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Engineering Hydrology

BE Civil 5th Semester

Chapter → 1

Introduction

Hydrology: Hydrology is derived from two words hydro and logos. 'Hydro' means water and 'logos' means study. simply, Hydrology is defined as the study of water. In other words hydrology may be also defined as the science, which deals with the occurrence, distribution and movement of water on the earth, including that in the atmosphere and below the surface of the earth.

Hydrological Cycle: Hydrological cycle may be defined as the process of transfer of moisture from the atmosphere to the earth in the form of precipitation, conveyance of the precipitated water by means of streams or river to the ocean and lakes and evaporation of water back to the atmosphere. This evaporation and precipitation continues for ever and there by, a balance is always maintained between the two. The cycle has no begining or end, and its many processes occur continuously.

The different process involved in the hydrological cycle are explained below.

- → Evaporation
- → Transpiration
- → Precipitation
- → Runoff

→ Infiltration
→ Interception

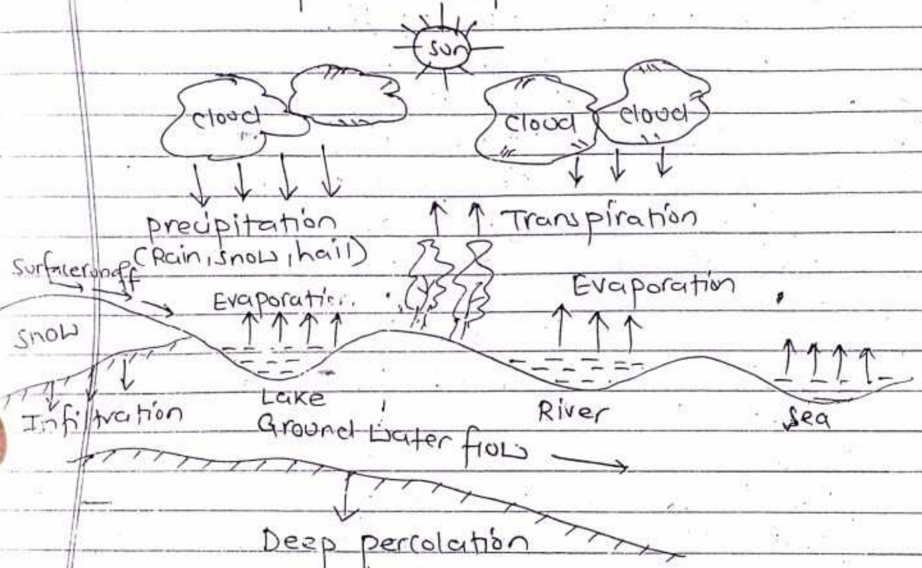

fig: Hydrological cycle

a) Evaporation: Evaporation is the process by which water from liquid or solid state is converted into vapour form.

b) Transpiration: Transpiration is the loss of water as vapour from the leaves of the plants.

c) Precipitation: The fall of water from the atmosphere to the earth in any forms like rain, snow, sleet, hail stone, drizzle etc is called precipitation.

d) Runoff: The portion of the precipitation which reaches the stream channel by a variety of paths above and below the surface of the earth is called runoff.

e) Infiltration: The process of movement of water into the ground from the ground surface is called infiltration.

f) Interception: It is the quantity of water that is intercepted by vegetation, building and other objects which finally gets evaporated without contributing to the runoff.

Scope of Hydrology:

→ Hydrology has scope in the following areas

- Estimation of water resources.
- Study of process like evaporation, precipitation infiltration and runoff and their interaction.
- Study of problems like flood and droughts.
- Study of ground water potential and its use.
- Environmental impact of hydraulic structure.

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- Understanding the properties of water in nature.

(*) Application of Hydrology in Engineering:

→ The Knowledge of hydrology is very much useful in the design and operation of various water resources engineering projects like watersupply, water power engineering, navigation and flood control. The application can be summarized below.

- For the determination of max^m flow expected at dam, reservoir, spillway, bridges etc.
- Estimation of a total volume of water that may be available from a drainage basin.
- In determining the minimum capacity of reservoir sufficient to meet the hydropower irrigation and water supply demands.
- For planning of hydropower plants.
- In determining the effect of river water level before and after completion of reservoir and other irrigation structure.
- To study of ground water potential and its use.
- For dimensioning of the navigation channel during various seasons.

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- To study the interaction of flood wave and the hydraulic structure.

(*) Factors to be considered for planning and design of Water Resources projects.

→ The different factors to be considered during the planning and design of water resources project are as follows:-

- Max^m flow which are expected to occur at a place.
- Minimum flow which can occur during any dry period.
- Minimum reservoir capacity to be fixed to meet all water demand.
- possible regulation of floods at the down stream reaches.
- Environmental impacts of a hydraulic structure.
- Study of ground water potential and its use.
- effect of water level before and after completion of reservoir.
- possible supply of water from a river to meet demands for water resources projects.

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(*) Water Balance Equation:

→ The water balance equation is the statement of the law of conservation of mass. In other words, it is the balance of input and output of water within a given area taking into account net changes of storage. It can be defined by the following relationship.

Mass inflow – Mass outflow = Change in storage

The main source of inflow is precipitation (P) and outflows are surface runoff (R), evaporation (E), Transpiration etc.

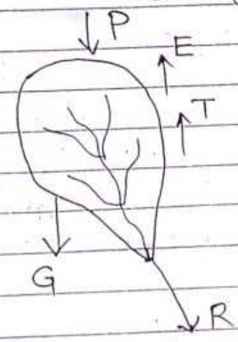

fig. Water balance in a basin

The general water balance equation is

$$P-(R+G+E+T) = \Delta S$$

Where,
ΔS = Change in storage
G = Net ground water flow

In case of other inflow besides precipitation, the water balance equation is

$$(P+I) - (R+G+E+T) = \Delta S$$

Where,
I = other inflow.

(1) A river reach had a flood wave passing through it. At a given instant the storage of water in the reach was estimated as 15.5 ha.meter. What would be the storage in the reach after an interval of 3 hours if the average inflow and outflow during the time period are 14.2 m^3/sec and 10.6 m^3/sec resp.

→ Given:

Initial storage = 15.5 ha. = 15.5×10^4 m^3
Inflow rate = 14.2 m^3/sec
outflow rate = 10.6 m^3/sec
Time (t) = 3hrs

Inflow volume in 3hrs = $14.2 \times 3 \times 60 \times 60$
= 153360 m^3

outflow volume in 3hrs = $10.6 \times 3 \times 60 \times 60$
= 114480 m^3

Final storage = ?

From water balance equation,
Inflow − outflow = change in storage

$153360 - 114480 = \text{Final storage} - 15.5 \times 10^4$

∴ Final storage = 193880 m^3

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(2) Estimate the constant rate of with drawl of water from a 1375 ha. reservoir in a month of 30 days during which the reservoir level dropped by 0.75m in spite of an average inflow into the reservoir of 0.5 million m^3/day. During the month the average seepage loss from the reservoir was 2.5 cm, total precipitation was 18.5 cm and total evaporation was 9.5 cm.

