

## Introduction:

Floods :- Floods are natural occurrences where an area of land that is normally dry abruptly becomes submerged in water. In simple terms, flood can be defined as an overflow of large quantities of water onto a normally dry land. Flooding happens in many ways due to overflow of streams, rivers, lakes & oceans & as a result of Excessive rain.

→ Whenever flooding takes place, there is the possibility of loss of life, hardship to people, and Extensive damage to property. This is because flooding can carry bridges, cars, houses, and Even people. Flooding also destroys crops and can wipe away trees and other important structures on land. Some floods occur abruptly and recede quickly whereas others take several days & Even months to form and to recede because of Variation in size, duration, and the area affected.

\* "European Union Floods Directive" defines a flood as a covering by water of land not normally covered by water. In the sense of "flowing water", the word may also be applied to the inflow of the tide.

## \* Causes of Flooding:

→ Many conditions result in flooding. Hurricanes, clogged drainages, and rainfall are some of the conditions that have led to flooding in various regions across the globe. Here are the leading causes of flooding.

### 1. > Rain:

→ Rain is the leading contributor to most of the flooding cases witnessed across the world. Too much rain causes water to flow overland contributing to flooding. In particular, It is due to high rainfall intensity over a prolonged period.

→ Depending on the rainfall distribution, the amount of rain, and soil moisture content, short rainfall period can also result in flooding. Light rains for longer periods - several days & weeks, can also result in floods. The rain water erosive force can weaken the foundations of buildings, causing tumbles and cracks.

## 2. River overflows:-

→ Rivers & streams can overflow their banks. This happens when the river & stream holds more water upstream than usual, and it flows downstream to the neighbouring low-lying areas, typically referred to as the flood plains. As a consequence, this creates a sudden discharge of water into the adjacent lands leading to flooding.

Dams in rivers may also at times overwhelm rivers when the carriage capacity is exceeded, causing the water to burst and get into the flood plains. Flood caused by river overflow has the potential of sweeping everything in its path downstream.

## 3. Lakes and coastal flooding:-

→ Lake and coastal flooding occurs when large storms & tsunamis cause the water body to surge inland. These overflows have destructive power since they can destroy ill-equipped structures to withstand water's strength such as bridges, houses and cars.

In the coastal areas, strong and massive winds and hurricanes drive water on to the dry coastal lands and give rise to flooding. The situation is even worsened when the winds blowing from the ocean carry rains in them. Sea water from the tsunami & hurricane can cause widespread damage.

## 4. Dam Breakage:-

→ Dams are man-made structures used to hold water from flowing down from a raised ground. The potential energy stored in the dam water is used to generate electricity. At times, the walls can become weak and break because of overwhelming carriage capacity. Due to this reason, breakage of the dam can cause extensive flooding in the adjacent areas.

Flooding occurs when the embankments built along the sides of the river to stop high water from flowing onto the land break. Sometimes, the excess water from the dam is deliberately released from the dam to prevent it from breaking thereby causing floods.

## 5. Melting of the Glaciers and Mountain tops:

→ In the cold regions, ice and snow build up during the winters. When the temperature rises in summer, the accumulated (snows) snows and ice are subjected to melting resulting in vast (moments) movements of water into lands that are normally dry. Regions with mountains that have ice on top of them also (provided) experience the same outcome when the atmospheric temperature rises. This type of flooding is usually termed as snowmelt flood.

## 6. Clogged drainages:

→ Flooding also takes place when snowmelt or rainfall runoff cannot be channeled appropriately into the drainage systems forcing the water to flow overland. Clogged or lack of proper drainage system is usually the cause of this type of flooding.

\* The areas remain flooded until the stormwater systems or waterways are rectified. Instances where the systems or waterways are not rectified, the areas remain flooded until the excess water evaporates or is transpired into the atmosphere by plants.

## \* Effects of floodings:

### → primary Effects:

- > The primary effects of flooding include loss of life and damage to buildings and other structures, including bridges, sewerage systems, roadways and canals.
- > Floods also frequently damage power transmission and sometimes power generation, which then has knock-on effects caused by the loss of power.
- > This includes loss of drinking water treatment and water supply, which may result in loss of drinking water or severe water contamination.
- > It may also cause <sup>the</sup> loss of sewage disposal facilities.
- > Lack of clean water combined with human sewage in the flood waters raises the risk of waterborne diseases, which can include typhoid, giardia, cryptosporidium, cholera and many other diseases depending upon the location of flood.
- > Damage to road and transport infrastructure may make it difficult to mobilize aid to those affected or to provide emergency health treatment.

→ Flood waters typically inundate (flood) farmland, making the land unworkable and preventing crops from being planted or harvested, which can lead to shortages of food both for humans and farm animals. Entire harvests for a country can be lost in extreme flood circumstances. Some tree species may not survive prolonged flooding of their root systems.

→ Secondary and long-term effects:

> Economic hardship due to a temporary decline in tourism, rebuilding costs, or food shortages leading to price increases is a common after-effect of severe flooding. The impact on those affected may cause psychological damage to those affected, in particular where deaths, serious injuries and loss of property occur.

> Urban flooding can cause chronically wet houses, leading to the growth of mold and resulting in adverse health effects, particularly respiratory symptoms.

> Urban flooding also has significant economic implications for affected neighbourhoods

- In the United States, industry experts estimate the wet basements can lower property values by 10-25% and are cited among the top reasons for not purchasing a home.
- According to the U.S. Federal Emergency Management Agency (FEMA), almost 40% of small businesses never reopen their doors following a flooding disaster.
- In the United States, insurance is available against flood damage to both homes and businesses.

\* Benefits:

- > floods can also bring many benefits, such as recharging ground water, making soil more fertile and increasing nutrients in some soils.
- > Flood waters provide much needed water resources in arid and semi-arid regions where precipitation can be very unevenly distributed throughout the year and kills pests in the farming land.

- > Freshwater floods particularly play an important role in maintaining Ecosystems in river corridors and are a key factor in maintaining flood plain biodiversity.
- > Flooding can spread nutrients to lakes and rivers, which can lead to increased biomass and improved fisheries for a few years.

(> ~~for~~ some fish species, an undervalued)

- > periodic flooding was essential to the well-being of ancient communities along the Tigris - Euphrates River, the Nile River, the Indus River, the Ganges and the Yellow River among others. The viability of hydropower, a renewable source of energy, is also higher in flood prone regions.

### \* Flood Frequency Studies:

→ Flood frequency:- It denotes the likely hood of a flood being equal (or) exceeded.

→ Recurrence Interval ( $T_{RI}$ ):- Denotes the no. of years in which a given flood can be expected once.

$$T_{RI} = \frac{100}{F}$$

→ calculation of R.I.:-

1. California Method:-  $T_{RI} = \frac{N}{M}$

2. Hazen's Method:-  $T_{RI} = \frac{2N}{2m-1}$

3. Weibull's Method:-  $T_{RI} = \frac{N+1}{m}$

4. Gumbel's Method:-  $T_{RI} = \frac{N}{m+c-1}$

c → Gumbel's correction

\* depends on  $\frac{m}{N}$  ratio.

### \* Gumbel's correction:

$\frac{m}{N}$	1	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2	0.1	0.08	0.06
c	1	0.95	0.88	0.865	0.78	0.73	0.66	0.59	0.52	0.4	0.38	0.28

→ probability of occurrence (p) :-

> the probability of Event being Equal & Exceeded in any one year is the probability of occurrence (p) has for a period of  $T_r$  years.

$$p = \frac{1}{T_r}$$

\* the probability that the flood will not occur in an year is known as probability of Non occurrence.

$$q = 1 - p$$

Examples

\* the following flood discharge data is available at a catchment. Estimate ' $T_r$ ' for a flood of  $6000 \text{ m}^3/\text{sec}$  - use all formulae.

Year	flood discharge ( $\text{m}^3/\text{s}$ )	data in order	Rank
1991	10,200	11,640	1
92	6000	10,200	2
93	7200	9600	3
94	2500	8270	4
95	9600	7200	5
96	5400	6000	6
97	3600	5400	7
98	4950	4950	8
99	8270	3600	9
2000	11,640	2500	10

i) California Method,  $T_r = \frac{N}{m}$

$$T_r = \frac{10}{6} = 1.67 \approx 2 \text{ years}$$

ii) Hazen's Method,  $T_r = \frac{N+1}{m}$

$$T_r = \frac{11}{12-1} = \frac{11}{11} = 1 \text{ year}$$

(6)

iii) Weibull's Method,  $T_r = \frac{11}{6}$

$$T_r = 1.83 \text{ years.}$$

iv) Gumbell's Method,  $\frac{M}{N} = \frac{6}{10} = 0.6 \Rightarrow C = 0.78$

$$T_r = \frac{10}{6 + 0.78 - 1}$$

$$T_r = 1.73 \text{ years}$$

\* Rational formulae

$$Q = KAR \text{ (i), } Q = CIR$$

$$Q = F_c KAR$$

$F_c \rightarrow$  factor depending on units taken.

'K' varies for different regions

$$K_{eff} = \frac{K_1 A_1 + K_2 A_2 + \dots + K_N A_N}{A_1 + A_2 + \dots + A_N}$$

\* Estimation of flood discharges.

→ Design flood: - It is the flood discharge adopted for the design of a hydraulic structure after careful consideration of hydrological aspects and design aspects.

→ Standard project flood: (SPP)

> the flood obtained by severe combination of Meteorological and hydrological factors.

Extreme combinations are not considered.

→ probable Maximum flood:

> here Extreme flood physically on a given catchment is considered.

Most severe combinations of hydrological and Meteorological factors are considered. Rare combinations are also considered.

Methods of Estimation of flood as follows. they are,

1) By physical indicators of past flood.

2) By using Empirical formulae - Flood discharge formulae.

3) Flood frequency service.

4) By unit graph method

5) By rational method.

\* By physical indicators of past floods

→ Marks/identification on surrounding areas

→ Investigation of surrounding people.

→ From the dimension of rivers, mean velocity discharge can be calculated.

\* Empirical formulae

→ the Equations of the form,  $Q = kA^n$

$k$  → flood discharge coefficient.

Dickers formulae

$$Q = CA^{3/4}$$

used for North central and Eastern India.

$C'$  depends on catchment location.

Central India:- 13.9-19.5

Eastern India:- 22.2-25

Northern India:- 11.4

Ryvels formulae

$$Q = CA^{2/3}$$

used for Madras catchment.

$C'$  depends on location.

Area within 24 km from the coast - 6.45

Area within 24-161 km from coast - 8.45

limited area near hills - 10.1

Inglis formulae

$$Q = \frac{123A}{\sqrt{A+10.4}} \approx 123\sqrt{A} \text{ (approx)}$$

→ this is applicable for former Bombay presidency.

⑧

### Nawab Jung Bahadur formula:

$$Q = CA(0.993 - \frac{1}{16} \log A)$$

C value ranges from 40-60

applicable to old Hyderabad catchments.

### Fanning's formula:

(F.P.S)

$$Q = CA^{5/6}$$

applicable for American catchments.

$$C = 2.54$$

### Peller's formula:

$$Q = CA^{0.8} (1 + 0.8 \log T) (1 + 2.67A^{-0.2})$$

T → No. of years after which such a flood is to reoccur.

Q → Max. discharge ( $m^3/s$ )

C → (0.185 to 1.3)

A → Area of catchment in  $km^2$

### \* Methods used to control floods:

→ floods can cause many problems and in serious cases, lives may be taken as well.

this is why it is important to take preventive measures to stop the floods from happening in the first place. Flood control is referred to as measures taken to prevent floods from happening.

\* Flooding has many negative impacts, from damaging property to even taking lives. Flooding also transports other sediments to other places. This pollutes the habitats that wildlife may reside in. If the floods are to make their way into urban areas then it may cause disruption to traffic, interfere with drainage and electrical systems. This causes millions of dollars in damage. Thus, it would be less costly and safe to prevent the flood from happening in the first place.

## \* Methods used to prevent floods: Structural Methods ↪

→ "Dams" are one way that can be used for flood control. Dams are barriers that control the flow of water from a big water source such as a river or reservoir. Unlike other barriers, dams are used to retain water. The advantage with dams is that they are able to generate electricity through hydropower.

