y_1}{8} \times 6$

$$\boxed{y_4 = \cancel{27.5} \text{ km}}$$ 23.75 km

$A_1 = \frac{5 + 11.25}{2} \times 2 = 16.25 \text{ km}^2$

$A_2 = \frac{11.25 + 20}{2} \times 2 = 31.25 \text{ km}^2$

$A_3 = \frac{20 + \cancel{27.5}^{23.75}}{2} \times 2 = \overset{43.75}{\cancel{47.5 \text{ km}^2}}$

$A_4 = \frac{^{23.75}\cancel{27.5} + 30}{2} \times 2 = \overset{53.75}{\cancel{57.5}} \text{ km}^2$

$$P_{AV} = \frac{\frac{P_1+P_2}{2} \times A_1 + \frac{P_2+P_3}{2} \times A_2 + \cdots}{A_1 + A_2 + \cdots}$$

$$= \frac{\frac{\overset{10}{\enclose{circle}{5}} + \overset{15}{\enclose{circle}{11.25}}}{2} \times 16.25 + \frac{\overset{15}{\enclose{circle}{11.25}} + \overset{12}{\enclose{circle}{20}}}{2} \times 31.25 + \frac{20+27.5}{2} \times 47.5 + \frac{27.5+30}{2} \times 57.5}{16.}$$

$$\boxed{\bar{P} = 14.5 \text{ cm}}.$$

② Determine the mean rainfall and the runoff volume with the following data- Runoff Co-eff = $\frac{R}{P} = 0.4$

Isohyet Value	Area
Station - 16 cm	80 km^2
16 - 14 cm	100 km^2
14 - 12 cm	92 km^2
12 - 10 cm	180 km^2
10 - 8 cm	110 km^2

Soln:

$$\bar{P} = P_{AV} = \frac{\frac{16+16}{2} \times 80 + \frac{16+14}{2} \times 100 + \frac{14+12}{2} \times 92 + \frac{12+10}{2} \times 180 + \frac{10+8}{2} \times 110}{80 + 100 + 92 + 180 + 110}$$

$R = KP$

$= 12.36$ cm

$$\text{Runoff Volume} = \sum A \times R \times \overset{C}{(0.4)}$$

$\sum A \times R$
$= [80 + 100 + 92 + 180 + 110] \times 10^6$
$\times 0.4 \times 0.1236$
$= 27.26 \times 10^6 \; m^3$

$= 27.78 \times 10^6 \; m^3$

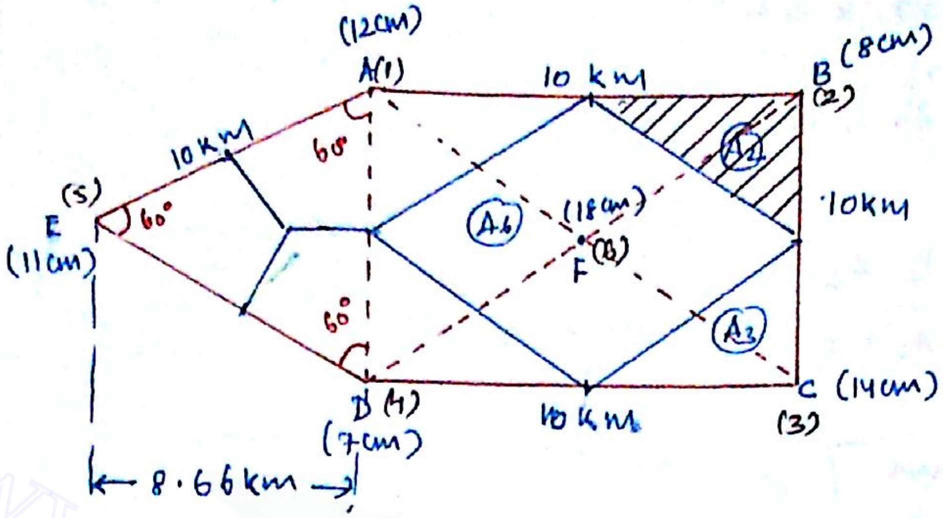
$= 27.78$ MCM

$$\text{Run off discharge} = \frac{\text{volume of Runoff}}{\text{time.}}$$

$$\boxed{Q = \frac{Vol}{t}}$$

② rainfall

shown in fig. below.



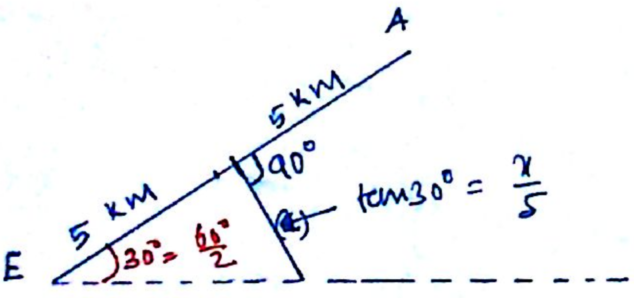
$$\bar{P} = P_{AV} = \frac{P_1 A_1 + P_2 A_2 + \cdots}{A_1 + A_2 + \cdots}$$

$$A_2 = \frac{1}{2} \times 5 \times 5 = 12.5 \text{ km}^2$$

$$A_3 = 12.5 \text{ km}^2$$

$$A_6 = \sqrt{5^2+5^2} \times \sqrt{5^2+5^2}$$

$$= 50 \text{ km}^2$$



$$\Rightarrow x = 5 \tan 30^\circ$$
$$= 2.89 \text{ km}$$

$$A_5 = 2 \times \frac{1}{2} \times 5 \times 2.89$$

$$= 14.43 \text{ km}2$$

$$A_1 = \frac{\left(\frac{1}{2} \times 10 \times 8.66\right) - 14.43}{2} + \frac{1}{2} \times 5 \times 5$$

$$A_1 = \overset{26.93}{\cancel{14.435}}\ km^2$$

$$A_4 = \overset{26.93}{\cancel{14.435}}\ km^2$$

$$\bar{P} = \frac{P_1 A_1 + P_2 A_2 + \cdots}{A_1 + A_2 + A_3 + \cdots}$$

$$\boxed{\bar{P} = 12.88\ cm}$$

ABSTRACTION FROM PRECIPITATION

→ Water losses

→ Infiltration, Evaporation, interception
(2014)

Infiltration

① Infiltration is the flow of water into the ground through the soil surface.

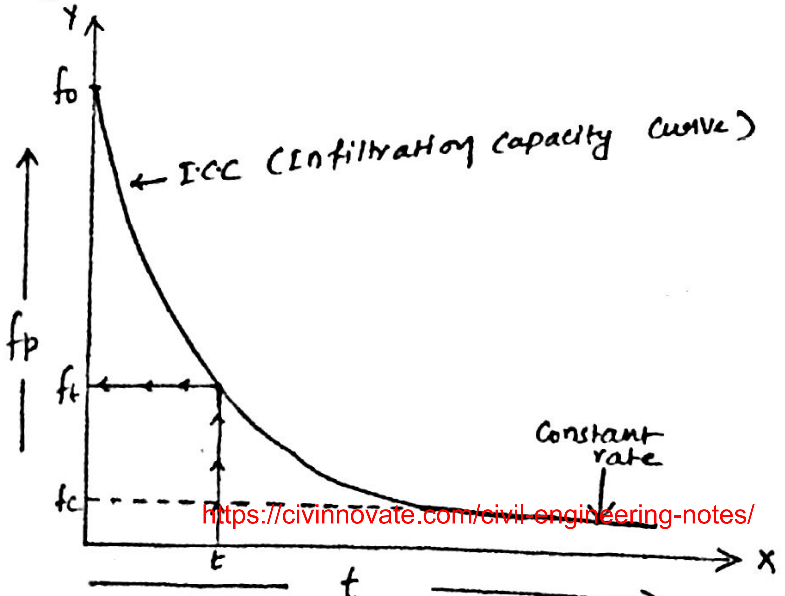
② The maximum rate of absorption of water by the soil mass below the ground surface at a particular time is known as infiltration capacity represented by symbol f_p. It's unit is cm/hr.

The infiltration capacity is not constant, i.e it is highest at the initial stage and it goes on decreasing and then a constant rate is achieved.

It was developed by Mr. Horton (1933).

(infilt. cap)

(2014) According to Mr. Horton, the decrease in infiltration capacity is exponential as given below



Horton's Equation

(2014) (Kwe)

$$\boxed{f_t = f_c + (f_o - f_c)\, e^{-K_h t}}$$

Where,

f_t → Infiltration capacity at any time 't' (cm/hr) (mm/hr)

f_c → constant rate of infiltration (cm/hr)

f_o → Initial infiltration capacity (cm/hr)

K_h → Horton's decay co-efficient, which is dependent on type of soil and vegetative cover (grass)

Unit → $\frac{1}{hr}$

→ The value of K_h is determined by considering

$$\boxed{t = \infty}$$

The total depth of infiltration at a given time is determined by integration of Horton's equation, i.e

$$F_p = \int_0^t f_t \times dt$$

$$= \int_0^t \left[f_c + (f_o - f_c)\, e^{-K_h t}\right] dt$$

$$F_p = f_c \times t + \frac{(f_o - f_c)\, e^{-K_h t}}{-K_h}$$

NOTE: The total depth of infiltration in first half of duration is always more than the total depth of infiltration during the second half of duration.

$$F_{p_1} = \int_0 f_t \times dt$$

$$F_{p_2} = \int_{t'}^{t} (f_t \times dt)$$

$$\boxed{F_p = F_{p_1} + F_{p_2}}$$

* (2014) The average rate of infiltration,

$$f_t = \frac{F_p}{t} = \frac{F_{p_1} + F_{p_2}}{t}$$

$$f_t = \frac{\int_0^t [f_c + (f_0 - f_c)e^{-K_h t}]\, dt}{t}$$

The value of Horton's decay co-efficient (K_h) is determined as below

$$\overset{(cm)}{F_p} = \int_0^t [f_c + (f_0 - f_c)e^{-K_h t}]\, dt$$

$$= \int_0^t f_c\, dt + \int_0^t (f_0 - f_c)e^{-K_h t} \cdot dt$$

$$= f_c t + \int_0^\infty (f_0 - f_c)e^{-K_h t} \cdot dt$$

$$= f_c t + \left[\frac{(f_0 - f_c)e^{-K_h t}}{K_h}\right]^\infty$$

$$= f_c t + \left(\frac{1}{-K_h}\right)\left[(f_0 - f_c)\left(e^{(-)K_h\infty} - e^{-0K_h}\right)\right]$$

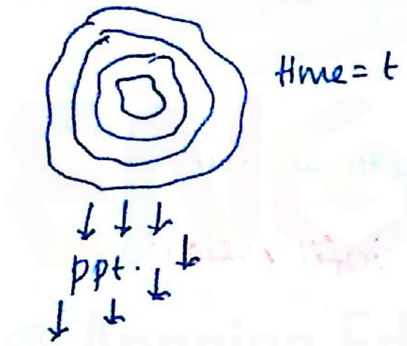
$$= f_c t - \frac{1}{K_h} \times (f_0 - f_c)\left[0 - 1\right]$$

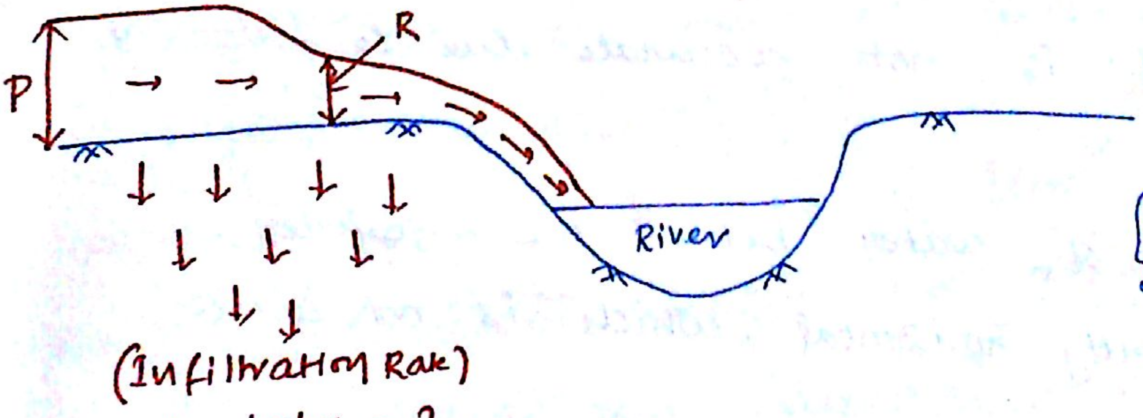
$$F_p = f_c t + \frac{f_0 - f_c}{K_h}$$

$$\frac{f_0 - f_c}{K_h} = F_p - f_c t$$

$$* \boxed{K_h = \frac{f_0 - f_c}{F_p - f_c t}}$$

$\left(\frac{1}{hr}\right)$





(Infiltration Rate)

[φ-index & w-index Same in lack of hydrograph]

φ - index = ?

w - index = ?

$$w\text{-Index} = \frac{P - R}{t_{Rain}} \text{ or } \frac{P - R - I_{loss}}{t_{Rain}}$$

← Initial loss.

① To measure the infiltration rate (infiltration capacity) the methods used are -

(i) Flooding type infiltrometer

(ii) 2 concentric ring type infiltrometer

(iii) Rain simulator

② In flooding type infiltrometer a vertical cylinder is placed below G.L having dia 30 cm and length of 60 cm. The 50 cm length is kept below G.L and remaining 10 cm is above G.L. The water level in the hollow cylinder is 5 cm below the top of the cylinder (5cm above G.L).

The rate of supply of water in hollow cylinder to maintain constant level is called infiltration rate (or) infiltration capacity.

This method is not accurate due to following reasons -

(a) Infiltration of water below the cylinder becomes partly horizontal, which is not correct way.

(b) There is no effect of rain beating (a natural phenomenon is not achieved)

(c) The water is pure water which is not correct, ie it should become turbid (dirty)

rainfall.

③ An arrangement on flooding type infiltrometer is adopted where 2 concentric rings having inner dia 30 cm and outer dia 60 cm is adopted. The length of the concentric rings is 25 cm where 15 cm length is below G.L and 10 cm is above the G.L. The water level is 5 cm below the top of the ring. The water available in outer ring satisfies the horizontal infiltration below the bottom of the ring. The rate of supplying water to the inner ring to maintain a constant level of water in the inner ring is called the infiltration capacity.

This method is also not accurate one because there is no effect of rain beating as well as turbidity of water.

④ The most accurate method of determination of infiltration rate (or) capacity is Rain simulator. In this method a piece of land having surface area 2m x 4m is adopted and a no. of nozzles are placed above the selected area. The intensity of sprinkling water through the nozzles is varied to make an analogous natural rainfall. The surface ~~rai~~ runoff is collected from the selected area. A Hyetograph (or) a hydrograph is obtained and then infiltration rate is determined.

⑤ The infiltration rate achieved from flooding type infiltrometer is always higher (greater) than the infiltration rate achieved from Rain Simulator.

* $\boxed{f_f > f_r}$

⑥ If the intensity of rainfall is less than infiltration capacity then the actual rate of infiltration is equal to the intensity of rainfall.

But if the intensity of rainfall is more than the infiltration capacity then the actual rate of infiltration is equal to infiltration capacity.

$$f = i \quad , \quad \text{if } i < f_c$$

$$f = f_c \quad , \quad \text{if } i > f_c$$

*** ⑦ φ - Index

φ- index is an average rate of infiltration of water below the ground surface and depending upon intensity of rainfall and Runoff quantity.

φ-index can be defined as ~~the~~ a particular intensity of rainfall above which the rainfall volume is equal to runoff volume, But if the rainfall is less than that intensity then there will be no runoff, i.e. only infiltration

The φ- index is determined with the help of Hyetograph where the area of hyetograph above the φ- line gives* depth of effective rainfall (ER), depth of rainfall excess (RE), Direct runoff (DRO), Runoff (R).

If the area of a catchment (m^2) is multiplied with the effective area of hy~~dro~~etograph (m) (above φ- line) (m) then it gives the area of direct run off hydrograph (DRH) which is nothing but total volume of Run off.

In lack of sufficient data, i.e hyetograph is not available, the φ- index ~~Can~~ may be taken as,

$$\boxed{\phi\text{-index} = \frac{P-R}{t}}$$

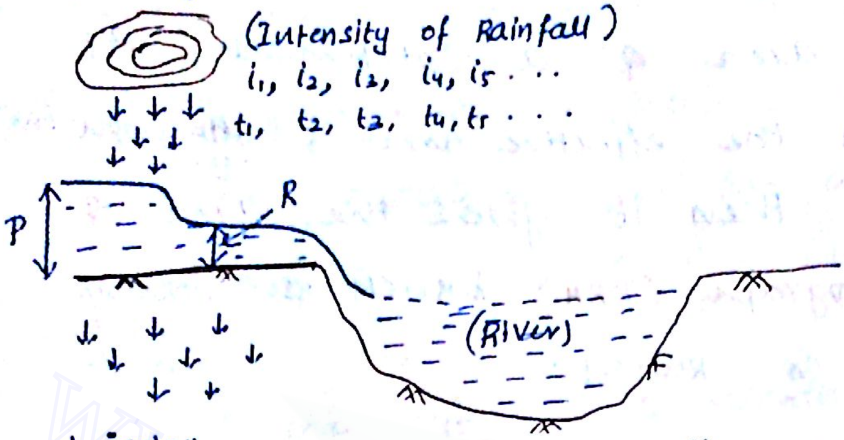
Where, P → total depth of rainfall (nothing but total ~~area~~ area of Hyetograph)

R → Total depth of Runoff (DRO) (ER) (RE) which is equal to $\frac{\text{total Vol. of Run off}}{\text{Catchment area}}$

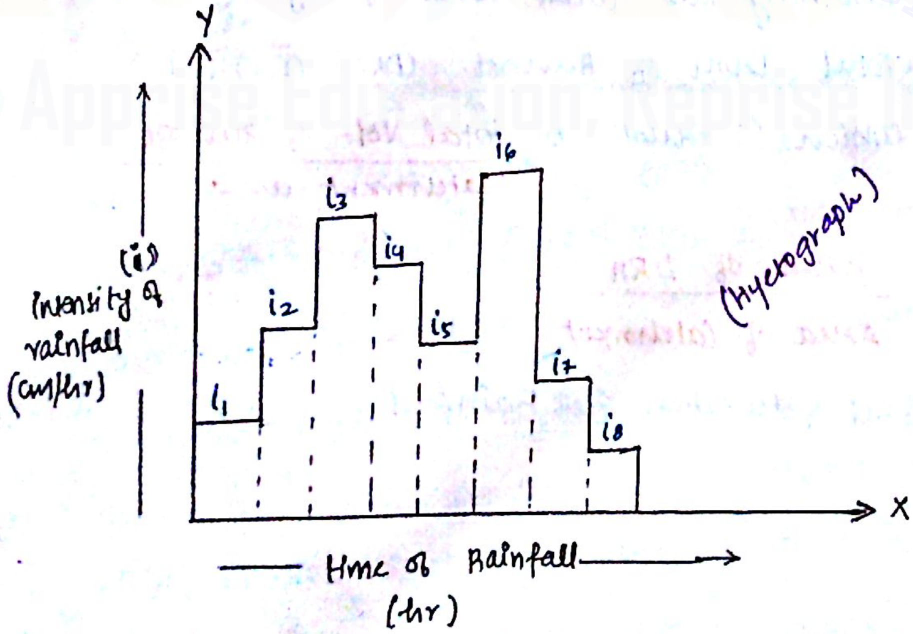
$$= \frac{\text{Area of DRH}}{\text{Area of Catchment}}$$

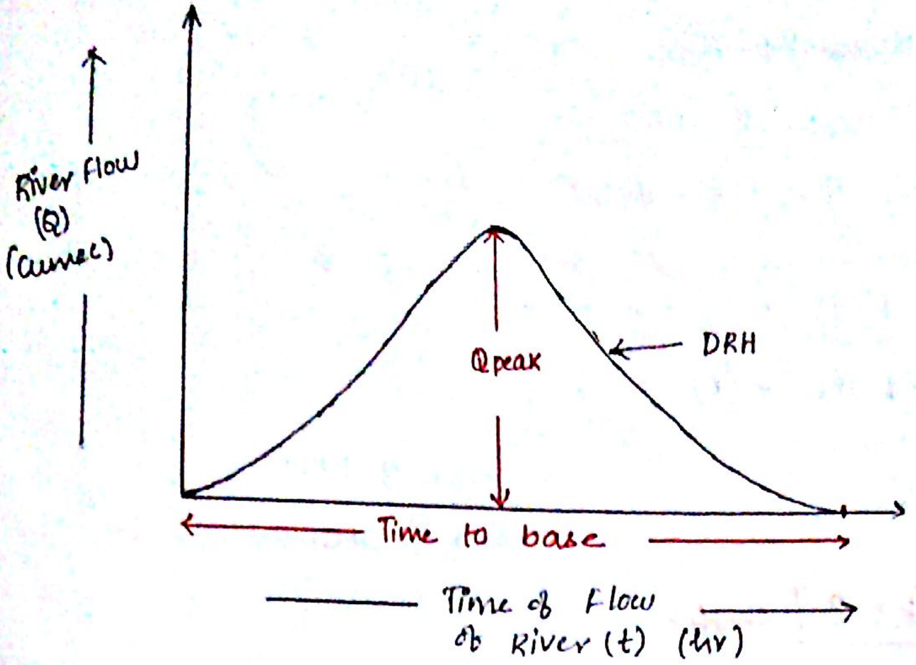
t → Total duration of Rainfall.

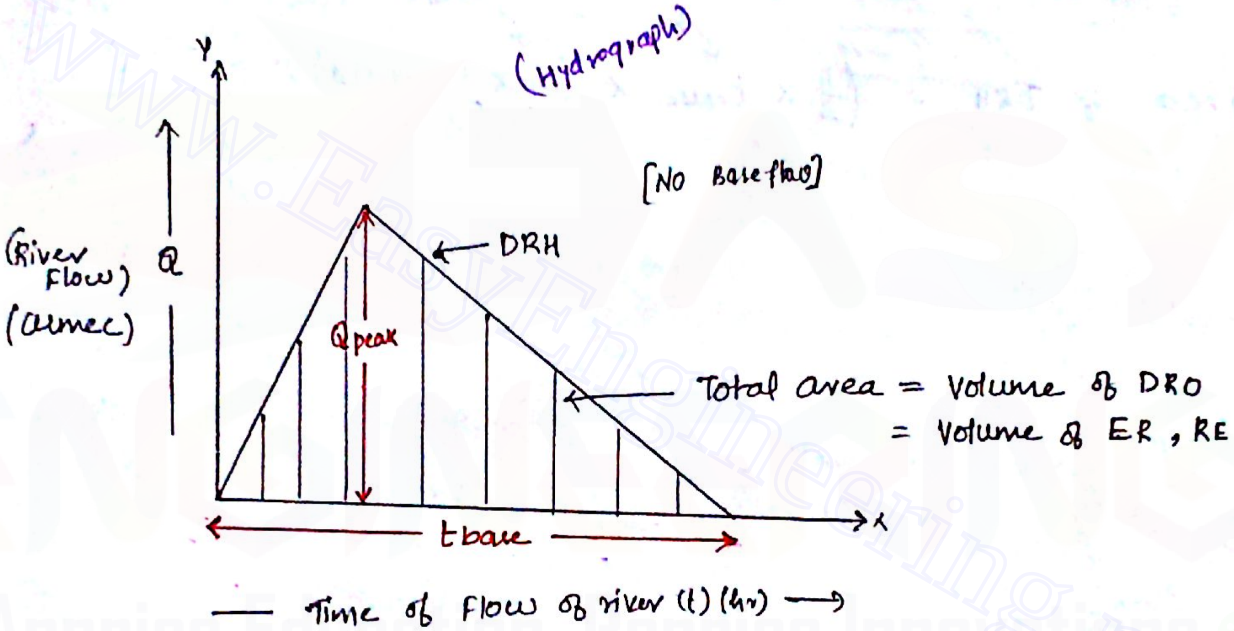
If no data is available regarding rainfall and runoff then the value of ϕ-index may be assumed as 0.1 cm/hr to determine flood discharge.

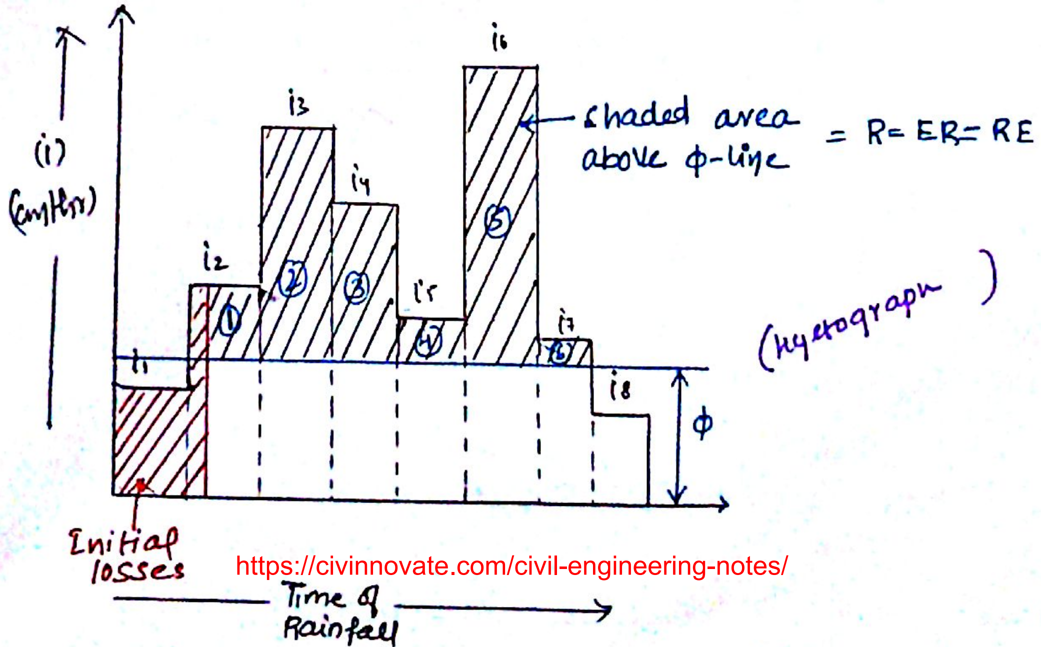


Time (t)	Direct Runoff discharge (Q)
0	0
t_1	Q_1
t_2	Q_2
t_3	Q_3
⋮	⋮




River flow
(Q)
(cumec)
Q peak
DRH
Time to base
Time of Flow of River (t) (hr)


(Hydrograph)
Y
(River Flow)
Q
(cumec)
[No Base flow]
DRH
Q peak
Total area = Volume of DRO
= Volume of ER , RE
x
t base
Time of Flow of river (t) (hr)


i6
i3
i4
i2
i5
i1
i7
i8
(i)
(cm/hr)
Shaded area above φ-line = R= ER= RE
①
②
③
④
⑤
⑥
(hyetograph)
φ
Initial losses
Time of Rainfall

= Depth of DRO

= $R = ER = RE$

$$(i_2 - \phi) t_2 + (i_3 - \phi) t_3 + (i_4 - \phi) t_4 + (i_5 - \phi) t_5 + (i_6 - \phi) t_6 + (i_7 - \phi) t_7 = R$$

$$= \frac{\text{Area of DRH}}{\text{Area of Catchment}}$$

$\rightarrow \boxed{\phi = ?}$ cm/hr

$$\text{Area of DRH} = \left(\frac{1}{2} \times \overset{hr}{t_{base}} \times \overset{m^3/hr}{Q_{peak}}\right) \rightarrow (m^3)$$

NUMERICALS

① The intensity of rainfall in 1st storm is 4 cm/hr and the duration of storm is 5 hr. The Runoff is 12 cm. The duration of 2nd storm is 8 hr on the same place having runoff depth 14 cm. Determine –

(i) ϕ - index

(ii) intensity of 2nd storm.

Soln :

(i) $$\phi\text{- index} = \frac{P-R}{t} = \frac{P}{t} - \frac{R}{t} = i - \frac{R}{t}$$

$$= 4\,cm/hr - \frac{12\,cm}{5\,hr}$$

$$= 1.6\ cm/hr$$

(ii) ϕ- index $= \phi_2$

$$1.6 = \frac{P_2 - R_2}{t_2} = i_2 - \frac{R_2}{t_2}$$

$$1.6 = i_2 - \frac{14}{8}$$

$$\Rightarrow i_2 = 3.35\ cm/hr$$

② The Horton equation is given as below.

(2019) Determine -

(i) Depth of infiltration in 1st half an hour

(ii) Depth of infiltration in 2nd half an hour.

(iii) Total depth of infiltration

(iv) Average rate of infiltration

$$f_t = \left[3 + 1.2e^{-1.5t}\right]$$

$\downarrow$

(cm/hr) $\quad t \to hr$

$$\boxed{t = 1\ hr}$$

(1) $F_{p_1} = \int_0^{t_1} f_t \times dt$

$$= \int_0^{\frac{1}{2}hr} (3 + 1.2\, e^{-1.5 \times 0.5 t})\, dt$$

$= 1.92$ cm

(ii) $F_{p_2} = \int_{t_1}^{t_2} (f_t \times dt)$

$$= \int_{1/2}^{1} (3 + 1.2\, e^{-1.5t})\, dt$$

= ~~3~~ 1.7 cm

(iii) Total depth of infiltration (or) $F_p = F_{p_1} + F_{p_2}$

(or) $F_p = \int_0^1 (3 + 1.2\, e^{-1.5t})\, dt$

$= 1.92 + 1.7$

$= 3.62$ cm

(iv) Avg. rate of infiltration $= \frac{F_p}{t} = \frac{3.62\ cm}{1\ hr}$

$= 3.62$ cm/hr

(2) Determine Horton's decay co-efficient, with the

given data -

(i) initial infiltration capacity = 6cm/hr

(ii) Duration of rainfall = 8hrs.