→ Given:

Reservoir area (A) = 1375 ha.

or, $A = 1375 \times 10^4 \ m^2$

Inflow into the reservoir = $0.5 \times 10^6 \ m^3/day$

Inflow in the month = $0.5 \times 10^6 \times 30$

$= 15 \times 10^6 \ m^3$

Inflow interms of depth = (I) = $\frac{15 \times 10^6}{1375 \times 10^4}$

or, I = 1.091 m

Change in storage (ΔS) = 0.7m

Seepage loss (G) = 2.5cm = 0.025m.

precipitation (P) = 18.5 cm = 0.185m

evaporation (E) = 9.5 cm = 0.095 m

From water balance equation,

$(P + I) - (Q + G + E) = \Delta S$

or, $(0.185 + 1.091) - (Q + 0.025 + 0.095) = 0.75$

or, Q = 0.406 m for a month.

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∴ Rate of withdrawl = $\frac{0.406 \times 1375 \times 10^4}{30 \times 24 \times 60 \times 60} = 2.15 \ m^3/s$

(3) A small catchment of area 150 ha received a rainfall of 10.5 cm in 90 minutes due to a storm. At the outlet of the catchment, the stream draining the catchment was dry before the storm and experience a runoff lasting for 10 hours with an average discharge value of 2 m^3/sec. The stream was again dry after the runoff event. (a) What is the amount of water which was not available to runoff due to combined effect of infiltration, evaporation and transpiration? (b) What is the ratio of runoff to precipitation.

→ Given:

Catchment area (A) = 150 ha = $150 \times 10^4 \ m^2$

precipitation = 10.5 cm = 0.105 m

Vol. of precipitation (P) = $0.105 \times 150 \times 10^4$

or, P = 157500 m^3

Runoff = 2 m^3/sec

Runoff volume in 10 hours (R) = $2 \times 10 \times 3600$

or, R = 72000 m^3

i) Amount of water loss (L) = ?

From water balance equation,

$P - (R + G + E + T) = \Delta S$

Where,

G + E + T = L and ΔS = 0 as the stream was dry before and after the storm.

$\therefore P-(R+L)=0$

or, $L=P-R$

or, $L=157500-72000$

or, $L=85500 m^3$

ii) Runoff / Rainfall = 72000/157500
= 0.457.

(*) Development of hydrological study in Nepal

→ Nepal has a short history in the field of hydrology. Government of Nepal started hydrological and meteorological activities in an organized way in 1962. The activities were initiated as a section under the department of electricity. Later on in 1988, a separate department of Hydrology and Meterology was established.

Some of the major historical events in the development of hydrological study in Nepal are summarized below.

- Hydrological studies in Nepal started after the Goverment of India initiated Koshi project in late 1940s.
- Hydrological stations on Koshi at Barahachhetra, Sunkoshi at Kampughat and Tamur at Mulghat were established in 1947.
- With the support of the goverment of India meterological observations stations were established in 1956.
- Nepal started hydrological and meterological activities in an organized way in 1962 from the Karnali basin.
- In 1966, Department of Hydrology and Meterology was established under the ministry of Water and Power.
- The department was merged with the Department of Irrigation in 1972.
- The department was finally separated in 1988 from the irrigation.

At present there are 120 number of hydrology stations in operation and 282 number of meteorological station in operation in the context of Nepal.

(*) Hydrology as a science of water:-

→ Water is one of the most important natural resources. Without it there would be no life on earth. The supply of water available for our use is limited by nature. Although there is plenty of water on earth it is not alway in the right place, at the right time and of the right quality. Adding to the problem is the increasing evidence that chemical wastes improperly discarded yesterday are showing up in our water supplies today. Hydrology has envolved as a science in response to the need to understand the complex water system of earth and help to solve water problem. It deals with the occurrence, circulation and distribution of water of the earth and earth's atmosphere. As a branch of earth science, it is concerned with the water in streams and lakes, rainfall and snowfall, snow and ice on the land and water coming or occuring below the earth's surface in the pores of the soil and rocks. Hydrology is a very broad subject of an inter-disciplinary nature drawing support from allied science such as meterology, geology, statistics chemistry, physics and fluid mechanics. It is basically an applied science which utilize the theories, formulas, principle from different branches of science to understand precipitation and evaporation process, infiltration, ground waterflow

surface runoff. Thus, Hydrology is the science of water that treats the water of the earth, their occurrence, circulation and distribution their chemical and physical properties and their relation with environment including their relations to living things.

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Physical Hydrology

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(*) **Precipitation:-** The fall of moisture from the atmosphere to the earth in any form like rain, drizzle, snow, sleet etc is called precipitation. It denotes all forms of water that reach the earth form the atmosphere. The magnitude of precipitation varies with time and place. This variation is responsible for many hydrological problems such as drought and floods.

Causes of Precipitation;

→ When water gets evaporated from the earth surface, river, ocean, lake in the form of vapour it gets collected in the atmosphere and behaves like a gas. As the evaporation increases the amount of water vapour also goes on increasing. But space can hold only a certain fixed amount of a water vapour. Finally, a state is reached where any further addition of water vapour will get condensed on the surface, forms the precipitation.

Forms of Precipitation:

→ The different forms of Precipitation are discussed below.

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- → Drizzle
- → Rain
- → Snow
- → Glaze
- → Sleet

i) **Drizzle:-** A fine sprinkle of numerous water d[illegible] of size less than 0.5mm and intensity less than 1mm/hr is known as drizzle. The drop[illegible] are so small that they appear to float in t[illegible] air.

ii) **Rain:-** Water droplet of size larger than 0.5m[illegible] is generally termed as rain. It is the princip[illegible] form of precipitation. The max^m size of the droplet is 6mm.

iii) **Snow:-** Snow is another important form of precip[illegible]tation. It consists of ice crystals.

iv) **Glaze:-** When drizzle or rain freezes as it comes in contact with cold objects, it is known as glaze.

v) **Sleet:** Precipitation of snow and rain simultaneously is called sleet

Types of Precipitation:

→ Moisture is always present in the atmosphere but it is condensed only when the air is cooled. Depending upon the way in which air is cooled to cause precipitation, it can be classified into three types. They are:

- → convective precipitation
- → orographic precipitation
- → cyclonic precipitation.

ⓐ Convective precipitation: The unequal (ie heating of earth surface is main responsible for convective precipitation. When the warmer air rises up due to heating, colder air from the surrounding takes up its place. The warm air continues to rise, undergoes cooling and results in precipitation. The precipitation occurs in the form of showers of high intensity and for short duration.

ⓑ Orographic precipitation: orographic precipitation is the most important precipitation and is responsible for most of the heavy rains. It is caused by the air masses, which strike some natural topographic barriers like hills, mountains and cannot move forward and hence rises up causing condensation and precipitation. The greatest amount of precipitation falls on the windward side and the leeward side often has very little precipitation.

ⓒ Cyclonic precipitation:- Cyclonic precipitation occurs in the form of drizzle, intermittent rain or steady rain. It is caused by lifting of air mass due to the pressure difference. If low pressure occurs in the area, air will flow horizontally from the surrounding area, causing the air in the low pressure area to lift. This displaced air move in upward direction and undergoes cooling and results in precipitation.

Rain Gauges:- Rain gauge is a device which is used to collect and measure the amount of rainfall. It consists of a cylindrical vessel assembly kept in the open to collect rain. The amount of rainfall varies from place to place so, it is necessary to establish no. of ~~rainfall~~ rain gauge station at various key points.