→ Another type of method of flood control that can be used would be "flood gates". Flood gates are systems that have adjustable gates to control the flow rate of a river. The water can either be stored or routed depending on the situation. Also, flood gates can also lower the water levels from canal channels or the main river channel. This allows more water to flow into a storage area if a flood is being predicted.

→ The next method that can be used for flood control would be a "flood wall". Similar to dams, flood walls serve the purpose of containing water of rivers or other water channels. However, flood walls are only temporary. They are used in areas where there is limited space or if a construction of other barriers would interfere with the surrounding environment. Flood walls are made out of fabricated concrete materials. These flood walls may sometimes have flood gates which allow people and vehicles to pass through. They are only closed in case of a flood.

→ In Asian countries, "flood diversion" is used to divert the flood away from more populated cities.

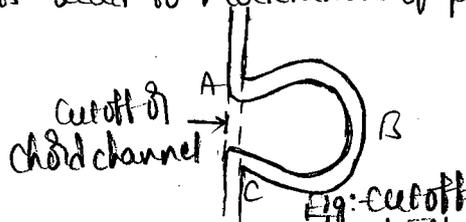
→ To control coastal flooding, there are some methods such as sea walls, beach nourishment and barrier islands that can be built along to coast to prevent flooding from the ocean.

> Sea walls may corrode over time and collapse.

> Beach nourishment is the addition of sand to the existing coast so that the tides will not reach the populated areas that quickly. However, it is a costly method that takes a considerable amount of time to complete.

## → Water Shed Management:-

- > Land treatment and watershed management in the basin aims to cutting down and delaying the runoff before it gets into the river. watershed management measure includes developing the vegetative and soil cover in conjunction with land.
- > Treatment works like check dams, contour bunding, terraces etc. these treatment cause increase in infiltration, increase in evapotranspiration and reduction in soil erosion. these all treatments lead to moderation of peak flows and increasing of dry weather flows.



## \* Non-structural method:-

- (a) Flood plain zoning:- when the channel discharges are very high, it is to be expected that the channel will overflow its banks and spill into the flood plains. flood plain management identifies the flood prone areas of a river and regulates the land use so that it restricts the damage due to flood.

Zone	Flood Return period	Example of uses
1	100 years	Residential houses, offices, factories etc.
2	25 years	Parks
3	Present	No construction/Encroachments.

- (b) Flood Forecasting and warnings Forecasting of floods should be done in advance, so that it enables us to give warning to the affected people and take appropriate precautionary measures. These techniques can be divided on the basis of time of forecasting.

- (i) Short-range forecasts:- This method gives advance warning 12-48 hours for flood.
- (ii) Medium-range forecasts:- This method gives advance warning of 2-5 days for floods, by using rainfall-runoff relationship.
- (iii) Long-range forecasts using ocean and meteorological satellite data, time of occurrence of event are predicted well in advance.

## (c) Evacuation and Relocation

- (d) Flood Insurance: Flood insurance provides a mechanism to modify the impact of loss burden. (11)

## \* Types of flood:-

→ Large area flood

→ Small area flood.

### (i) Large area flood:-

→ This type of flood occur due to storm of low intensity having a duration of a few days to several weeks. Sometimes, snow melt may also be a reason to cause large area floods. The time period of maximum total precipitation do not necessarily coincide with the time of occurrence of large area floods.

### (ii) Small Area flood:-

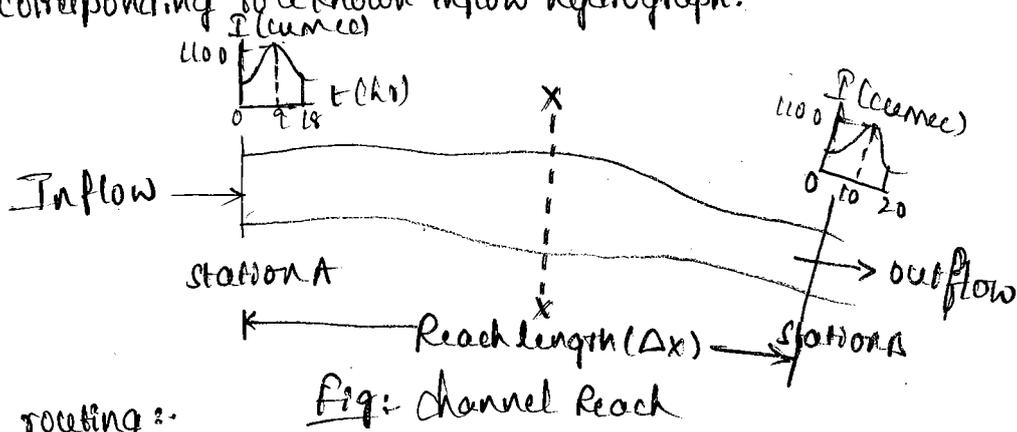
→ Storm of high intensity and duration of one day or less causes small area floods.

Such flows cause good damage to agricultural land in the form of excessive soil erosion. This is the major cause of sedimentation in reservoir and river. In this type, the total flood volume is not much but such floods cause serious local area damage.

→ The importance of structure and economic development of surrounding area. dictate the design criteria for choosing the flood magnitude.

### \* Flood routing:-

→ Flood routing is a process of determining peak discharge or stage of outflow hydrograph, corresponding to a known inflow hydrograph.



### \* Uses of flood routing:-

- In Estimation of design flood.
- In designing of reservoir.
- In designs of flood control structure.
- In determining adequacy of spillway
- In study of flood wave.

### \* flood wave:-

→ flood wave in open channels are also recognised as long progressive waves with the only difference that the movement of water in downstream direction, is also associated with the movement of water in open channel or reservoir.

- By associating flood wave with water movement give two type of flood waves:
  - > the wave which travel downstream is known as primary wave.
  - > the wave which travel upstream is known as Secondary wave.

### \* Types of flood routing:-

→ on the basis of application, flood Routing can be classified as

- (i) Reservoir Routing
- (ii) channel Routing

### (i) Reservoir Routing:

→ In reservoir routing the effect of a flood wave entering a reservoir is studied to predict the variations of reservoir elevation and outflow discharge with time.

this form of routing is used:

- (a) In the design of capacity of spillways and other reservoir-outlet structures.
- (b) In locating and sizing of the capacity of reservoir to meet specific requirements.

### (ii) Channel Routing:

→ In channel routing the change in the shape of a hydrograph as it travel down a channel is studied to predict the flood hydrograph at various section of the reach.

### \* Various Method of Reservoir Routing:-

→ Modified pul's method:-

> The equation used before is rearranged as

$$\left(\frac{I_1 + I_2}{2}\right) \Delta t + \left(S_1 - \frac{O_1 \Delta t}{2}\right) = \left(S_2 + \frac{O_2 \Delta t}{2}\right)$$

At the starting of flood routing, the initial storage and outflow discharge are known, which means all the terms of left hand side are known.

> The various steps are to be taken:

(i) For the 1<sup>st</sup> time interval  $\Delta t$ ,  $\left(\frac{I_1 + I_2}{2}\right) \Delta t$  and  $\left(S_1 - \frac{O_1 \Delta t}{2}\right)$  are known and hence.

$\left(S_2 + \frac{O_2 \Delta t}{2}\right)$  is determined.

(ii) The water surface elevation corresponding to  $\left(S_2 + \frac{O_2 \Delta t}{2}\right)$  is found by using curve of storage - Elevation and discharge Elevation. The outflow discharge  $O_2$  at the end of time step  $\Delta t$  is found from the curve outflow discharge - Elevation.

(iii) Deducting  $O_2 \Delta t$  from  $\left(S_2 + \frac{O_2 \Delta t}{2}\right)$  gives  $\left(S - \frac{O \Delta t}{2}\right)$  for the beginning of the next time step.

(iv) The procedure is repeated till entire inflow hydrograph is routed.

## \* Hydrological channel Routing

→ In channel routing the storage is a function of both outflow and inflow discharge.

$$S = f(I, Q)$$

→ the water surface in a channel reach is not only unparallel to the channel bottom but also varies with time. the flow in a channel during a flood is spatially varied unsteady flow.

→ the total volume stored in a channel reach can be considered under two categories.

(i) prism storage

(ii) wedge storage.

### \* prism storages

→ It is the volume formed by an imaginary plane parallel to channel bed, drawn upstream from the outflow section to the inflow section.

### \* wedge storages

→ It is the triangular volume enclosed between the actual water surface profile and the top surface of the prism storage.

→ At fixed depth at a downstream section of a river reach, the prism storage is constant while the wedge storage changes from a positive value at an advancing flood (in this case depth increases) to a negative value during receding flood (in this case depth decreases).

→ the prism storage is steady so it is similar to reservoir and expressed as  $S_p = f(I)$

the wedge storage can be expressed as  $S_w = f(Q)$

→ the total storage.

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

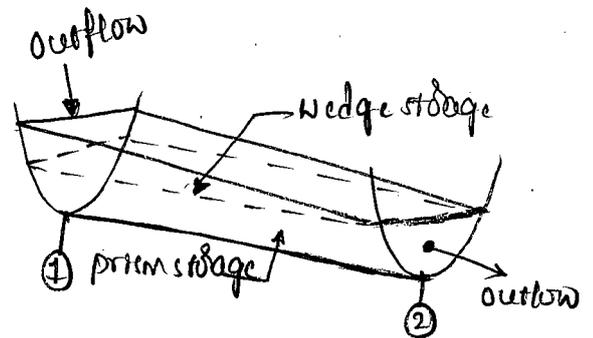


Fig: storage in a channel reach.

## \* Muskingum Equation

→ Considering the above Equation of total storage:

>  $M$  is taken as 1, as found for natural river,

$$S = K[xI + (1-x)O]$$

$K$  = a coefficient parameter

$M$  = a constant Exponent

$x$  = weightage constant.

Now the storage have linear relationship with inflow and outflow.

>  $x$  is a dimensionless weighing factor between inflow and outflow, its value varies between 0 to 0.5. for nature river 0.1 to 0.3.

> when  $x=0.5$ , inflow will not cause any influence on storage, and storage will become a function of outflow only, now called a linear storage or linear reservoir.

>  $K$  is known as storage-time constant. It is approximately equal to the time of travel of a flood wave through the channel reach.

the muskingum Equation can be write down as

$$Q_2 = C_0 I_2 + C_1 I_1 + C_2 O_1$$

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

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

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

NOTE:-  $C_0 + C_1 + C_2 = 1$

## Examples:

→ The storage in a stream reach has been studied, and  $x$  and  $K$  have been identified as 0.28 and 1.6 days. If the inflow hydrograph in the stream reach, as the flood starts coming in and passes, is given by the following table, complete the outflow hydrograph (plotting is not needed).

Hours	0	6	12	18	24	30
$I$ ( $m^3/sec$ )	35	55	92	130	160	140

Q. Given,  $K = 1.6$  days  
 $= 1.6 \times 24h = 38.4$  days  
 $x = 0.28$

Coefficients:-

$$C_0 = - \left[ \frac{38.4 \times 0.28 + 0.5 \times 6}{38.4 - 38.4 \times 0.28 + 0.5 \times 6} \right] = \frac{-7.752}{30.648} = -0.253$$

$$C_1 = \frac{38.4 \times 0.28 + 0.5 \times 6}{38.4 - 38.4 \times 0.28 + 0.5 \times 6} = \frac{13.752}{30.648} = 0.449$$

$$C_2 = \frac{38.4 - 38.4 \times 0.28 - 0.5 \times 6}{38.4 - 38.4 \times 0.28 + 0.5 \times 6} = \frac{34.648}{30.648} = 0.806$$

Check:-  $C_0 + C_1 + C_2 = -0.253 + 0.449 + 0.806 = 1.0$

The outflow  $Q_2$  at the end of each interval by using the Muskingum Equation may be calculated from the Equation

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

The calculations are carried out in table.