(iii) constant infiltration capacity = 1.2 cm/hr

(iv) Total depth of infiltration = 15 cm

Soln:

$$F_p = \int_0^t f_t \times dt = \int_0^t \left[f_c + (f_0 - f_c) e^{-K_h \times t} \right] dt$$

$$= f_c t + \int_0^\infty (f_0 - f_c) e^{-Kt} \cdot dt$$

$$F_p - f_c \times t = (f_0 - f_c) \left(\frac{e^{-kt}}{-k} \right)_0^\infty$$

$$\underset{(5.4)}{F_p - f_c \times t} = \frac{f_0 - f_c}{-K} \left[e^{-\infty} - e^0 \right]$$

$$= \frac{(f_0 - f_c)(0 - 1)}{-K}$$

$$5.4 = \frac{f_0 - f_c}{K} = \frac{6 - 1.2}{K}$$

$$\boxed{K = 0.89 \ /hr}$$

④ Determine the depth of runoff with the foll. data if φ-index is 1.2 cm/hr

Time (Min)	Rainfall.	
0	0	
30	3 cm	6 cm/hr
1hr	4 cm	8 cm/hr
1.5 hr	2 cm	4 cm/hr
2 hr	1.5 cm	3 cm/hr

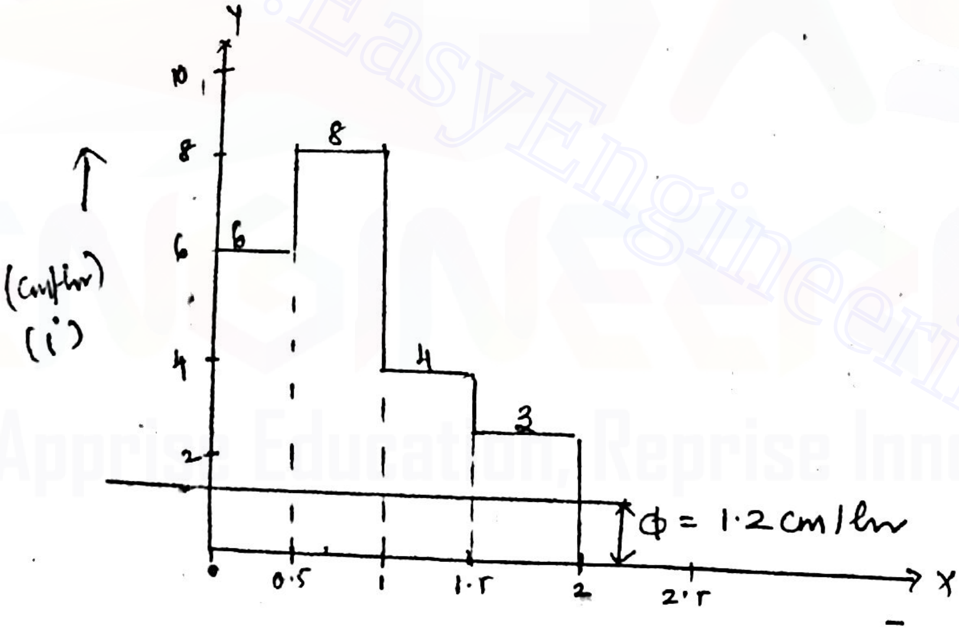
Soln: (PRO)(R)(RE)(ER)

Depth of runoff

$= (6-1.2)\times 0.5 + (8-1.2)0.5$

$\quad + (4-1.2)0.5 + (3-1.2)0.5$

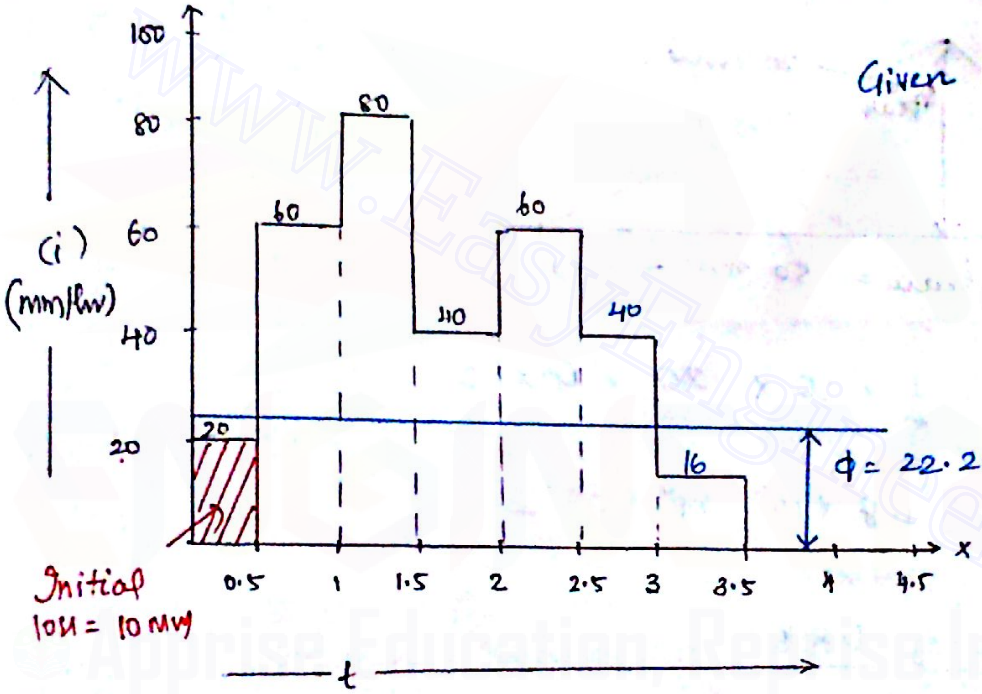
$= 8.1$ cm



⑤ Determine ϕ - index with the foll

Take initial loss ~~10 cm~~ 10 mm. E.R = 80 mm

Time (hr)	0.5	1	1.5	2	2.5	3	3.5
Cumulative ppt (mm)	10	40	80	100	~~35~~ 130	150	158
(i) (mm/hr)	20	60	~~160~~ 80	~~200~~ 40	~~260~~ 60	~~300~~ 40	~~316~~ 16



Given R = 80 mm

$$\phi = \frac{P-R}{t} = \frac{158-80}{3.5} = 22.29 \text{ cm/hr} \quad (\text{Assumed})$$

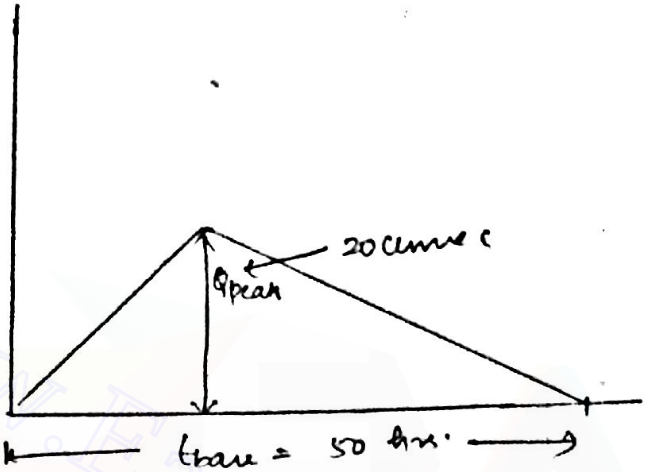
$$\text{Shaded Area} = (60-\overset{\phi}{\cancel{22.29}})0.5 + (80-\overset{\phi}{\cancel{22.29}})0.5 + (40-\overset{\phi}{\cancel{22.29}})0.5 + (60-\overset{\phi}{\cancel{22.29}})0.5 + (40-\overset{\phi}{\cancel{22.29}})0.5 = 80 \text{ mm}$$

= ~~83.665 cm = R.~~

$$\therefore \frac{158-83.661}{3.5} = \quad \Rightarrow \phi = 24 \text{ cm/hr}$$

Determine the value of φ-index if the total depth of rainfall is 15 mm (P) in duration of 4 hr. The peak of DRH is 20 cumec and the time $\overset{\text{to}}{\text{of}}$ base is 50 hrs. Take Catchment area ~~1000~~ 1400 sq.km

Soln:



$$\text{Area of DRH} = \frac{1}{2} \times 50 \times 20 \times 60 \times 60$$

$$= 1.8 \times 10^6 \ m^3$$

$$R = \frac{\text{Area of DRH}}{\text{Area of Catchment}}$$

$$= \frac{1.8 \times 10^6}{1400 \times 10^6} \times 1000$$

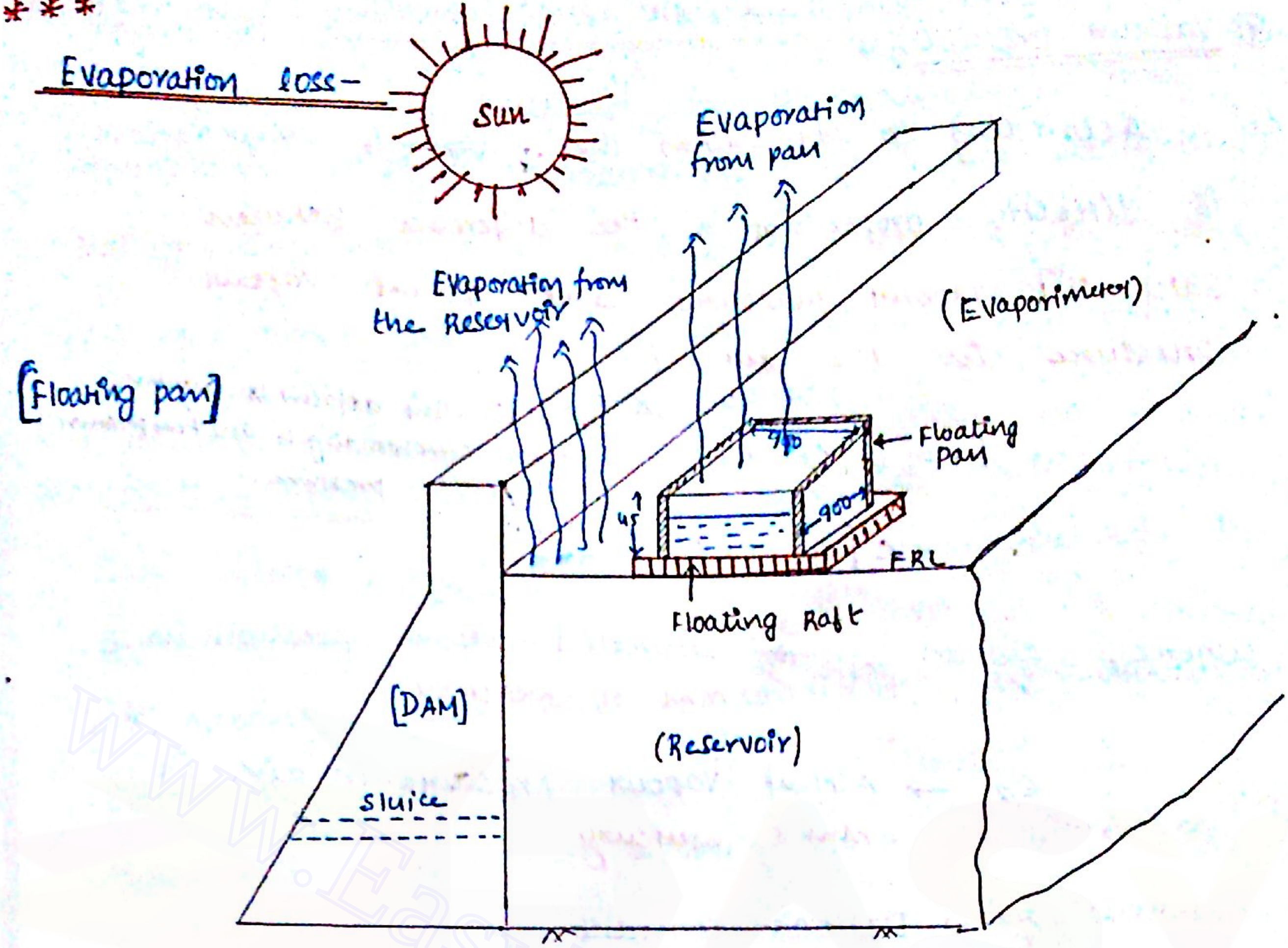
$$= 1.286 \ mm$$

$$\phi = \frac{15 - 1.286}{4}$$

$$\boxed{\phi = 3.43 \ mm/hr}$$

***

Evaporation loss-



① Evaporation is a process where a liquid is converted into gaseous state at the free surface below the boiling point due to transfer of heat energy

② Evaporation is a cooling process where internal heat transfer is about 586 calories per gram called latent heat of vaporisation.

③ There are following factors affecting evaporation –

(a) Vapour pressure

(b) Temperature

(c) Wind velocity

(d) Barometric pressure (local atmospheric pressure)

(e) Solution of salt (specific gravity)

(f) Type of water body (like sea, ocean)

④ <u>Vapour pressure</u> –

(i) According to Mr. Dalton the depth of evaporation is directly proportional to the difference between saturated vapour pressure and actual vapour pressure in the air, i.e

(Thin air film adjacent to water surface corresponding to the temperature = saturation pressure.

$$E = c\,[e_s - e_a]$$

$$= c\,[e_w - e_a]$$

where, e_w (or) e_s → saturated vapour pressure in mm of mercury

e_a → Actual vapour pressure in air in mm of mercury

c → Dalton's constant.

E → depth of evaporation from water body in mm/day

→ Area large → Evaporation higher.

(ii) <u>Temperature</u> – If the temperature of water is raised, then the evaporation increases.

(iii) <u>Wind Velocity</u> – If the wind velocity is increased upto critical wind velocity then the evaporation increases. The wind velocity is measured by Anemometer.

The wind velocity is directly proportional to $(h)^{1/7}$, where h → height of water above GL, i.e.

$$U_w \propto h^{1/7}$$

$$U_w = C h^{1/7}$$

* $$\boxed{\frac{U_{w_1}}{U} = \frac{(h_1)^{1/7}}{(h)^{1/7}}}$$

(iv) Barometric pressure

At the high altitude (top of mountain) the atmospheric pressure decreases (fall down), due to which boiling point come down and hence evaporation increases. On the other hand, if the atmospheric pressure increases then evaporation decreases.

→ Humidity more ⇒ Evaporation less.

(v) Salt solution - Due of presence of salt in sea water, the vapour pressure gets affected due to which evaporation decreases.

It means under identical conditions, the evaporation from the sea surface is less than the evaporation from the river water surface by 2 to 3%. The specific gravity of sea water is 1.025 (1025 kg/m3), but the specific gravity of pure water (or) river water (Free from salt) is 1.000 (1000 kg/m3). It means specific gravity of sea water is higher and hence evaporation is lower.

(vi) Water body - In case of ocean (largest water body) the heat storage is higher due to which evaporation is higher during winter season from the ocean.

→ Humidity measured by Hygrometer

③ Measurement of evaporation -

The evaporation is measured by following methods -

(i) Evaporimeter
(ii) Pan method
(iii) Atmometer
(iv) Emperical formulae method
(Dalton &, Mayer and Rover method)
(v) Water budget equation (Analytical method)
(vi) Heat transfer method (Energy method)
(Analytical method)
(vii) Mass transfer method (Based on turbulence of water)
(Analytical method)

Pan Method -

① Pan is a container having cylinder or square cross section where water is filled to desired depth and then at the end of observation (one day or 7 days), the depth of water loss is measured.

To determine the actual depth of evaporation from the nearby water body, pan co-efficient is adopted.

pan co-efficient (C_p) is the ratio of depth of evaporation in reservoir to depth of evaporation in the pan.

$$\boxed{C_p = \frac{E_{Reservoir\,(lake)}}{E_{pan}}}$$

Generally it is less than 1

Depth of Evaporation in reservoir, $E_{Reser} = C_p \times E_{pan}$

Volume of evaporation $= E_{Reser} \times A_{surface}$

$= C_p \times E_{pan} \times A_{surface}$

	Name of pan	Size of pan	Placement of pan	Average pan co-efficient **	Range of C_p
①	United States Bureau land Class A pan [Class A pan]	Dia = 1·21 m (1210 mm) height = 25·5 cm = 255 mm Made up of galvanized iron (or) Steel	Above the ground surface resting on timber block	0·7	0·6 to 0·8
②	ISI pan (BIS pan) Bureau of Indian Standard (Modified Class A pan)	Dia = 1·22 m height = 25·5 cm (255 mm) Made of Copper	Above the ground level resting on timber block	0·8	0·65 to 1·1
③	Colorado sunken pan	920 mm x 920 mm x 460 mm (L x B x H)	Below the ground level totally, & water is at ground level	0·78	0·72 to 0·86
④	United States geological Floating pan. (USGF)	900 mm x 900 mm x 450 mm (L x B x H)	On the top surface of the reservoir supported by floating raft	0·8	0·76 to 0·82

$$\boxed{C_p = \frac{E_{res}}{E_{pan}}}$$

E_{res} → standard.

E_{pan} → changes.

Class A pan ⇒ $0.7 = \frac{E_{res}}{E_{pan}}$ —① ⇒ $E_{pan} = \frac{E_{res}}{0.7}$

ISI pan ⇒ $0.8 = \frac{E_{res}}{E_{pan}}$ —② ⇒ $E_{pan} = \frac{E_{res}}{0.8}$

$$\Rightarrow \boxed{\underset{(ISI)}{E_{pan}} < \underset{(class A)}{E_{pan}}}$$

NOTE:

pan co-efficient of ISI pan is greater than pan co-efficient of US class A pan by 14 %.

→ Methods to Control (minimise) Evaporation loss

(i) Selection of site of the Reservoir -

The selection of site for a reservoir should be such that the site should have cup-shaped profile, ie depth of storage should be large and surface area should be low to minimize the evaporation loss.

(ii) By providing artificial rain -

Above the reservoir surface thick layer of cloud with rain should be made artificially by spraying silver iodide on existing cloud (or) spraying compressed CO_2 gas. It is a very costly method, which is practically difficult to adopt

(iii) **By providing cover** - The top of the reservoir surface should be covered by temporary shed to minimise the evaporation loss, but it is ~~a~~ very costly and hence it is not feasible.

* (iv) **By spraying Certain chemicals** - The most popular chemicals used are - (i) Cetyl Alcohol (Hexa decanol)
(ii) ~~Styreal~~ Stearyl Alcohal (octa decanol)

These chemicals are sprayed on the water surface where the quantity of the chemical is about 3.5 N / Hectare, ~~which~~ It can control evaporation upto 60%. [3.7N/Ha/day = 377gm/Ha/day]

NUMERICALS -

① A canal has average top width 15 m and the length of the canal is 10 km, Determine the vol. of water lost due to evaporation if the evaporation loss in class A pan is 5 mm/day.

Soln:

$$C_p = \frac{E_{can}}{E_{pan}}$$

$$0.7 = \frac{E_{can}}{5}$$

$$E_{can} = 3.5 \text{ mm/day}$$

$$= 3.5 \times 10^{-3} \text{ m/day.}$$

$$\text{Vol. water lost} = 15 \times 10 \times 1000 \times 3.5 \times 10^{-3}$$

$$= 525 \text{ m}^3/\text{day}$$

$$= 525000 \text{ litres/day}$$

$$= 0.525 \text{ Million litre/day}$$

② The plan area of a reservoir is 1 sq.km The water level in the reservoir is observed to decline by 20 cm. the inflow in the reservoir is 10 Ha-m, outflow from reservoir is 20 Ha-m. The pan evaporation (class A) is 12 cm and the rainfall is 3 cm. Determine the seepage lost in Ha-m

$$0.7 = \frac{E_{Res}}{0.12}$$

$$E_{Res} = 0.084 \text{ m}$$

Soln :

Inflow vol. = outflow vol. + change in storage.

$10 \times 10^4\ m^3 + 0.03 \times 10^6 = 20 \times 10^4\ m^3 + ~~(0.084 \times 10^6)\ m^3~~ (0.084 \times 10^6) + (-0.2 \times 10^6) + (G \times 10^6)$

$G = 0.016\ m$

$G = 0.016 \times 10^6$

$\boxed{G = ~~1~~.6\ Ha\text{-}m}$

$G = 4.6\ Ha\text{-}m$

③ A reservoir has average surface area of 20 sq. km, The inflow in month of June is 10 cumec and outflow in the month of June is 15 cumec. The average depth of rain fall is 10 cm. The Seepage loss is 1.8 cm. The change in storage is 16 MCM. Determine the depth of Evaporation lost in that month

Soln: $(10 \times 30 \times 24 \times 60^2) + (0.1 \times 20 \times 10^6) = (15 \times 30 \times 24 \times 60^2) + 0.018 \times 20 \times 10^6 \overset{(-)}{\cancel{+}} 16 \times 10^6 + E \times 20 \times 10^6$

(27.92×10^6)

~~$E = (-)1.366\ m$~~

$\boxed{E = ~~13.66\ cm~~}$

$\Rightarrow E = 0.2340\ m$

$E = 23.40$

DISCHARGE MEASUREMENT (STREAM FLOW MEASUREMENT)

① The branch of science which deals with stream flow measurement is known as Hydrometry (science of water)

② The stream (river) is an open channel where the surface runoff from the nearby catchment area (drainage basin) enters in it (drains in it).

③ The stream flow measurement plays an important role in -

(i) To find the capacity of a reservoir

(ii) To find the flood flow

(iii) To find the depth of surface runoff which helps in finding the infiltration capacity of the soil.

(iv) To find LWW, so that length of a bridge can be decided [$LWW = 4.75\sqrt{Q}$]

(v) To find the elevation of weir (or) barrage across a river.

④ There are 2 categories of stream flow measurement technique -

(i) Direct measurement (method)

(ii) Indirect measureme~~thod~~nt

Direct Measurement consists of following methods -

(a) Area velocity method

(b) Dilution method [chemical method]

(c) Electromagnetic method

(d) Ultra-sonic method

Indirect Measurement consists of following methods -

(a) Hydraulic structure method like weir, Notch, gates

(b) Slope area method

⑤ To determine the discharge of a stream in modern age (practically adopted), the Rating curve, which is nothing but stage-discharge relationship is adopted.

The rating curve is a graph plotted in between the water surface elevation (stage) and the corresponding discharge in the stream (river). If the stage rises then the discharge also increases and vice versa, i.e

$$\boxed{Q_R > Q_F}$$

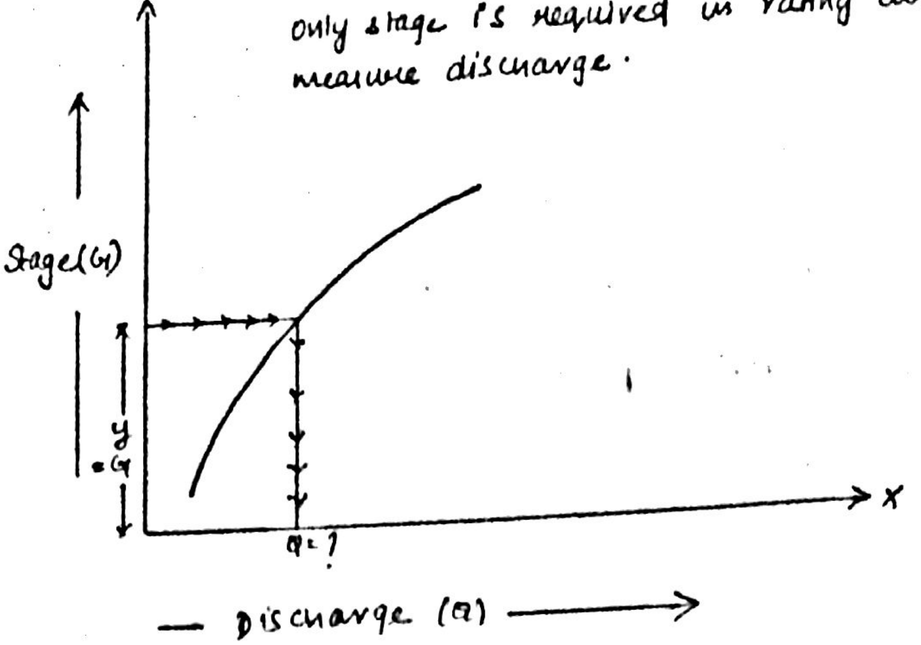
discharge in rising stage > discharge in falling stage.

The stage of the river water is the elevation of river water surface above known datum (MSL).

The stage of river flow is determined by Automatic gauge known as Floating gauge where stilling well is required. Some times another automatic gauge called Bubble gauge is adopted.

The rate of flow in a stream is determined with the help of known value of stage of the river, where the rating curve is utilised to find the discharge corresponding to known value of stage only

only stage is required in rating curve
measure discharge.



⑥ The rate of flow is defined as the volume of water passing through a section in unit time.

$$Q = \frac{Vol}{time} = \frac{A \times L}{t} = A \times V$$

$$\boxed{Q = AV}$$

A → Cross-sectional area

V → Mean velocity

The mean velocity of Flow of an open channel (river) is determined with the help of an instrument called Current meter. The current meter consists of a vertical rod supporting one horizontal arm and one balancing weight. The horizontal arm supports a revolving cup facing the opposite to the flow direction.

The revolution of the cup due to striking of the current of water is noted in ~~terms~~ terms of rps. (revd. per second)

The velocity of flow at a particular depth is given by relation, $\boxed{V = aN_s + b}$ V → m/sec a → Multiplying constant

b → additive Constant

There are 2 types of current meter –

(i) price meter

(ii) Pigmy meter (Pigmimeter)

In case of pricemeter the dia of cup is larger, ie 12.5 cm which is used for finding velocity of large river. The instrument constant of ~~prince~~ pricemeter are multiplying constant $\boxed{a = 0.65}$, additive constant, $\boxed{b = 0.03}$

$$\boxed{V = 0.65 N_s + 0.03}$$

m/sec

→ In case of pigmimeter the dia of the cup is 5 cm having instrument constants, $\boxed{a = 0.3}$ $\boxed{b = 0.003}$

The instrument constants are determined with the help of 2 algebraic equations,

$$V_1 = aN_1 + b \text{ —— } ①$$

$$V_2 = aN_2 + b \longrightarrow ②$$

$$\Rightarrow a = ? \quad b = ?$$

substitute, in

$$v = aN_3 + b = ?$$

The mean velocity of river flow (channel flow) is based on following methods –

(a) <u>one point method</u> – This method is adopted for a shallow river having depth of flow upto 3m. In this method the current meter is placed at a depth 0.6 times total depth of flow from the top and the revolution of the cup is noted at that depth,

$$\bar{V} = V_{AV} = V_{0.6d} = V_{0.6} = (a N_{s(0.6d)} + b) \rightarrow \text{(from top)}$$

(or) $V_{AV} = V_{0.4d}$ (from bottom)

(b) <u>Two point method</u> - This method is widely used, especially for moderate depth of flow. It is the most accurate method to find mean velocity of flow. In this method the current meter is placed vertically at 2 different depths,

(i) At 0.2d from the top

(ii) At 0.8d from the top

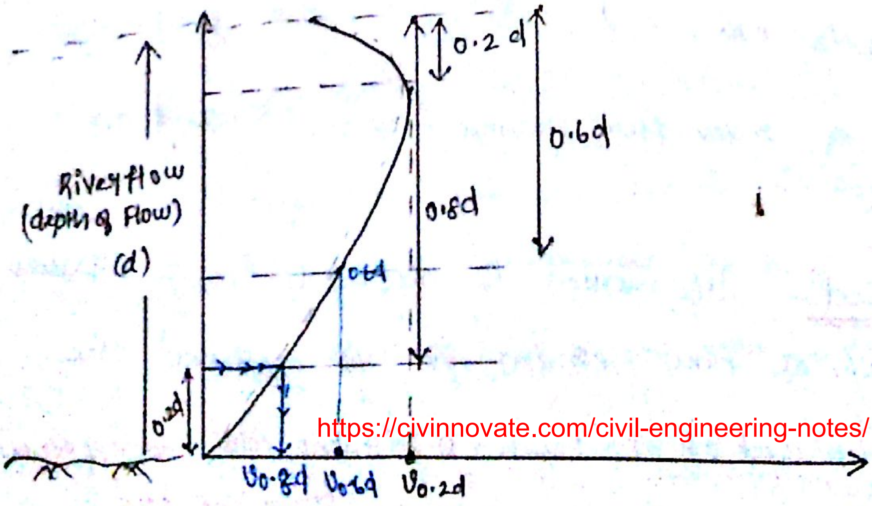
$V_{0.2d} = a N_{(0.2d)} + b \rightarrow$ (0.8d from bottom)

$V_{0.8d} = a N_{(0.8d)} + b \rightarrow$ (0.2d from bottom)

$$\therefore \boxed{\bar{V} = V_{AV} = \frac{V_{0.2d} + V_{0.8d}}{2}}$$ *
*

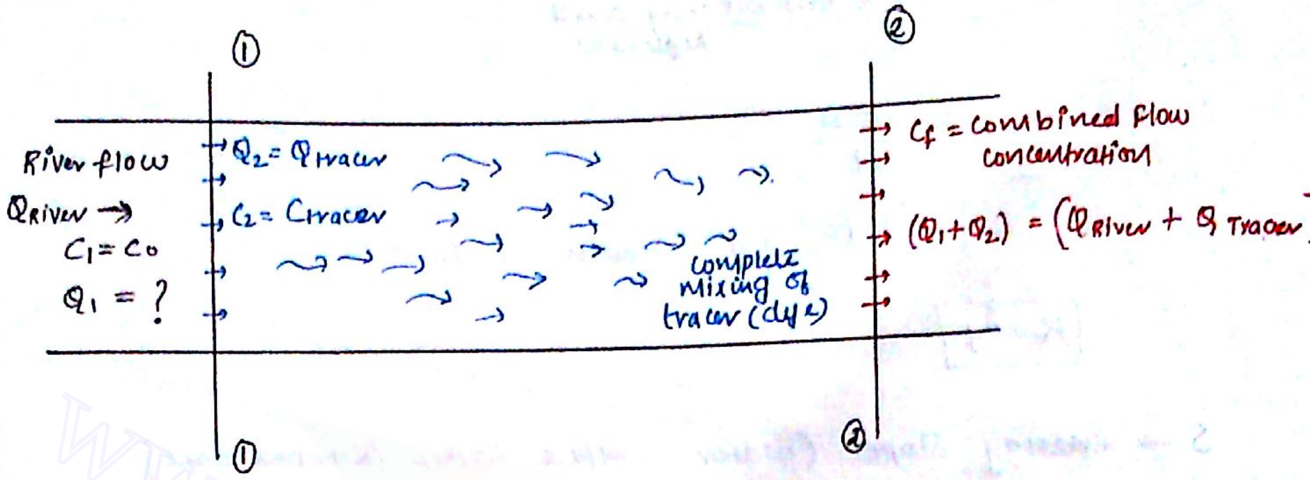
(c) <u>Three point method</u> -

$$V_{AV} = \frac{V_{0.2d} + V_{0.6d} + V_{0.8d}}{3}$$



① This method is mainly applicable for small turbulent stream (or) river in mountaineous region (zone).