The following points should be kept in mind while selecting a site for a rain gauge station.

- The ground must be level and site should be open space.
- The gauge must be set near the ground as far as possible to reduce wind effect.
- No object should be nearer to the instrument.
- A fence should be errected to protect the gauge from external agencies like cattles.

Types of Rain gauge:

→ Raingauge are broadly classified into two categories. They are

- → Non recording raingauge
- → Recording raingauge

i) **Non recording Raingauge:** The gauge which is read manually is called non-recording raingauge. This type of rain gauge only collects the rain but donot record any rain. The collected rain is then measured by means of graduated cylinders in terms of depth of water. The depth of rainfall is given by

$$= \frac{\text{Vol. of Water Collected}}{\text{Area of aperture of gauge}}$$

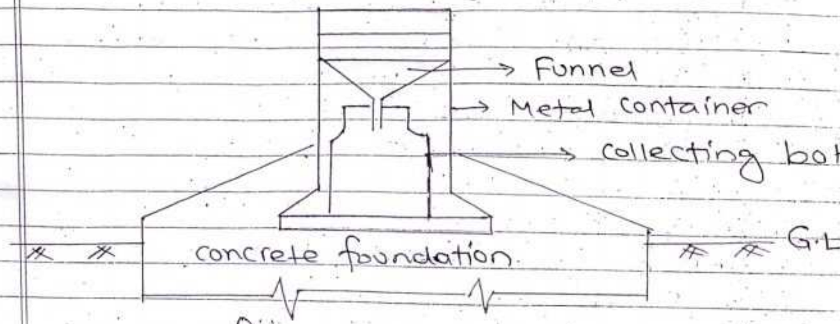

fig. Non recording raingauge

ii) **Recording raingauge:** Recording type rain gauge gives us direct rainfall record without any bottle reading. In this type of gauge a man has not to go to the gauge to measure or to read the amo[unt] of rain fallen. There is a mechanical arrangement by which the total amount of rainfallen, since th[e] record was started gets recorded automatically [on] a graph paper. The gauge thus produce a record of cumulative rain Vs time In the form of a graph which is known as the mass curve of rain fallen.

Figure shows a tipping bucket type of raingauge. It consists of a cylindrical ~~vessel~~ receiver with a funnel inside. Just below the funnel a pair of tipping bucket is pivote[d] such that one of the bucket receives a rainfall of 0.25mm, it tips and bring the other one in position and the process is repeated. The tipping of the bucket actuates an electrically driven pen to trace a record on clock driven chart. The water collected in the storage can is measured at regular intervals to provide total rainfall and also serve as a check. The record from tipping bucket gives data on the intensity of rainfall.

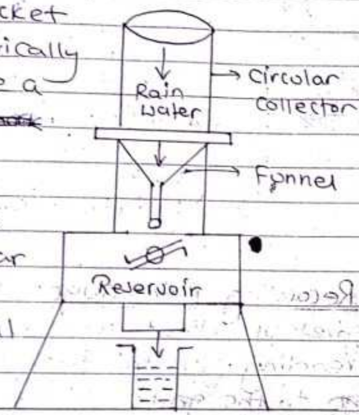

fig. Tipping Bucket type of rain gauge.

Estimation of missing rainfall data:-

⇒ Sometimes the rainfall amount at a certain rain gauge station for a certain day, may be missing due to the absence of some observer or instrumental failure.

In such cases it might be needed to estimate the missing rainfall amount by approximating the value from the data of the near by rain gauge station.

Suppose, m = no. of surrounding stations

$P_1, P_2, P_3 \ldots P_m$ = annual precipitation at neighbouring $1, 2, 3 \ldots m$ station.

P_x = rainfall of missing station 'x'.

$N_1, N_2, N_3 \ldots N_m$ = Normal annual rainfall at m - surrounding stations.

N_x = Normal annual rainfall at missing station 'x'

Then

If the normal annual rainfall at various stations are within 10% of normal annual rainfall at station 'x' under consideration, then missing data may be estimated as the simple arithmetic average as follows,

$$P_x = \frac{1}{m}\left(P_1 + P_2 + P_3 + \cdots + P_m\right)$$

otherwise, normal ratio method is used

$$P_x = \frac{N_x}{m}\left[\frac{P_1}{N_1} + \frac{P_2}{N_2} + \frac{P_3}{N_3} + \cdots + \frac{P_m}{N_m}\right]$$

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① The normal annual rainfall at stations A, B, C, D in a basin are 80.97, 67.59, 76.28, 92.01 cm resp. In the year 1975, the station D, was inoperated due to the storm and the station A, B and C annual precipitation of 91.11, 72.23 and 79.89 cm resp. Estimate the rainfall at station 'D' in that year.

→ Given,

Normal annual precipitation at A (N_1) = 80.97 cm
" " " " B (N_2) = 67.59 cm
" " " " C (N_3) = 76.28 cm
" " " " D (N_x) = 92.01 cm

In year 1975,
Precipitation at A (P_1) = 91.11 cm
" " B (P_2) = 72.23 cm
" " C (P_3) = 79.89 cm
" " D (P_x) = ?

No. of stations with no missing precipitation (m) = 3

Then, A/c to Normal ratio method,

$$P_x = \frac{N_x}{m}\left[\frac{P_1}{N_1} + \frac{P_2}{N_2} + \frac{P_3}{N_3}\right]$$

or, $$P_x = \frac{92.01}{3}\left[\frac{91.11}{80.97} + \frac{72.23}{67.59} + \frac{79.89}{76.28}\right]$$

or, P_x = 99.40 cm

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✱ Test for consistency of recorded rainfall data (Double Mass curve)

→ The trend of rainfall records at a station may slightly change after some years due to
- shifting of a raingauge station to a new location
- change in ecosystem due to landslide, forest
- Replacement of old instrument with new one
- occurrence of observational error from certain data.
- change in wind pattern due to planting or cutting forest.

This gives rise to inconsistency of rainfall data of that station. Hence, the inconsistency data should be checked. Double Mass curve technique is used to test the consistency of rainfall record at any rain gauge station which is suspected to contain certain discrepancy. It is the plot of accumulated annual rainfall of a particular station versus the accumulated annual value of mean rainfall of surrounding stations. This technique is based on the principle that "when each recorded data comes from the same parent population, they are consistent". If the double mass curve is straight line, the rainfall of the particular station is said to be consistent and if there is break in the slope of the plot, then the rainfall of that particular station is inconsistent.