Time (1)	Inflow (cumecs) (2)	$C_0 I_2$ $= (-) 0.253 I_2$ (3)	$C_1 I_1$ $= 0.449 I_1$ (4)	$C_2 Q_1$ $= 0.806 Q_1$ (5)	$Q_2$ in cumecs (3) + (4) + (5) (6)
0	35	—	—	—	35.00*
6	55	$-0.253 \times 55$ $= -13.915$	$0.449 \times 35$ $= 15.715$	$0.806 \times 35$ $= 28.21$	29.94*
12	92	$-0.253 \times 92$ $= -23.276$	$0.449 \times 55$ $= 24.695$	$0.806 \times 29.94$ $= 24.072$	25.49**

18	130	$-0.253 \times 130$ $= -32.89$	$0.449 \times 92$ $= 41.308$	$0.806 \times 25.49$ $= 20.494$	28.91**
24	160	$-0.253 \times 160$ $= -40.48$	$0.449 \times 130$ $= 58.37$	$0.806 \times 28.91$ $= 23.266$	41.13
30	140	$-0.253 \times 140$ $= -35.42$	$0.449 \times 160$ $= 71.84$	$0.806 \times 41.13$ $= 33.068$	69.49

The outflow hydrograph ordinates are thus, obtained in col. (6) w.r.t. time in col. (4)

2nd sum:

→ Route the following flood through a river reach for which the Muskingum coefficients  $k$  and  $x$  are 22h and 0.25, respectively. At time  $t=0$ , the outflow discharge is  $40 \text{ m}^3/\text{s}$ .

Hours	0	12	24	36	48	60	72
Inflow ( $\text{m}^3/\text{s}$ )	40	65	165	250	240	205	170

plotting the hydrograph is not needed.

A. using Muskingum's Equations, we have

$$C_0 = \frac{-kx + 0.5\Delta t}{k - kx + 0.5\Delta t}$$

$$C_1 = \frac{kx + 0.5\Delta t}{k - kx + 0.5\Delta t}$$

$$C_2 = \frac{k - kx - 0.5\Delta t}{k - kx + 0.5\Delta t}$$

$$C_0 + C_1 + C_2 = 1 \text{ (for verification)}$$

and

Here, in the given Equation

$$k = 22 \text{ h}$$

$$x = 0.25$$

$$\Delta t = 12 \text{ h (time interval)}$$

$$C_0 = \frac{(-)(22 \times 0.25 - 0.5 \times 12)}{22.5} = 0.022$$

$$C_1 = \frac{(22 \times 0.25 + 0.5 \times 12)}{22.5} = 0.511$$

$$C_2 = \frac{(22 - 22 \times 0.25 - 0.5 \times 12)}{22.5} = 0.467$$

check:  $C_0 + C_1 + C_2 = 1.0$  (which is correct).

Now, the outflow ordinates are worked out in col. (6) of table using

$Q_2 = C_0 I_2 + C_1 I_1 + C_2 Q_1$ , assuming the initial value of outflow =  $40 \text{ m}^3/\text{s}$  as given the table is otherwise self explanatory.

NOTE:  $I_2$  stands for inflow at the end of the interval and  $I_1$  at the start of interval.

Time from start (h) (1)	Inflow in cumecs (2)	$C_0 I_2$ $= 0.222 I_2$ (3)	$C_1 I_1$ $= 0.511 I_1$ (4)	$C_2 Q_1$ $= 0.467 Q_1$ (5)	$Q_2$ in cumecs (3) + (4) + (5) (6)
0	40	—	—	—	40* (given)
12	65	$-0.222 \times 65$ $= 14.3$	$0.511 \times 40$ $= 20.44$	$0.467 \times 40$ $= 18.68$	40.55
24	165	$-0.222 \times 165$ $= 36.3$	$0.511 \times 65$ $= 33.32$	$0.467 \times 40.55$ $= 18.94$	55.79
36	250	5.5	84.32	26.05	115.84
48	240	5.28	127.75	54.11	187.14
60	205	4.51	122.64	87.39	214.54* (peak)
72	140	3.74	104.76	100.19	208.69

NOTE: the above routing shows that the peak which occurred at  $t=36$  hrs at upstream point of river reach, now occurs at  $t=60$  hrs at downstream point, i.e., at a lag of 24 hrs. The peak discharge also reduces from 250 cumecs to 214.54 cumecs.



# GROUND WATER AND WELL IRRIGATION

## UNIT - V

### INTRODUCTION

①

Ground water hydrology is the science of occurrence, distribution and movement of water below the surface of earth. The largest available source of fresh water lies underground. The total ground water potential is estimated to be one third the capacity of oceans.

Aquifer: Aquifers are the permeable formations having structures which permit appreciable quantity of water to move through them under ordinary field conditions thus these are the geologic formations in which ground water occurs (ie sands and gravels)

Aquiclude: Aquicludes are the impermeable formations which contain water but are not capable of transmitting and supplying a significant quantity (eg, clays)

Aquifuge: Aquifuge is an impermeable formation which neither contains water nor transmits any water  
ex: Granite, shale.

Aquitard: Saturated geological formation poorly permeable and hence does not yield water freely into the well  
It may transmit vertically appreciable quantity of water to/from adjacent aquifer

Ex: Sandy clay.

## Occurance of ground water:

The ground water strata formation possesses

- i) Porosity
- ii) Permeability

Porosity: Porosity ( $n$ ) is defined as the ratio of the volume of openings or pores (or voids)  $V_u$  in the material to its total volume  $V$  and is expressed as percentage

$$n = \frac{V_u}{V} \times 100$$

$n > 20$	large	Mere porosity
$5 < n < 20$	Medium	alone cannot
$n < 5$	low	given G.W.

Permeability: Capacity to permit water through soil

Permeability of unconsolidated sedimentation is to transmit water through it

Transmissibility: Which represents the same physical meaning and differ mathematically

Def: Capability of entire soil formation of full depth and unit width is known as transmissibility

Specific Yield: The specific yield of an aquifer is defined as the ratio expressed as percentage, of the volume of water which after being saturated, can be drained by gravity to its own volume

$$\text{Specific yield} = \frac{\text{Volume of water drained by gravity}}{\text{Total volume}}$$

(8)

$$S_y = \frac{w_y}{V} \times 100$$

Specific Retention: The specific retention ( $S_r$ ) of an aquifer is the ratio, expressed as a percentage, of the volume of water it will retain after saturation against the force of gravity to its own volume.

$$S_r = \frac{w_r}{V} \times 100$$

$w_r$  = volume of water retained

$$\text{Porosity } n = \frac{V_w}{V} \times 100 = \frac{w}{V} \times 100$$

$w$  = volume of water =  $V_w$  in a saturated aquifer  
 $= w_y + w_r$

$$n = S_y + S_r$$

Divisions of Subsurface water:

- a) Zone of Aeration
- b) Zone of saturation

## Zone of aeration:

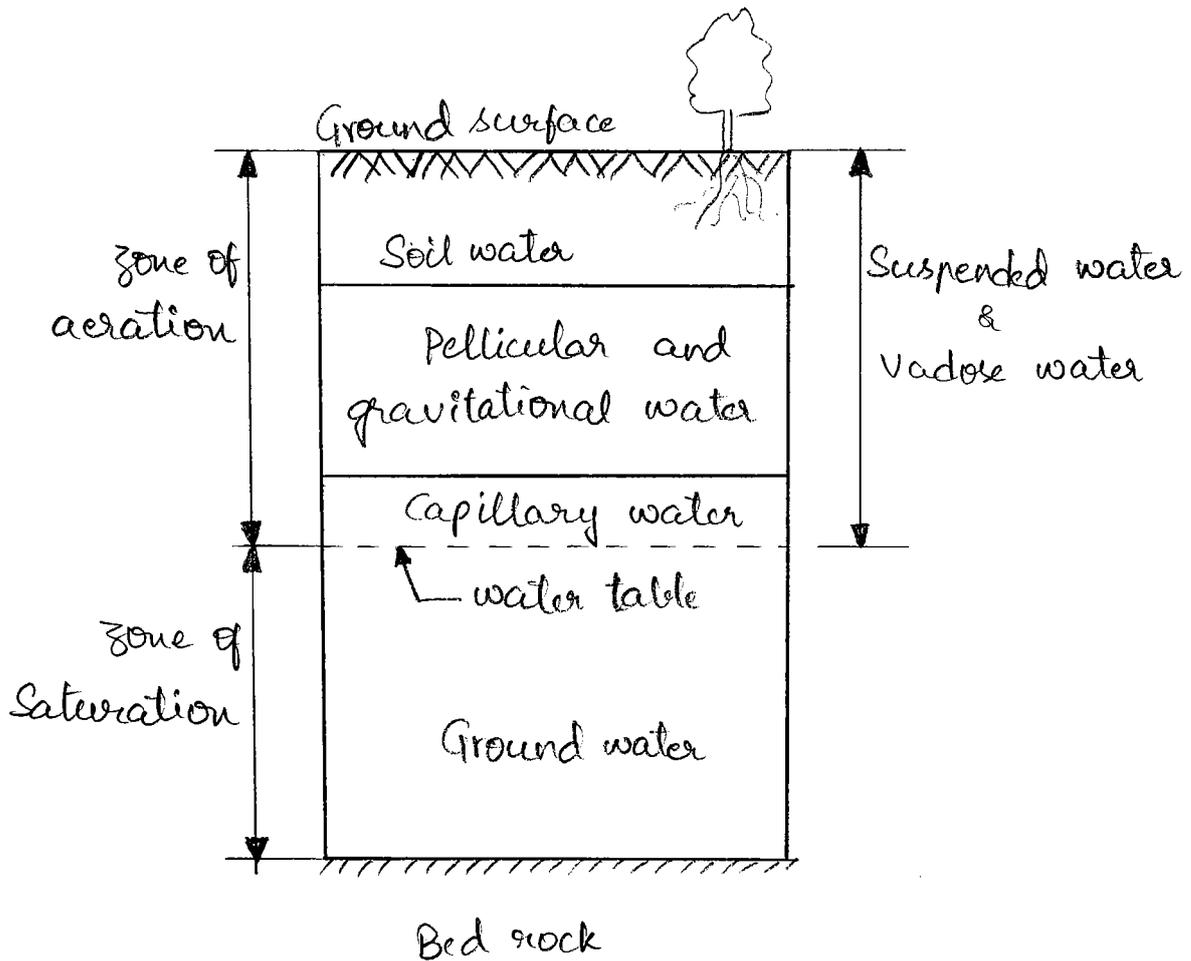
consisting of :

- i) Soil water zone : Soil water
- ii) Intermediate zone: Pellicular and gravitational water
- iii) Capillary zone: Capillary water

## Zone of saturation:

Ground water fills all the interstices in the saturated zone

Water table: In the absence of the confining impermeable layer, the static level of water in wells penetrating the zone of saturation is called the water table



DIVISIONS OF SUBSURFACE WATER

Aquifer: An aquifer is an underground body of rock or sediment that serves as a storage reservoir for ground water <sup>(3)</sup>

### Types Of Aquifers:

Aquifers are mainly of two types:

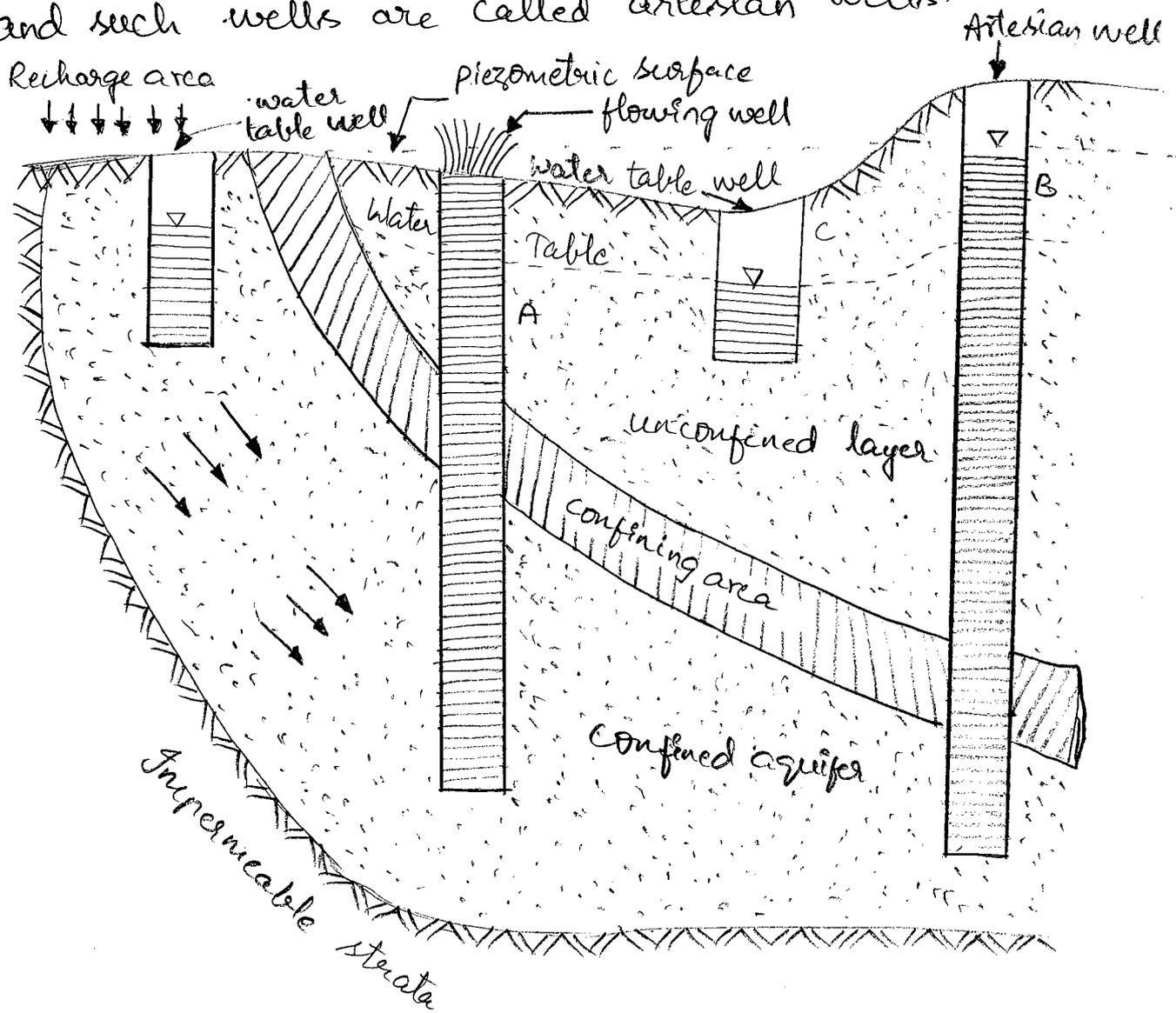
1. Unconfined aquifer
2. Confined aquifer (artesian aquifer)

Unconfined aquifer: Unconfined aquifer or water table aquifer is the one in which a water table serves as the upper surface of the zone of saturation. It is also sometimes known as the free, phreatic or non-artesian aquifer. In such an aquifer, the water table varies in undulating form and in slope. Rises and falls in the water table corresponds to changes in the volume of water in storage within unconfined aquifer.

Confined aquifer: A second common type of aquifer is a confined aquifer which is isolated from pressure communication with overlying or underlying geologic formations and with the land surface and atmosphere by one or more confining layers or confining units.

Confined aquifers differ from unconfined aquifers in two

fundamental and important ways. First, confined aquifers are typically under considerable pressure which may be derived from recharge at higher elevation or from the weight of the overlying rock and soil (known as the overburden). In some cases the pressure is high enough that wells drilled into the aquifer are free flowing. This condition requires that the water pressure in the aquifer is sufficient to drive water up the well bore and above the land surface and such wells are called artesian wells.



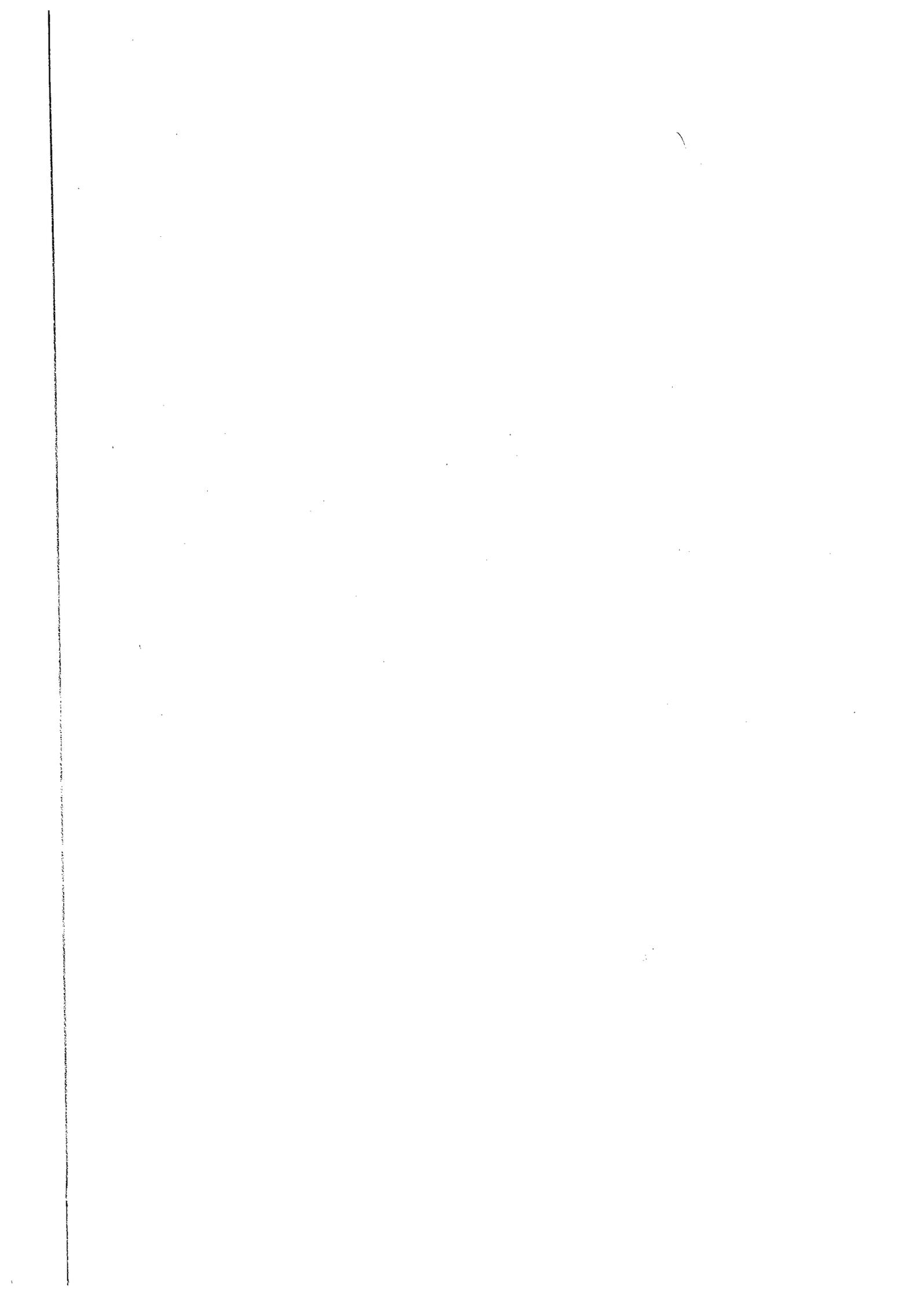
CONFINED AND UNCONFINED AQUIFERS

Perched aquifer: It is a special type of (aquifer) unconfined aquifer, and occurs where a ground water body is separated from the main ground water by a relatively impermeable stratum of small aerial extent and by the zone of aeration above the main body of ground water.

### Storage Coefficient :

The water yielding capacity of a confined aquifer can be expressed in terms of its storage coefficient.

Storage coefficient is defined as the volume of water that an aquifer releases from or takes into storage per unit surface area of aquifer per unit change in the component of head normal to that surface.



## Acquifer parameters:

### Coefficient of permeability (K):

It is defined as the velocity of flow which will occur through the total c/s area of the soil (or) aquifer under a unit hydraulic gradient.

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

### Coefficient of Transmissibility (T):

It is defined as the rate of flow in ( $m^3/day$ ) through a vertical strip of aquifer of width 1m, extending full saturation height under unit hydraulic gradient.

$$T = K \times b$$

### Specific yield and specific retention:

Specific yield ( $S_y$ ): volume of water drained from aquifer by gravity force divided by total volume of aquifer drained.

Specific Retention ( $S_{ri}$ ): volume of water held by aquifer against gravity force divided by total volume of aquifer drained.

$$S_{ri} + S_y = n$$

Specific capacity: specific capacity of well is defined as rate of flow from a well per unit of draw down.

### Coefficient of storage (A):

→ Defined for confined aquifer's only.

It is defined as the volume of water that a confined aquifer releases (or) stores in per unit surface area of aquifer

or unit change in the component of head perpendicular to the surface.

→ Range varies from 0.00005 to 0.005

formulae:

I. confined aquifer:

$$\rightarrow Q = \frac{2.72T(H-h)}{\log_{10}\left(\frac{R}{r}\right)} \quad [T=kb]$$

$$\rightarrow Q = \frac{2.72TS}{\log_{10}\left(\frac{R}{r}\right)}$$

for observation wells by thiem's equation:

$$\rightarrow Q = \frac{2.72T(h_2-h_1)}{\log_{10}\left(\frac{r_2}{r_1}\right)} \Rightarrow \frac{2.72T(s_1-s_2)}{\log_{10}\left(\frac{r_2}{r_1}\right)}$$

for straight length:

$$\rightarrow Q = \frac{2.72Kbs}{\log_{10}\left(\frac{r_2}{r_1}\right)} \Rightarrow \frac{2.72Kbs}{\log_{10}\left(\frac{R}{r}\right)}$$

Coefficient of Transmissibility:

$$T = \frac{Q \log_{10}\left(\frac{r_2}{r_1}\right)}{2.72(s_1-s_2)}$$

for 1 log cycle:

$$T = \frac{Q}{2.72\Delta s}$$

## II. unconfined aquifer:

$$\rightarrow Q = \frac{1.36 K (H^2 - h^2)}{\log_{10} \left( \frac{R}{r} \right)} \quad (\text{Dupit's equation})$$

$$\rightarrow Q = \frac{1.36 K S (S + 2h)}{\log_{10} \left( \frac{R}{r} \right)}$$

$$\rightarrow Q = \frac{2.72 K S \left( h + \frac{S}{2} \right)}{\log_{10} \left( \frac{R}{r} \right)} \Rightarrow \frac{2.72 K S \left( L + \frac{S}{2} \right)}{\log_{10} \left( \frac{R}{r} \right)}$$

[L  $\rightarrow$  strainer length, L=h]

for observation wells by Thiem's equation:

$$\rightarrow Q = \frac{1.36 K (h_2^2 - h_1^2)}{\log_{10} \left( \frac{R}{r} \right)}$$

Coefficient of transmissibility:

$$T = \frac{Q \log_{10} \left( \frac{R}{r} \right)}{2.72 \Delta s'}$$

for 1 log cycle:

$$T = \frac{Q}{2.72 \Delta s'}$$

## Well hydraulics:

Darcy's law: The percolation of water through soil was first studied by Darcy (1856) who demonstrated experimentally that for laminar flow conditions in a saturated soil, the rate of flow, (Q) the discharge per unit time is proportional to the hydraulic gradient, and it could be expressed as follows:

$$Q = KiA$$

(Q)

$$v = \frac{Q}{A} \Rightarrow Ki$$

Where  $Q$  = rate of flow

$i$  = hydraulic gradient

$K$  = Darcy's coefficient of permeability

$A$  = Total cross-sectional area of soil mass

perpendicular to the direction of flow.

$v$  = flow velocity

Darcy's law is valid only for laminar flow. Because of very small pore dimensions in fine grained soils, a laminar flow should exist, but in coarse grained soils turbulent flow may be expected under certain conditions. It has been borne out by experiments that the limits of validity of Darcy's law may be fixed with respect to particle size, velocity of flow and hydraulic gradient.

## Steady radial flow to a well : Dupit's theory :

③

When a well is penetrated into an extensive homogeneous aquifer, the water table initially remains horizontal in the well. When the well is pumped, water is removed from the aquifer and the water table (or) piezometric surface, depending upon the type of the aquifer, is lowered resulting in a circular depression in the water table (or) the piezometric surface.