② In this method the Law of continuity is adopted to find rate of flow directly.



$$Q_1C_1 + Q_2C_2 = (Q_1+Q_2)\,C_{final}$$

$$Q_RC_R + Q_tC_t = (Q_1+Q_2)\,C_{final}$$

$$\Rightarrow \boxed{Q_R = ?}$$

** Slope - Area Method -

(i) This method is mainly used for finding flood discharge of river having high water mark.

(ii) In this method, Mr. Manning formula is adopted,

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

N → Manning's constant (Rugosity co-efficient) (Roughness co-efficient) having value 0·02 to 0·03 for earthen con bed, 0·013 → for lined bed.

R → Hydraulic mean depth (HMD)

$$= \frac{\text{Area (wetted)}}{\text{perimeter (wetted)}}$$

For very wide river,

HMD is taken as actual depth of flow,

$$\boxed{R = y}$$

$$R = \frac{A}{P} = \frac{BD}{B + \underbrace{D + D}_{\text{very small (neglected)}}}$$

$$= \frac{BD}{B}$$

$= D = y =$ depth of flow.

$$\therefore \boxed{R = y}$$

$S \rightarrow$ energy slope (water surface slope) (~~Not bed slope~~)

$$Q = A \times V$$

$$= A \times \frac{1}{N} \times R^{2/3} \times S^{1/2}$$

$$= A \times \frac{1}{N} \times R^{2/3} \times \sqrt{S}$$

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

$K \rightarrow$ conveyance $= A \times \frac{1}{N} \times R^{2/3}$

(m3/sec)

$$\boxed{Q \propto \sqrt{S}}$$

$$** \boxed{\frac{Q_1}{Q_2} = \frac{\sqrt{S_1}}{\sqrt{S_2}}}$$

For very wide river, $\boxed{R = y}$ $Q = A\,V = B \times y \times \frac{1}{N} \times \overset{y}{R}^{2/3} \sqrt{S}$

$= B\,y^{5/3} \times \frac{1}{N} \sqrt{S}$

*** change in discharge $\rightarrow \frac{Q_2 - Q_1}{Q_1} \times 100$ *

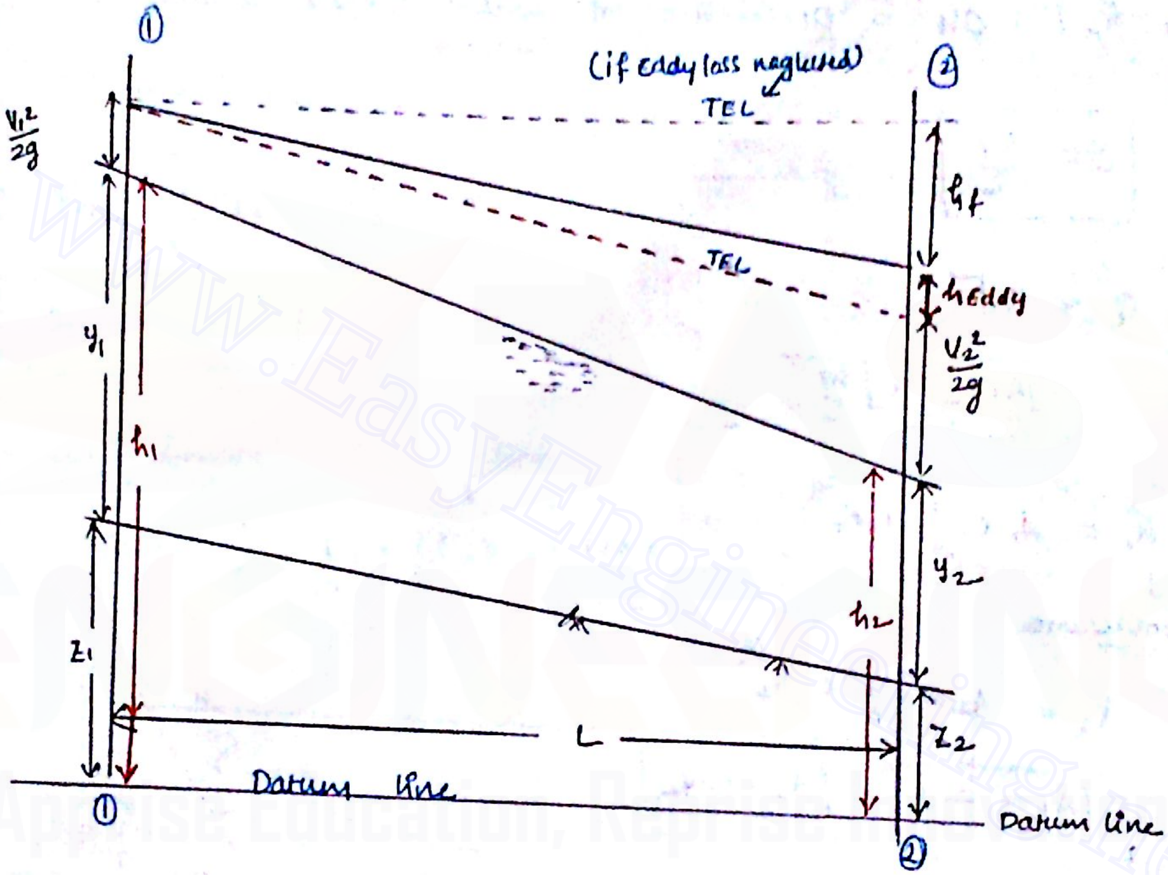
$$** \quad \frac{dq}{q} \times 100 = \frac{\frac{1}{N} B (y_2)^{5/3} \sqrt{S_2} - \frac{1}{N} B (y_1)^{5/3} \sqrt{S_1}}{\quad} \times 100$$

$$\boxed{\frac{Q_2-Q_1}{Q_1}\times 100 = \frac{(y_2)^{5/3}\sqrt{S_2} - (y_1)^{5/3}\sqrt{S_1}}{(y_1)^{5/3}\sqrt{S_1}}\times 100}$$

Application of Bernoulli's theorem to determine energy slope

⇓ (Friction loss per unit length)

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



Applying Bernoulli's equation b/w ①-① and ②-②

$$E_1 = E_2 + \text{losses}.$$

$$(z_1 + y_1) + \frac{V_1^2}{2g} = (z_2 + y_2) + \frac{V_2^2}{2g} + h_{eddy} + h_f$$

$$h_1 + \frac{V_1^2}{2g} = h_2 + \frac{V_2^2}{2g} + h_{eddy} + h_f$$

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

$h_{eddy} = K \left| \frac{}{2g} \quad \frac{}{2g} \right|$

$$V_1 = \frac{Q}{A_1} = \frac{Q}{B_1 y_1}$$

$$V_2 = \frac{Q}{A_2} = \frac{Q}{B_2 y_2}$$

$(h_1 - h_2) = \Delta h$ = Difference in water surface elevation

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

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

$$= \sqrt{K_1 K_2} \times \sqrt{\frac{h_f}{L}}$$

$$K_1 = A_1 \times \frac{1}{N} \times R_1^{2/3}$$

$\downarrow$

Conveyance

$$K_2 = A_2 \times \frac{1}{N} \times R_2^{2/3}$$

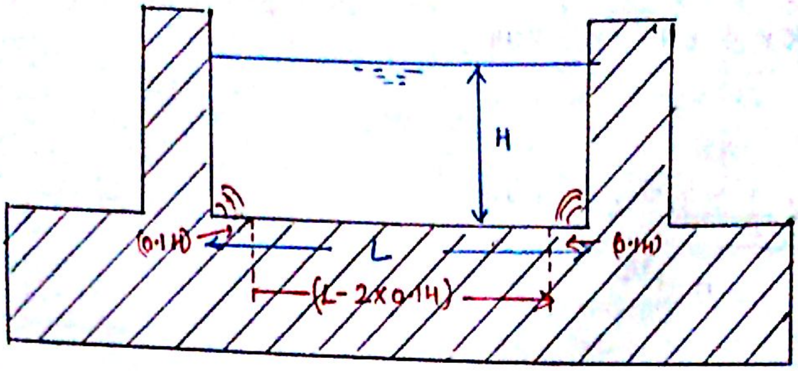
$\boxed{A_1 = B_1 y_1}$ $\boxed{A_2 = B_2 y_2}$

$$R_1 = \frac{A_1}{P_1} = \frac{B y_1}{B + 2y_1}$$

$$R_2 = \frac{A_2}{P_2} = \frac{B y_2}{B + 2y_2}$$

Percentage error in discharge due to error in water head measurement over a weir -

(i) Rectangular weir

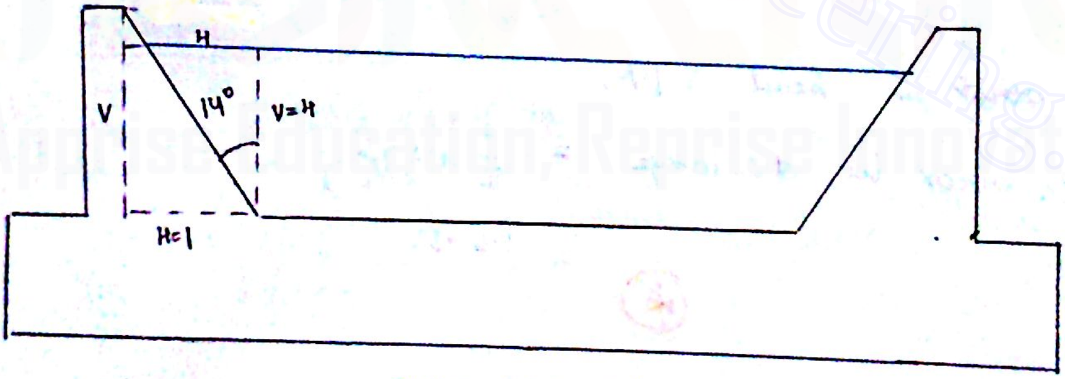


$$Q_{Rect} = \frac{2}{3} C_d \times L \times \sqrt{2g} \times H^{3/2}$$

$$\boxed{Q_{Rect} \propto H^{3/2}}$$

Francis formula,

$$Q_{Rect} = \frac{2}{3} C_d \times (\overset{\text{left}}{\downarrow} L - 0.1H \times 2) \sqrt{2g} \times H^{3/2}$$



CIPPOLLETI WEIR

$$\tan 14° = H/V$$

$$0.25 = H/V$$

$$\boxed{\frac{1}{4} = H/V}$$

$$Q \propto H^{3/2}$$

$$Q = K H^{3/2} \qquad K = \frac{2}{3} C_d \times L \sqrt{2g}$$

differentiate w.r.t H

$$\frac{dQ}{dH} = K \times \frac{3}{2} H^{3/2 - 1}$$

$$dH \qquad \qquad 2$$

$$dq = k \times \frac{3}{2} \times H^{3/2} \times H^{-1} \times dH$$

$$\frac{dq}{q} = \frac{k \times \frac{3}{2} \times H^{3/2} \times H^{-1} \times dH}{Q}$$

$$\frac{dQ}{Q} = \frac{k \times \frac{3}{2} \times H^{3/2} \times H^{-1} \times dH}{k\,H^{3/2}}$$

$$\frac{dQ}{Q} = \frac{3}{2} \times \frac{dH}{H}$$

$$\frac{dq}{q} \times 100 = \frac{3}{2} \times \frac{dH}{H} \times 100$$

$$\Downarrow \qquad \qquad \Downarrow$$

$$** \boxed{P_{discharge} = \frac{3}{2} \times P_{head}}$$

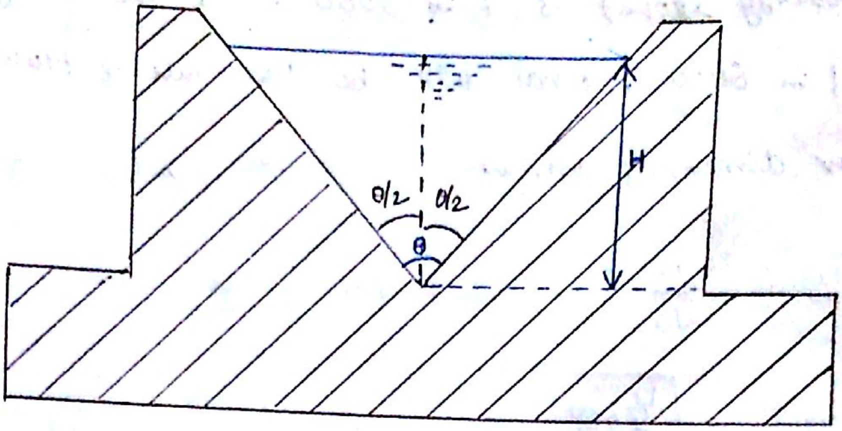
% error in discharge = $\frac{3}{2}$ x % error in head measurement

If % error in head = 1%,

then % error in discharge = $\frac{3}{2} \times 1\%$

= 1.5% ⊛

ii) Triangular weir (Notch)



$$Q = \frac{8}{15} \times C_d \times \sqrt{2g} \times \tan\frac{\theta}{2} \times H^{5/2}$$

$$\boxed{Q \propto H^{5/2}}$$

$$\boxed{Q = K H^{5/2}}$$

$$\frac{dQ}{dH} = K \times \frac{5}{2} \times H^{5/2 - 1}$$

$$dq = K \times \frac{5}{2} \times H^{5/2} \times H^{-1} \times dH$$

$$\frac{dq}{Q} = \frac{K \times \frac{5}{2} \times H^{5/2} \times H^{-1} dH}{K H^{5/2}}$$

$$\frac{dQ}{q} = \frac{5}{2}\left(\frac{dH}{H}\right)$$

$$\frac{dq}{Q} \times 100 = \frac{5}{2} \times \frac{dH}{H} \times 100$$

$$\boxed{\text{\% error in discharge} = \frac{5}{2} \times \text{\% error in head}}$$

→ If % error in head = 1%, then

% error in discharge = $\frac{5}{2} \times 1\%$

(✱) = 2.5%

NUMERICALS -

① The rate of flow in a river is 100 cumec if the bed slope (energy slope) is 1 in 3000. If the energy slope becomes 1 in 6000, what will be the rate of flow, keeping other dimension constant.

Soln:

$$\frac{Q_1}{Q_2} = \frac{\sqrt{S_1}}{\sqrt{S_2}}$$

$$\frac{100}{Q_2} = \frac{\sqrt{1/3000}}{\sqrt{1/6000}}$$

$$\boxed{Q_2 = 70{\cdot}71 \text{ cumec}}$$

* ② If the depth of flow of a river is increased by 40% and the energy slope is decreased by 20%, what will be the % change in discharge?
(imp)

Assume very wide channel.

Soln:

$$\frac{Q_2 - Q_1}{Q_1} = \frac{(y_2)^{5/3}\sqrt{S_2} - (y_1)^{5/3}\sqrt{S_1}}{(y_1)^{5/3} \times \sqrt{S_1}}$$

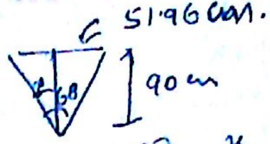
$$= \frac{(1{\cdot}4\, y_2)^{5/3}\sqrt{0{\cdot}8 S_1} - (y_1)^{5/3}\sqrt{S_1}}{(y_1)^{5/3}\sqrt{S_1}}$$

$$\frac{Q_2 - Q_1}{Q_1} = (+)\, 0{\cdot}4059$$

$= (+)\, 46{\cdot}591\cdot$ (Increase in discharge)

A triangular channel has depth of flow equal to 90 cm and the apex angle is 60°. The velocity of flow at 18 cm from top and 72 cm from top are 0.6 m/sec and 0.4 m/sec. Determine the rate of flow.

Soln : $d = 72 + 18 = 90\text{ cm}$



$\tan(\theta/2) = \frac{x}{90}$

30

$\bar{v} = \frac{0.6 + 0.4}{2} = 0.5\text{ m/sec}$

$q = A \times \bar{v} = \frac{1}{2} \times 0.9 \times 0.5196 \times 0.5$

×2

$= 0.117\text{ cumec}$

×2

$Q = 0.234\text{ cumec}$

④ The no. of revolution of cup in 50 seconds at a place ~~where~~ was 25 and 35. The velocity of flow were 0.3 m/sec and 0.45 m/sec. If the no. of revolutions is 50 in 60 seconds. V = ?

$N_1 = \frac{25}{50} = 0.5$ $0.3 = a \times 0.5 + b$

$N_2 = \frac{35}{50} = 0.7$ $0.45 = a \times 0.7 + b$

$a = 0.75$ $b = -0.075$

$v_3 = 0.75 \times \frac{50}{60} + (-0.075)$

$v_3 = 0.55\text{ m/sec}$

⑤ Determine rate of flow in a small turbulent river with the help of foll. data –

initial conc. of river = 0

conc. of tracer = 25 gm/lit

discharge of ~~press~~ tracer = 50 cc/sec

final conc. = 10 p p billion

$$\frac{10}{10^9} = \frac{10\times10^5\ m}{}$$

$$\frac{10\,mg}{10^9}$$

$$\frac{10\,mg}{10^9}$$

$$Q_R C_R + Q_T C_T = (q_1+q_2)\,c_f$$

$$0 + \frac{50\times25}{\times10^6\ 000} = (Q_1+Q_2)\times\frac{10}{1000}\ mg/lit$$

$$\underline{\underline{Q = 125\ \text{cumec}}}$$

*****

HYDROGRAPH (RUNOFF)

① Hydrograph is a plot (graph) in b/w the discharge of a river (Y-axis) and the time of flow of the river in chronological order (increasing) (x-axis). A hydrograph is also known as flood hydrograph (or) storm hydrograph which consists of direct runoff and base flow (combinely)

② Type of Hydrographs -

Name of Hydrograph	Inventor name	Description
(i) Hydrograph (Flood hydrograph) (Storm hydrograph)	—	A plot (graph) between discharge of river and time of flow of river (Direct runoff + Base flow) (DRO + Baseflow)
(ii) Direct runoff Hydrograph (DRH) (Flood hydraph − base flow) (Starts from origin terminates on x-axis)	—	A plot b/w the discharge of a river without Base flow and time of flow of river (it consists of only DRO, having depth of flow ~~more~~ other than 1cm) (R) (Not 1cm) (other than 1 cm)
(iii) Unit hydrograph (UH)	Mr. Sherman	A plot b/w discharge of a river without base flow having * 1cm depth of DRO and time of flow of river. $\boxed{R = 1cm}$ (unit depth) (No Base flow) (Graph always starts from origin & terminates on x-axis)

(iv) Summation hydrograph (S-curve)	—	A· Plot b/w the discharge of a river due to infinite duration of rainfall, having DRO = 1cm (R=1cm) and the time of flow of the river. It consists of cumulative discharge of unit hydrograph. It has an equilibrium discharge which is constant after certain interval of flow of river.
(v) Distribution hydrograph	Mr. Bernard	A plot b/w % of total runoff (Y-axis) and the time of flow (X-axis)
(vi) Synthetic unit hydrograph	Mr. Snyder	A plot b/w discharge based on unit depth of runoff on a catchment and the time of flow. It is totally ~~bed~~ based on empirical formulae for particular region of catchment developed by Mr. Snyder. It is a type of unit hydrograph based on empirical formulae.
(vii) Instantaneous unit hydrograph (IUH)	Mr. Nash	A plot b/w the discharge due to unit depth of DRO where the duration of rainfall is very very small (infinite decimal small) and the flow of the river

③ The direct runoff (DRO) includes -

(i) Surface runoff

(ii) prompt interflow (without delay) (verysoon)

(iii) Precipitation on the channel surface

It's symbol is R,

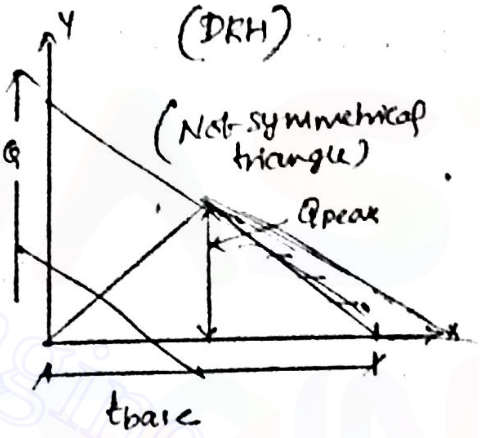
$$\phi\text{-index} = \frac{P-R}{t}$$

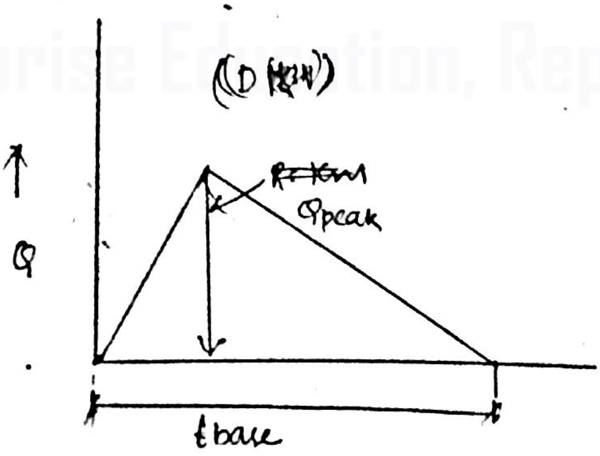
$$\Rightarrow \boxed{R = P - \phi t}$$

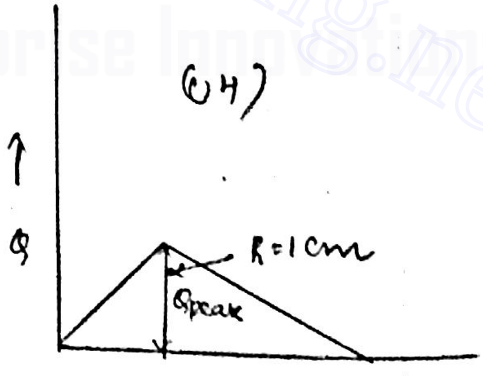
$$R = \frac{\text{Volume of DRO}}{\text{Catchment area}}$$

$$= \frac{\text{Area of DRH}}{\text{Catchment area}}$$

$$= \frac{\frac{1}{2} \times t_{base} \times Q_{peak}}{\text{Area of Catchment}}$$







(DRH)

(UH)

$$R:1 = \frac{\frac{1}{2} \times t_{base} \times Q_{peak}}{\text{Catchment area}}$$

④ The base flow of a flood hydrograph is essentially delayed ground water flow.

* ⑤ The base flow is subtracted from the flood hydrograph to determine effective rainfall (either volume of eff. rainfall (or) depth of effective rainfall)

The effective rainfall (ER) is also called Rainfall excess (or) DRO

$$\left[R = DRO = \overset{\text{Effective}}{E}R = R\overset{\text{Excess}}{E} \right]$$

⑥ In India the water year begins when the rainfall exceeds evapotranspiration.

In India the beginning of water year occurs at 1st June and it ends on 31st May of the following calendar year.

⑦ Types of river

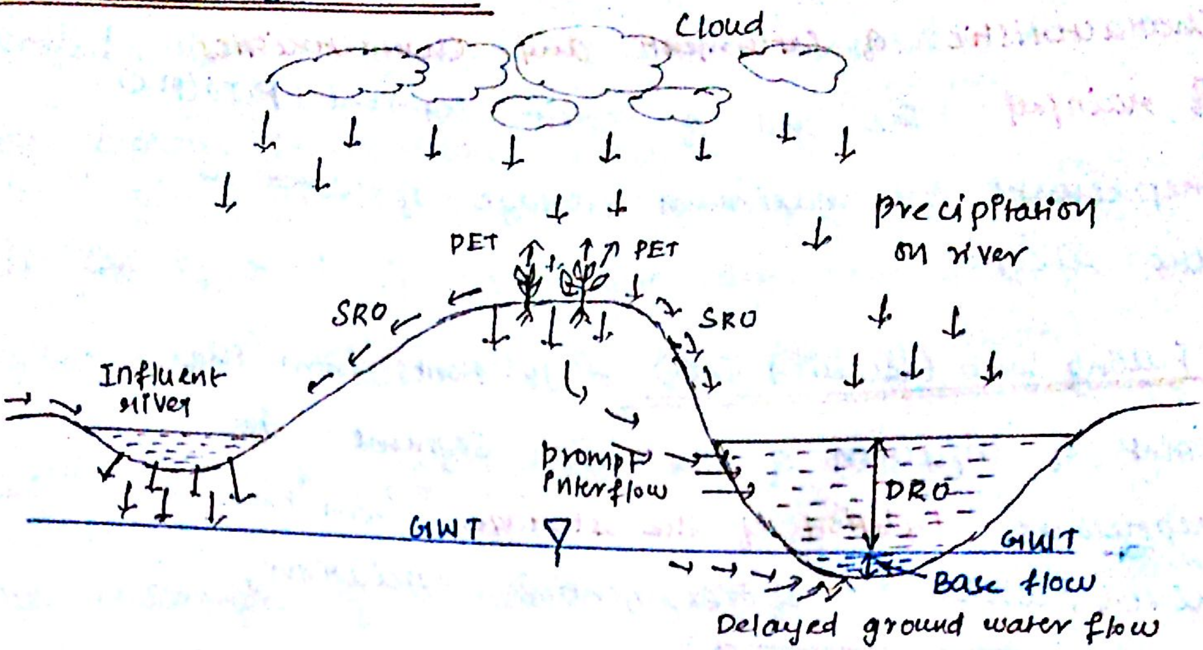
(i) Perennial river - A river which always gets water on the ground water source and the surface runoff, i.e the water table is always above the bed of the river is known as perennial river where the Flood hydrograph (Hydrograph) also always ~~stra~~ starts from Y axis (ordinate) and it does not touches X-axis

(ii) Intermittent river - A river gets water from ground water source and from the Surface during the wet climate, ie the ground water table is above the bed of the river only during the wet season and it is below the bed of river during dry season, ie it has a limited base flow and hence the hydrograph may touch the x-axis during dry season (Cauvery)

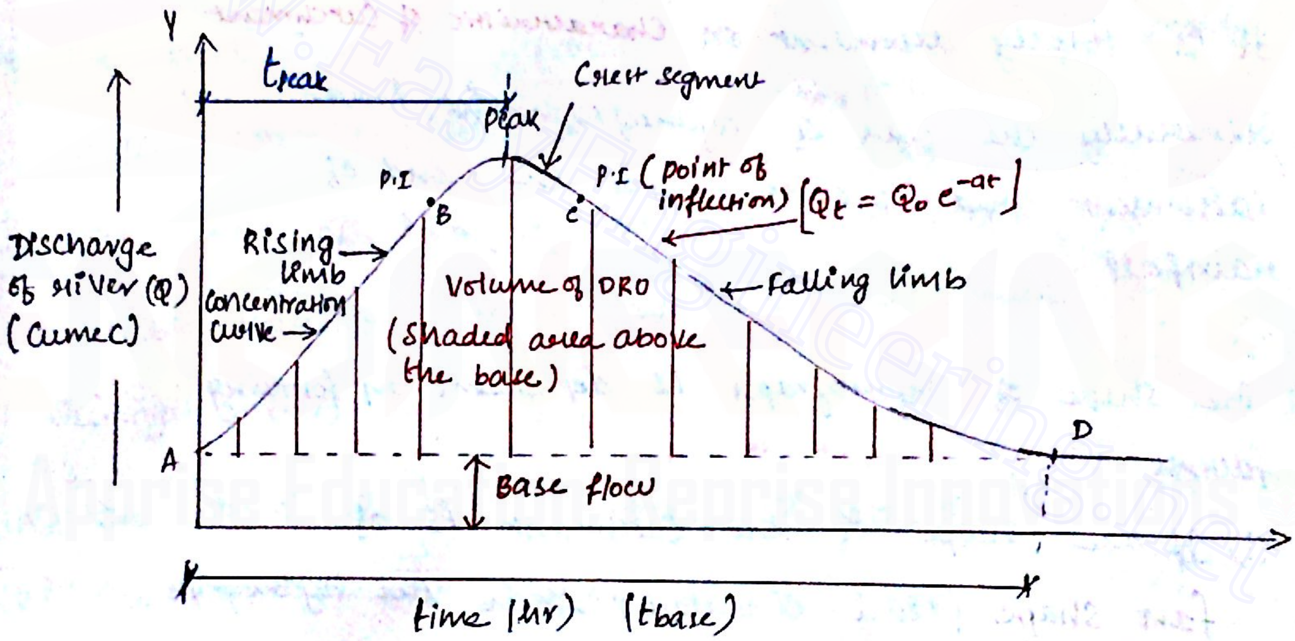
(iii) Ephemeral river - A river which does not have base flow contribution (There is no base flow at all), the ground water table is always below the bed of the river and hence the hydrograph starts from the x-axis and terminates at the ~~a~~ x-axis.