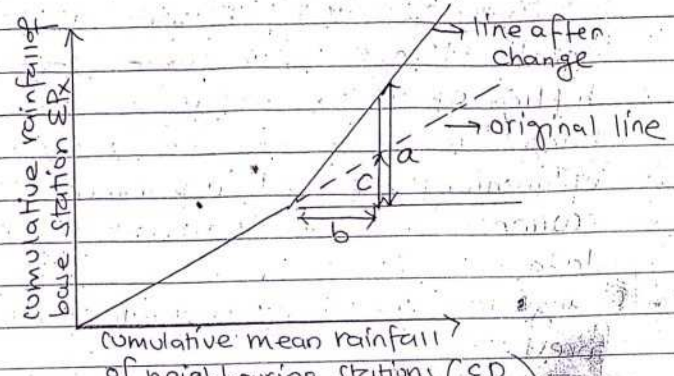


fig. Double mass curve

Procedure:-

- A group of 5 to 10 base station in the neighbourhood should be selected.
- Accumulated Precipitation of the station X, ΣP_x and the accumulated value of the average of the group of the base stations ΣP_{av} are calculated.
- Value of ΣP_x in y-axis and ΣP_{av} in x-axis are plotted for consecutive time period. The curve is usually plotted starting from the latest record which is assumed to be correct.
- A break in the slope of resulting plot indicates the change in the precipitation regime of the station.
- The precipitation value at the station beyond the period of change in regime is corrected by using the formula

$$P_{cx} = P_x \frac{M_c}{M_a} = P_x \frac{c}{a}$$

Where,
P_{cx} = corrected precipitation of station X.
P_x = original precipition of station X.
M_c = slope of original line = c/b
M_a = slope of line after change of regime. = a/b

correction should be applied for change in slope exceeding 10% of original line.

(*) The annual precipitation of station 'X' and the average annual precipitation at 10 neighbouring stations are as follows. Use double mass curve to correct for any data inconsistent at the station.

Year	Precipitation at X (mm)	10 station avg (mm)	Year	Precipitation at X (mm)	10 station avg. (mm)
1972	35	28	1980	30	26
1973	37	29	1981	31	31
1974	39	31	1982	35	36
1975	35	27	1983	38	39
1976	30	25	1984	40	44
1977	25	21	1985	28	32
1978	20	17	1986	25	30
1979	24	21	1987	21	23

⇒

Year	Precipitation at 'x' mm	10 station avg (mm)	Cumulative rainfall at x	Cumulative rainfall of 10 st.
1972	35	28	35	28
1973	37	29	72	57
1974	39	31	111	88
1975	35	27	146	115
1976	30	25	176	140
1977	25	21	201	161
1978	20	17	221	178
1979	24	21	245	199
1980	30	26	275	225
1981	31	31	306	256
1982	35	36	341	292
1983	38	39	379	331
1984	40	44	419	375
1985	28	32	447	407
1986	25	30	472	437
1987	21	23	493	460

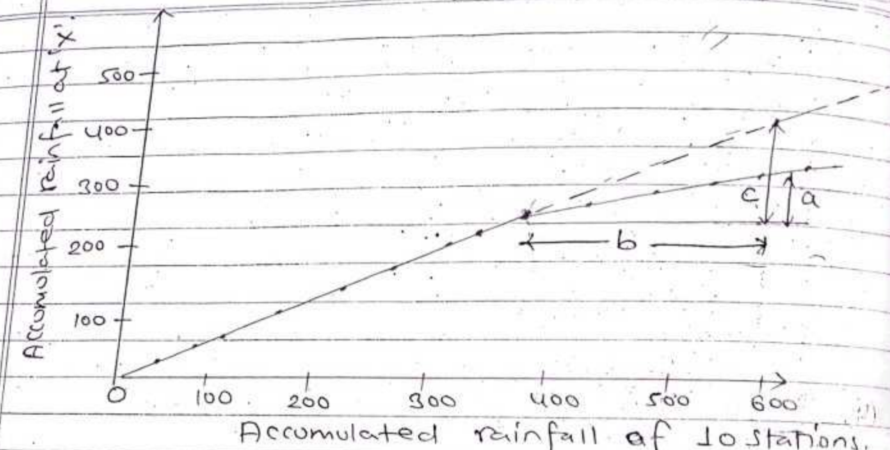
These data are plotted in the graph. It is seen that there is break in slope of line after 1980. So, the rainfall data at 'x' is inconsistent after 1980.

From graph,

$a = 200$

$c = 280$

$b = 215$



slope of original line $(M_c) = \frac{c}{b} = \frac{280}{215} = 1.3023$

slope of line after 1980 $(M_a) = \frac{a}{b} = \frac{200}{215} = 0.93$

∴ Change in slope $= \frac{1.3023 - 0.93}{1.3023} \times 100 = 28\%$

As the difference is more than 10%, so correction should be applied to it.

Correction ratio $= \frac{M_c}{M_a} = \frac{1.3023}{0.93} \approx 1.4$

Thus, the yearly precipitation value of station 'x' from year 1981 to 1987 are corrected by multiplying the original value by 1.4.

Year	Precipitation at x (mm)	Corrected precipitation at x
1981	31	43.4
1982	35	49
1983	38	53.2
1984	40	56
1985	28	39.2
1986	25	35
1987	21	29.4

(#) Mean Precipitation over an area:-

→ The different methods employed to find out the mean rainfall on an area are

- → Arithmetic - Mean Method
- → Thiessen polygon Method
- → Isohyetal Method.

(a) Arithmetic Mean Method:-

→ This is the simplest method in which the average precipitation over the catchment is taken as the arithmetic mean of the station values.

If $P_1, P_2, P_3 \ldots P_n$ are the rainfall values in a given period in N-stations within a catchment, then

the value of mean precipitation 'p' over the catchment is calculated as

$$P = \frac{P_1 + P_2 + P_3 + \ldots + P_n}{N}$$

If the rainfall is uniformly distributed over the whole catchment, this method gives better result. It does not take into account the topography and other influences.

(b) Thiessen Polygon Method:-

Thiessen polygon method is a common method of weighing the raingauge observation according to area. This method is also called weighted mean method. Thiessen assumed that rainfall at any point within the polygon is same as that of the nearest gauge. For this method all the gauges in and around the basin are considered.

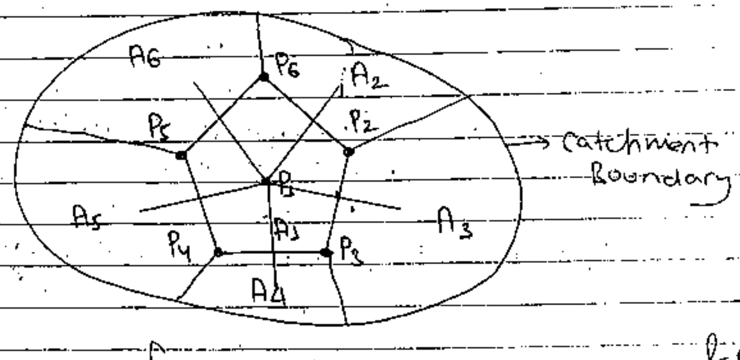


fig. Thiessen polygon

- consider a catchment with rainfall station ($P_1, P_2 \dots$).
- connect the adjacent rainfall station by straight lines forming triangle.
- Draw perpendicular bisector to each of the lines.
- The perpendicular bisector forms the boundary of polygon.
- Multiply the area of each Thiessen polygon by the raingauge value of the enclosed station.
- Finally divide by the total area 'A' of the basin to get the average precipitation or rainfall.

or, $$\bar{P} = \frac{P_1A_1 + P_2A_2 + \dots + P_6A_6}{A_1 + A_2 + \dots + A_6}$$

or, $$\bar{P} = \frac{\Sigma P_i A_i}{\Sigma A_i}$$

This method is superior to arithmetic mean method. It does not consider topographic effect. It assumes linear variation of precipitation between stations.

(c) Isohyetal Method: An isohyet is a line joini[ng] points of equal rainfall magnitude.