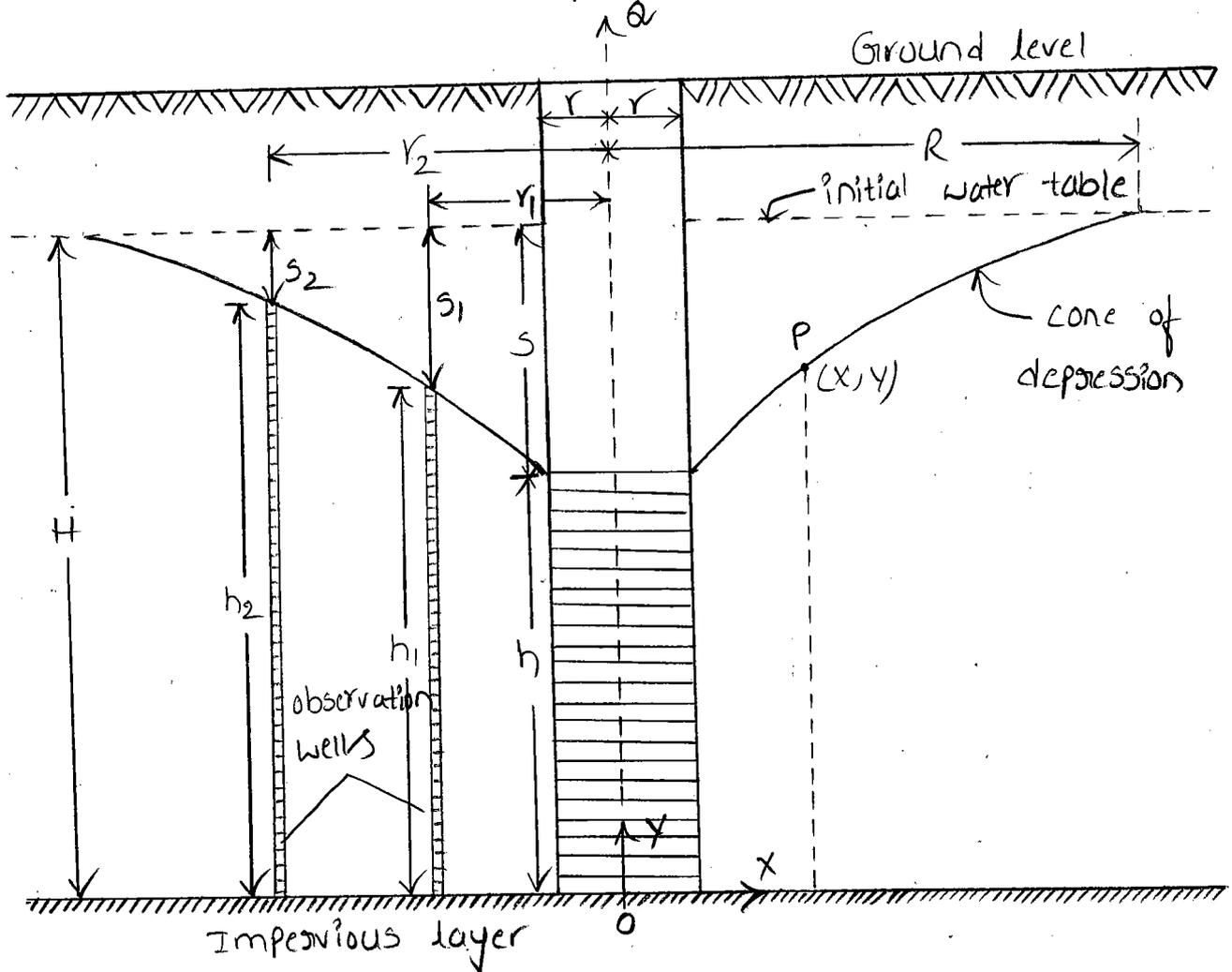
This depression is called the cone of depression (or) the drawdown curve. At any point, away from the well, the drawdown is the vertical distance by which the water table (or) the piezometric surface is lowered. The analysis of such radial flow towards a well was originally proposed by Dupit in 1863 and later modified by Thiem (1906). For the sake of analysis, we shall take 2 cases.

- 1) Well in unconfined aquifer, and
- 2) Well fully penetrating a confined aquifer.

### 1) unconfined aquifer:

following fig shows a well penetrating an unconfined (or) free aquifer to its full depth.

unconfined aquifer



Where  $R$  = Radius of influence

$s$  = Draw down

$h$  = depth of water in main well

$H$  = Total depth of water

$r_1$  = Radius of observation well (1)

$r_2$  = Radius of observation well (2)

$h_1$  = Height/depth of water in observation well (1)

$h_2$  = Height/depth of water in observation well (2)

$s_1$  = Draw down of well (1)

$s_2$  = Draw down of well (2)

$$Q = \frac{1.36 K (H^2 - h^2)}{\log_{10} \left( \frac{R}{r} \right)}$$

$$Q = \frac{1.36 K (h_2^2 - h_1^2)}{\log_{10} \left( \frac{r_2}{r_1} \right)}$$

$$Q = \frac{2.72 K S \left( L + \frac{S}{2} \right)}{\log_{10} \left( \frac{R}{r} \right)}$$

### Assumptions and Limitations of Dupit's theory:

Dupit's theory of flow for unconfined aquifer is based on the following assumptions:

→ The velocity of flow is proportional to the tangent of the hydraulic gradient instead of sine.

→ The flow is horizontal and uniform everywhere in the vertical section.

→ Aquifer is homogeneous, isotropic and of infinite axial extent.

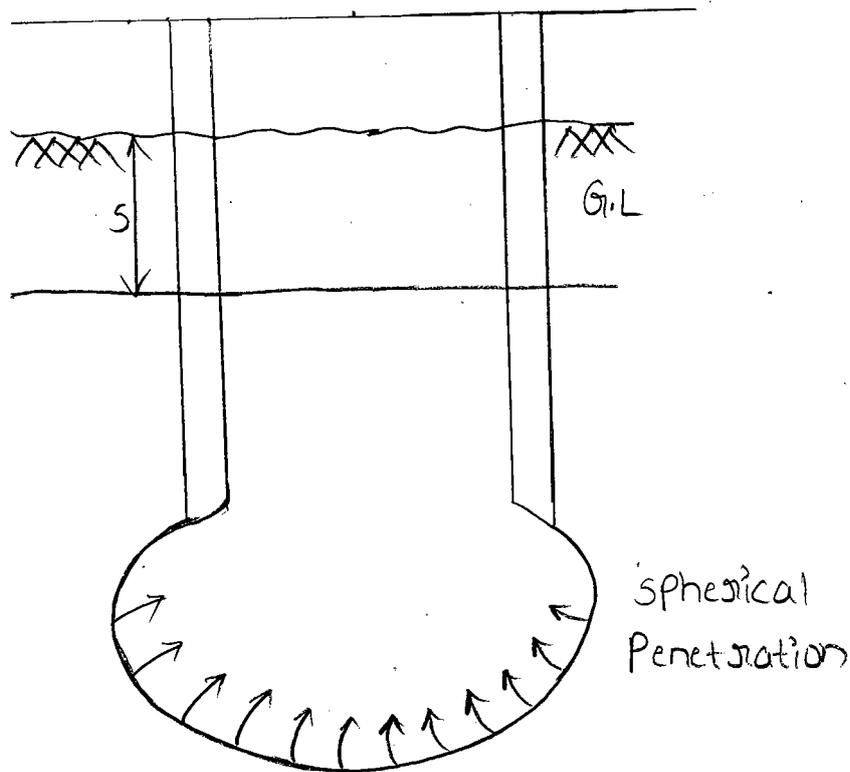
→ The well penetrates and receives water from the entire thickness of the aquifer.

→ The coefficient of transmissibility is constant at all places and at all times.

→ Natural ground water regime affecting an aquifer remains constant with time.

→ flow is laminar and Darcy's law is applicable.

## constant level pumping test:



→ using pump, heavy discharge is taken out to create depression head (s).

→ The discharge to be such that outward discharge should be same as inward discharge. (spherical penetration).

from Darcy's equation

$$\begin{aligned} AV &= Q = KiA \\ &= K \cdot \frac{s}{L} \cdot A \\ &= \frac{K}{L} \cdot s \cdot A \end{aligned}$$

$$i = \frac{s}{L}$$

c → depends on soil formation.

$$\boxed{Q = cSA}$$

c → Percolation intensity coefficient

$$Q \propto s \quad \text{as} \quad Q = AV$$

$$\sqrt{LS}$$

Area of c/s at bottom =  $\frac{4}{3}$  (Actual area) in case a spherical cavity is formed.

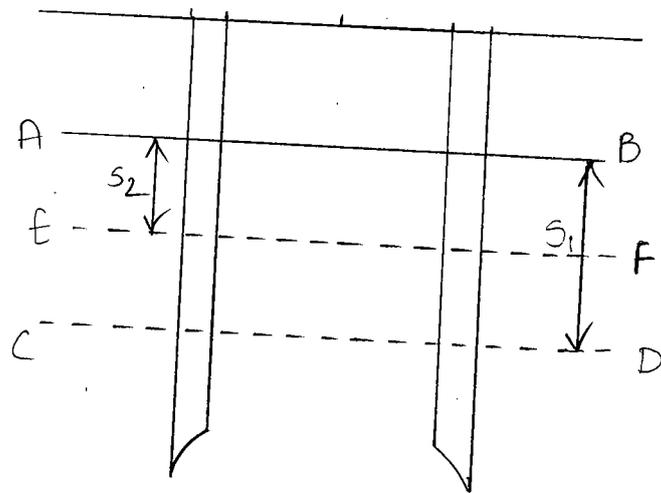
- Recharge velocity cannot be increased beyond the critical velocity (depends on soil formation; varies from soil to soil).
- Head corresponding to critical velocity is called critical head.

$$\text{Working Head} = \frac{1}{3} (\text{critical head})$$

- $Q_{\text{max}}$  from an open well corresponds to critical depression head and max. safe yield corresponds to working head.

### Recuperation test:

- When constant head (or discharge) cannot be maintained (unable to maintain) Recuperation test is performed.



- Water is pumped heavily, water level (head) drops from AB to CD.
- Wait for time 't' water level recuperate to EF (recharges)
- As drawdown varies for every interval of time, discharge too varies

AB  $\rightarrow$  Initial water level

CD  $\rightarrow$  water level in the well when pumping was stopped.

S  $\rightarrow$  Depression head (s) drawdown when pumping.

tt  $\rightarrow$  water level at time 't' after pumping was stopped.

Let (ds) be change for (dt)

volume of water entering in  $dV$  = Area of  
Small time (dt) = c/s  $A \times (ds)$

Let x-x be water level at any time 't'.

S  $\leftrightarrow$  drawdown corresponding to x-x.

If 'Q' is rate of recharge in small time (dt).

volume of water recharged  $dV = Q \cdot dt$

We know  $Q < S$

$$Q = c's$$

$$dV = c'sx dt \rightarrow \text{eqn } ①$$

$$dV = A \times ds \rightarrow \text{eqn } ②$$

equating ① and ②

$$-c's \cdot dt = A \times ds$$

C  $\rightarrow$  is introduced because as 't'  $\uparrow$  's'  $\downarrow$

$$- \frac{c'}{A} \cdot dt = \frac{ds}{s}$$

by integrating on both sides we get

$$\frac{c'}{A} \int_0^T dt = - \int_{s_1}^{s_2} \frac{ds}{s}$$

$$\text{at } t=0, s=s_1$$

$$t=T, s=s_2$$

$$\frac{c'}{A} (t)_0^T = \left[ \log_e s \right]_{s_1}^{s_2}$$

$$\frac{c'}{A}(T) = \log_e \frac{s_1}{s_2}$$

$$\frac{c'}{A}(T) = 2.303 \log_{10} \left( \frac{s_1}{s_2} \right)$$

$$\frac{c'}{A} = \frac{2.303}{T} \log_{10} \left( \frac{s_1}{s_2} \right)$$

$\frac{c'}{A}$  is called specific capacity (or) specific yield of the formation (Aquifer) and its units are

$$\frac{\text{m}^3/\text{sec}}{\text{discharge}} \left| \frac{\text{m}^2}{\text{area}} \right| \frac{\text{m}}{\text{draw down}}$$

We have,  $Q = c's$

$$Q = \left( \frac{c'}{A} \right) \cdot A \cdot s \quad (\times \& \div \text{ by } A)$$

$$Q = \frac{2.303}{T} \log_{10} \left( \frac{s_1}{s_2} \right) \cdot A \cdot s$$

Experimental values: Given by "massiot"

<u>Type of soil</u>	<u><math>(c'/A)</math> (<math>\text{m}^3/\text{hr}) / \text{m}^2/\text{m}</math>)</u>
clay	0.25
sand	0.5
coarse sand	1.0

These values can be assumed by default if NO time to perform experiment.

while solving if data is insufficient (rare cases).