(iv) Influent river - A river which supplies water to ground water table (~~to~~ recharge the ground water), ie water table is below the bed of river is known as influent river (Type of ephemeral) (No base flow)

(v) Effluent river - A river which gets water from the ground water table, ie the ground water table is above the bed of river to get base flow is called an effluent river.



[DRO = SRO + prompt interflow + ppt on river surfac



(i) Rising limb (concentration curve) -

It is the initial part of hydrograph which represents the building of storage of water and it is dependent on -

(a) Characteristic of catchment (land) (basin) (watershed) (drainage)

(b) Characteristic of rainfall (pptn.)

(c) Climatic factor

(ii) Crest Segment - It is dependent on characteristic of catchment and characteristics of rainfall. The end of crest segment (P.I)(Pt.c) represents the maximum storage of water in the river.

(iii) Falling limb (Recession limb) - It starts from the point of inflection of the crest segment. It represents depletion of the storage from the river, which is of exponential variation,

$$\boxed{Q_t = Q_0 e^{-at}}$$

It is independent of rainfall characteristic. It is totally dependent on characteristic of catchment.

* Generally the peak of hydrograph for large catchment area occurs after the end of rainfall.

⑨ The shape of hydrograph is dependent on following factors -

(i) If the shape of the catchment is of fan shape (semi-circular) then the hydrograph has large peak and narrow shape

(ii) If the shape of the catchment is of elongated shape (Fern leaf) then the hydrograph has low peak and broad shape

(iii) If the catchment has steep slope, then the hydrograph has sharp rise, high peak and narrow shape having lower time base

(iv) If the soil conservation ~~method~~ is measures are adopted, then infiltration is higher resulting low peak and broad hydrograph

(v) If the land is of urban zone then the peak of hydrograph is larger and it has narrow shape

⑩ Steps to analyse hydrograph -

(i) The discharge of a river plotted on a graph, where the discharge is kept on Y-axis and the time of flow of river on x-axis.

(ii) The base flow is separated out from the hydrograph to get direct runoff hydrograph (DRH)

(iii) The area of DRH is determined with the help of suitable relation (simpson's rule, Trapezoidal rule or triangle rule)

(iv) The area of DRH is called Volume of DRO

(v) The depth of DRO (ER = RE = R) is determined, i.e

$$R = \frac{\text{Volume of DRO}}{\text{Area of Catchment}} = \frac{\text{Area of DRH}}{\text{Area of Catchment}}$$

(vi) The ordinate of DRH is divided by 'R' to get UHO (unit hydrograph ordinate)

⑪ Unit Hydrograph- (UH) (Mr. sherman)

(i) It is a type of DRH having depth of direct runoff always unit (R=1cm)

(ii) The value of UHO of a particular river for a given duration of rainfall ~~will~~ shall always be the same irrespective of when it occurs which is known as time invariance

(iii) The total discharge in a river due to successive rainfall can be obtained by linear interpolation, ie if input is $x_1(t)$ which produces output $y_1(t)$, if another input $x_2(t)$ produces $y_2(t)$ then the total input ~~is~~ $x_1(t) + x_2(t)$ will produce total output is $y_1(t) + y_2(t)$, It is known as linear response.

(iv) There are 2 basic assumptions of unit hydrograph theory -

(2014)

(a) Time invariance

(b) Linear response (The most important assumption)

**

(v) If the rainfall is 6 hour and DRO = 1cm then the UH is called 6 hour UH having time base 60 hour (say). If the rainfall becomes 12 hour on the same catchment having DRO 1cm then the time base of 12 hour UH will become 66 hour UH (60+6)

But the peak value of 12 hour UH decreases

But if the duration of rainfall decreases then the peak of UH increases.

* (vi) The time base of UH increases (dependent on) with increase of rainfall duration only, i.e the time base of UH is independent of Depth of DRO, but the peak of DRH is dependent on depth of DRO, ie the ordinate of DRH is equal to {ordinate of UH × R},

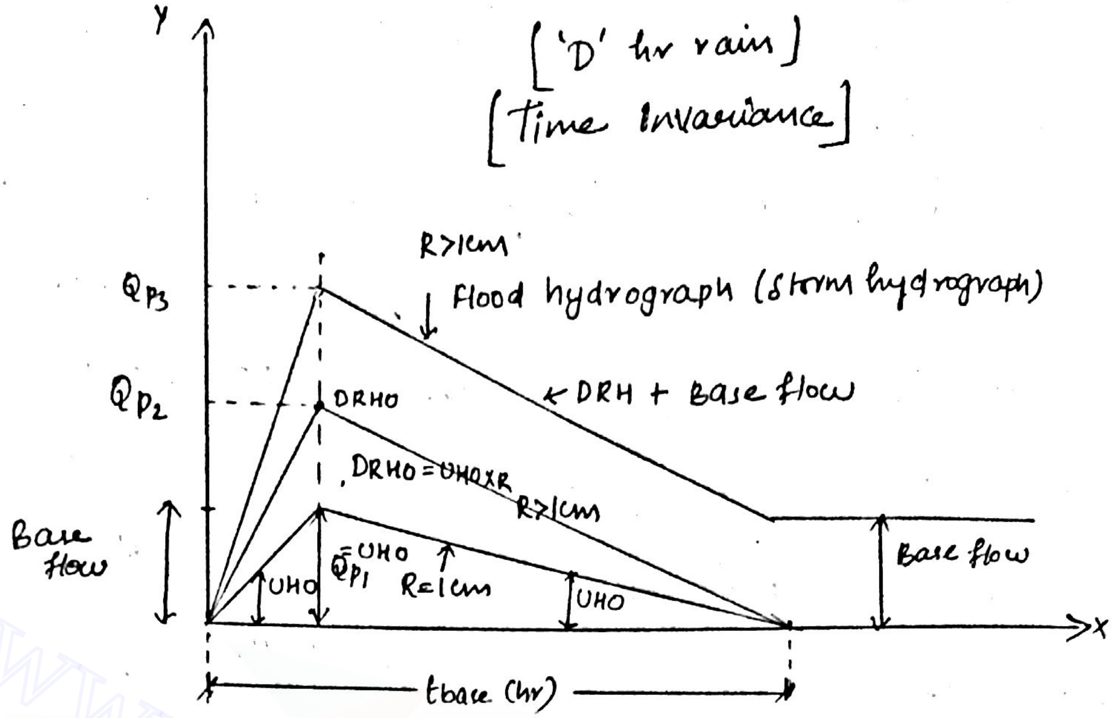
ie. DRHO = UHO × R (or) UHO = $\frac{DRHO}{R}$

If the effective area of Hyetograph is multiplied with area of catchment then it is equal to Area of DRH (Volume of DRO)

** ⑫ Drainage density - It is defined as the ratio of length of drainage (channel) (river) to the area of the catchment. If the drainage density increases then there will be quick disposal of runoff and hence the hydrograph shall have larger peak and narrow shape. on the other hand for low drainage density, the peak of hydrograph will be low and the shape of hydrograph will be ~~ta~~ broad (large time base)

* Unit Hydrograph – Theory applicable for a catchment area less than (or) equal to (Not more than) 5000 sq.km.

⑬ Co-relation among UH, DRH and SH



$$\boxed{Q_{P_2} = Q_{P_1} \times R}$$

$$\boxed{Q_{P_2} = UHO \times R}$$

~~Q_{P_3} = DR~~

$Q_{P_3} = Q_{P_2} +$ Base flow (only for time invariance)

$Q_{P_3} = (Q_{P_1} \times R) +$ Base flow

Flood hydrograph ordinate = UHO × R + Base flow

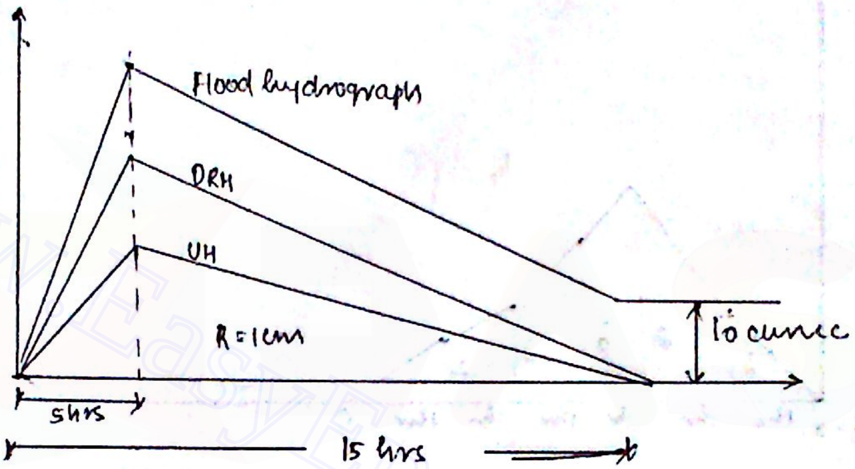
where $\boxed{R = P - \phi \times t}$

Numericals -

① The drainage area of a water shed is 50 sq. Kms. The ϕ-index is 0.5 cm/hr and the base flow is 10 cumec. one hour UH is triangular in shape, time base of 15 hours, The peak ordinate occurs at 5 hrs

(i) Determine the peak ordinate of UH

(ii) Determine the peak ordinate of hydrograph due to one hour duration rain having depth of rain 5.5 cms.



Every UH = DRH

Every DRH ≠ UH

Soln:

(i) $R = \dfrac{\text{Area of DRH}}{\text{Area of Catchment}}$

$$\frac{1}{100} = \frac{\text{Area of UH}}{\text{Catchment area}} = \frac{\frac{1}{2} \times (15 \times 60 \times 60) \times Q_{peak}}{50 \times 10^6 \text{ m}^2}$$

$Q_{peak} = 18.52$ ~~m~~ cumec

(ii) $P = 5.5$ cm

ϕ index $= 0.5$ cm/hr

ϕindex $= 0.5 = \dfrac{5.5 - R}{1}$

$R = P - \phi t$

$= 5.5 - 0.5 \times 1$

$R = 5$ cm =

$$\frac{5}{100} = \frac{\frac{1}{2} \times 15 \times 60 \times 60 \times Q_{peak}}{50 \times 10^6}$$

$Q_{peak} = 92.6$ cumec

$Q_{peak} = Q_{DRH}$ + Base flow

$= Q_{UHo} \times R$ + Base flow

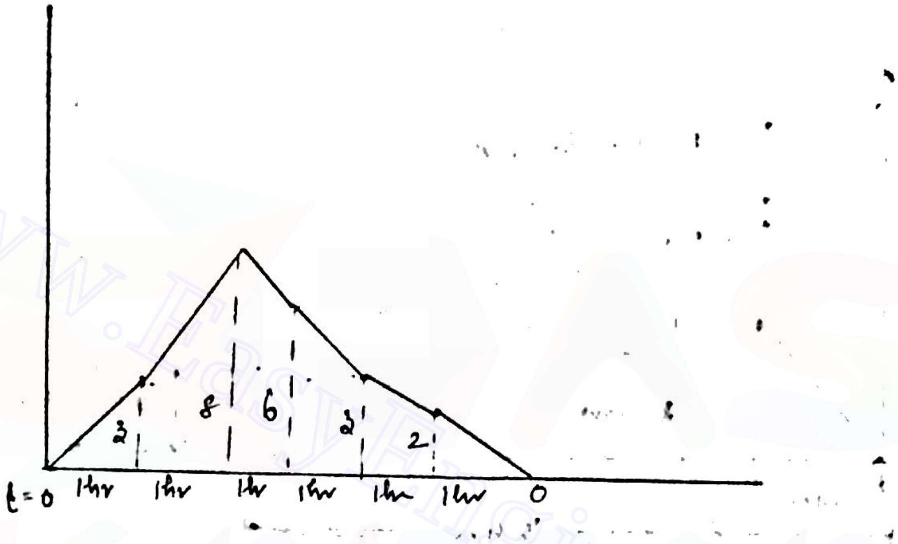
$= 18.5 \times 5 + 10$

$= 102.5$ cumec.

② The ordinates of a 2 hour UH at one hour interval are, 0, 3, 8, 6, 3, 2 and 0 cumec. Use trapezoidal rule

(i) Determine catchment area

(ii) peak flow of the storm hydrograph due to 6.6 cm depth of rain occurring ~~in~~ over the catchment in 3 hours and the ϕ-index is equal to 2 mm/hr and base flow is 5 cumec.



(i) $R = \dfrac{\text{Area of UH}}{\text{Area of Catchment}}$

$$\frac{1\text{ cm}}{100\text{ (m)}} = \frac{\left(\frac{1}{2}\times 1\times 3\times 60^2\right) + \left(\frac{3+8}{2}\times 60^2\right) + \left(\frac{8+6}{2}\times 60^2\right) + \left(\frac{6+3}{2}\times 60^2\right) + \left(\frac{3+2}{2}\times 60^2\right) + \frac{1}{2}\times 1\times 2\times 60^2}{A_{\text{catchment}}}$$

$$A_{\text{catchment}} = 7.92\times 10^6\ \text{m}^2$$

$$= 7.92\ \text{km}^2$$

(ii) $\overset{3\text{hr}}{Q_{\text{peak}}} = \overset{\text{Must be 3 hr.}}{Q_{\text{peak (UH)}}} \times R + \text{Base flow}$

$R = P - \phi t = 6.6 - 0.2\times 3$

$= 6$ cm

Base flow = 5 cumec

t	2 hour (Q)	S-curve ΣQ	lagged by 3 hr	S-curve ordinate	Scurve × 3hr decrease in Q	
0 →	0	0 →	Nil	0	$0 \times \frac{2}{3} = 0$	
1 →	3	0+3=3 →	Nil	3	$3 \times \frac{2}{3} = 2$	
2 →	8	3+8 = 11 →	Nil	11	$11 \times \frac{2}{3} = 7.33$	
③ →	6	11+6=17 →	0	17	$17 \times \frac{2}{3} = \boxed{11.33}$	peak value
4 →	3	17+3=20 →	3	17	$17 \times \frac{2}{3} = \boxed{11.33}$	
5 →	2	20+2 = 22	11	11		
6 →	0	22+0 =22	17	5		
		22	20	2		
		22	22	0		
		22	22	0		

$Q_{peak} = 11.33 \times 6 + 5$

$= 72.98$ cumec

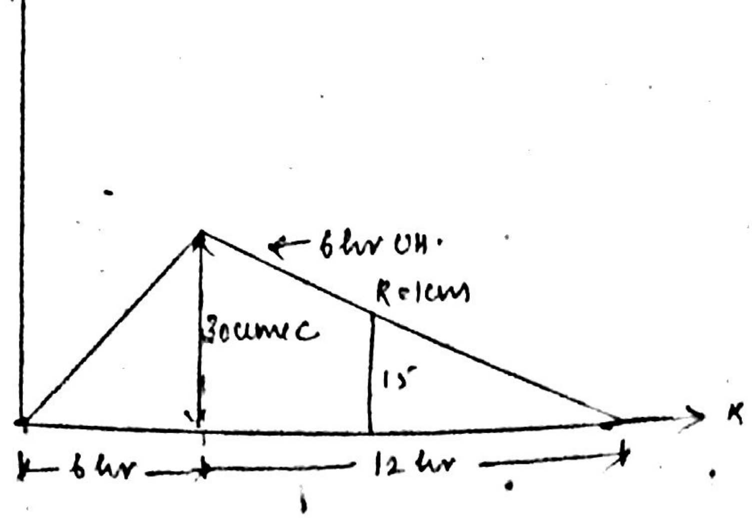
≈ 73 cumec

③ An average rainfall of ~~3cm~~ 16cm occurs over a catchment during a period of 12 hr with a uniform intensity. The 6 hour UH rises linearly from 0 to 30 cum in 6hr and then falls linearly from 30 to 0 cumec in next 12 hour. φ-index is 0.5 cm/hr. The base flow is 5 cum

Determine -

(i) Area of Catchment

(ii) peak discharge of DRH of 12 hr



(i) $R = \dfrac{\text{Area of UH}}{\text{Catchment Area}}$

$$A = \frac{\frac{1}{2} \times 18 \times 30 \times 60^2}{1/100}$$

$$= 97.2 \times 10^6 m^2$$

$$\boxed{A = 97.2 \ km^2}$$

(ii) $\underset{(12 hr)}{\overset{DRH}{Q_{peak}}} = \overset{12 hr}{Q_{UHO}} \times R$

= ~~28 x~~

Coz of (1 cm + 1 cm) = 2 cm cum

t	6 hr UHO	6 hr UHO lagged by 6 hr	Adding 12 hr DRH (2cm R)	12 hr UH ← (Always 1 cm)
0	0	Nil	0	0/2 = 0
6	30	0	30	30/2 = 15
12	15	30	45	45/2 = $\boxed{22.5}$
18	0	15	15	15/2 = 7.5
24	←	0	0	0/2 = 0

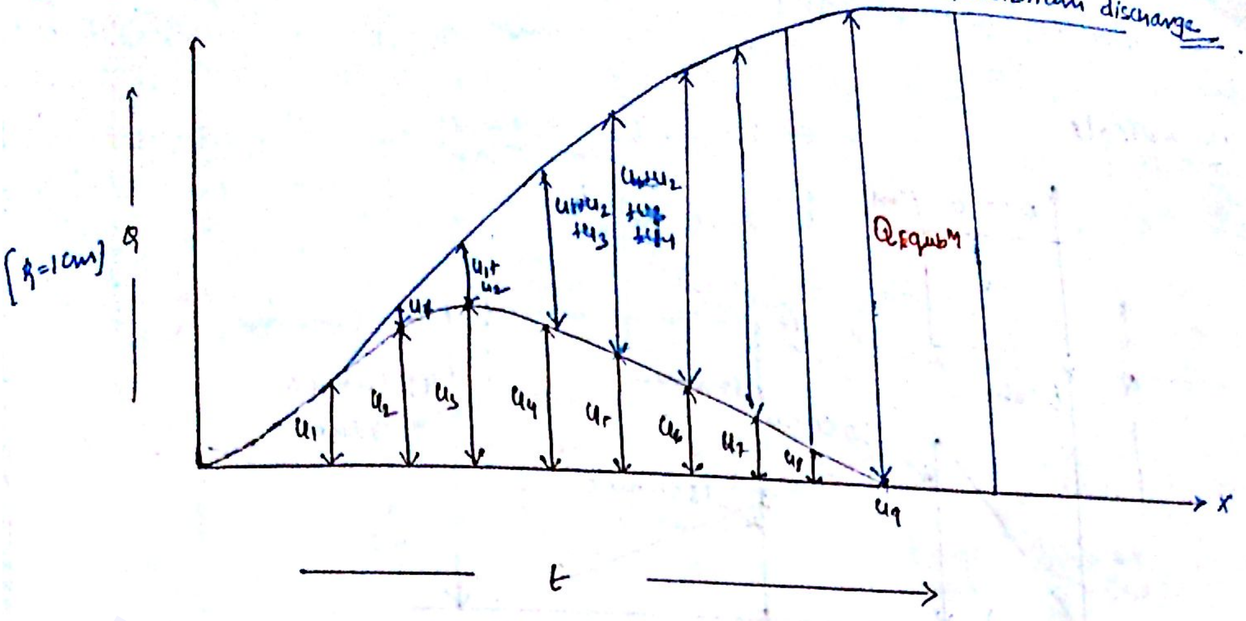
$$\underset{\substack{(DRH)\\(12 hr)}}{Q_{peak}} = 22.5 \times \cancel{R2} \ (P - \phi t)$$

$$= 22.5 \times (16 - 0.5 \times 12)$$

$$= 225 \text{ cumec} //$$

S-Curve :

…duration, discharge becomes constant ⇒ Equilibrium discharge



The s-curve (summation curve) is a hydrograph obtained by summation of ordinates of unit hydrograph (R=1cm) of a particular duration of rain as shown in above fig.

The duration of rain should be of infinity extent.

→ The equilibrium discharge is the maximum of total discharge which will be ~~the~~ constant even though the duration of rain increases, ie the equilibrium discharge occurs if the duration of rain is of infinity extent. The expression for equilibrium discharge is given by fundamental relation,

$$Q = \frac{\text{Volume of Runoff}}{\text{time}} = \frac{A \times R}{t} = \frac{A \times 1cm}{t}$$

$$Q = \frac{\overset{(km^2)}{A_{catchment}} \times 1cm}{t_{rain}(hr)} = \frac{A \times 10^6 \times \frac{1}{100}}{t \times 60 \times 60} = 2.778 \frac{A}{t}$$

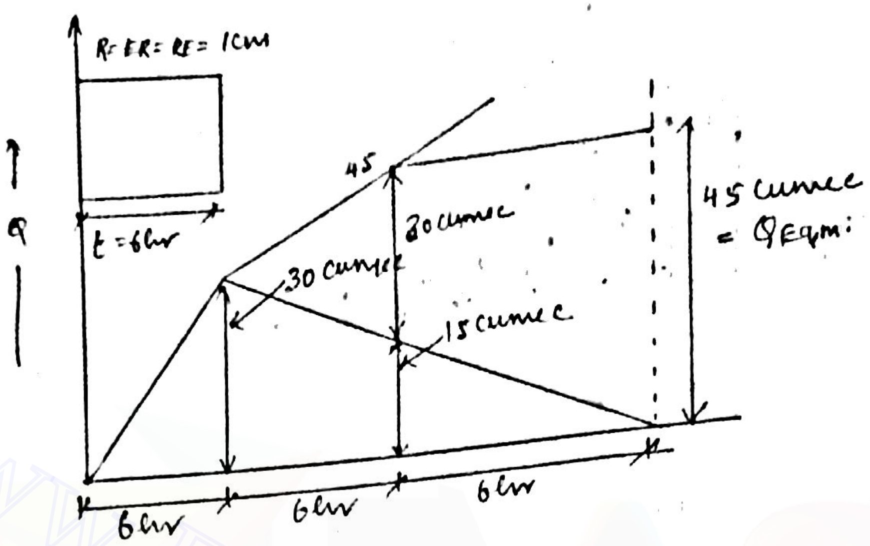
$$\boxed{Q = \frac{2.778A}{t}} \leftarrow km^2$$

A = Catchment area (km²) = Watershed area
= Drainage area

t = duration of rain (hr)

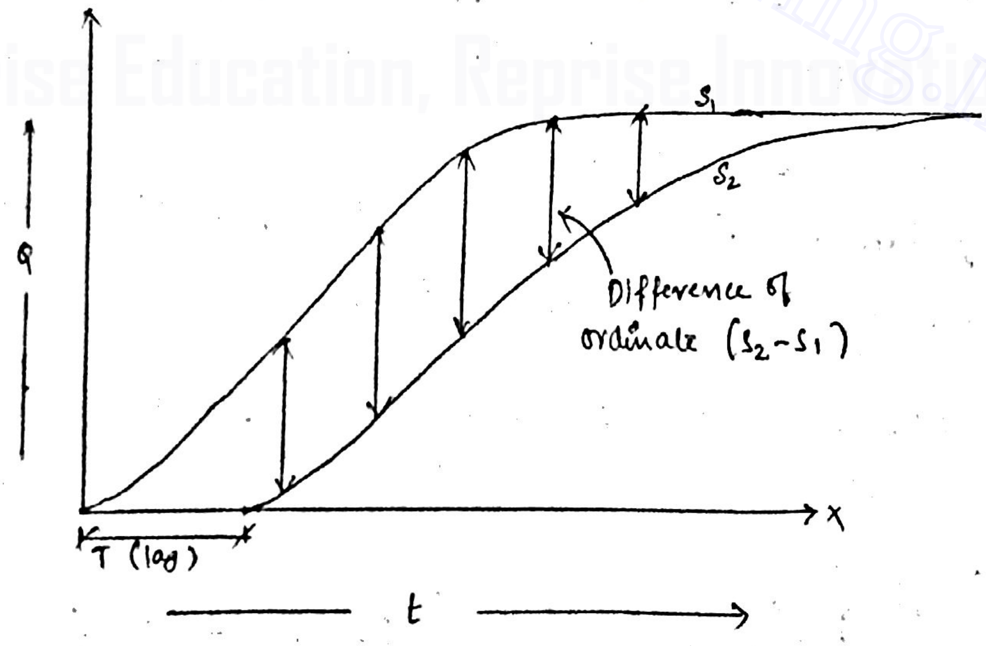
Numericals

Q-②



$$45 = \frac{2.778 \times A}{6\,hr} \leftarrow km^2$$

$$\Rightarrow \boxed{A_{catch} = 97.2\ km^2}$$

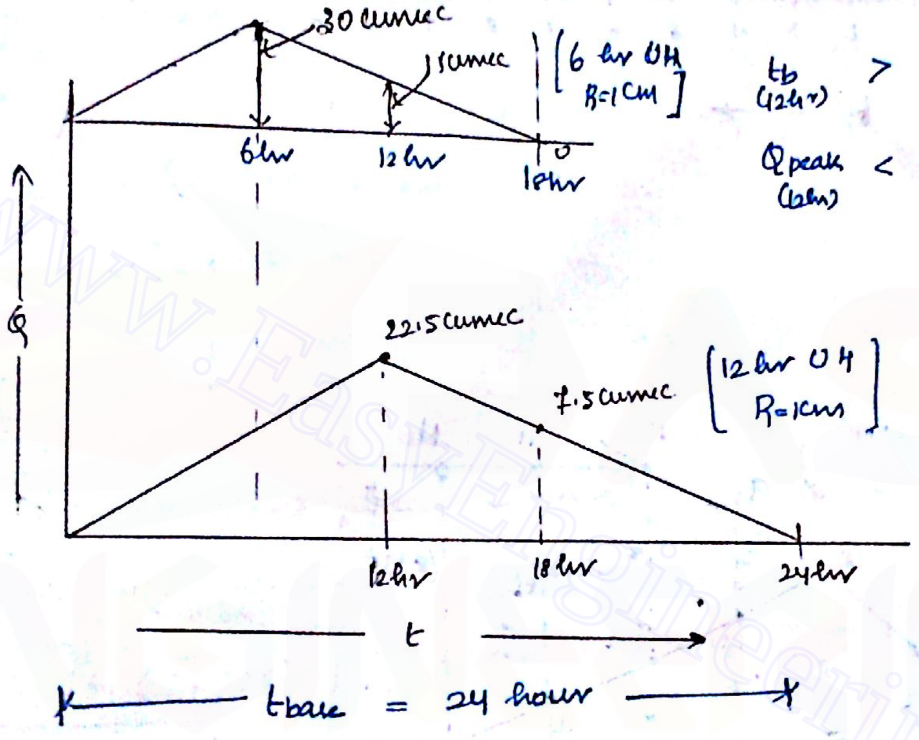


& $Q_T = (S_1 - S_2) \times \frac{D}{T}$ ← Given duration
← Req. duration

$$\boxed{Q_T = \frac{(S_1 - S_2)}{T/}}$$

(6hr) (12hr UHO)

Time	6 hr UHO	S-curve ordinate	Required time, T=12 hr	Subtraction of 2 S-curve	$q_T = (S_{12}) \times D/T$
0	0	0	–	0	0
6	30 cumec	30+0=30	–	30	15 cumec
12	15 cumec (By III^ar triangle)	30+15=45 →	0	45	22.5 cumec
18	0	45+0=45 →	30	15	7.5 cumec
24	45 (Q_{Eqm})	45 (Q_{Eqm}) →	45	0	0
30 (Not req.)	45	45 →	45	0	0

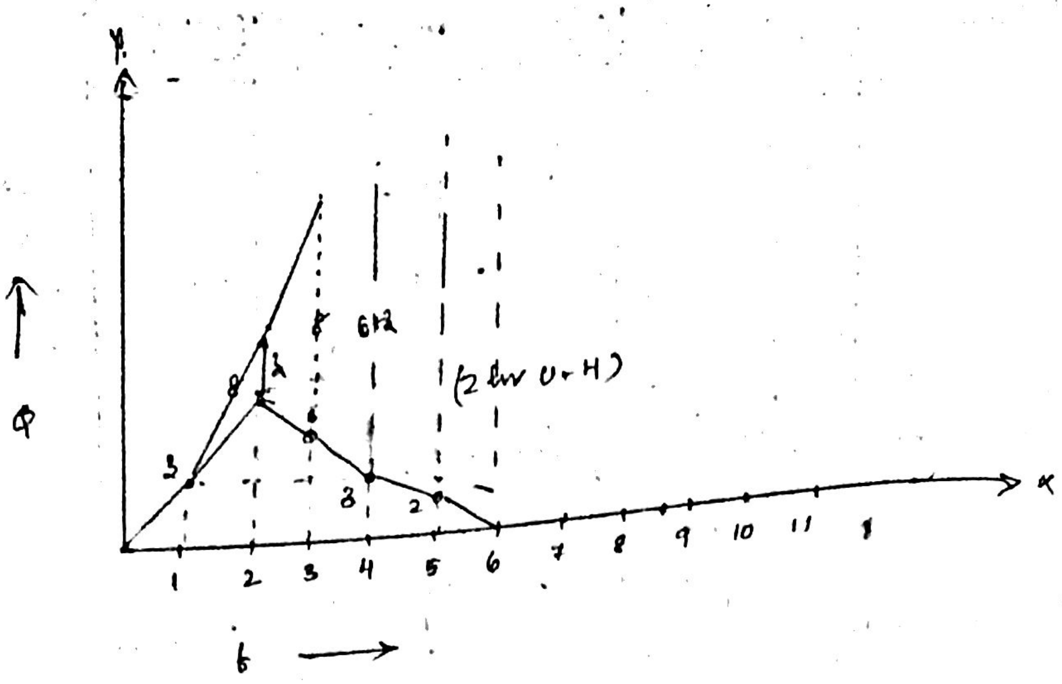


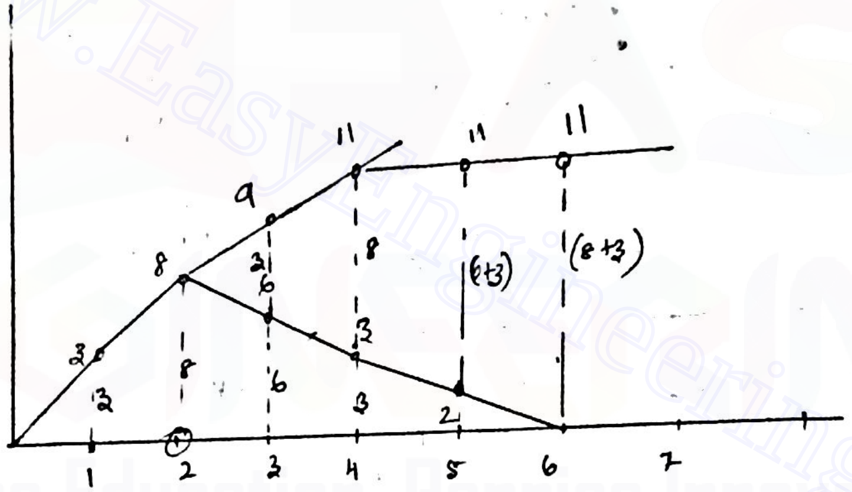
t_b (12 hr) > t_b (6 hr)

Q_{peak} (12 hr) < Q_{peak} (6 hr)

t_b (12 hr UH) > t_b (6 hr UH) by Exactly Extra 6 hour.