In the isohyetal method, the catchment area is drawn to scale and the raingauge stations are marked. The depth of rainfall at each station is measure. Isohyetal map is prepared for the storm causing the rainfall over the area using rainfall data. The area between successive isohyets is measured. Then the area is multiplied wi[th] average rainfall between the isohyet. Then, average rainfall is find using this formula,

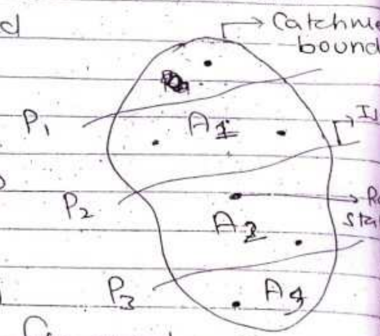


Fig. Isohyetal Metho[d]

$$\bar{P} = \frac{\Sigma A_i (P_i + P_{i+1})/2}{\Sigma A}$$

let isohyets represents rainfall $P_1, P_2, P_3 \dots$ in the area between $A_1, A_2, A_3 \dots A_{n-1}$. Then avg. precipitation is calculated by

$$\bar{P} = \frac{A_1\left(\frac{P_1+P_2}{2}\right) + A_2\left(\frac{P_2+P_3}{2}\right) + \dots + A_{n-1}\left(\frac{P_{n-1} \dots}{}\right)}{A_1 + A_2 + \dots + A_{n-1}}$$

This method is superior than other method.

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① Using Thiessen polygon method, compute the depth of average Precipitation over a basin using following data.

Raingauge	1	2	3	4	5	6
Area of polygon (cm²)	480	2860	2200	1600	710	270
Depth of precipitation (cm)	50	65	85	90	60	70

=> Here,

Raingauge	Area of polygon (A_i) (cm²)	Depth of precipitation (P_i) (cm)	$P_i A_i$
1	480	50	2400
2	2860	65	185900
3	2200	85	187000
4	1600	90	144000
5	710	60	42600
6	270	70	18900
Sum	$\Sigma A_i = 8120$		$\Sigma P_i A_i = 602400$

Average precipitation $(\bar{P}) = \frac{\Sigma P_i A_i}{\Sigma A_i}$

or, $\bar{P} = \frac{602400}{8120}$

or, $\bar{P} = 74.18$ cm

② For a drainage basin of 600 km², isohyets drawn for a storm gave the following data.

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Isohyetal Interval (cm)	15-12	12-9	9-6	6-3	3-1
Inter-isohyetal area (km²)	92	128	120	175	85

=> Here,

Rainfall in (cm) P_i	Area in km² (A_i)	$\frac{P_i + P_{i+1}}{2}$	$\left(\frac{P_i + P_{i+1}}{2}\right) A_i$
15		-	
12	92	13.5	1242
9	128	10.5	1344
6	120	7.5	900
3	175	4.5	787.5
1	85	2	170
Sum	$\Sigma A_i = 600$		$\Sigma\left(\frac{P_i + P_{i+1}}{2}\right) A_i = 4443.5$

Average Precipitation $(\bar{P}) = \frac{\Sigma\left(\frac{P_i + P_{i+1}}{2}\right) A_i}{\Sigma A_i}$

or, $\bar{P} = \frac{4443.5}{600}$

or, $\bar{P} = 7.41$ cm

③ consider a rectangular area whose coordinate are (0,0), (4,0), (0,4) and (4,4). The area

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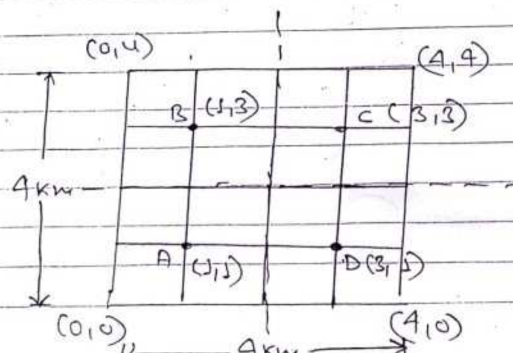
is 4km wide and 4km long. The area had 4 raingauge. The location of raingauge are given below. Determine the mean rainfall by Theissen polygon method.

Station	Gauge coordinate (km, km)	Rainfall (cm)
A	(1, 1)	5
B	(1, 3)	10
C	(3, 3)	8
D	(3, 1)	12

⇒ Here,
Rainfall at station A (P_A) = 5cm
" " " B (P_B) = 10cm
" " " C (P_C) = 8cm
" " " D (P_D) = 12cm

Area of polygon of for gauge station A
= 2×2
= 4km2



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Similarly, for gauge station B, Area = 4k
" " " C, Area = 4k
" " " D, Area = 4km

A/c to Theissen polygon Methon,

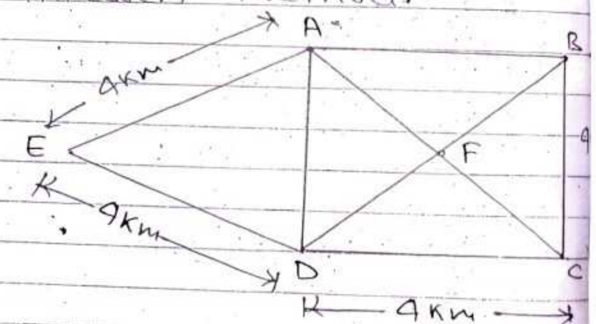
$$\text{Mean precipitation } (\bar{P}) = \frac{\Sigma P_i A_i}{\Sigma A_i}$$

$$\text{or, } \bar{P} = \frac{5\times4 + 10\times4 + 8\times4}{4+4+4+4}$$

$$\text{or, } \bar{P} = 8.75 \text{ cm.}$$

④ The area of catchment is composed of a square plus an equilaterial triangle of side 4km as shown in figure. The rainfall read at A, B, C, D, E, F are , 11cm, 8cm, 7cm, 4cm, 3cm and 5cm resp. Find the mean precipitation by Thiessen Method.

⇒ Here,
Precipitation at station A, B, C, D, E and F are

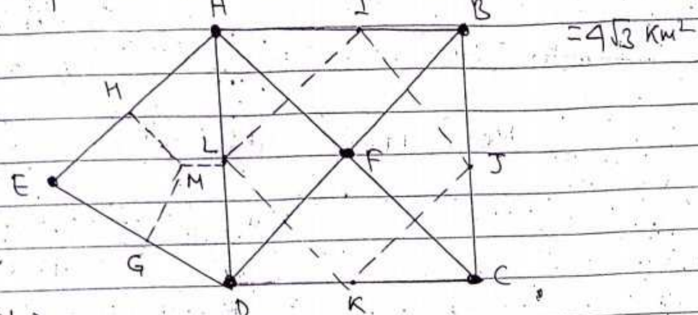


P_1 = 11 cm
P_2 = 8 cm
P_3 = 7 cm
P_4 = 4 cm
P_5 = 3cm
P_6 = 5cm resp. let us draw perpn bise

to each of the line.