### Problem-1:

Design a tube well for the following data.

yield required =  $0.08 \text{ m}^3/\text{s}$ , thickness of confined aquifer =  $30 \text{ m}$ , Radius of circle of influence =  $300 \text{ m}$ , permeability coefficient is  $60 \text{ m/day}$ , draw down  $5 \text{ m}$ .

Sol: Given that

$$K = \frac{60 \text{ m}}{\text{day}} = \frac{60}{86400} \text{ m/s}$$

$$K = 6.93 \times 10^{-4} \text{ m/s}$$

$$Q = 0.08 \text{ m}^3/\text{s}$$

$$b = 30 \text{ m}$$

$$R = 300 \text{ m}$$

$$s = 5 \text{ m}$$

$$Q = \frac{2.72 K b s}{\log_{10} \left( \frac{R}{r} \right)}$$

$$\log_{10} \left( \frac{300}{r} \right) = \frac{2.72 \times 6.93 \times 10^{-4} \times 30 \times 5}{0.08}$$

$$\left( \frac{300}{r} \right) = 10^{(3.534)}$$

$$r = 0.088 \text{ m}$$

$$\text{diameter, } d = 2r = 0.177 \text{ m} \approx 18 \text{ cm}$$

$$d = 18 \text{ cm}$$

### Problem-2:

A tube well penetrates fully on unconfined aquifer. Calculate discharge from the tube well under the following conditions.

Dia of well =  $30 \text{ cm}$ , draw down =  $2 \text{ m}$ , effective length of strainer under above draw down is equal to  $10 \text{ m}$ , coefficient of permeability of aquifer is  $0.05 \text{ cm/s}$ . Radius of zero draw down =  $300 \text{ m}$ .

sl: Given,

$$\text{dia of well} = 30\text{cm} = 0.3\text{m}$$

$$\text{Radius, } r = 0.15\text{m}$$

$$\text{Length of strainer, } L = 10\text{m}$$

$$\text{for confined aquifer, } (L=b) \Rightarrow b=10\text{m}$$

$$\text{draw down, } s = 2\text{m}$$

$$\text{coefficient of permeability, } k = \frac{0.05\text{cm}}{s} = 5\text{m/s} \times 10^{-4}$$

$$\text{Radius of zero draw down, } R = 300\text{m}$$

we have discharge,

$$Q = \frac{2.72 k s (L + s/2)}{\log_{10} \left( \frac{R}{r} \right)}$$

$$Q = \frac{2.72 \times 5 \times 2 \left( 10 + \frac{2}{2} \right) \times 10^{-4}}{\log_{10} \left( \frac{300}{0.15} \right)}$$

$$Q = 9.06 \times 10^{-3} \text{ m}^3/\text{s}$$

$$Q = 9.06 \text{ lit/sec}$$

Alternatively

$$s = 2\text{m}, L = 10\text{m}, H = 12\text{m}, h = 10\text{m}$$

$$Q = \frac{1.36 \times 5 \times 10^{-4} \left( (12)^2 - (10)^2 \right)}{\log_{10} \left( \frac{300}{0.15} \right)}$$

$$Q = 9.06 \times 10^{-3} \text{ m}^3/\text{s}$$

$$Q = 9.06 \text{ lit/sec}$$

### Problem-3:

An artesian tube well has a dia of 20cm. Thickness of aquifer is 30m, 'k' is 38m/day. find its yield under a draw down of 4m at well face. Also determine transmissivity of aquifer.

Sol: Given,

$$\text{diameter, } d = 20\text{cm}$$

$$r = 10\text{cm} \Rightarrow 0.1\text{m}$$

$$\text{thickness, } b = 30\text{m}$$

$$\text{permeability, } k = 38\text{m/day} \Rightarrow 38/86400\text{m/s}$$

$$k = 4.39 \times 10^{-4}\text{m/s}$$

$$\text{draw down, } s = 4\text{m}$$

Radius of influence 'R' from Sichardt's eq<sup>n</sup>

$$R = 3000 s \sqrt{k}$$

$$R = 3000 (4) \sqrt{4.39 \times 10^{-4}}$$

$$R = 251.5\text{m}$$

$$\text{yield/discharge, } Q = \frac{2.72 k b s}{\log_{10} \left( \frac{R}{r} \right)}$$

$$Q = \frac{2.72 (4.398) (30) (4)}{\log_{10} \left( \frac{251.5}{0.1} \right)} \times 10^{-4}$$

$$Q = 0.0422\text{m}^3/\text{s}$$

Coefficient of transmissibility

$$T = k \times b$$

$$T = (4.398 \times 10^{-4}) \times 30$$

$$T = 0.0132\text{m}^2/\text{s}$$

### Problem-4:

A well penetrates through 10m thick water bearing stratum of coarse sand having  $K = 0.005 \text{ m/s}$ . Radius of artesian well is 10cm and is to be worked under a draw down of 4m at well phase. Calculate discharge from the well. What will be % increase in discharge. If the radius of well is doubled. Take  $R = 300\text{m}$  in each case.

Sol: Given,

$$\text{Thickness, } b = 10\text{m}$$

$$\text{Permeability, } K = 0.005 \text{ m/s}$$

$$\text{Radius, } r_1 = 0.1\text{m}$$

$$\text{draw down, } s = 4\text{m}$$

$$\text{We have discharge, } Q = \frac{2.72 K b s}{\log_{10} \left( \frac{R}{r_1} \right)}$$

$$Q = \frac{2.72 \times (0.005) \times (10) \times (4)}{\log_{10} \left( \frac{300}{0.1} \right)}$$

$$Q = 0.156 \text{ m}^3/\text{s}$$

If radius is doubled,

$$r_1 = r_1, \quad r_2 = 2r_1$$

$$Q_1 = Q$$

$$Q_2 = \frac{2.72 (0.005) (10) (4)}{\log_{10} \left( \frac{300}{0.2} \right)}$$

$$= 0.1712 \text{ m}^3/\text{s}$$

$$\% \text{ increase } \Delta Q = \frac{Q_2 - Q_1}{Q_1} \times 100 \Rightarrow \frac{0.1712 - 0.156}{0.156} \times 100$$

$$\Delta Q = 9.79 \%$$

### Problem-5:

In order to determine the field permeability of a free aquifer pumping out test was performed and following observations were made. Dia of well is 20cm, discharge is  $240 \text{ m}^3/\text{hr}$ . R.L of original water surface before pumping 240.5m, R.L of water in the well at constant pumping 235.6m, R.L of impervious layer 210m, R.L of water in observation well is 239.8m. Radial distance of observation well from tube well is 50m. Calculate  $K$  also calculate

- 1) Error in  $K$  if observations are not taken in observation well and radius of influence is assumed as 300m.
- 2) Actual radius influence based on observations of observation well.

Sol: Given,

$$Q = 240 \text{ m}^3/\text{hr}$$

$$r = 0.1 \text{ m}$$

$$H = (240.5 - 210)$$

$$H = 30.5 \text{ m}$$

$$h = (235.60 - 210)$$

$$h = 25.60 \text{ m}$$

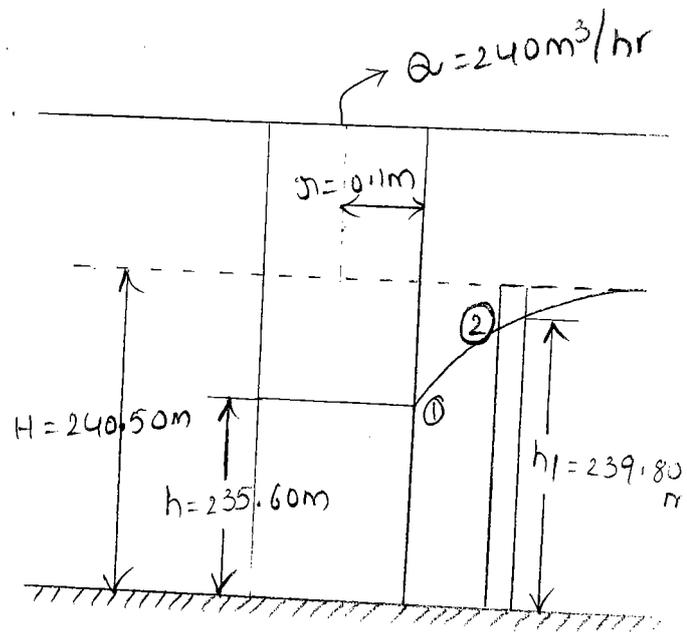
$$h_1 = (239.80 - 210)$$

$$= 29.80 \text{ m}$$

Now by applying thiem's eq<sup>n</sup> for point ① & ②

$$Q = \frac{1.36K(h_2^2 - h_1^2)}{\log_{10}(r_2/r_1)}$$

$$Q = \frac{1.36K((29.8)^2 - (25.6)^2)}{\log_{10}(50/0.1)} = 240$$



$$K_a = 2.046 \text{ m/hr}$$

1) Consider radius of influence,  $R = 300\text{m}$

Substitute in Dupit's eq<sup>n</sup>

$$Q = \frac{1.36 K ((30.5)^2 - (25.6)^2)}{\log_{10} \left( \frac{300}{0.1} \right)} = 240$$

$$K_o = 2.232 \text{ m/hr}$$

$$\% \text{ error in } K = \frac{K_{obs} - K_{act}}{K_{act}} \times 100$$

$$= \frac{2.232 - 2.046}{2.046} \times 100$$

$$= 9.1\%$$

2) Now actual radius of influence be ' $R_m$ '

$$Q = \frac{1.36 K_{act} (H^2 - h^2)}{\log_{10} \left( \frac{R}{r} \right)}$$

$$240 = \frac{1.36 (2.046) ((30.5)^2 - (25.6)^2)}{\log_{10} \left( \frac{R}{0.1} \right)}$$

$$\log_{10} \left( \frac{R}{0.1} \right) = 1538.41\text{m}$$

$$R = 153.84\text{m} \approx 154\text{m}$$

Problem-6:

A 60cm dia well is pumped at the rate of 2000lit/min. measurements is near by test wells were made at the same time were: At a discharge of 10m draw down is 4m at a discharge of 20m, the draw down is 2m. The thickness of unconfined aquifer is 40m.

a) find out ' $k$ '

b) If all the observed values are on Dupit's curve. What was

draw down in drain well during pumping.

c) specific capacity, Radius of influence.

d) what is maximum rate at which water can be drawn from main well.