[24 hr > 18 hr]





t	2 hr UHO (Given)	S-curve ordinate	S-curve lagged by 3 hr.	Diff. of S-curve	3 hr UHO (D/T) Diff of S-curve ×2/3
0	0	0	–	→ 0	0
1	3	3	–	→ 3	2
②	8	8+0=8	–	→ 8	5.33
3	6	6+3=9	0	→ 9	⑥ peak
4	3	3+8 =11	3	→ 8	5.33
5	2	2+(6+3)= 11	8	→ 3	2
6	0	0+(8+3)=11	9	→ 2	1.33
⑦ to increa.	⟶	11	11	→ 0	0

3 hr UH , $\boxed{Q_p = 6 \text{ cumec.}}$

$P = 6.6$ cm

$\phi = 0.2$ cm/hr

$t = 3$ hr

$$\phi = \frac{P - R}{t}$$

$$0.2 = \frac{6.6 - R}{3}$$

$$\boxed{R = 6 \text{cm}}$$

$$Q_{\text{peak flood}} = Q_{\text{peak}} \times R + \text{Base flow}$$

$$= (6 \times 6) + 5$$

$$= \underline{41 \text{ cumec}} \; //$$

④ For a catchment, the S-curve due to a rainfall of intensity 1cm/hr is given by,

$\boxed{Q = 1 - (1+t)e^{-t}}$, where $t \to$ hours , $Q \to$ cumec .

(i) What is the area of Catchment

(ii) What will be the ordinate of 2-hour UH at t=3hr.

Soln:

(i) For Equilibrium discharge, $\boxed{t = \infty}$ ⊛

$$Q = 1 - (1+\infty)e^{-\infty} = 1 - (1+\infty) \times \frac{1}{e^{\infty} \to 0}$$

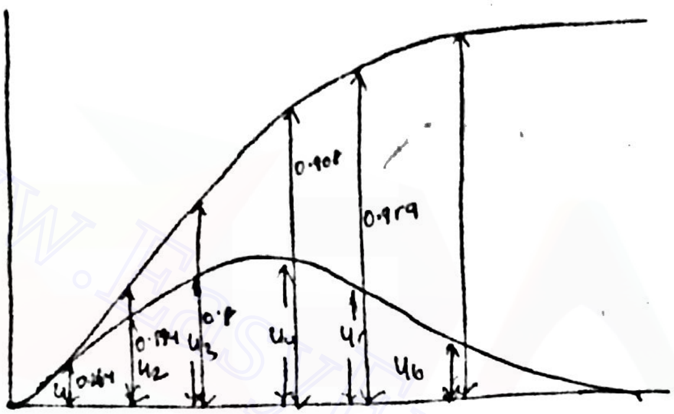
$$\boxed{Q_{eq.} = 1 \text{ cumec}}$$

$$Q = \frac{2.778 A}{t} \Rightarrow 1 = \frac{2.778 \times A}{1}$$

$$A = 0.36 \text{ km}^2$$

	$q = 1-(1+t)e^{-t}$		lagged by 1hr	1H = 2hr DRHO R = 1+1 = 2cm	2cm ↓ R = 1+1 = 2cm
0	0	0	-		0/2 = 0
①	0.264 cumec	0.264-0=0.264	0.264 0		0.264/2 = 0.132 cm.
2	0.594 cumec	0.594-0.264 = 0.33	0.264 0.594	0.594	0.297 cumec
3	0.8 cumec	0.206	0.33	0.54	3hr 0.27 cumec
4	0.908 cumec	0.108	0.206	0.318	0.159 cumec
5	0.959 cumec	0.051	0.108	0.159	0.079 cumec
⋮					

[Ans ⇒ 0.27 cumec (at t = 3hr) 2 hr UHO]



⑤ Ordinates of a 1hr U.H at 1hr intervals starting from t = 0 are 0, 2, 6, 4, 2, 1 and 0.

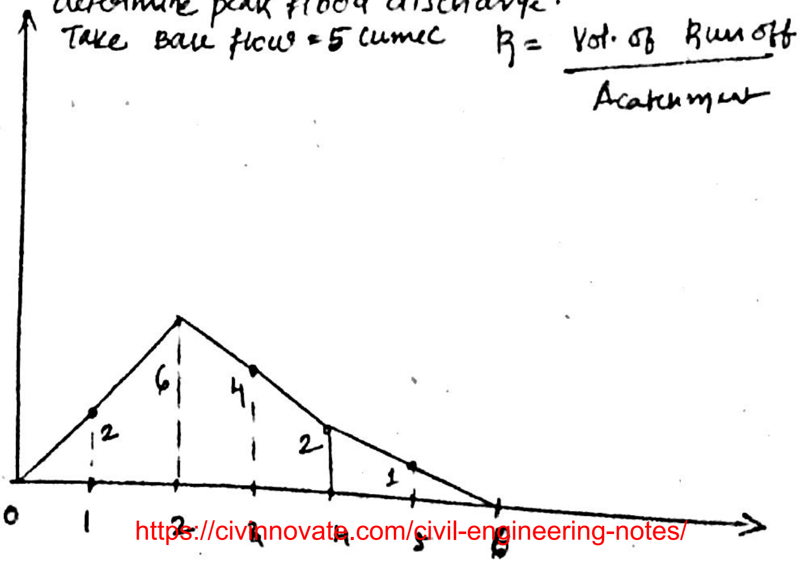
(i) Determine the catchment area

(ii) Determine the ordinate of 3 hr U.H at t = 3hr

(iii) If 2 successive 3 hr rain occurs having total rainfall depth 5cm and 4 cm respectively and the φ-index is 0.5 cm/hr, determine peak flood discharge.

Take base flow = 5 cumec

$R = \frac{\text{Vol. of Runoff}}{A_{catchment}}$



$$R = \frac{1\,cm}{100} = \frac{Area\ of\ DRH}{Area\ A_{catchment}}$$

$$= \frac{\left[\frac{0+2}{2} + \frac{2+6}{2}\times1 + \frac{6+4}{2}\times1 + \frac{2+1}{2} + \frac{1+0}{2}\right]\times60\times60}{A_{catchment}} = \frac{1}{100}$$

$$A_{catchment} = 5.4\ km^2$$

(ii) ordinate of 3hr UH at t = 3hr

1hr UHO		2nd hr UHO lagged by 1hr		3rd 1hr UHO lagged by 2hr	Add/HGY (1hr+1hr+1hr) = 3 hr DRHO, R = 1+1+1 (R = 3cm)		3hr UHO R = 1 cm = DRHO/3cm	DRHO due to 1st 3hr R₁	DRHO due to 2nd 3hr R₂
0	→	Nil	→	Nil	0		0/3 = 0		
2	→	0	→	Nil	2		2/3 = 0.67		
6		2	→	0	8		8/3 = 2.67		
4		6	→	2	12	→	12/3 = ④ ← peak		
		4	→	6	12		12/3 = 4		
		2	→	4	7		7/3 = 2.33		
		1	→	2	3		3/3 = 1		
		0	→	1	1		1/3 = 0.33		
				0	0		0		

Ans: 3hr UH ordinate at t = 3hr is 4 cumec

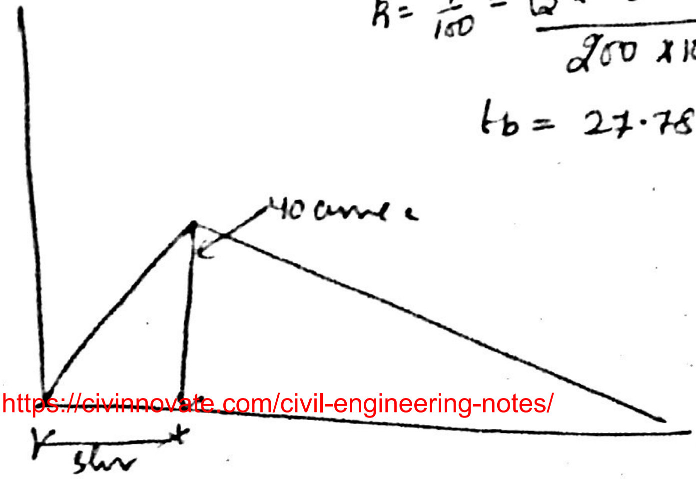
DRHO due to 1st 3hr $R_1 = P - \phi t$ $= 5 - 0.1 \times 3$ $= 3.5$ cm.	DRHO due to 2nd 3hr (lagged by 3hr) $R_2 = 4 - 0.1 \times 3$ $= 2.5$ cm.	Base flow. (cumec)	ordinates of hydrograph. = DRHO₁ + DRHO₂ + Base flow. $(R_1 + R_2 + \text{Base flow})$
			5
$0 \times 3.5 = 0$		5	7.345
$0.67 \times 3.5 = 2.345$		5	14.345
$2.67 \times 3.5 = 9.345$		5	19
$4 \times 3.5 = 14 \rightarrow$	$0 \times 2.5 = 0$	5	$\boxed{20.67}$ peak
$4 \times 3.5 = 14$	$0.67 \times 2.5 = 1.675$	5	19.82
$2.33 \times 3.5 = 8.155$	$2.67 \times 2.5 = 6.675$	5	18.5
$1 \times 3.5 = 3.5$	$4 \times 2.5 = 10$	5	16.155
$0.33 \times 3.5 = 1.155$	$4 \times 2.5 = 10$	5	5.825
$0 \times 3.5 = 0$	$2.33 \times 2.5 = 5.825$		2.5
	$1 \times 2.5 = 2.5$		0.825
	$0.33 \times 2.5 = 0.825$		

peak flood occurs at 4th hour = 20.67 cumec.

⑥ A 3hour UH has peak of 40 cumec and it is triangular in shape for a catchment area of 200 km². If 2nd 3 hour U.H has the same time base, then what will be the peak discharge, if the catchment area is 400 km².

$$R = \frac{1}{100} = \frac{\left[\frac{1}{2} \times t_b \times 40\right] \times (60 \times 60)}{200 \times 10^6}$$

$$t_b = 27.78 \text{ hr.}$$

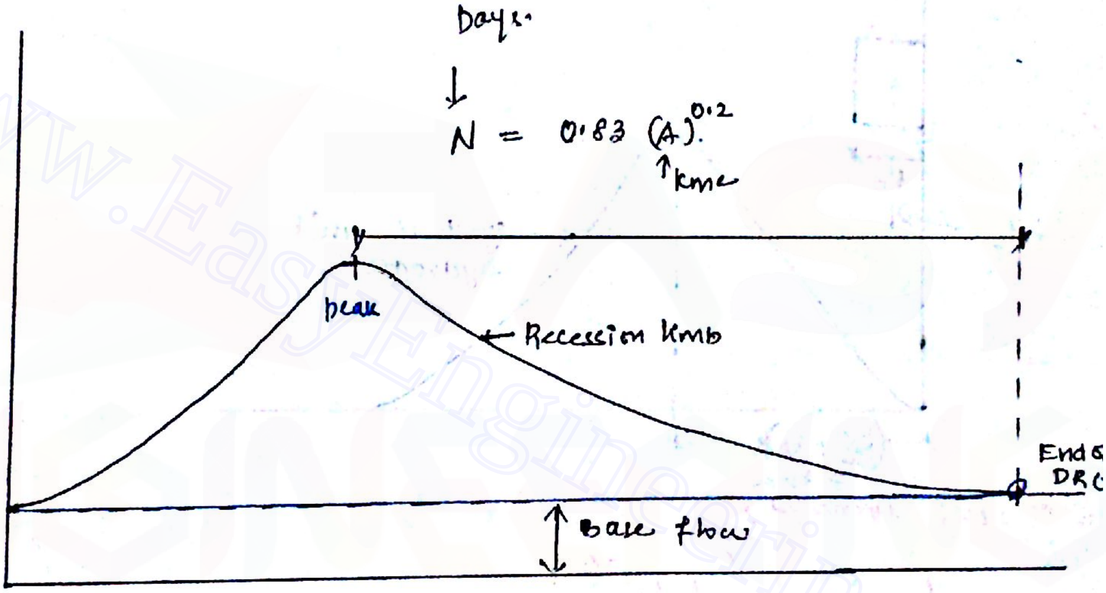


$K = \frac{\quad}{100}$ $\frac{\times Q_{peak}}{400 \times 10^{6}}$

$Q_{peak} = 80$ cumec

⑦ A flood hydrograph of a catchment area of 1000 km² is plotted. Determine the time from the peak, where the hydrograph terminates (end of Recession limb).

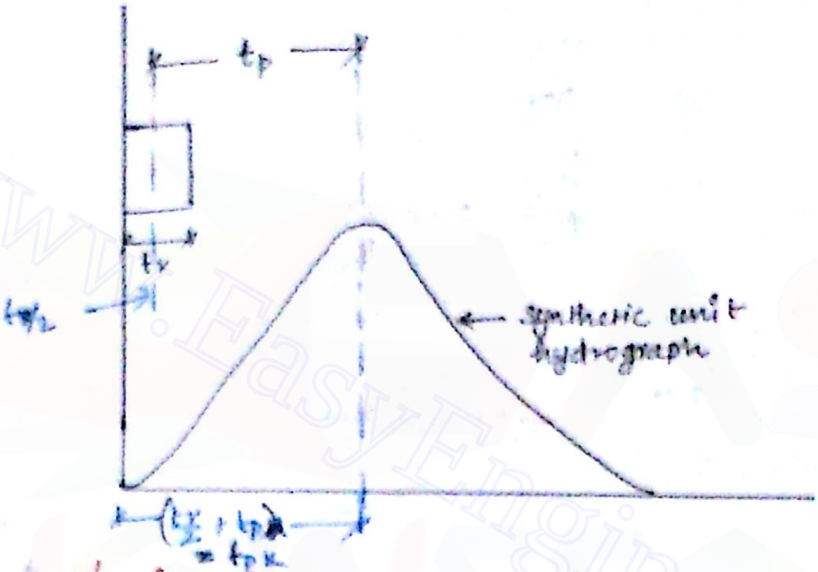
114 (end of DRO) (continuation of Base flow only)



$N = 0.83 (1000)^{0.2} = 3.3$ days

$= 79.2$ hours from the peak.

is 30 Km along
of the catchment is 15 km. Determine
and the centroidal distance
(i) Lag time (peak time) and the
peak discharge if the duration of rain
is 3 hours and catchment co-efficients
are, $C_1 = 1.2$ & $C_2 = 0.75$



Snyder's formula,

$$t_p = \underset{(C_1)}{C_t}\left[L L_c\right]^{0.3}$$

$$Q_p = \frac{2.778\, A \times \overset{(C_2)}{C_p}}{t}$$

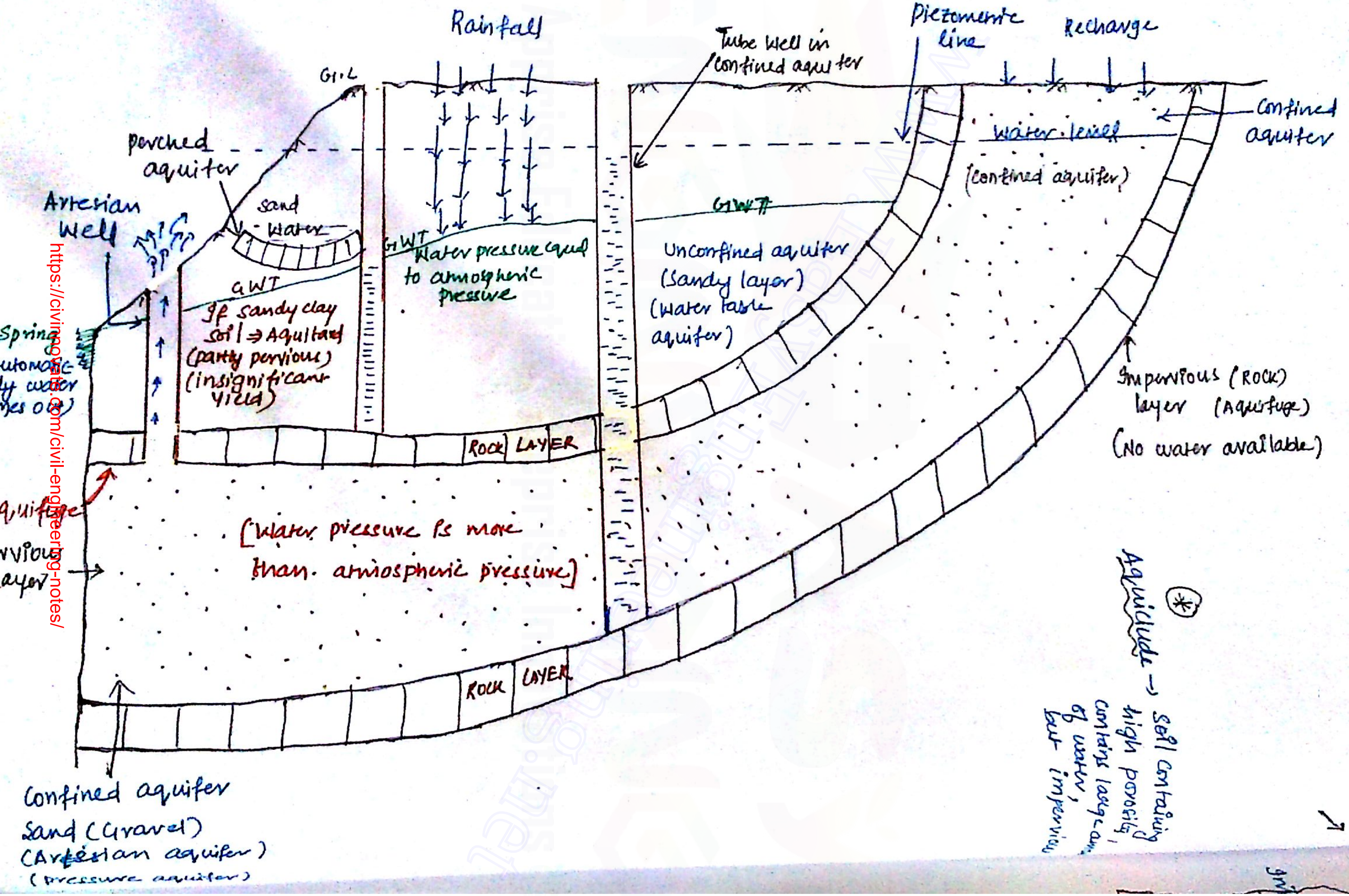
$$t_p = 1.2\left[30 \times 15\right]^{0.3} = 7.5 \text{ hr.}$$

$$Q_p = \frac{2.778 \times 300 \times 0.75}{3 \text{ hr}} = 208.35 \text{ cumec}$$

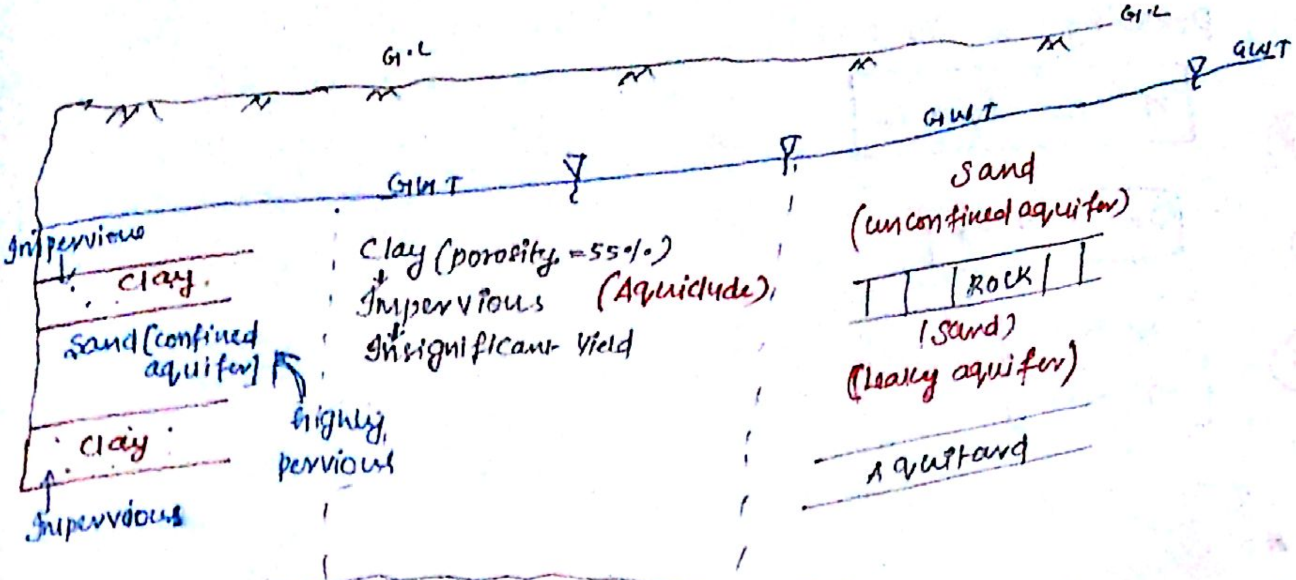
$$t_{pk} = t_p + \frac{1}{2} t_r = 7.5 + \frac{1}{2} \times 3$$

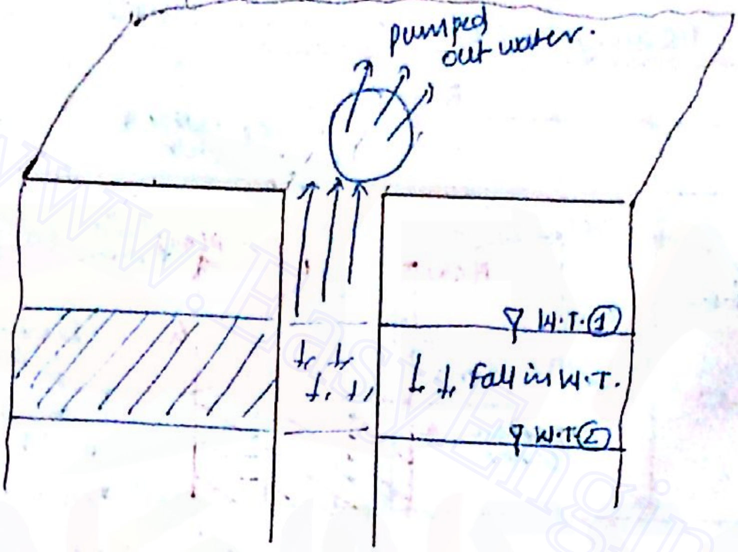
$$= 9 \text{ hrs}$$

TUBE WELL (Ground Water)



(*) Aquiclude → soil containing high porosity, contains large am. of water, but impervious





* **

Specific yield = $S_y = \dfrac{\text{Volume of water pumped out}}{\text{Vol. of fully sat. soil mass}}$

$M^0L^0T^0$ (No dimension) $= \dfrac{V_w \text{ (pumped)}}{\text{Surface Area} \times \text{Fall in W.T}} \times 100$

→ Specific retention = $S_r = \dfrac{\text{Vol. of water held against gravity}}{\text{Vol. of fully sat soil mass}}$

⇒ $M^0L^0T^0$ (No unit) (No dimension)

Porosity = $S_y + S_r$

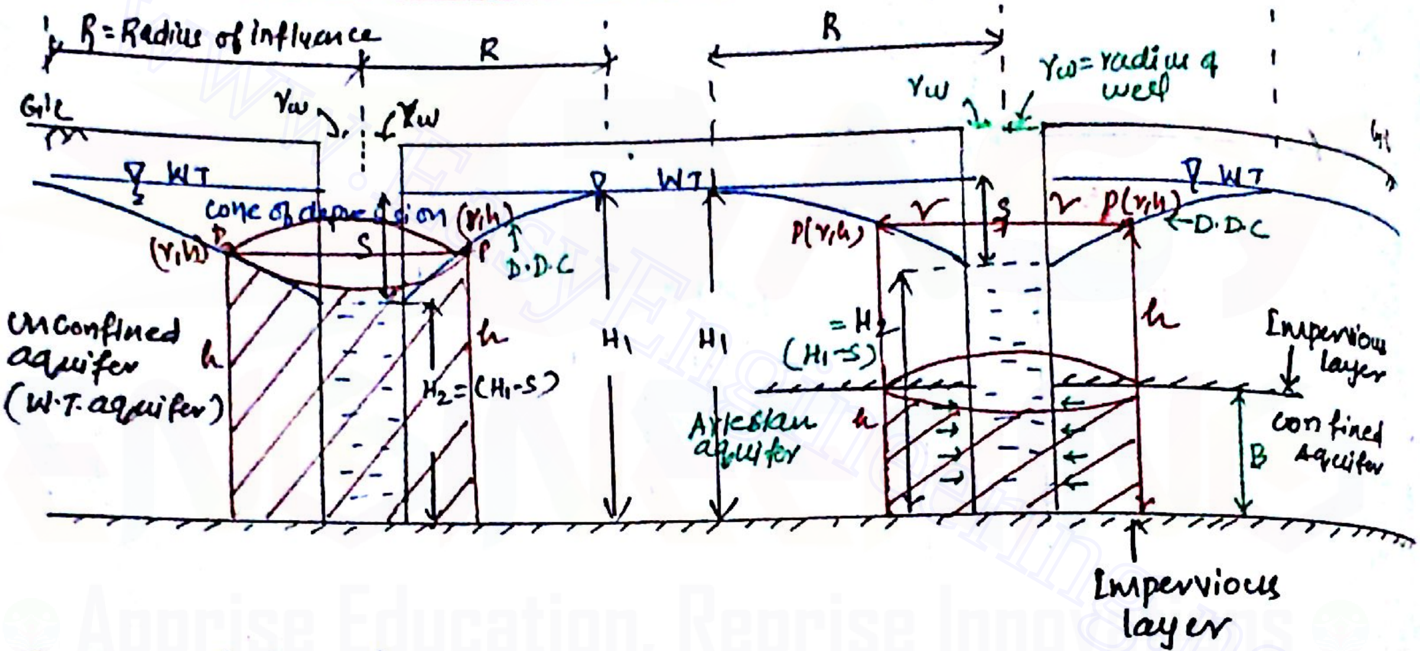
$$\boxed{n = S_y + S_r}$$ (i)

$$\boxed{S_r = n - S_y}$$

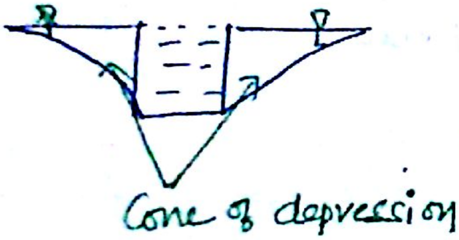
$$\boxed{\begin{matrix} S_r < n \\ S_y < n \end{matrix}}$$

***

Expression for Yield (discharge) from a tube well



D.D.C = Draw down curve

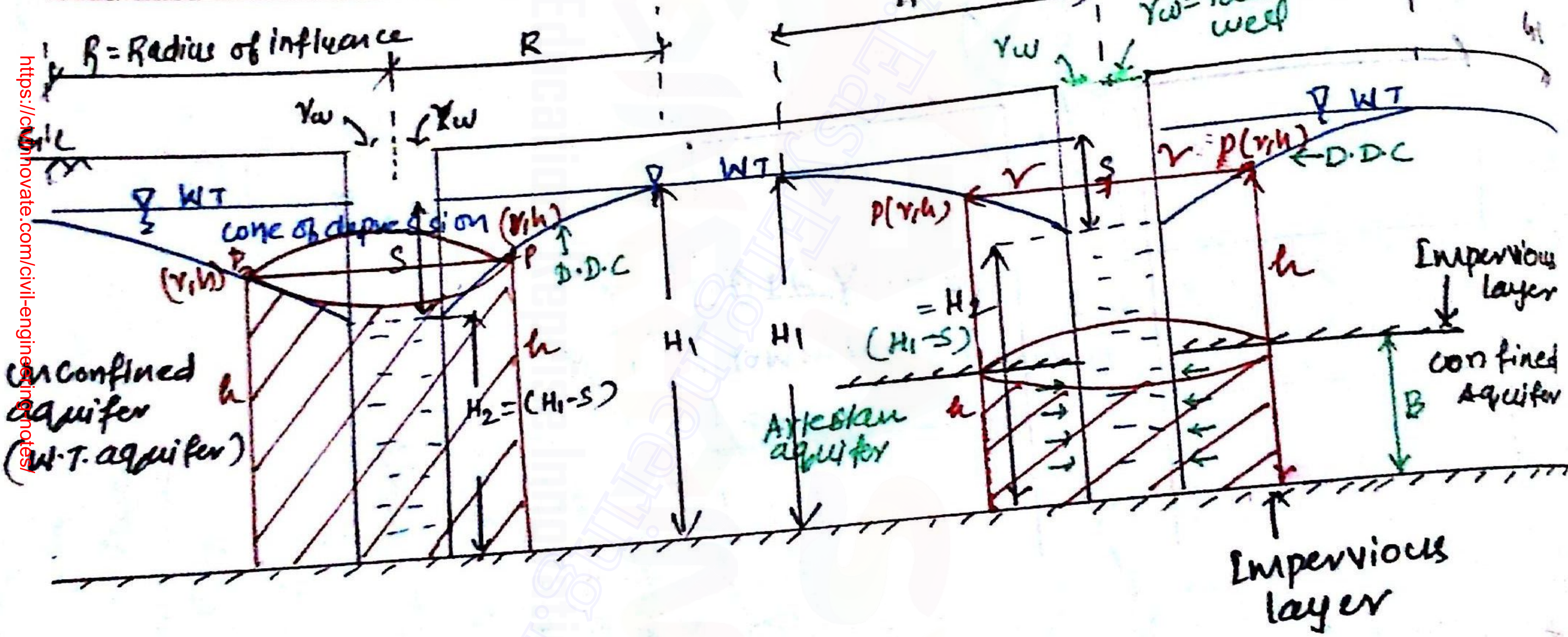


Unconfined aquifer was developed by Mr. Dupuit.