Area of equilaterial $\Delta ADE = a^2\sqrt{3}/4 = 4^2\sqrt{3}/4$

$= 4\sqrt{3}\ Km^2$



Now,

Area covered by gauge station A

$A_1 = (\text{Area of } \Delta\, ALI + \text{Area of Polygon } AHML)$

$A_1 = \left[\frac{1}{2}\times2\times2 + \frac{4\sqrt{3}}{3}\right]$ [Area of equilaterial triangle $= a^2\sqrt{3}/4$]

$A_1 = 4.30\ Km^2$

Similarly,

Area of Polygon B (A_2) = Area of $\Delta\, BIJ$

or, $A_2 = \frac{1}{2}\times2\times2 = 2\ Km^2$

Area of polygon C (A_3) = Area of $\Delta\, JCK$

$= \frac{1}{2}\times2\times2$

$= 2\ km^2$

Area of polygon D (A_4) = [Area of $\Delta\, DKL$ + Area of polygon LMGD]

$= \left[\frac{1}{2}\times2\times2 + \frac{4\sqrt{3}}{3}\right]$

$A_4 = 4.30\ Km^2$

Area of polygon E (A_5) = Area of polygon MHEG

or, $A_5 = \frac{4\sqrt{3}}{3} = 2.31\ Km^2$

and

Area of Polygon F (A_6) = Area of square IJKL

$= [4\times4 - (2\times4)]$

$= 8\ Km^2$

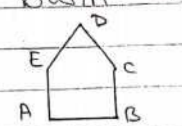
Then,

Mean precipitation $(\bar{P}) = \frac{\Sigma P_i A_i}{\Sigma A_i}$

or, $\bar{P} = \frac{(11\times4.30 + 8\times2 + 7\times2 + 4\times4.30 + 3\times2.31 + 5\times8)}{4.30+2+2+4.30+2.31+8}$

or, $\bar{P} = 6.17$ cm

(5) A basin has the area in the form of a pentagon with each side of 20Km as shown in figure. The five raingauges located at the corners A, B, C, D and E have recorded 60, 81, 73, 59 and 45mm of rainfall resp. compute the mean rainfall over the basin by Thiessen polygon method.



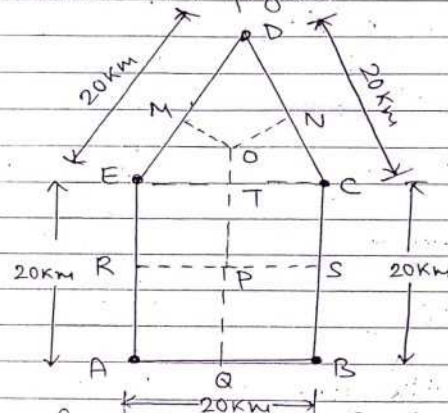
⇒ Here,

Given,

Precipitation at station (A) $P_1 = 60$ mm

Precipitation at station (B) $P_2 = 81$ mm
" " " C $(P_3) = 73$ mm
" " " D $(P_4) = 59$ mm
" " " E $(P_5) = 45$ mm

Now,
let us construct Thiessen Polygon as shown in figure.



Area of polygon A (A_1) = Area of sq. AQPR
or, $A_1 = 10 \times 10 = 100 km^2$

Area of polygon B $(A_2) = 10 \times 10 = 100 Km^2$

Area of polygon C (A_3) = [Area of sq. PSCT + Area of polygon CTON]

$$= \left[10 \times 10 + \frac{20^2\sqrt{3}}{4 \times 3}\right] = 157.73 \ Km^2$$

$$\left[\because \text{Area of equilaterial } \Delta = \frac{a^2\sqrt{3}}{4}\right]$$

Area of polygon D = Area of polygon DNOM

$$= \frac{20^2\sqrt{3}}{4 \times 3}$$

$A_4 = 57.73 Km^2$

and
Area of polygon E = [Area of sq. ETPR + Area of polygon ETOM]

or, $A_5 = \left[10 \times 10 + \frac{20^2\sqrt{3}}{4 \times 3}\right]$

or, $A_5 = 157.73 \ Km^2$

Hence,
Mean Precipitation $(\bar{P}) = \frac{\Sigma P_i A_i}{\Sigma A_i}$

$$\text{or, } \bar{P} = \frac{60 \times 100 + 81 \times 100 + 73 \times 157.73 + 59 \times 57.73 + 45 \times 157.73}{100 + 100 + 157.73 + 57.73 + 157.73}$$

or, $\bar{P} = 62.9$ mm

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⊛ Optimum Number of Rain Gauge

→ If there are number of raingauge which are already installed in the region, the information obtained from these gauges can be used to determine the optimum number of raingauge required for the area. The optimum number (N) depends upon the coefficient of variation (CV) of mean rainfall at the existing station and the allowable degree and is given by

$$N = \left(\frac{C.V}{X}\right)^2$$

Where,
N = optimum no of raingauge
C.V = Coeff. of variation
X = allowable degree of error (in %)

The different steps involved are as follows:

- Calculate total rainfall,

$$\Sigma P = P_1 + P_2 + \dots + P_n$$

Where,
n = total no. of existing rainfall in catchment.

- Calculate mean rainfall

$$\bar{P} = \frac{\Sigma P}{n}$$

- Calculate the sum, $\Sigma(P-\bar{P})^2$

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- Calculate S.D, $\sigma = \sqrt{\frac{\Sigma(P-\bar{P})^2}{n-1}}$
- Calculate coeff of variation $C.V = \frac{\sigma}{\bar{P}} \times 100\%$
- optimum no. of raingauge $(N) = \left(\frac{C.V}{X}\right)^2$

Where X = % of error

- Additional no. of raingauge $= (N - n)$

① A catchment has seven raingauge stations. In a year the annual rainfall in cm recorded by raingauge are as follows: 130, 142.1, 118.2, 108.5, 165.2, 102.1, 146.9. For a 5% error in the estimation of rainfall, calculate the minimum no. of additional station required to be established in the catchment.

⇒ Here,
No. of raingauge station (n) = 7

Rainfall (P_i) (cm)	$(P_i - \bar{P})$	$(P_i - \bar{P})^2$
130	−0.43	0.18
142.1	11.67	136.19
118.2	−12.23	149.57
108.5	−21.93	480.92
165.2	34.77	1208.95
102.1	−28.33	802.59
146.9	16.47	271.26
ΣP_i = 913		$\Sigma(P_i-\bar{P})^2$ = 3049.66

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Mean $(\bar{p}) = \frac{\Sigma P_i}{n} = \frac{913}{7} = 130.43$ cm

Standard deviation $(\sigma) = \sqrt{\frac{\Sigma(P_i - \bar{p})^2}{n-1}}$

or, $\sigma = \sqrt{\frac{3049.66}{7-1}}$

or, $\sigma = 22.54$

coeff. of variation $(C.V) = \frac{\sigma}{\bar{p}} \times 100\%$

or, $C.V = \frac{22.54}{130.43} \times 100\%$

or, $C.V = 17.28\%$

Error in estimation $(x) = 5\%$

$\therefore$ optimum no. of stations $(N) = \left(\frac{C.V}{x}\right)^2$

or, $N = \left(\frac{17.28}{5}\right)^2$

or, $N = 11.95 \simeq 12$

Hence,
No. of additional station $= N - n$
$= 12 - 7$
$= 5$

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(2) A catchment has 8 rain gauges. The an[illegible] rainfall recorded by the raingauge are follows. If it is desired to limit the e[illegible] in the mean value of rainfall not to exceed 10 cm, calculate the optimum no. raingauge stations.