sol: Given,

$$\begin{aligned} (Q)_{\text{tube well}} &= 2000 \text{ lit/min} \\ &= 0.033 \text{ m}^3/\text{s} \\ &= 2880 \text{ m}^3/\text{day} \end{aligned}$$

$$r = 0.3 \text{ m}$$

$$\text{draw downs, } s_1 = 4 \text{ m}$$

$$s_2 = 2 \text{ m}$$

$$\therefore h_1 = H - s_1 = 40 - 4 = 36 \text{ m}$$

$$h_2 = H - s_2 = 40 - 2 = 38 \text{ m}$$

$$\begin{aligned} a) \quad Q &= \frac{1.36 K (h_2^2 - h_1^2)}{\log_{10} \left( \frac{r_2}{r_1} \right)} \\ &\Rightarrow \frac{1.36 K ((38)^2 - (36)^2)}{\log_{10} \left( \frac{20}{10} \right)} = 2880 \text{ m}^3/\text{day} \end{aligned}$$

$$K = 4.307 \text{ m/day}$$

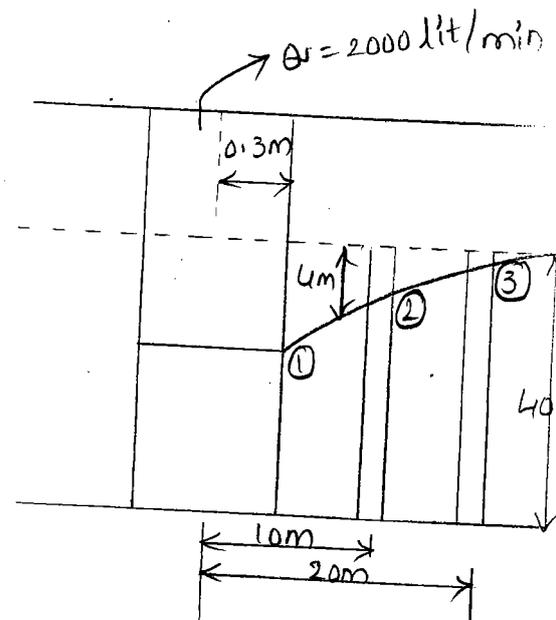
b) Apply thiem's eq<sup>n</sup> at point ① & point ②

$$Q = \frac{1.36 K (h_2^2 - h_1^2)}{\log_{10} \left( \frac{r_2}{r_1} \right)}$$

$$Q = \frac{1.36 (4.307) ((36)^2 - (h)^2)}{\log_{10} \left( \frac{10}{0.3} \right)} = 2880$$

$$h = 23.39 \text{ m} \approx 23.4 \text{ m}$$

$$\text{draw down, } s = H - h = 16.6 \text{ m}$$



Specific capacity,  $\frac{C'}{A} = \frac{2.303}{T} \log_{10}\left(\frac{s_1}{s_2}\right)$

$$T = K \times b \Rightarrow (4.307 \times 40) \Rightarrow 172.28 \text{ m}^2/\text{day}$$

$$\frac{C'}{A} = \frac{2.303}{172.28} \log_{10} \quad (s_1, s_2 \rightarrow \text{not available})$$

We know that, specific discharge of the well is discharge per unit draw down

$$\text{i.e. } s=1 \Rightarrow h = H - s \Rightarrow 40 - 1 \Rightarrow 39 \text{ m}$$

apply dupit's eq<sup>n</sup>,  $Q_s = \frac{1.36 K (H^2 - h^2)}{\log_{10}(R/s)}$

$$Q_s = \frac{1.36 (4.307) (40^2 - 39^2)}{\log_{10}\left(\frac{41.5}{0.3}\right)}$$

$$Q_s = 216.14 \text{ m}^3/\text{day} \quad (\text{m depression head})$$

we have,  $Q_s =$

d) Radius of influence,  $\log_{10}\left(\frac{R}{s}\right) = \frac{1.36 K (H^2 - h^2)}{Q_s}$

$$\log_{10}\left(\frac{R}{0.3}\right) = \frac{1.36 (4.307) (40^2 - (23.4)^2)}{2880}$$

$$\left(\frac{R}{0.3}\right) = 138.20 \text{ m}$$

$$R = 41.46 \text{ m} \approx 41.5 \text{ m}$$

maximum discharge of well (at  $h=0$ )

$$Q_w = \frac{1.36 K (H^2)}{\log_{10}(R/s)}$$

$$Q_w = \frac{1.36 (4.307) (40)^2}{\log_{10}\left(\frac{41.5}{0.3}\right)}$$

$$Q_w = 4377.55 \text{ m}^3/\text{day}$$

## UNIT-VI Advanced Topics in Hydrology.

### CHOW'S METHOD:-

Chow gave a method of solution which avoids curve fitting. He (de)introduced a function  $F(u)$  given by the relation:

$$F(u) = \frac{W(u) \cdot e^u}{2.303}$$

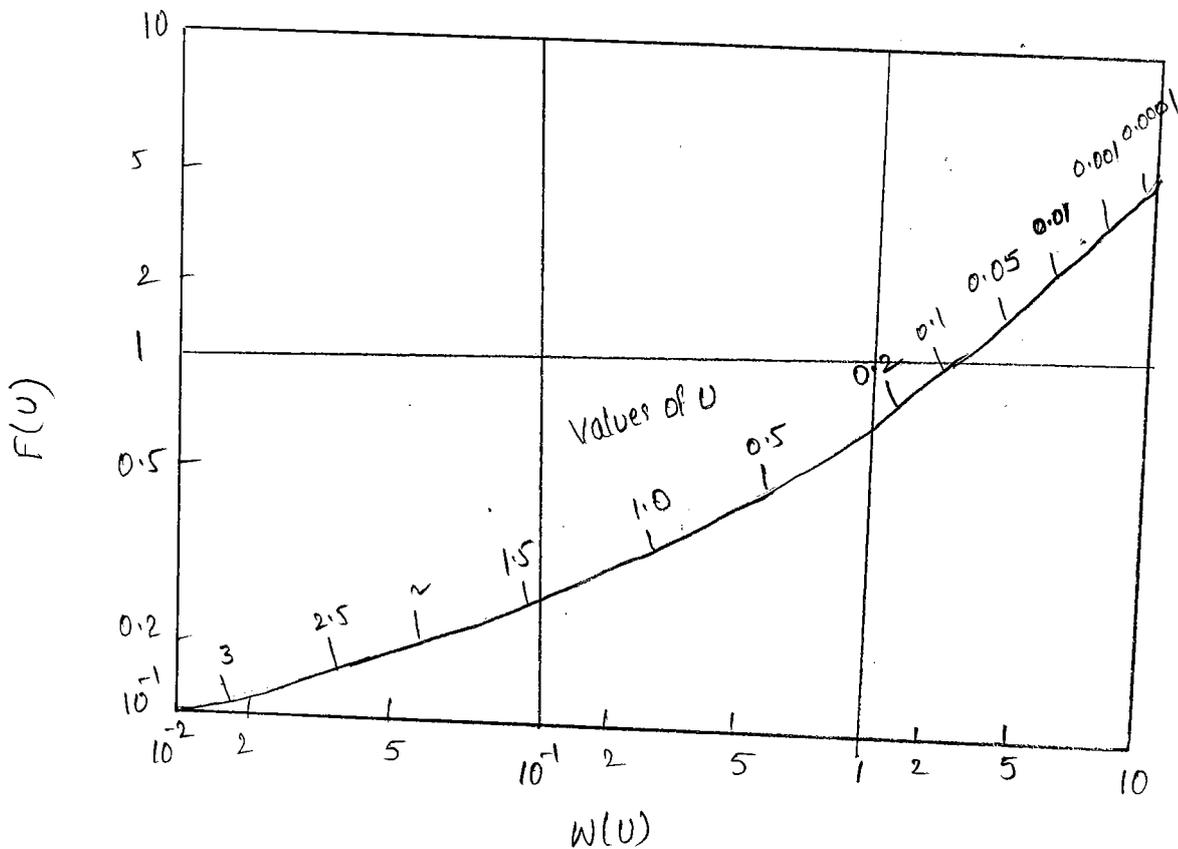


Fig 1. Function  $F(u)$

The relation between  $F(u)$ ,  $W(u)$  and  $u$  is shown in Fig. with the  $u$  scale on the curve. The function  $F(u)$  is introduced to relate  $w(u)$  and  $u$  to a certain combination of sand. The procedure is as follows:

1. Select an observation well near the pumped well and observe the drawdown ( $s$ ) at all times.
2. plot a graph between  $s$  and  $\log_{10} t$  as shown in Fig 2. Join the points by a smooth curve.

- on the plotted graph, choose an arbitrary point  $p$  and note its co-ordinates  $s$  and  $t$ .
- Draw the tangent of Curve at the chosen point  $p$  and determine the drawdown difference  $\Delta s$  per log cycle of time.
- Compute the function  $F(u)$  by the relation.

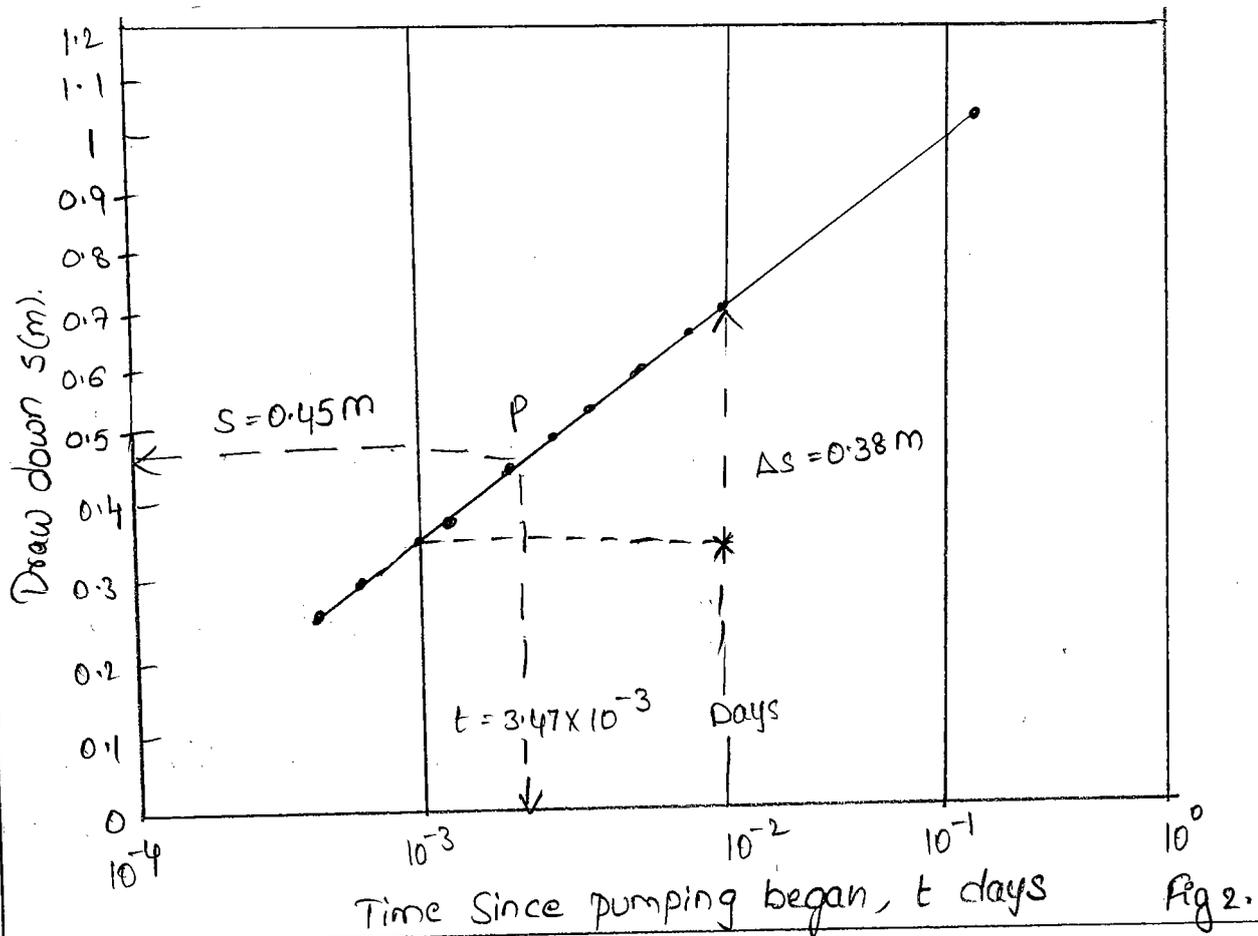
$$F(u) = \frac{s}{\Delta s}$$

- knowing  $F(u)$  and using Fig 1. find  $u$  and  $w(u)$ .
- Compute  $s$  and  $T$  from the following relations:

$$T = \frac{Q}{4\pi S} w(u) \quad \text{and} \quad s = \frac{4uTt}{r^2}$$

Note: For small values of  $u$ , we have from Jacob's method:

$$\Delta s = \frac{2.303Q}{4\pi T} \log_{10} \frac{t_2}{t_1} = \frac{2.303Q}{4\pi T} \quad (\text{when } t_2 = 10t_1).$$



$$\text{Also } S = \frac{Q}{4\pi T} W(u)$$

$$F(u) = \frac{S}{\Delta S} = \frac{W(u)}{2.303}$$

For  $F(u) < 2$ , the above Equation can safely be used. For  $F(u) > 2$ ,  $u$  becomes large. Hence (Eq.) above Equation should be used.