Confined aquifer → developed by Mr Thiem.

***

Expression for Yield (discharge) from a tube well



$Q = KiA$

$K =$ coeff of permeability

= Hydraulic conductivity

$i = \frac{dh}{dr}$

$A = (2\pi r) \times h$

$Q = K \times \frac{dh}{dr} \times (2\pi r) \times h$

$$\int_{r_w}^{R} Q \times \frac{dr}{r} = \int_{H_2}^{H_1} K \times 2\pi h \times dh$$

$$Q \times \left[\log_e(r)\right]_{r_w}^{R} = 2\pi K \left[\frac{h^2}{2}\right]_{H_2}^{H_1}$$

$$Q \times \left[\log_e(R) - \log_e r_w\right] = \pi K \left[H_1^2 - H_2^2\right]$$

$$Q \times \log_e\left(\frac{R}{r_w}\right) = \pi K \left[H_1^2 - H_2^2\right]$$

$$\boxed{Q = \frac{\pi K \left[H_1^2 - H_2^2\right]}{\log_e(R/r_w)}}$$

Drawdown

Main W

$$\boxed{H_2 = H_1 - S}$$

Water table height

(ii) Confined aquifer [Steady State Equilibrium Condition] {Thiem's

$Q = KiA = K \times \frac{dh}{dr} \times \{2\pi r \times B\}$ *

$$\int_{r_w}^{R} Q \times \frac{dr}{r} = 2\pi K B \int_{H_2}^{H_1} dh$$

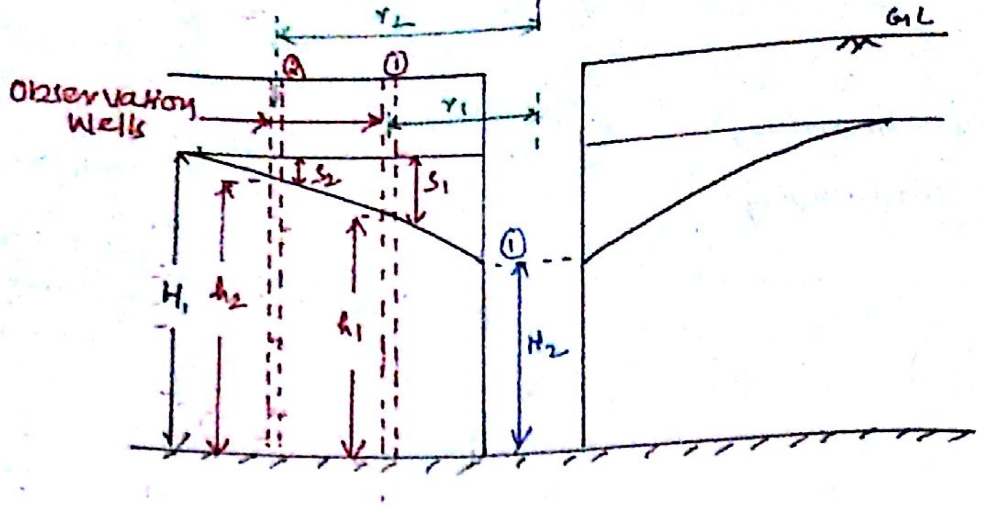
$$Q \log_e\left(\frac{R}{r_w}\right) = 2\pi K B \left(H_1 - H_2\right)$$

$$\boxed{H_1 - H_2 = S}$$

$$\boxed{Q = \frac{2\pi K B (H_1 - H_2)}{\log_e(R/r_w)}}$$

IIIly, $\boxed{Q = \frac{2\pi K B (h_2 - h_1)}{\ln(r_2/r_1)}}$

$\boxed{Q = \frac{2\pi K B \left[h_1 - H_2\right]}{\ln(r_1/r_w)}}$ or $\boxed{Q = \frac{2\pi K B \left[H_1 - h_2\right]}{\ln(R/r_2)}}$



Un (Confined)

$$\boxed{Q = \frac{\pi k\left[h_2^2 - h_1^2\right]}{\ln\left(\frac{r_2}{r_1}\right)}}$$

$h_1 = H_1 - S_1$

$h_2 = H_1 - S_2$

Any 2 points can be taken as per data

$$Q = \frac{\pi k\left[H_1^2 - H_2^2\right]}{\ln\left(r_1/r_w\right)} \quad \text{(or)} \quad Q = \frac{\pi k\left[H_1^2 - h_2^2\right]}{\ln\left(R/r_2\right)}$$

Un Confined

$H_1 - H_2 = S_{w\,(\text{main well})}$ = Drawdown in the Main well

KB = Transmissibility

= T = Transmissivity

Unit → m^2/sec (or) $\frac{m^3/sec}{m}$

Dimension → L^2T^{-1}

$$\therefore \boxed{Q = \frac{2\pi TS}{\ln(R/r_w)}}$$ → confined aquifer yield.

$$\boxed{\frac{Q}{S} = \frac{2\pi T}{\ln(R/r_w)}}$$

⇓

$\frac{Q}{s}$ = specific capacity of well

= performance of well = $\frac{2\pi T}{\ln(R/r_w)}$

The specific capacity of a well is the discharge from a well per unit drawdown. It is also known as the performance of a well. If the drawdown is increased, the specific capacity decreases.

→ The storage co-efficient ← storativity of a confined aquifer is defined as vol. of water released from unit area due to unit decrease in piezometric head (only for confined)

It has no dimension ($M^0L^0T^0$) [symbol = S] $\boxed{S = \frac{V_{water}}{A_{confined} \times H}}$

The co-efficient of transmissibility [T] and the storage co-efficient (no dimension) are called formation constant of confined aquifer. [TS]

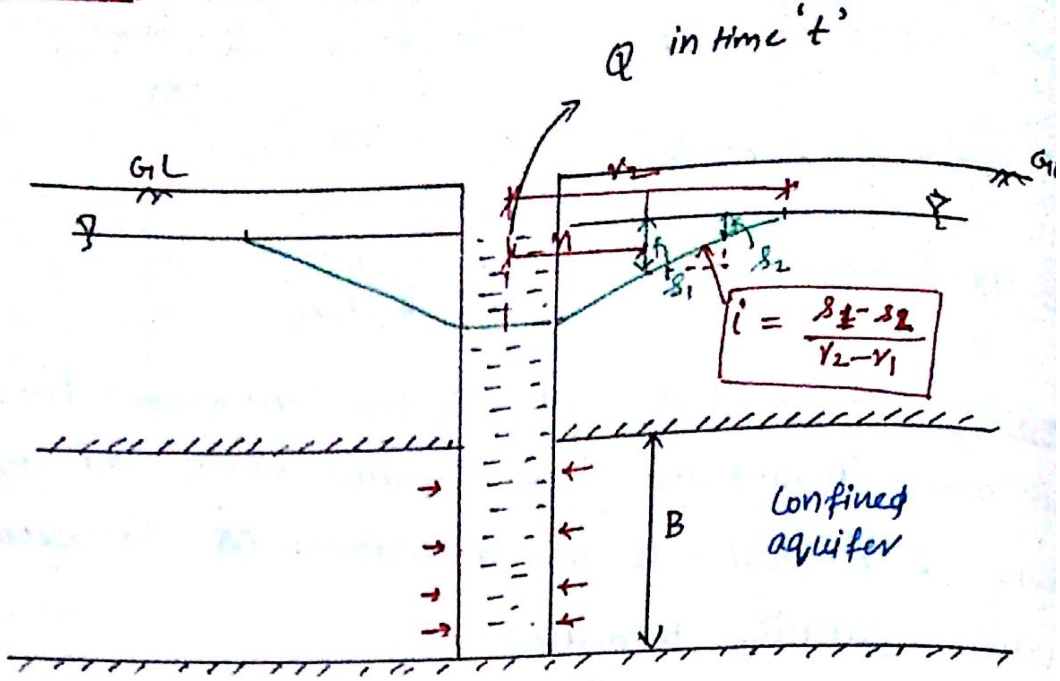
* The specific storage can be defined as the volume of water released from unit volume of aquifer due to unit decrease in piezometric head.

It's symbol is S_s

$$\boxed{S_s = \frac{V_{water}}{V_{soil} \times H}}$$

Unit → $\frac{1}{m}$

Dimension L^{-1}



$$i = \frac{s_1 - s_2}{r_2 - r_1}$$

According to Mr. Jacob and Mr. Thies, (unsteady state)

$$\boxed{s = \frac{Q}{4\pi T}\ln\frac{2.25\,Tt}{r^2 S_{coeff}}}$$ [for confined aquifer]

Drawdown

where, s → Drawdown or piezometric head

T → co-efficient of transmissibility

t → time of pumping

r → radial distance from centre of well where drawdown is required.

NOTE: The difference in 2 drawdowns (s_1 and s_2) at

$$\boxed{s_1 - s_2 = \frac{Q}{4\pi T}\ln\frac{2.25\,Tt}{r_1^2 S} - \frac{Q}{4\pi T}\ln\frac{2.25\,Tt}{r_2^2 S}}$$

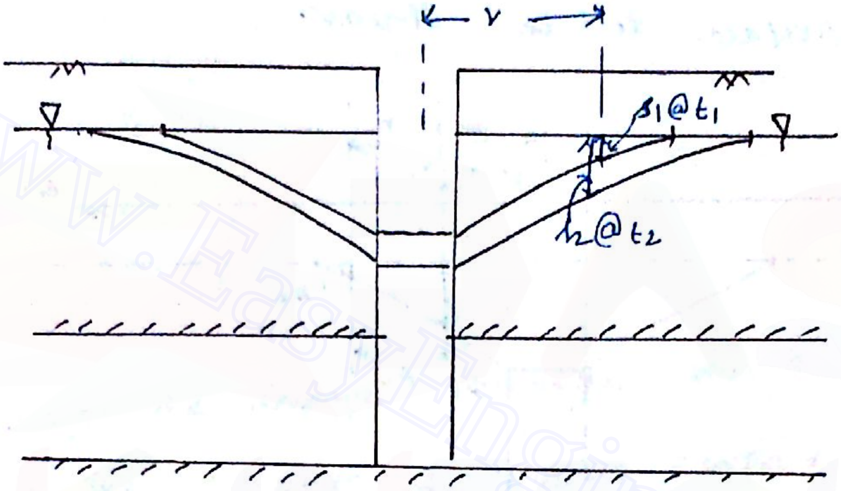
at same time for 2 different places.

$$s_1 - s_2 = \frac{Q}{4\pi T}\left[\ln\left(\frac{2.25Tt}{r_1^2 S}\right) \Big/ \left(\frac{2.25Tt}{r_2^2 S}\right)\right]$$

$$\boxed{s_1 - s_2 = \frac{Q}{4\pi T}\left[\ln \frac{r_2^2}{r_1^2}\right]}$$ → Time constant, 2 different places

(i) Time is variable, but same place (r is constant)

$$s_1 - s_2 = \frac{Q}{4\pi T} \times \ln \frac{2.25\,T \times t_1}{r^2 S} - \frac{Q}{4\pi T} \times \ln \frac{2.25\,T \times t_2}{r^2 S}$$



$$\frac{s_2}{s_1} = \frac{\ln \frac{2.25\,T t_2}{r^2 S}}{\ln \frac{2.25\,T t_1}{r^2 S}}$$

Suppose, $s_2 = s_1$

$$\Rightarrow \frac{Q}{4\pi T} \ln \frac{2.25\,T t_2}{r^2 S} = \frac{Q}{4\pi T} \ln \frac{2.25\,T t_1}{r^2 S}$$

$$\Rightarrow \boxed{t_1 = t_2}$$

If 2 different places different time

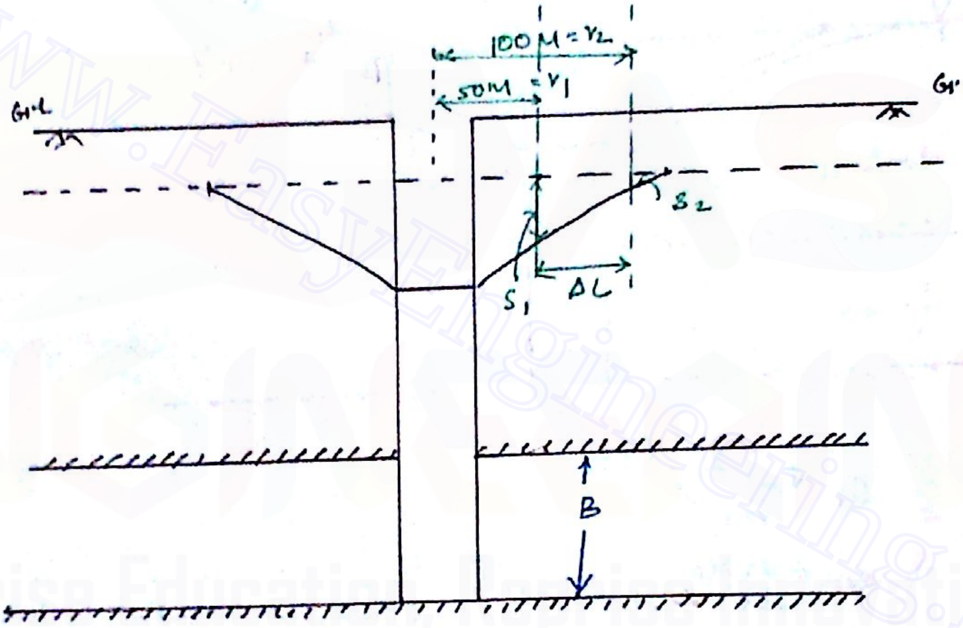
Suppose $s_2 = s_1$

$$\frac{t_2}{r_2^2} = \frac{t_1}{r_1^2}$$

$$\boxed{\frac{r_1}{r_2} = \sqrt{\frac{t_1}{t_2}}}$$

Numericals

① A 30 cm dia pumping well starts to pump water as 6 A.M. at a rate of 2000 lpm from a confined aquifer having thickness 30m and permeability 30 m per day. The storage co-efficient is 0.005. Find out the $\boxed{\text{Slope}}$ i.e.) of the hydraulic gradient at 9 P.M on the same day b/w 2 observation wells located at distance of 50 m and 100 m from the pumping well. Assume the piezometric surface to be linear.



$$s_1 = \frac{Q}{4\pi T} \times \ln \frac{2.25\, T t_1}{r_1^2 \times S}$$

$\boxed{t_1 = 15 \text{ hrs. (6 AM to 9 PM)}}$

$T = KB = 30 \times 30 = 900 \text{ m}^2/\text{day}$

$= 37.5 \text{ m}^2/\text{hr}$

~~Sr~~ $Q = 2000 \frac{\text{lit}}{\text{min}}$

$= 2000 \times 10^{-3} \times 60$

$= 120 \text{ m}^3/\text{hr}$

$S = 0.0095$

$$\therefore s_1 = \frac{120}{4\pi \times 37.5} \times \ln \frac{2.25 \times 37.5 \times 15}{50^2 \times 0.0095}$$

$$= 1.175 \text{ m}$$

$$s_2 = \cancel{0}\ \frac{120}{4\pi \times 37.5} \times \ln \frac{2.25 \times 37.5 \times 15}{100^2 \times 0.005}$$

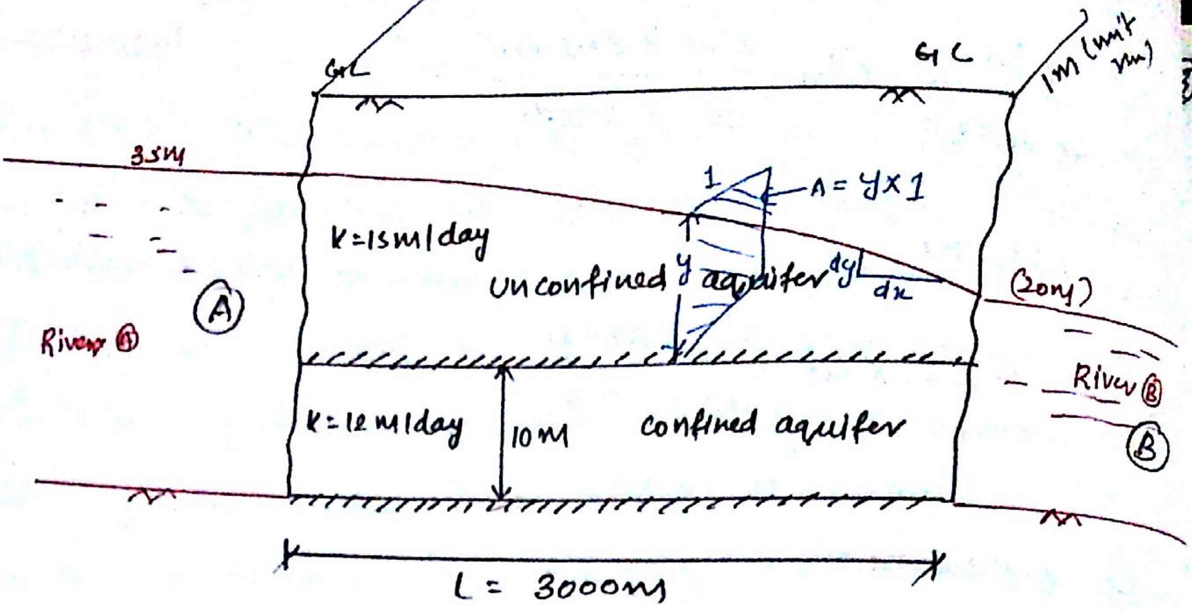
$$= 0.823 \text{ m}$$

$$i = \frac{s_1 - s_2}{r_2 - r_1} = 7.04 \times 10^{-3}$$

$$\boxed{i = \frac{1}{142}}$$

4)**

② Determine the seepage discharge b/w 2 rivers, 3km apart. The elevation of water surface in upper river is 35 m and in the lower river it is 20 m. The thickness of confined aquifer is 10m. The hydraulic conductivity of confined aquifer is 12 m per day and of unconfined aquifer, it is 15 m per day.



(i) Confined aquifer

$$Q_1 = kiA = \frac{12}{24} \times \frac{(35-20)}{3000} \times (10 \times 1)$$

$$= 0.025\ m^3/hr \qquad = 0.6\ m^3/day$$

(ii) Unconfined aquifer

$$Q_2 = kiA = K \times \frac{dy}{dx} \times (y \times 1)$$

$$\int_0^{3000} Q \times dx = \int_{10}^{25} k \times y \times dy$$

$$Q\,(x)_0^{3000} = k \times \left[\frac{y^2}{2}\right]_{10}^{25}$$

$$Q \times 3000 = \frac{15}{24} \times \frac{1}{2} \times \left[25^2 - 10^2\right]$$

$$Q_2 = 0.054\ m^3/hr \qquad = 1.296\ m^3/day$$

Total discharge = $Q = Q_1 + Q_2$

$$= 0.079\ m^3/hr = 1.9\ m^3/day$$

③ Determine dia 20cm driven in confined aquifer having thickness 30m permeability 20 m/day. Take drawdown in the well 6m and radius of influence 300m

<u>soln</u>:
$$Q = \frac{2\pi KB\,[\overset{\downarrow S}{h_1 - h_2}]}{\ln(R/r_w)}$$

$$= \frac{2\pi \times 20 \times 30\,[6]}{\ln\left(\frac{300}{0.1}\right)}$$

$= 2975$ m3/day = 0.10

$= 34.43$ lps = 0.0344 cumec.

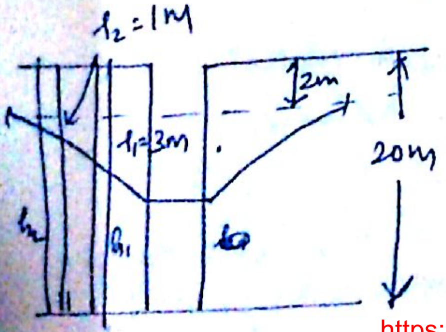
$H_1 = 18$m

④ Determine the yield from a tube well having dia 30cm driven in unconfined aquifer. The water table is 2m below GL. & total thickness of sandy soil mass is 20m. The drawdown in 2 observation wells located at 25 m and 75 m from the centre of well are 3m and 1m. Take conductivity as 15 m/day

<u>soln</u>:
$$Q = \frac{\pi K\,[h_2^2 - h_1^2]}{\ln(r_2/r_1)} = \frac{\pi \times 15\,[17^2 - 15^2]}{\ln(75/25)}$$

$= 2745.22$ m3/day

$= 31.77$ lps



$h_1 = 20 - 2 - 3 = 15$m

$h_2 = 20 - 3 = 17$m

(5) Determine the co-efficient of permeability, co-efficient of intrinsic permeability and Reynold number with the help of foll. data –

(i) Time required by tracer to travel a distance of 50m b/w 2 wells is 10 hours.

(ii) Dia of soil grain = 2mm

(iii) Porosity of soil = 0.3

(iv) Kinematic viscosity of water = 0.01 stoke

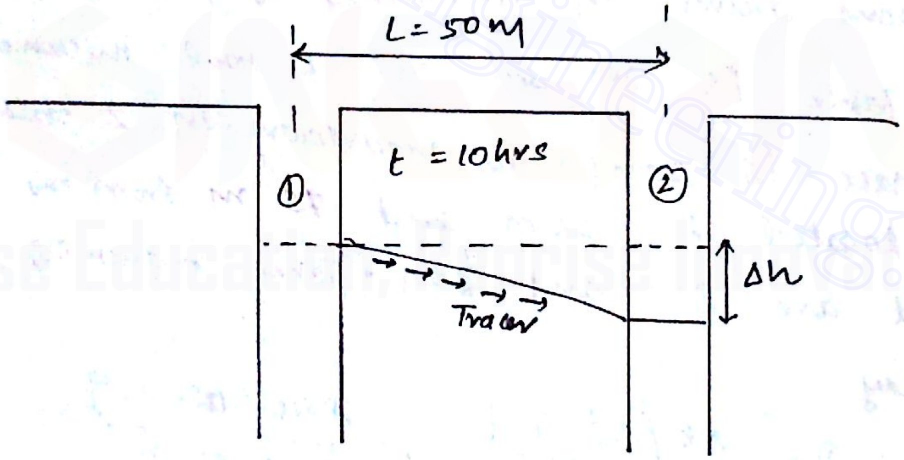
(v) Total surface area = 3 km² (3 sq. km)

(vi) Volume of water pumped out = ~~1M~~ 1 MCM

(vii) Fall in water table = 2 m

Find the specific yield. Take diff. of water level in b/w 2 wells as 50 cm.

Soln:



(i) Actual velocity of seepage

$$V_{sep} = \frac{L}{t} = \frac{50 \times 100}{10 \times 60^2} = ~~1.39 cm/sec~~ = 0.139 \text{ cm/sec}.$$

(ii) $V_{Dary} = \frac{0.139 \times 0.3}{\cancel{0.3}} =$ ~~0.463 cm/sec~~. $= 0.0417$ cm/sec

$v = Ki$ ⇒ ~~0.463~~ 0.0417 $= k \times \frac{50}{50 \times 100}$

$k = 4.17$ cm/sec

(iii) Intrinsic permeability

$$K = D_{10}^2 \times \underbrace{\left(\frac{e^3}{1+e}\right)}_{\leftarrow C} \times \frac{\gamma}{\mu}$$

$$\boxed{K = \underbrace{(C\, D_{10}^2)}_{\Downarrow} \times \left(\frac{\gamma}{\mu}\right)} \leftarrow \text{Fluid property}$$

soil property

$$\boxed{K = K_0 \times \frac{\gamma}{\mu}}$$

$$\boxed{K_0 = \text{Intrinsic permeability}}$$

$\Downarrow$

func. of soil property only.

$$K_0 = \frac{K\mu}{\gamma} = \frac{K\mu}{\rho g} = \frac{K\nu}{g}$$

$$= \frac{\overset{4.17\,cm/sec}{\overset{cm/sec}{\cancel{0.0417}}} \times 0.01\ \frac{cm^2}{sec}}{9.81 \times 100\ \frac{cm}{sec^2}}$$

$$\boxed{K_0 = 4.25 \times 10^{-5}\ cm^2}$$

Unit of $k_0 \longrightarrow$ Darcy.

1 Darcy $= 9.87 \times 10^{-13}\ m^2$

$= 9.87 \times 10^{-9}\ cm^2$

$$\Rightarrow k_0 = \frac{4.25 \times 10^{-5}}{9.87 \times 10^{-9}}$$

$= 4305$ ~~cm²~~ Darcy.

$$\text{(iv)}\quad R_e = \frac{\rho V \overset{Darcy}{D_{10}}}{\mu} = \frac{VD}{\nu} \neq \frac{\cancel{4.17} \times D}{\cancel{\nu}}$$

$$= \frac{\cancel{\frac{50 \times 100}{90 \times 60^2}}^{\ cm/sec} \times \cancel{\frac{2}{10}}\ cm}{0.01\ cm^2/sec} \neq$$

$$R_e = \frac{\rho \bar{V} d_{10}}{\mu} = \frac{\bar{V} d_{10}}{\nu}$$

$$= \frac{0.0417 \times d_{10}^{\leftarrow \frac{2}{10}}}{0.01} = 0.834 < 1$$

$\boxed{R_e < 1} \Rightarrow$ laminar flow.

Darcy law valid //

Specific Yield (S_y) = $\frac{\text{Vol. water pumped}}{A \times H}$ = $n - S_r$

$$= \frac{1 \times 10^6 \ m^3}{3 \times 10^6 \times 2} = \frac{1}{6}$$

$$\boxed{S_y < n}$$

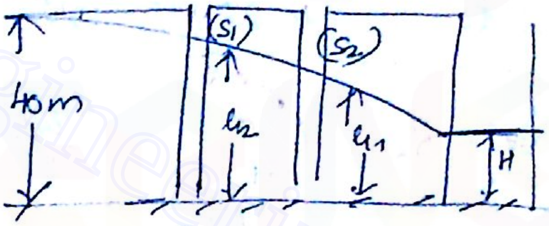
$$= 0.167 < n$$

$S_r = n - S_y = 0.3 - 0.167$
$= 0.133$ (No dimension) //

⑥ A 30 cm dia tube well penetrates 40 m thick aquifer resting on hard stratum. The steady state discharge is 1500 lpm. The drawdown in 2 observation wells located at 25 m and 75 m from the centre of well are 3 m and 2.25 m. Determine —

(i) coefficient of transmissibility

(ii) Drawdown in the main well.