→ Here,
no. of raingauge stations $(n) = 8$

Station	Rainfall (P_i) (cm)	$(P_i - \bar{P})$	$(P_i - \bar{P})^2$
A	70	−45	2025
B	90	−25	625
C	170	55	3025
D	100	−15	225
E	100	−15	225
F	140	25	625
G	130	15	225
H	120	5	25
Sum	$\Sigma P_i = 920$		$\Sigma(P_i - \bar{P})^2 = 7000$

Mean $(\bar{p}) = \frac{\Sigma P_i}{n} = \frac{920}{8} = 115$ cm

Standard deviation $(\sigma) = \sqrt{\frac{\Sigma(P_i - \bar{P})^2}{n-1}} = \sqrt{\frac{7000}{8-1}}$

or, $\sigma = 31.62$

$C.V = \frac{\sigma}{\bar{p}} \times 100\% \Rightarrow \frac{31.62}{115} \times 100\% \Rightarrow 27$

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error in estimation $(X) = \frac{10}{\bar{P}} \times 100\%$

or, $X = \frac{10}{115} \times 100\%$

or, $X = 8.7\%$

Then,
optimum no. of raingauge station

$$N = \left(\frac{C.V}{X}\right)^2$$

or, $N = \left(\frac{27.5}{8.7}\right)^2$

or, $N = 9.9 \simeq 10$

Hence,
Additional raingauge station $= N - n$
$= 10 - 8$
$= 2$

(3) In certain catchment, there are four rain gauge station, with their normal annual precipitation as 800, 520, 440 and 400mm resp. Determine the optimum no. of raingauge in the catchment, if it is desired to limit the error in mean rainfall calculation as 12%.

=> Here,
no. of raingauge station $(n) = 4$
error in estimation $(X) = 12\%$

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Station	Rainfall (P_i) (cm)	$(P_i - \bar{P})$	$(P_i - \bar{P})^2$
A	800	260	67600
B	520	-20	400
C	440	-100	10000
D	400	-140	19600
Sum	$\Sigma P_i = 2160$		$\Sigma(P_i - \bar{P})^2 = 97600$

Mean $(\bar{P}) = \frac{\Sigma P_i}{n} = \frac{2160}{4} = 540$

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

or, $\sigma = \sqrt{\frac{97600}{4-1}} = 180.37$

$C.V = \frac{\sigma}{\bar{P}} \times 100\% = \frac{180.37}{540} \times 100\% = 33.40\%$

optimum no. of raingauge station $(N) = \left(\frac{C.V}{X}\right)^2$

or, $N = \left(\frac{33.40}{12}\right)^2$

or, $N = 7.75 \simeq 8$

∴ No. of additional raingauge station
$= N - n$
$= 8 - 4$
$= 4$

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⊛ Presentation of Rainfall Data:

→ There are different methods for presentation of rainfall data.

- Mass curve
- Hyetograph
- point Rainfall

a) **Mass curve:–** The mass curve is a plot between the accumulated rainfall at a station against time plotted in chronological order. It is very ~~usf~~ useful in extracting the information on the intensity, duration, magnitude, starting & ending of a storm. A typical mass curve of rainfall at a station is shown in the figure below.

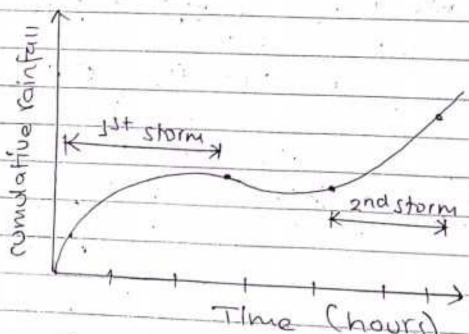

fig. Mass curve of rainfall

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(b) **Hyetograph:**

→ Hyetograph is a plot of intensity of rainfall against the time interval. The hyetograph is derived from the mass curve and usually rep-sented as a barchart. The graph represen the characteristics of storms and is useful in predicting the floods. The area under hyetograph gives total rainfall occured i that period.

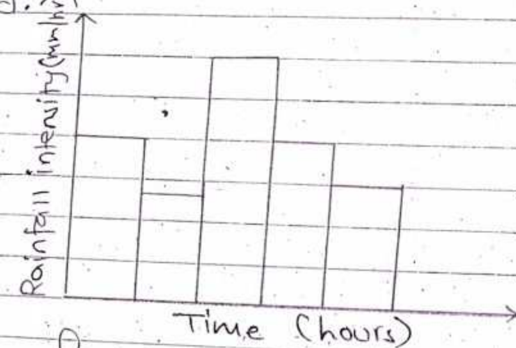
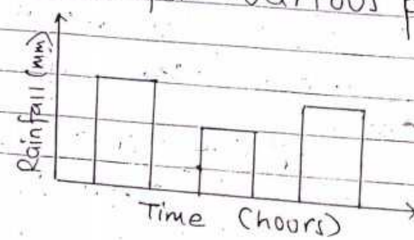


fig. Hyetograph of a storm

(c) **Point Rainfall:** point rainfall is a plot of rai versus time in bar diagram of a station o certain duration. Depending upon the need d can be listed as daily, weekly, monthly, seas or annual values for various periods.



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Intensity Duration Frequency (IDF) curve:

→ Intensity duration curve is a plot of average rainfall intensity (mm/hr) and duration. The most important use of this curve is to find the maximum intensity for any duration.

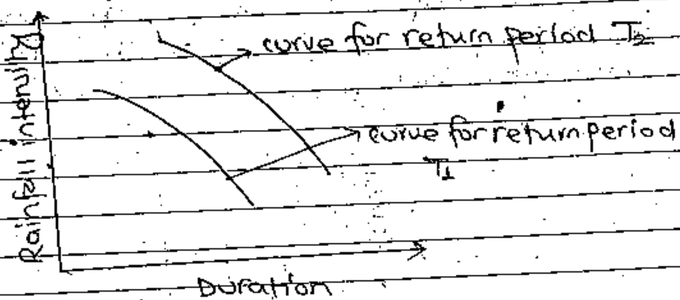

fig. IDF curve

A typical intensity duration curve is shown in above figure. From the curve it is seen that the intensity of storm decrease with the increase in storm duration. A storm of any given duration will have a larger intensity if its return period is large.

IDF curve can be expressed as equation in the exponential form given by

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

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Where,

i = intensity
T = return period or frequency
D = duration
K, x, a, n = constants.

① The rainfall depth with time duration is shown below.

Time (hr)	10:00	10:30	11:00	11:30	12:00	12:30	13:00	13:30	14:00	14:30	15:00	15:30	16:00
Rainfall (mm)	0	6	11	16	24	29	38	51	57	61	66	67	67

i) construct the hyetograph of storm using uniform time interval of 30 minutes and 2 hours.

ii) compute the maxm avg. intensities of rainfall for the duration of 30min, 1hr, 3hr and 5hr in this storm and plot the intensity duration curve.