### INSTANTANEOUS UNIT HYDROGRAPH:-

We have seen earlier that each unit hydrograph representing 1cm of direct runoff, is for some unit duration  $T_0$ . For a catchment, there can be a number of unit hydrographs corresponding to various values of unit duration  $T_0$ . To obtain the runoff hydrograph resulting from a storm of varying duration and varying intensities, it is preferable to have a unit hydrograph of very short unit duration. Theoretically, the shortest unit duration is zero. If the duration of rainfall excess becomes infinitesimally small, the resulting unit hydrograph is called "INSTANTANEOUS UNIT HYDROGRAPH". The IUH is designated as  $u(t, 0)$  or simply as  $u(t)$ .

If two S-curves are drawn at a time lag of  $t_0$ , the ordinate of unit hydrograph of  $t_0$  hour unit duration at any time  $t$  is given by  $u(t, t_0) = \frac{T_0}{t_0} (S_t - S_{t-t_0}) \rightarrow \text{①}$

where,  $u(t, t_0)$  = ordinate of unit hydrograph of unit duration  $t_0$ .

$T_0$  = unit duration of unit hydrograph from which S-curve has been obtained.

$S_t$  = ordinate of S-curve at any time  $t$ .

$(S_{t-t_0})$ ,  $S_{t-t_0}$  = ordinate of shift S-curve, shifted by  $t_0$ .

If  $t_0$  is taken as  $\Delta t$ , the ordinates of resulting unit hydrograph of  $\Delta t$  unit duration is given by

$$u(t, \Delta t) = \frac{T_0}{\Delta t} [S_t^{T_0} - S_{t-\Delta t}^{T_0}]$$

(a)

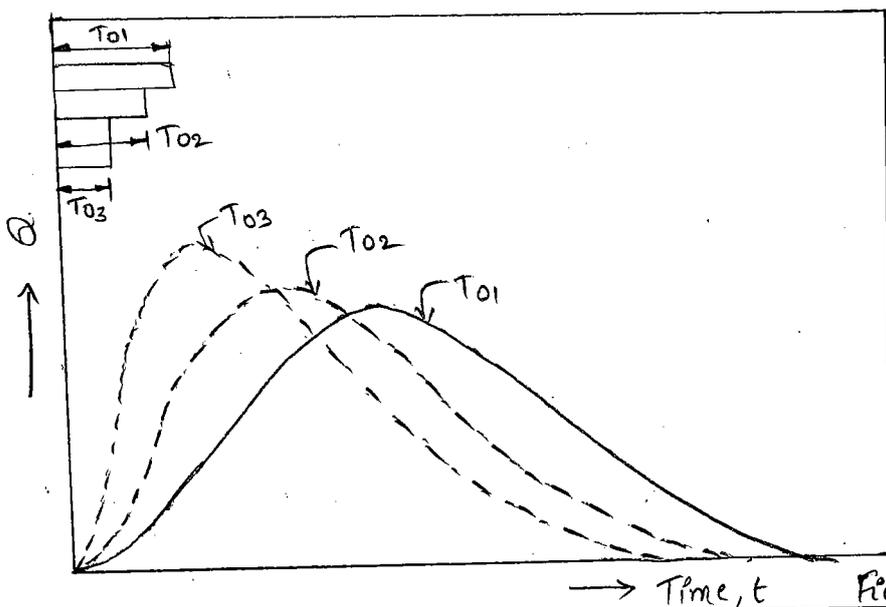
$$u(t, \Delta t) = T_0 \frac{\Delta S_t^{T_0}}{\Delta t}$$

Where  $S_t^{T_0}$  is the S-curve ordinate derived from unit hydrograph of  $T_0$  unit duration. In the limit  $\Delta t \rightarrow 0$ , we get the UH, given by

$$\lim_{\Delta t \rightarrow 0} u(t, \Delta t) = T_0 \frac{dS_t^{T_0}}{dt}$$

$$(b) \quad u(t) = T_0 \frac{dS_t^{T_0}}{dt} = \frac{L}{R} \frac{dS_t^{T_0}}{dt}$$

Hence  $\left\{ \begin{array}{l} \text{The ordinate of} \\ \text{UH at any time 't'} \end{array} \right\} = T_0 \left\{ \begin{array}{l} \text{the slope of S-curve derived from } T_0 \\ \text{hour unit hydrograph at time 't'} \end{array} \right\}$



In the above expression,  $R_0$  is the intensity of rainfall excess, given by  $R_0 = 1/T_0$

If  $R_0 = 1 \text{ cm}$ , we get

$$u(t) = \frac{ds_t}{dt}$$

Where  $s_t$  is the ordinate of S-curve of intensity  $1 \text{ cm/hr}$ . Thus, the ordinate of IUH at any time 't' is the slope of S-curve of intensity  $1 \text{ cm/hr}$ .

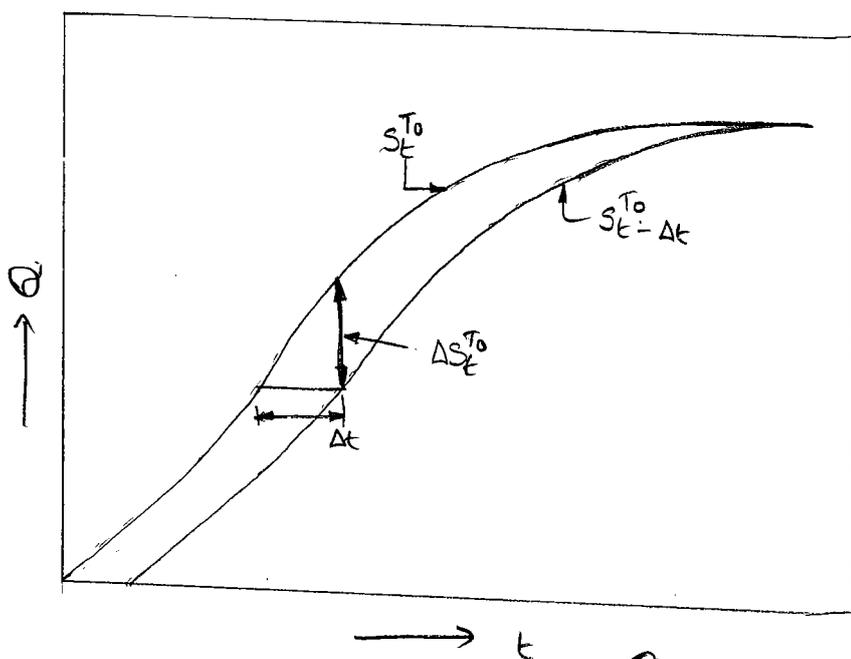


Fig 4.

An IUH, designated by  $u(t)$  is a single peaked hydrograph with a finite base width. It has the following properties:

1.  $0 \leq u(t) \leq a$  positive value at  $t > 0$ .

2.  $u(t) = 0$  at  $t \leq 0$

3.  $u(t) \rightarrow 0$  as  $t \rightarrow \infty$

4.  $\int_0^{\infty} u(t) dt = \text{unit depth of over catchment}$

and 5. (Time to peak)  $<$  (Time to Centroid of Curve).

It is interesting to note that IUH is a unique demonstration of a particular Catchment's response to rain, independent of duration, just as unit hydrograph is its response to rain of a particular unit duration. IUH is not time dependant. It is a geographical expression of the integration of the catchment, such as length, shape, slope etc. that control such a response.

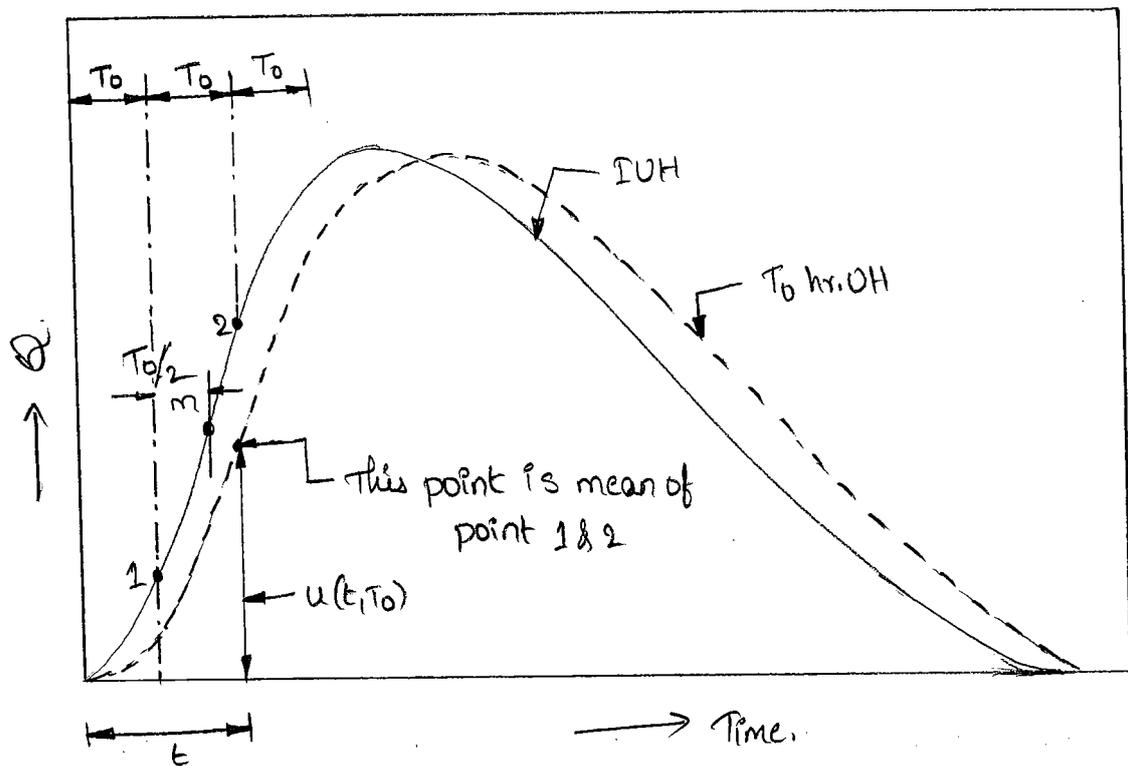


Fig. 5

→ The IUH can be developed either directly from the observed data or by adopting conceptual models.

→ When once IUH is available for a catchment, unit hydrographs of various unit durations can be easily derived.

Let us derive unit hydrograph  $u(t, T_0)$  of  $T_0$  unit (hydrograph) duration.

We already know that,

$$u(t, T_0) = S_t - S_{t-T_0}$$

$$(a) \quad u(t, T_0) = \frac{1}{T_0} \left[ \int_0^t u(z) dz - \int_0^{t-T_0} u(z) dz \right]$$

$$(a) \quad u(t, T_0) = \frac{1}{T_0} \int_{t-T_0}^t u(z) dz$$

Hence,

$$\left\{ \begin{array}{l} \text{ordinate of U.H. of} \\ T_0 \text{ at any time } t \end{array} \right\} = \frac{1}{T_0} \left\{ \begin{array}{l} \text{area of UH in the limits} \\ \text{between } (t-T_0) \text{ and } t \end{array} \right\}$$

If UH is assumed to be linear between  $(t-T_0)$  and  $t$ , the above Equation reduces to

$$u(t, T_0) = u\left(t - \frac{T_0}{2}\right).$$

Thus the ordinate of  $T_0$  unit hydrograph at any time  $t$  is the average of UH for  $(t-T_0)$  hours and that of 't' hours.

From fig 5., it is clear that if UH is divided into  $T_0$  hour time interval, and if the averages of the ordinates at the beginning and the end of each interval are plotted at the end of the interval, we get ordinate of  $T_0$  unit hydrograph.

$$\text{Thus, } u(t, T_0) = \frac{1}{2} [u(t) + u(t-T_0)]$$