Soln:

$$Q = \frac{\pi K (h_2^2 - h_1^2)}{\ln (r_2/r_1)}$$

$$Q = 1500 \text{ lpm} = \frac{1500 \times 10^{-3}}{60} = 0.025 \text{ cumec}$$

$r_1 = 25 \text{ m}$ $\quad r_2 = 75 \text{ m}$

~~$h_1 = 40 - 2.25 = 37$m~~ 37.75m

~~$h_2 = 40 - 3 = 37.75$~~
$= 37$m

$$0.025 = \frac{\pi K (37.75^2 - 37^2)}{\ln \left(\frac{75}{25} \right)}$$

$h_1 = 40 - 3 = 37$m

$h_2 = 40 - 2.25 = 37.75$m

$$\boxed{K = 1.56 \times 10^{-4} \ m/sec}$$

⇒ ~~$K = 2.85 \times 10^{-6}$ m/sec~~

(i) Co-efficient

$$= 1.56 \times 10^{-4} \times 40$$

$$= 6.24 \times 10^{-3} \ m^2/sec.$$

(ii)

$$\phi = \frac{\pi k [h_1^2 - H_1^2]}{\ln(25/0.15)}$$

$$\Rightarrow 0.025 = \frac{\pi \times 1.56 \times 10^{-4} [37^2 - H_1^2]}{\ln\left(\frac{25}{0.15}\right)}$$

$$\boxed{H_1 = 33.287 m}$$

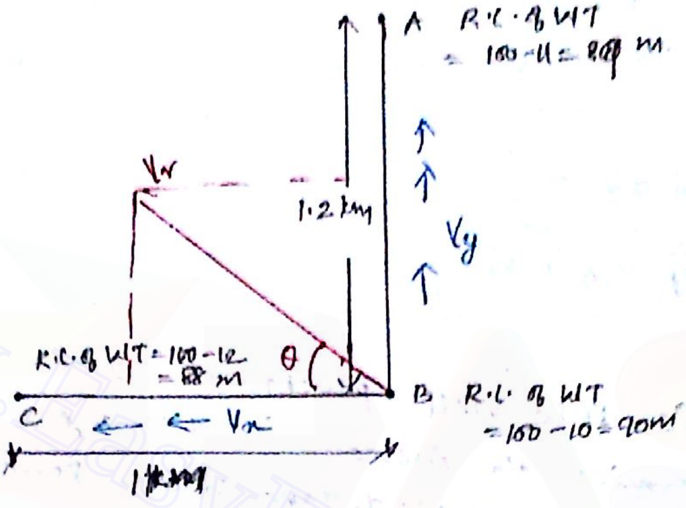
$$\boxed{S = 40 - H_1 = 6.713 m}$$

⑦ A 30 cm dia tube well penetrates 25 m thick confined aquifer. Determine % increase in discharge if the dia of tube well is doubled. Take coeff of permi. = 15 m/day. Take constant radius of influence = 300 m & drawdown in main well is 6 m.

$$\frac{\frac{\cancel{2\pi K B}[H_2 - H_1]}{\ln(R/r_w)} - \frac{\cancel{2\pi K B}(H_2 - H_1)}{\ln(R/r_w)}}{\frac{\cancel{2\pi K B}[H_2 - H_1]}{\ln(R/r_w)}} \leftarrow 6$$

$$\frac{\frac{1}{\ln\left(\frac{300}{0.3}\right)} - \frac{1}{\ln\left(\frac{300}{0.15}\right)}}{\frac{1}{\ln\left(\frac{300}{0.15}\right)}} \times 100 = 10.03 \%$$

drilled located at origin, North and West directions. The R.L. of G.L is 100 m. The R.L. of water table at the origin is 10 m below G.L, at the North 11 m below G.L and at the West 12 m below G.L. The position of Northern well is 1.2 km and the position of Western well is 1 km



$V = Ki$

$$V_{y(North)} = K_y \times \frac{\Delta H}{L} = k \times \frac{(90-89)}{1200} = \frac{k}{1200} \text{ m/sec}$$

$$V_x = K_x \times \frac{90-88}{1000} = \frac{k}{500} \text{ m/sec}$$

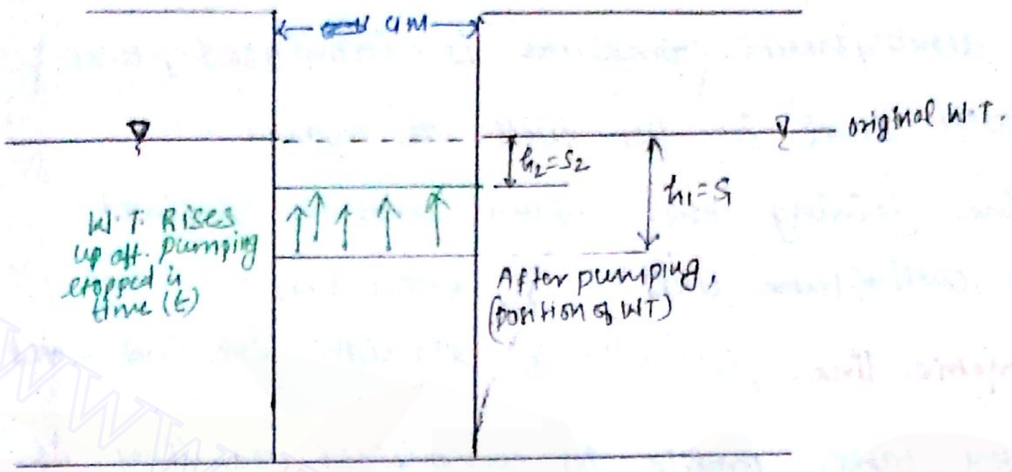
$$V = \sqrt{V_x^2 + V_y^2} = \sqrt{\left(\frac{k}{1200}\right)^2 + \left(\frac{k}{500}\right)^2}$$

$$\boxed{V = K \times 2.17 \times 10^{-3}} \rightarrow \text{Resultant velocity.}$$

$$\tan\theta = \frac{V_{\cancel{x}y}}{V_{\cancel{y}x}} = \frac{\cancel{k/500}}{\cancel{k/1200}} \quad \frac{k/1200}{k/500}$$

$$\boxed{\theta = \cancel{22.6}^\circ \text{ (or) } N\,68^\circ\,W}$$

where time required to recuperate (Reoccupy) water ~~o~~ from 2.5m drawdown (depression) to 1.2m drawdown (depression) is 90 minute.
consider drawdown as 2.5m.



$$Q = A \times V$$

$$= A \times ki = kiA$$

$$= K \times \frac{H}{L} \times A$$

$$= \frac{K}{L} \times S \times A$$

$$= C \times S \times A$$

$$= \left[\frac{1}{t} \log_e\left(\frac{h_1}{h_2}\right)\right] \times S \times A$$

$$= \left[\frac{1}{90 \times 60} \log_e\left(\frac{2.5}{1.2}\right)\right] \times 2.5 \times \frac{\pi}{4} \times 4^2$$

$$= 4.27 \times 10^{-3} \; m^3/sec$$

$$\boxed{Q = 4.27 \; lps}$$

If 2 tube wells are very close, then

$$Q = \frac{2\pi k b \overset{\leftarrow (H_2 - H_1)}{S}}{\log_e \frac{R^2}{r_w \times \text{spacing of tube well}}}$$

[Radius of influence co-incide with each other]

NOTE :

① If the atmospheric pressure is increased, then the water level in the well decreases.

② The line joining the water level in confined aquifer well → (tube well) is known as Piezometric line, which is a straight line (not curved).

③ The water table profile in unconfined aquifer is Parabolic [without recharge of ground water].

But if there is recharge of ground water (heavy rain) then the profile of water table is Elliptical (semi-elliptical)

④ The porosity of clay is highest (about 45 to 50 %) but it's specific yield (S_y) is the lowest, i.e. (about 1 % to 10 %). The porosity of sand is about 35 % and its specific yield is the highest (about 30 %)

⑤ The well loss is directly proportional to square of discharge from well. (Turbulent flow) $\left[\frac{fLV^2}{2gd}\right]$ but the formation loss (laminar flow is directly proportional to discharge, power 1 (Q^1)], Total loss in the well,

$$\underbrace{h_L}_{\text{Total loss}} = \underbrace{C_1 Q^1}_{\text{Formation loss}} + \underbrace{C_2 Q^2}_{\text{Well loss}}$$

⑥ The radius of influence,

$$\boxed{R = 3000 S\sqrt{K}}$$

S → drawdown (m)

K → permeability (m/sec)

$$R = 3000 \times 6\sqrt{2\times10^{-4}}$$

$$\boxed{R = 254.56 m}$$

FLOOD AND FLOOD ROUTING

① In India the flood damage per year is about 5000 crore rupees.

② There are 2 methods of flood control –

(i) <u>Structural method</u> –

~~(ii)~~ (a) Construction of Reservoir and dams

(b) Flood Control valley (levees) (Dyke)

(c) Channel improvement

(d) Soil conservation

(ii) <u>Non – structural measure</u>

(a) Flood plains zoning

(b) Flood warning

(c) Evacuation

(d)

* ③ If the probability of drought is in a range of 20% to 40% (0.2 to 0.4) then it is known as drought zone.

The aridity index, $A.I = \frac{PET - AET}{PET} \times 100$

$= 0$ to $-ve$ → No arid zone

$= 1$ to 25% → Mild arid zone

$= 26$ to 50% → Moderate arid zone

$= >50\%$ → Severe arid zone

* ④ There are 3 types of flood –

(i) <u>Probable Maximum Flood (PMF) (MPF)</u> –

It is a physically possible flood. It takes place due to Severe most combination of Hydrological and Meteorological factors creating catastrophes

It is about 40 to 60% of PMF. It takes place due to $\boxed{\text{Severe combination}}$ of hydrological and meteorological factors.

(iii) **Design Flood** – The Flood which is actually adopted for designing Hydraulic structure is called design Flood.

Hydraulic structure	Design Flood
① Large dam and spillway and large reservoir, {Capacity of Reservoir > 60 MCM}	PMF, if it is not available, then 1000 yr. Flood frequency flood
② small reservoir and low height spillway [capacity < 60 MCM]	(i) SPF (or) 100 yr Frequency Flood whichever is greater
③ Small weir (Pick up weir)	100 yr. frequency flood
④ Aqueduct	100 yr. Frequency flood for designing foundation, 50 yr Frequency flood for designing LWW
⑤ Scanty data (insufficient)	Empirical formulae

⑤ **Empirical formula** – (to find peak flood)

(i) **Mr Dicken formula** (North India) and (Central India)

$$\boxed{Q_{peak} = C(A)^{3/4}} \quad (\text{cumec})$$

$A \to km^2$ $\quad C = 6$ to 6.5 (Indo Gangetic plain)

(ii) **Mr. Ryve formula** –

$$\boxed{Q_{peak} = C(A)^{2/3}}$$

Southern part of India, Esp. Tamil Nadu

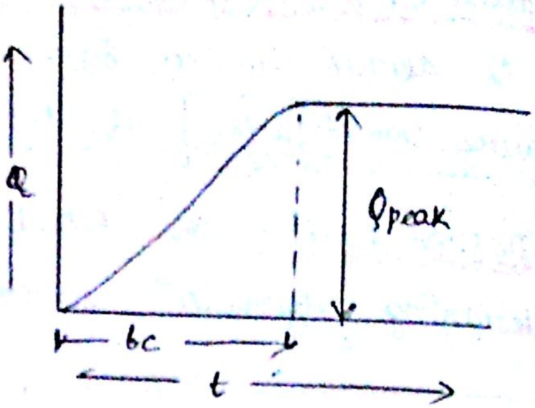
(iii) **Mr. Inglis formula** – (Western ghats) (Mainly for Maharashtra)

$$\boxed{Q_{peak} = \frac{124A}{\sqrt{A+10.4}}}$$

$A \to Km^2$

$Q \to$ cumec

$$\boxed{Q_{peak} = C i A}$$



$$\boxed{i = \frac{P}{t_c}}$$

$$\boxed{t_c = 0.01947 (L)^{0.77} (S)^{-0.385}}$$

L → length of catchment (m)

S → slope of catchment

t_c → minutes

$L = 950\,m \quad S = 0.006 \quad \Rightarrow t_c = 27.39$ min.

$A = 90$ Ha

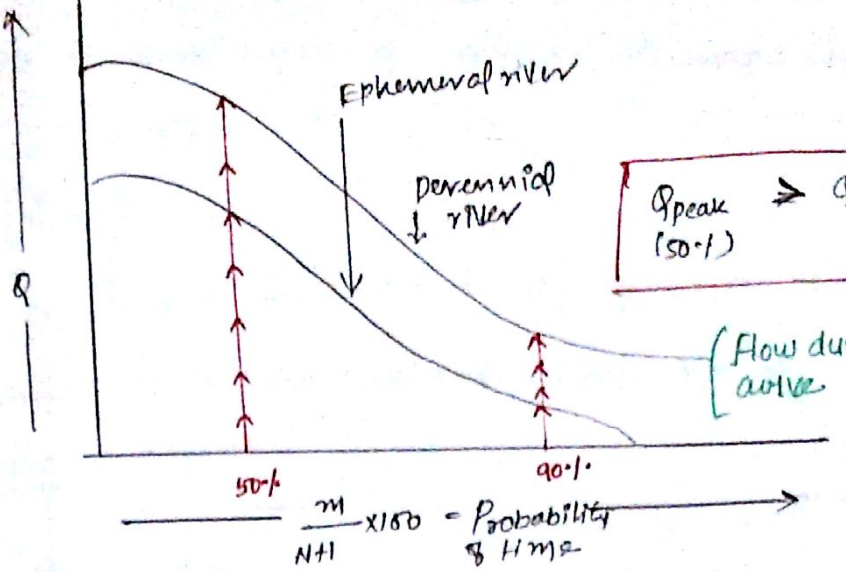
t (min)	0	5	10	15	20	25	30
P (cm)	0	6	8	12	16	21	28 cm

$25 \quad 21 + / \underline{(30-27)} \qquad 21 + \frac{(7)}{5} \times 2.39$

$\Rightarrow P$ ~~= 23.39~~

$= 24.3616$ cm

$Q_{peak} = CiA = 40$ cumec

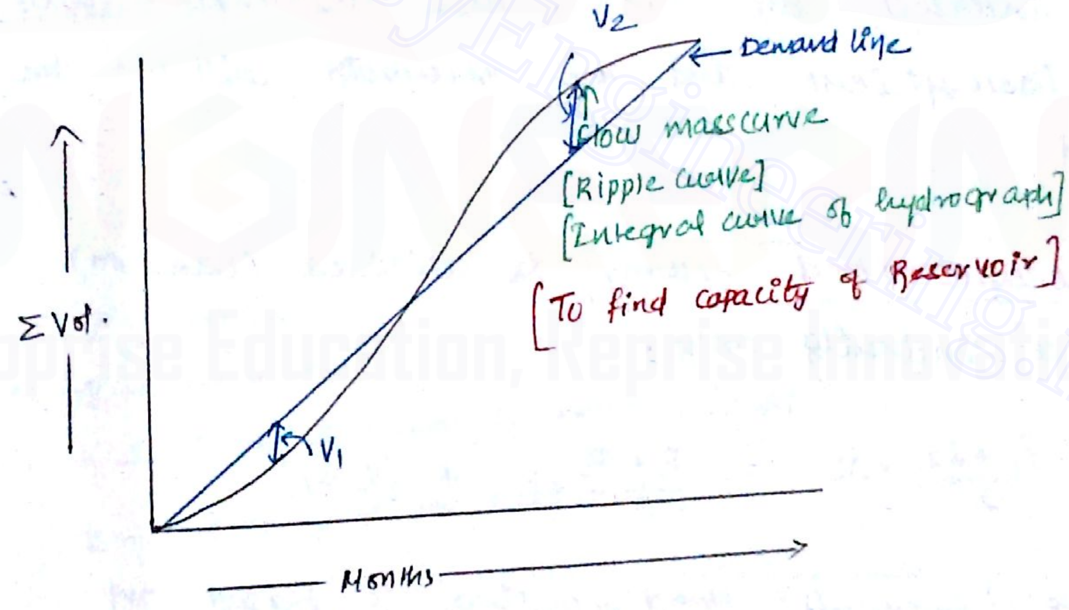


Q_{peak} (50%) > Q_{peak} (90%)

[Flow duration curve q and % time]

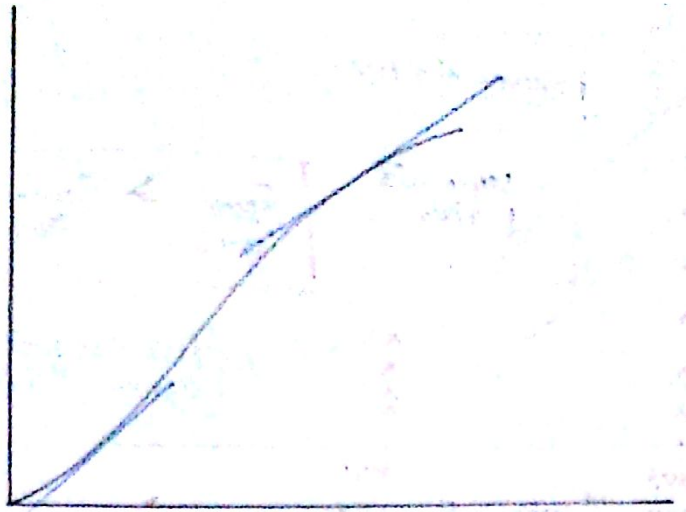
$$\boxed{P = \frac{1}{T} \times 100}$$

Weibull formula: $\boxed{T = \frac{N+1}{m}}$



Capacity of Reservoir = $V_1 + V_2$

Virgin flow - No human intervention (Natural flow)



If a demand line is drawn at the bottom-most of sag point and it does not intersect the flow mass curve at initial stage, then the reservoir will be Unfilled (dry) at initial stage.

If a demand line is drawn on the crest point (peak point) of the flow mass curve and it does not intersect the curve, then the inflow will be insufficient and the reservoir will not be filled.

⑦ Hydrologic flood routing is utilises (based on) Law of continuity only,

$$\frac{I_1+I_2}{2} \times \Delta t = \frac{Q_1+Q_2}{2} \times t + [S_2-S_1]$$

But the Hydraulic flood routing is based on continuity equation and equation of motion of unsteady flow. (BOM)

⑧ A

storage is a function of outflow discharge only, i.e

$S = f(Q)$

$\boxed{S \propto Q}$

→ But a [linear channel] is a type of channel where discharge is always constant and hence it is a fictitious channel.

**

⑨ Mr. Muskinghum theory is mainly applicable for flood routing through a natural channel (natural river) which is based on [Hydrologic channel routing], where the prism storage is function of outflow and the wedge storage (above prism) is a function of inflow in a channel and the total combination is called channel storage.

According to Mr. Muskinghum,

$$\boxed{S = K\left[x \underset{\downarrow \; I^m}{I^1} + (1-x) Q^1\right]}$$

I^m (m=1) by Muskinghum

I → Inflow

Q → outflow

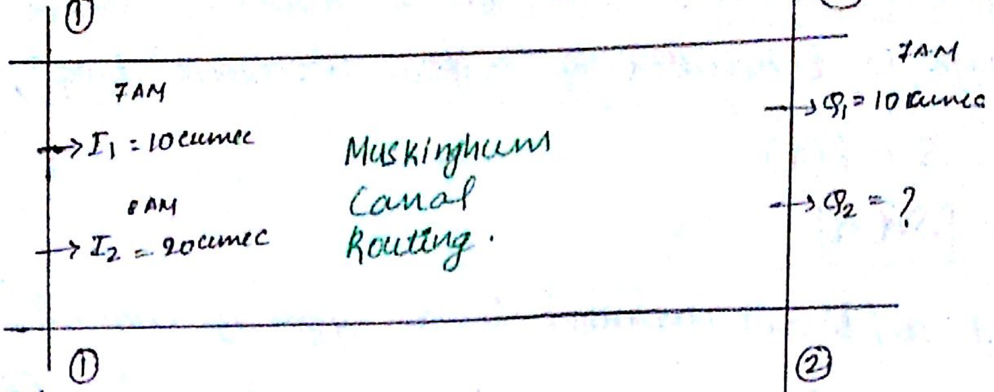
K → Time of travel of water (hours)

x → Weighing factor (No unit)

If $x = 0 \Rightarrow \boxed{S = KQ} \Rightarrow$ Linear Reservoir.

(func of outflow only).

Q ↓ outflow discharge

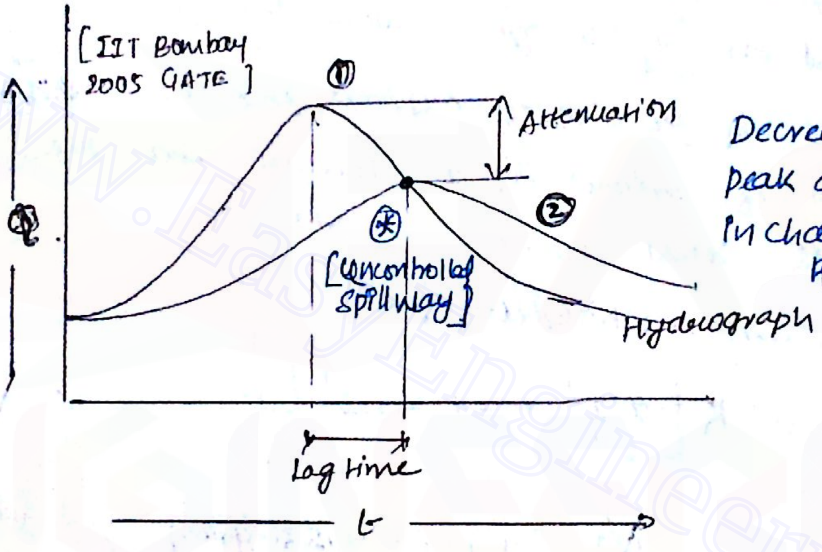


By Muskinghum

$$\boxed{Q_2 = C_1 I_1 + C_0 I_2 + C_2 Q_1} \Rightarrow \text{continuity equation along with storage equation}$$

$$\boxed{C_1 + C_0 + C_2 = 1} \qquad C_0, C_1, C_2 = f(\Delta t, K)$$

[IIT Bombay 2005 GATE]



Decrease in flood peak due to storage in channel (or) Reservoir.

The peak of outflow hydrograph lies at the intersection point with inflow hydrograph only when there is uncontrolled spillway

*****

FLOOD ROUTING

① Flood routing is the technique of determining the hydrograph at any section of the river with the help known data on upper stream of the section of river li inflow and storage.

② Flood routing is helpful to design ~~determine~~ –

(i) Capacity of spillway
(ii) Flood forecasting
(iii) Spillway length
(iv) Elevation of water on ~~spillw~~ the crest of spillway
(v) The surface area of the reservoir at known elevation of wa
(vi) Expected flood discharge on downstream on a particula elevation of water
(vii) Attenuation of flood hydrograph
(viii) The time lag of inflow & outflow hydrograph
(ix) Flood protection of downstream area.

③ There are 2 types of flood routing –

(a) Flood routing through a reservoir (Reservoir routing)
(b) Flood routing through a channel (Channel routing)

RESERVOIR ROUTING - (Level pod routing)

① Reservoir routing is the technique where the elevation of water surface above the crest of the spillway is determined with the help of inflow data like inflow discharge, duration of inflow, storage in reservoir and outflow discharge.

② Reservoir routing is adopted to design spillway a flood protection of downstream area

③ The method (basic logic) adopted in reservoir routing is Hydrolo routing, continuity equation (water Budget equation)

④ The data required for reservoir routing are as follows

(a) Inflow discharge into reservoir at time t_1 (I_1 at t_1)

(b) Inflow discharge at time t_2 (I_2 at t_2)

(c) Surface area of reservoir (Mean surface area)

(d) Water surface elevation at time t_1 (h_1 at t_1)

(e) Water surface elevation above the crest of the spillway at time t_2 (h_2 at t_2)

(f) Equation of outflow, $\boxed{Q = \frac{2}{3} C_d \, l_{eff} \sqrt{2g} \, h^{3/2}}$

h → head of water above spillway.

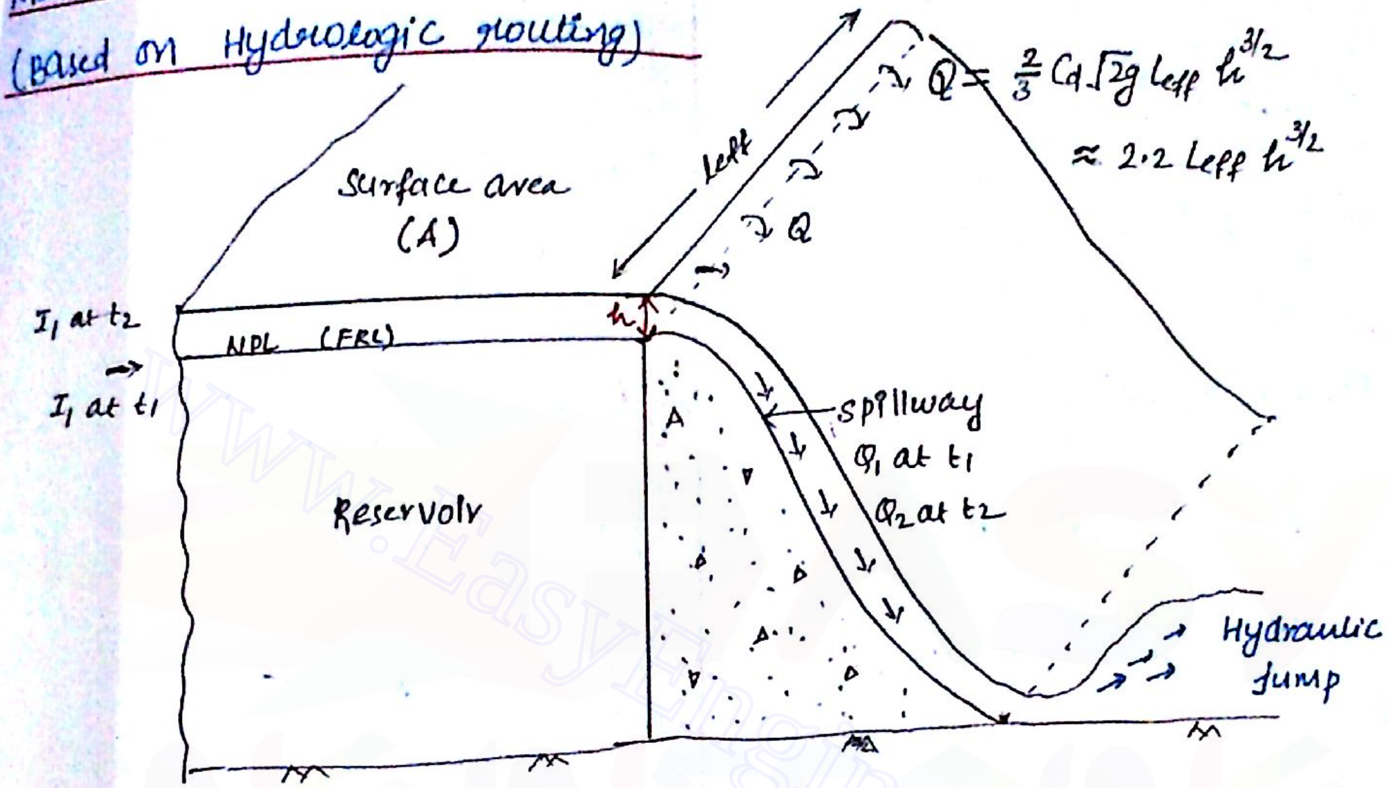
l_{eff} → effective length of water way above spillway by considering end contraction developed by Mr. Francis

($l_{eff} = L - 0.1H \times 2$)

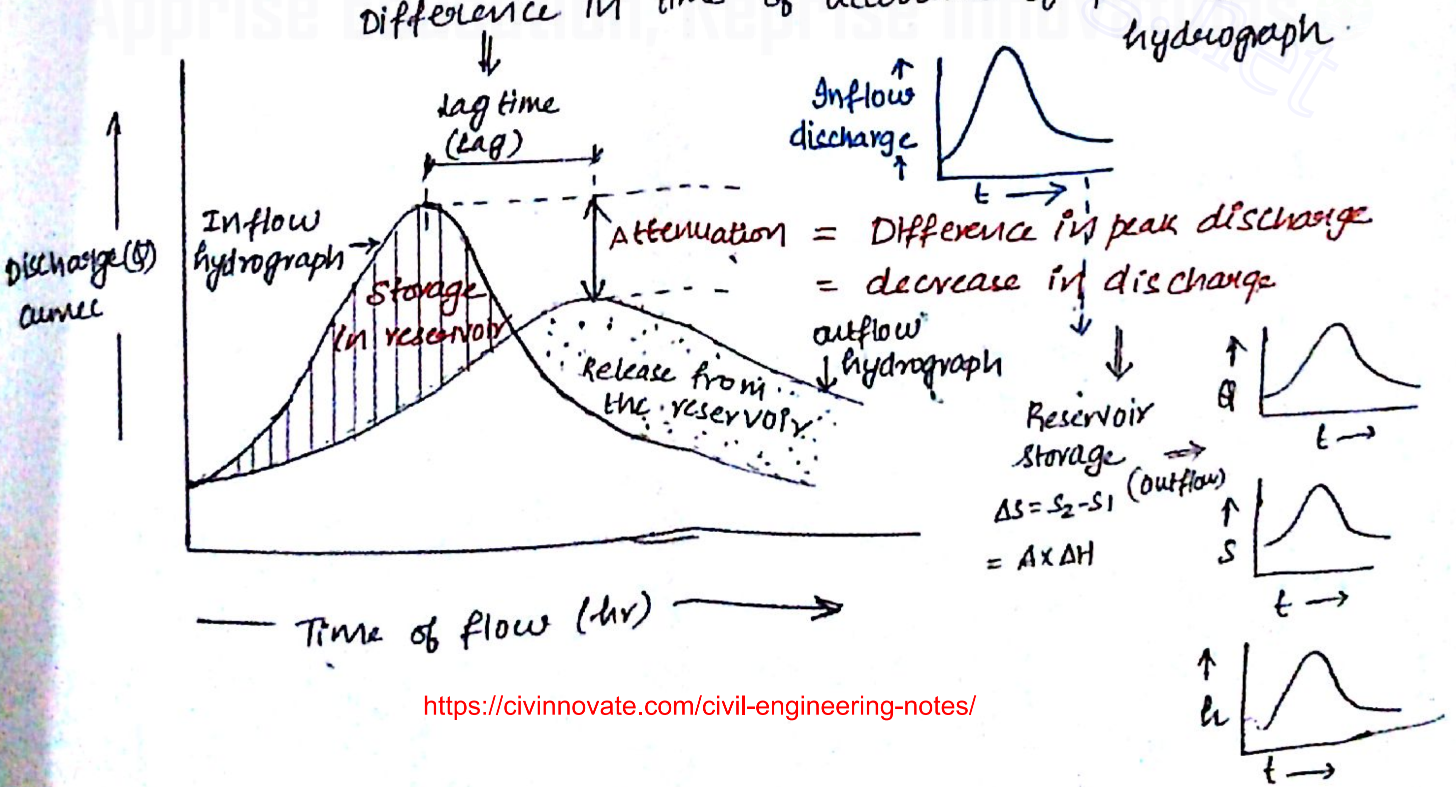
C_d → co-efficient of discharge over spillway (or) $Q = 2.2 \times l_{eff} \times h^{3/2}$ $\boxed{C_d = 0.74}$

(or) $q = 2.2 \times h^{3/2}$

Mathematical expression for Reservoir routing (Based on Hydrologic routing)



Difference in time of occurrence of peak of two hydrograph

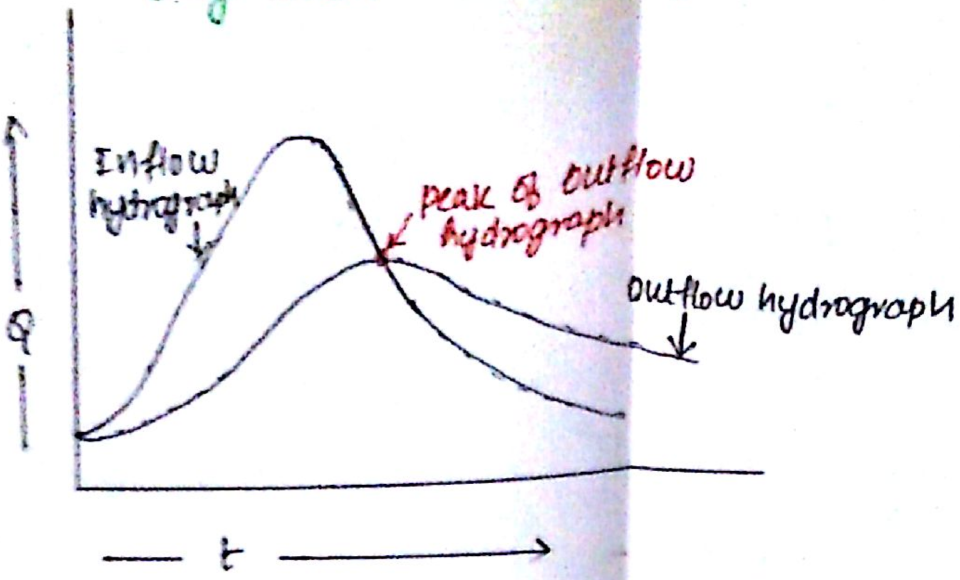


Difference in time of occurrence of peak of two hydrograph ⇓ Lag time

Attenuation = Difference in peak discharge
= decrease in discharge

$\Delta S = S_2 - S_1$
$= A \times \Delta H$

Only when the spillway is uncontrolled



Continuity equation (water Budget equation) (Hydrology equation)

$$I\Delta t = \bar{Q}\Delta t + \Delta S \qquad \boxed{\Delta t = t_2 - t_1}$$

$$\frac{I_1+I_2}{2}\times\Delta t = \frac{Q_1+Q_2}{2}\times\Delta t + (S_2-S_1)$$

$$\rightarrow \frac{I_1}{2}+\frac{I_2}{2} = \frac{Q_1}{2}+\frac{Q_2}{2} + \frac{(S_2-S_1)}{\Delta t}$$

$$\rightarrow \left(\frac{I_1}{2}-\frac{S_1}{\Delta t}\right) + \left(\frac{I_2}{2}+\frac{S_2}{\Delta t}\right) = \frac{Q_1}{2}+\frac{Q_2}{2}$$

$$\frac{I_1+I_2}{2}\Delta t = \left[\frac{CLh_1^{3/2} + CLh_2^{3/2}}{2}\right]\Delta t + \left[A_2h_2 - A_1h_1\right]$$

$$\rightarrow \boxed{h_2 = ?} \text{ at time } t_2$$

~~rcises~~ - ③ 2.052×10

① An reservoir has avg. surface area of 5000 Ha. The eff length of spill way is 20 m. The inflow at Initial stage is 50 cumec and after 6hr it becomes 140 cumec. Det. water surface elevation at crest of spillway, assuming initial outflow ~~ø~~ zero, Initial head zero and R.L. of Bed of reservoir is 155.55 m. Also determine the water surface elevation at end of 12th hr if inflow at 12th hr is 300 cumec. Take spillway constant = 2.

$$2.052\times10^6 = 4.752\times10^5\, h_2^{3/2} + 5000\times10^4\times h_2$$

$$\Rightarrow h_2 = 0.041\ m \qquad \underline{h_2 = 4.1\ cm}$$

Case ii.

$$4.752\times10^6 = (4.752\times10^5\, h^{3/2} + 3945.05) + [5000\times10^4\times h_2 - 5000\times10^4\times0.041]$$

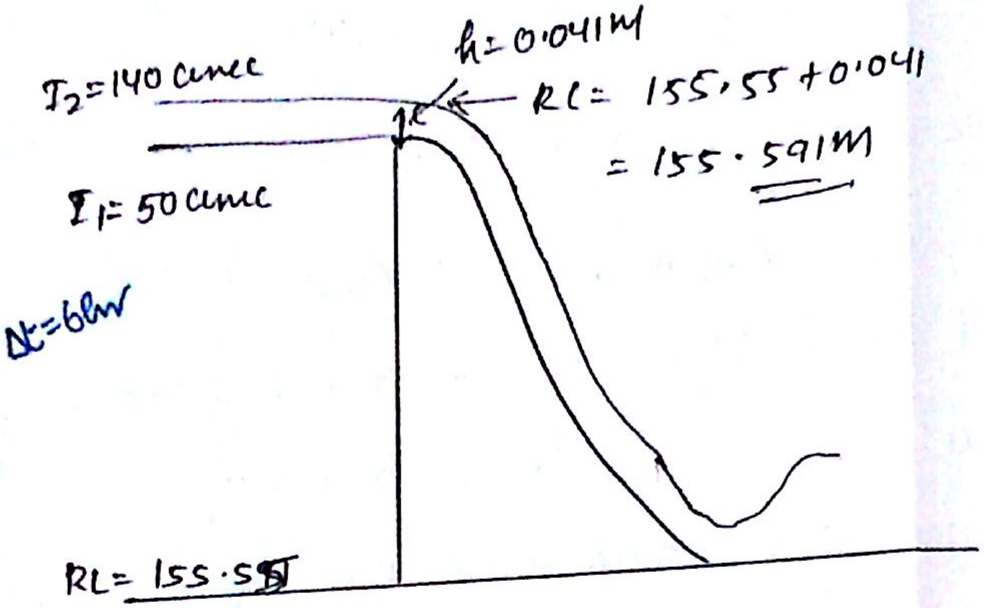
$$\boxed{2.698\times10^6}$$

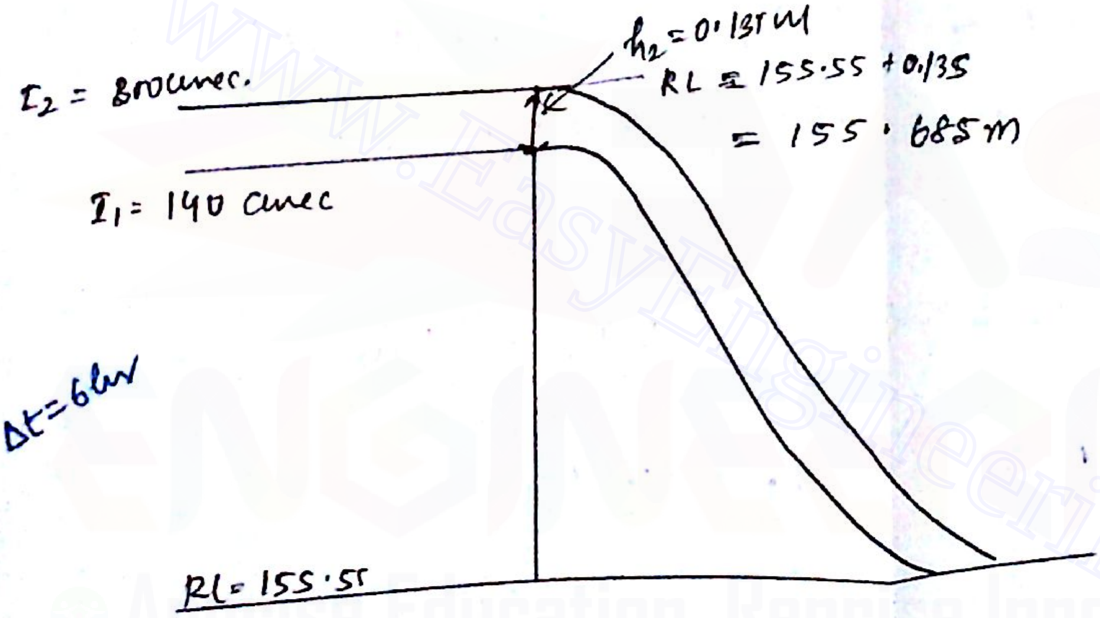
$$\underline{h_1 = 0.041 m}$$

$$\Rightarrow \cancel{h_2 = 0.05}$$

$$4.752\times10^5\,[0.041^{3/2} + h_2^{3/2}]$$

$$\boxed{h_2 = 0.135 m}$$


h= 0.041M
RL= 155.55+0.041
= 155.591M
I2=140 cumec
I1= 50 cumec
Δt=6hr
RL= 155.55


h2 = 0.135 m
RL = 155.55 + 0.135
= 155.685 m
I2 = 800 cumec.
I1 = 140 cumec
Δt=6hr
RL= 155.55

Channel Routing -

① In the channel routing the flood wave is studied, i.e the inflow hydrograph is determined at upper stream, which helps to determine the outflow hydrograph at any section towards downstream of a given reach.

In channel routing the attenuation and time lag are computed.

② Channel routing plays an important role in

(a) Flood forecasting
(b) Flood protection
(c) Flood plane zone

③ The channel routing was developed by Mr. Muskingham.

④ The flood flow in a channel comes under Gradually varied unsteady flow.

⑤ The storage in a channel is a function of inflow and outflow (But in case of reservoir, the unique storage, linear reservoir is a function of outflow discharge only)

⑥ There are 2 types of storage in a channel -

(a) Prism storage
(b) Wedge storage

⑦ **Prism Storage** - It is the storage in a channel obtained by considering a plane parallel to the bed of the channel at the outlet section.

The prism storage is a function of outflow only (like a linear reservoir), i.e storag

$$\boxed{S_{Prism} = f(Q)}$$

The prism storage is always positive, it cannot be negative.

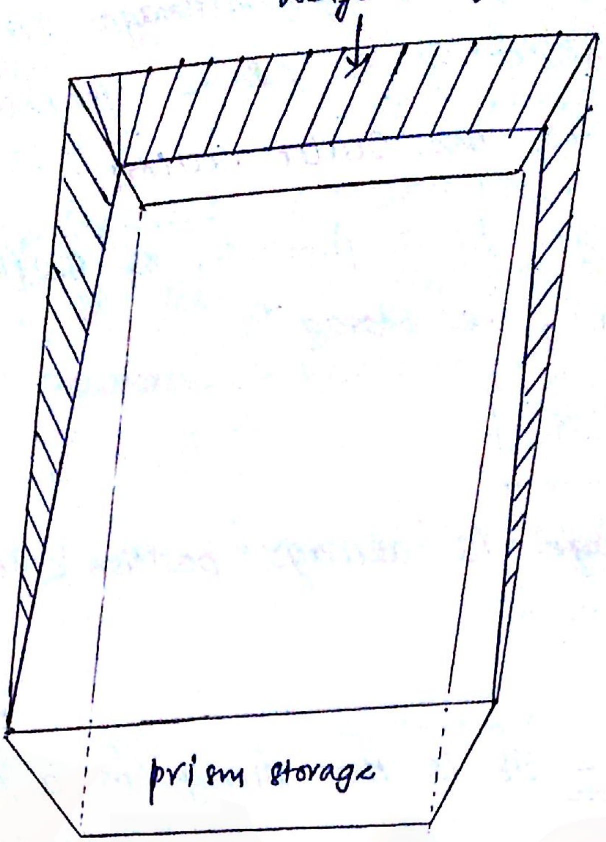
⑧ **Wedge storage** - It is the storage in a channel obtained between the top of prism storage and the top of the water surface in the channel. It is just like the shape of a wedge.

The wedge storage is ~~always~~ always a function of inflow only

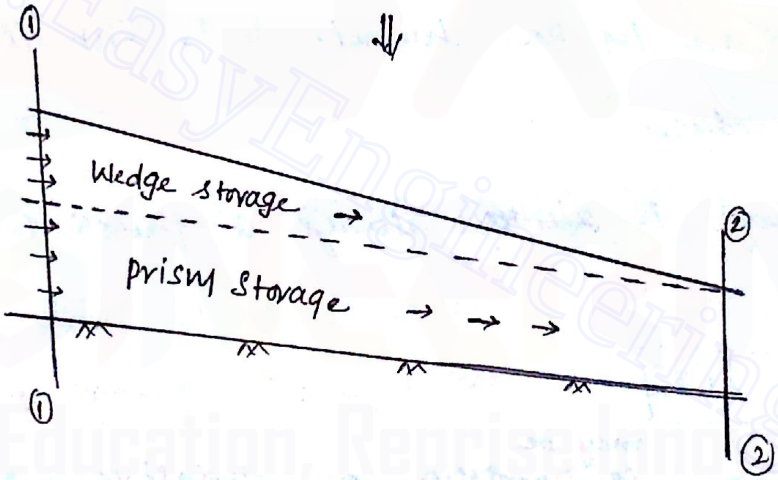
$$\boxed{S_W = f(I)}$$

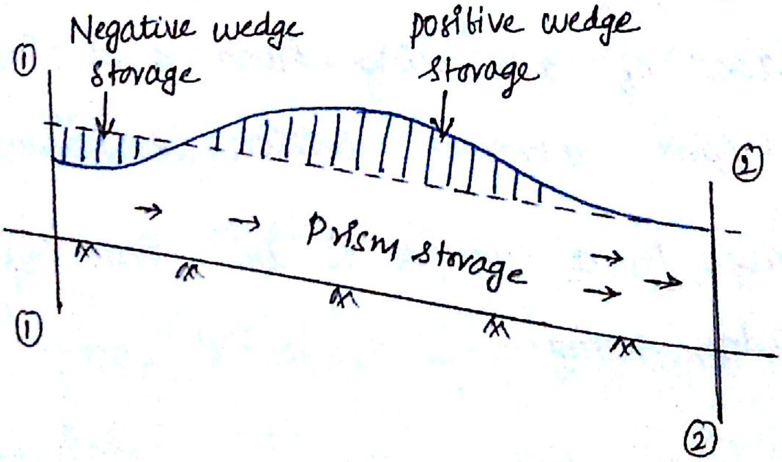
The wedge storage ~~is~~ may be positive or negative. It is positive in case of advancing flood and it is negative in upper stream section ($\overset{(-ve)}{\text{Backward}}$)

⑨ The total storage in a channel is the sum of prism storage and wedge storage.


prism storage

⇓


1
Wedge storage
prism storage
2
1
2


Negative wedge storage
positive wedge storage
1
2
Prism storage
1
2

$$S = S_P + S_W$$

$$= f(Q) + f'(I)$$

$$* \quad \boxed{S = K\left[x I^m + (1-x) Q^m\right]} \quad *$$

where, K → Time required by channel flow b/w 2 sectio of the channel having dimension of time (T

$\boxed{\text{Unit} \rightarrow hr}$

x → weighting factor having value 0 to 0.5

I → Inflow discharge at section-① of upper strean (cumec)

Q → Outflow discharge at section-② [cumec] [down stream section]

m → Exponent, having value = 1 → Natural channel

= 0.6 → Rectangular chann

Muskingham Flood Routing

① The Muskingham method of channel routing (or) (Muskingham method) is mainly applicable for the natural stream (natural channel) (river).

② In the Muskingham flood routing method, the value of exponent of equation of storage is always 1 (one)

$\boxed{m = 1}$

③ The Muskingham flood routing method (channel routing) is based on the storage equation,

$$S = K\left[x I^1 + (1-x) Q^1\right]$$

$\boxed{S = k[xI + (1-x)Q]}$ x → weighting factor (No dimension

In above formula the value of x lies in a range of 0 to 0.5.

If $x = 0 \Rightarrow S = k[0 \times I + (1-0)Q]$

$$\boxed{S = kQ}$$

$$\boxed{S = f(Q)}$$

Such storage, where the storage is a function of outflow discharge only, is known as a **Linear Reservoir**; where the storage is totally independent of inflow.

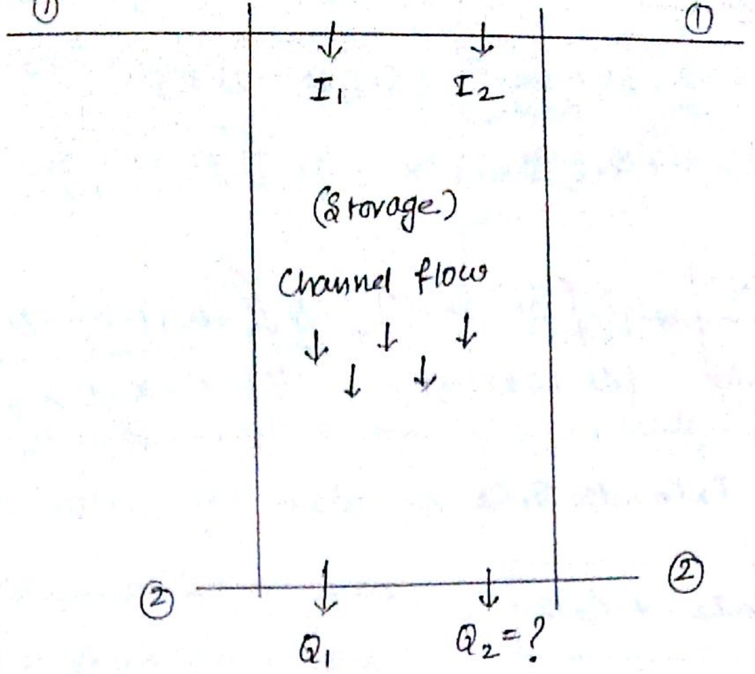
If $x = 0.5 \Rightarrow S = k[0.5I + 0.5Q]$,

then the storage is dependent on inflow as well as outflow discharge, i.e. Both inflow and outflow are of equal importance.

If $x = 0.5$, then the attenuation takes place.

(*) → A linear channel is a fictitious channel, where Attenuation is Zero. It means the inflow and outflow hydrographs are the same. (No storage) ~~Practica~~.

The Muskingum flood routing method is a channel routing, which is based on hydrologic flood routing. (water budget eqn.) (continuity eqn.) as given below.



Applying Water Budget equation, where the storage in the channel is the storage equation given by Mr. Muskinghum.

$$\frac{I_1+I_2}{2} \times \Delta t = \frac{Q_1+Q_2}{2}\Delta t + (S_2 - S_1)$$

$$(I_1+I_2)\Delta t = (Q_1+Q_2)\Delta t + 2\left[K\{xI_2 + (1-x)Q_2\} - K\{xI_1 + (1-x)Q_1\}\right]$$

$$I_1\Delta t + I_2\Delta t = Q_1\Delta t + Q_2\Delta t + 2KxI_2 + 2K(1-x)Q_2 - 2KxI_1 - 2K(1-x)Q_1$$

$$I_1\Delta t + I_2\Delta t = Q_1\Delta t + 2KxI_2 - 2KxI_1 - 2K(1-x)Q_1 + Q_2\left[2K(1-x) + \Delta t\right]$$

$$Q_2\left[2K(1-x) + \Delta t\right] = -Q_1\Delta t - 2KxI_2 + 2KxI_1 + 2K(1-x)Q_1 + I_1\Delta t + I_2\Delta t$$

$$Q_2\left[2K(1-x) + \Delta t\right] = Q_1\left[2K(1-x) - \Delta t\right] + 2Kx\left[I_1 - I_2\right] + \left[I_1 + I_2\right]\Delta t$$

$$= Q_1\left[2K(1-x) - \Delta t\right] + I_2\left[\Delta t - 2Kx\right] + I_1\left[\Delta t + 2Kx\right]$$

$$Q_2[\Delta t + 2k(1-x)] = I_1[\Delta t + 2kx] + I_2[\Delta t - 2kx]$$
$$+ Q_1[2k(1-x) - \Delta t]$$

$$Q_2 = I_1\left[\frac{\Delta t + 2kx}{\Delta t + 2k(1-x)}\right] + I_2\left[\frac{\Delta t - 2kx}{\Delta t + 2k(1-x)}\right] + Q_1\left[\frac{2k(1-x) - \Delta t}{\Delta t + 2k(1-x)}\right]$$

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

$$\boxed{Q_2 = C_1 I_1 + C_0 I_2 + C_2 Q_1}$$

where $c_1 = \dfrac{\Delta t + 2kx}{\Delta t + 2k(1-x)}$

$$c_0 = \frac{\Delta t - 2kx}{\Delta t + 2k(1-x)}$$

$$c_2 = \frac{2k(1-x) - \Delta t}{\Delta t + 2k(1-x)}$$

$$C_0 + C_1 + C_2 = \frac{\cancel{\Delta t} + 2\cancel{k}x + \Delta t - 2\cancel{k}x + 2k(1-x) - \cancel{\Delta t}}{\Delta t + 2k(1-x)}$$

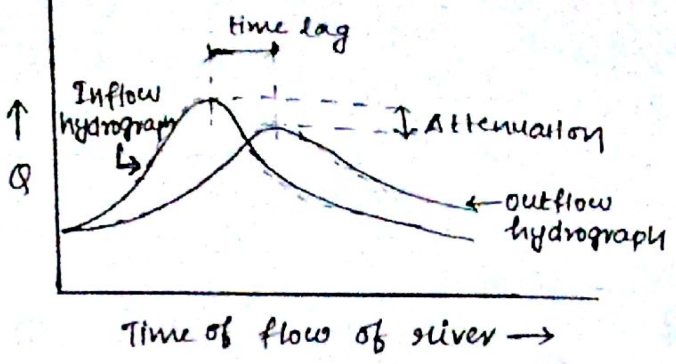
$$= \frac{\Delta t + 2k(1-x)}{\Delta t + 2k(1-x)}$$

$$\boxed{C_0 + C_1 + C_2 = 1}$$

$$\boxed{C_2 = 1 - C_1 - C_0}$$

For the best result,

$$\boxed{K > \Delta t > 2kx}$$



Numerical -

(14)

① Determine the outflow hydrograph (Route the flood through a channel) with the help of foll. data -

(i) Time constant (K) = 12 hrs.

(ii) weighting factor (x) = 0.2

(iii) Inflow hydrograph

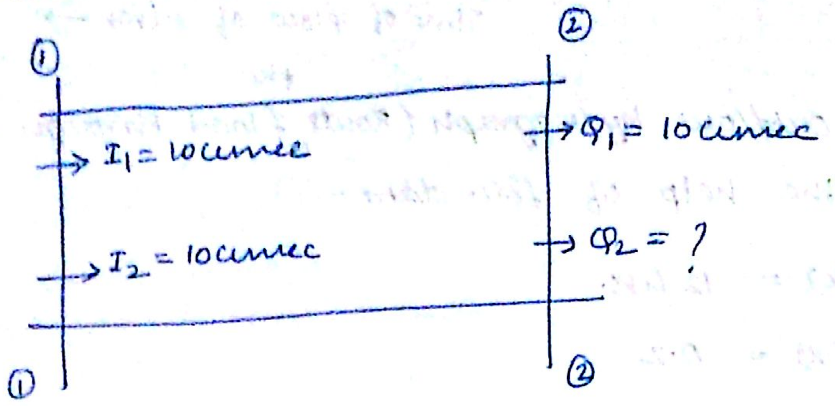
Time (hr)	0	6	12	18	24	30	36	42	48	54
Discharge (cumec)	10	20	50	60	55	45	35	27	20	12

Assume initial outflow as 10 cumec.

Soln. $C_1 = \frac{6 + 2\times 12\times 0.2}{6 + 2\times 12(1-0.2)} = 0.428 = 3/7$ $C_0 = \frac{6 - 2\times 12\times 0.2}{6 + 2\times 12(1-0.2)} = \frac{1}{21} = 0.048$ $C_2 = \frac{2\times 12(1-0.2) - 6}{6 + 2\times 12(1-0.2)} = 11/21 = 0.524$

Time (hr)	discharge	$C_1 I_1$	$C_0 I_2$	$C_2 Q_1$	Q_2
0	10 I_1	—	—	—	10 cumec
6	20 Q_1	4.28	0.96	5.24	10.48 10.
12	50	8.56	2.4	5.49	16.45
18	60 = x(1)	21.4	2.88	8.62	32.89 (Q_1)
24	55	25.68	2.64	17.29	45.55
30	45	23.54	2.16	23.87	49.57 Soln
36	35	0.428×45 ~~14.98~~	0.048×35 1.29	0.124×49.17 25.97	46.92 ~~46.24~~
42	27	~~11.55~~	0.96	22.13	40.86 ~~34.64~~
48	20	8.56	0.576	18.15	~~27.286~~ 33.92
54	12	~~8~~.			26.91

Eg Trial-①

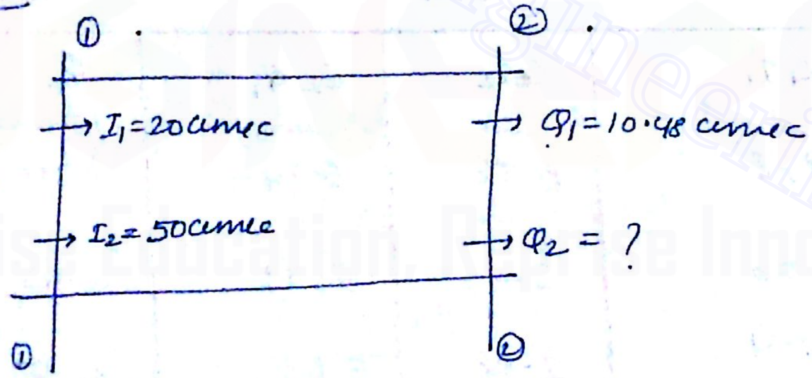


$$Q_2 = C_1 I_1 + C_0 I_2 + C_2 Q_1$$

$$= 0.428 \times 10 + 0.048 \times 20 + 0.524 \times 10$$

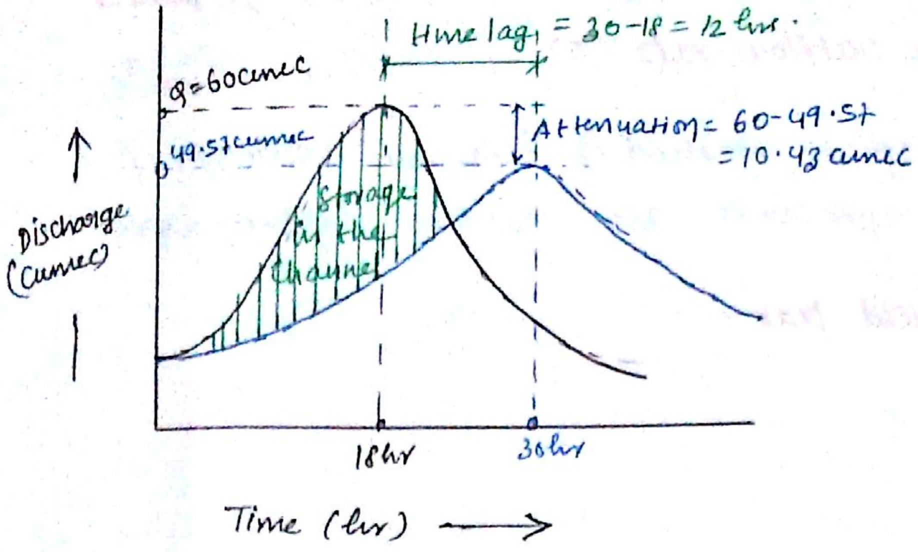
$$= 10.48 \text{ cumec}$$

Trial-②



$$Q_2 = 0.428 \times 20 + 0.048 \times 50 + 0.524 \times 10.48$$

$$= 16.45 \text{ cumec}$$ (low discharge due to storage)



IMPORTANT OBJECTIVES -

① The prism storage in a river during the passage of a flood wave is outflow function only.

② The wedge storage in a river during the passage of a flood wave is positive during rising phase.

③ The Muskinghum method of flood routing assumes the storage, $S = K[xI + (1-x)Q]$

④ If the Muskinghum method of channel routing is $x = 0.15$, $K = 12$ hrs, $\Delta t = 4$ hrs, the co-efficient C_0 is -

$$C_0 = \frac{\Delta t - 2Kx}{\Delta t + 2K(1-x)} = \frac{4 - 3.6}{4 + 24(1-0.15)} = \frac{0.4}{4 + 20.4}$$

$$\boxed{C_0 = 0.016}$$

24 × 0.15
240
3.6

24
0.15
120
24
3.60

24
× 0.85
120
192
20.40

$\frac{0.4}{24.4}$

$\frac{4}{244}$ $\frac{1}{61}$

0.016

61) 100 (.016
61
390

⊛

⑤ A linear Reservoir is one, in which the storage varies nearly with the outflow rate (Q) $(x=0) \Rightarrow (\underline{\underline{S = kQ}})$

⊛

⑥ In the Muskinghum method of channel routing, if $x = 0.5$, it represents an outflow hydrograph that has Reduced peak.

Civinnovate

Discover, Learn, and Innovate in Civil Engineering