→ Here,

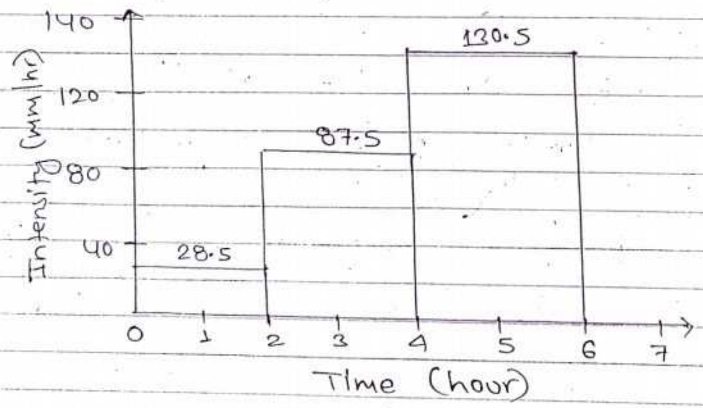
For 2 hour interval

Time in interval (hr)	Rainfall (mm)	Rainfall intensity (mm/hr)
2	57	28.5
4	175	87.5
6	261	130.5

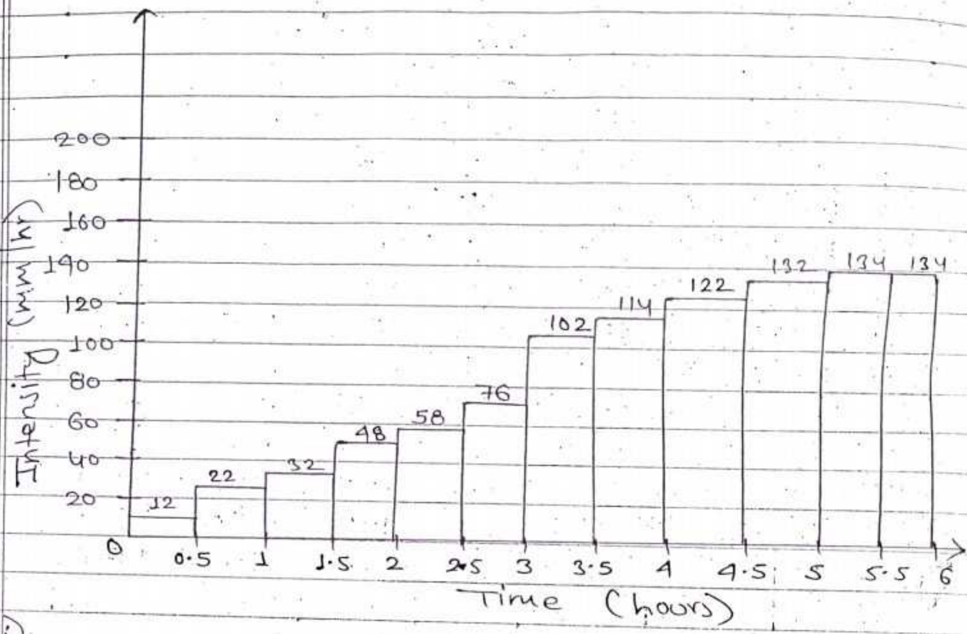
For 30min interval

Time interval (hr)	Rainfall (mm)	Intensity (mm/hr)
0.5	6	12
1	11	22
1.5	16	32
2	24	48
2.5	29	58
3	38	76
3.5	51	102
4	57	114
4.5	61	122
5	66	132
5.5	67	134
6	67	134

Then, (Hyetograph for 2hrs interval)



(Hyetograph for 30min interval)



ii)

P.T.O

Time interval (hr)	Rainfall (mm)	Cumulative rainfall (mm)	Rainfall in any Possible duration (mm)			
			30 min	1hr	3hr	5hr
10:00	0	0	–	–	–	–
10:30	6	6	6	–	–	–
11:00	11	17	11	17	–	–
11:30	16	33	16	27	–	–
12:00	24	57	24	40	–	–
12:30	29	86	29	53	–	–
13:00	38	124	38	67	124	–
13:30	51	175	51	89	169	–
14:00	57	232	57	108	215	–
14:30	61	293	61	118	260	–
15:00	66	359	66	127	302	359
15:30	67	426	67	133	340	420
16:00	67	493	67	134	369	476

From above table, max^m. rainfall and corresponding intensity for different duration is given below.

hour	Max^m rainfall (mm)	Max^m intensity (mm/hr)
0.5	67	134
1	134	134
3	369	123
5	476	95.20

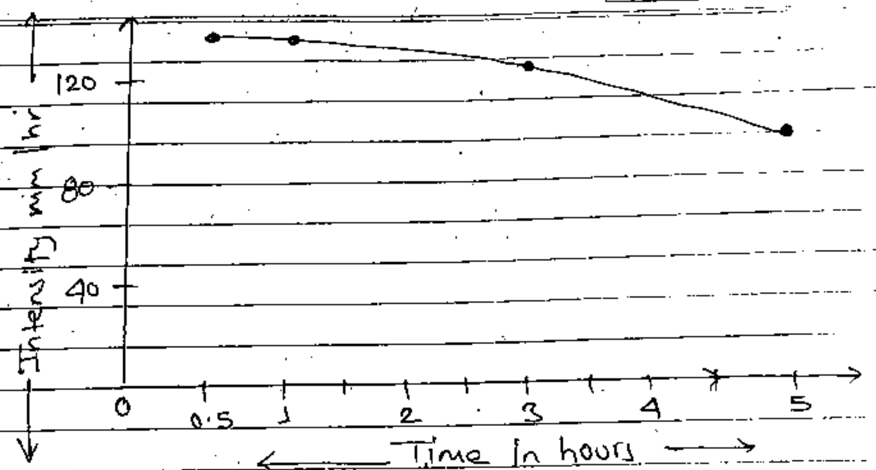


Fig. Intensity duration Curve.

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Hydrological Losses

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Initial Losses:

In hydrology we study about the occurrence, distribution and movement of water on earth, including that in the atmosphere and below the surface of earth. While studying various hydrological phenomenon rainfall losses also come into account. Rainfall losses or hydrological losses are generally considered to be the result of evaporation of water from land surface, interception of rainfall by vegetal cover, infiltration of water into the soil matrix and depression storage on the land surface. Out of these various factors for hydrological losses interception and depression storage comes under initial losses.

Interception:- Interception refers to the precipitation that doesnot reach the soil but is instead intercepted by leaves and branches of plants, buildings etc. The intercepted water finally gets evaporated without contributing to runoff.

Depression Storage:- Depression storage capacity in soil is the ability of a particular area of land to retain

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water in pits and depression, thus preventing it from flowing.

(ii) Evaporation:- Evaporation is the process by which the water is changed into vapour. The vapour goes to the atmosphere and forms cloud. Evaporation occurs when molecules of water attain high kinetic energy to eject themselves from the watersurface into the atmosphere.

(iii) Factors affecting Evaporation:-

→ The various factors affecting the rate of evaporation are discussed below:-

(a) Temperature:- As the temperature increases the kinetic energy of water molecules also increases due to which the rate of evaporation also increases. But for same temp. evaporation in colder months is less than in summer due to environmental conditions.

(b) Wind:- Wind helps to carry moisture away as it evaporates and thus accelerates the rate of evaporation. The rate of evaporation increases with the wind speed up to a critical speed beyond which any further increase in wind speed has no influence on rate of evaporation

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...liation: Radiation is the direct transfer energy by ~~en~~ means of electromagne- waves. This directly influences the ...perature of evaporating surface.

...mospheric pressure:- Increase in the ...mospheric pressure decrease the rate evaporation and viceversa.

...midity: Amount of water vapour in air called humidity. If humidity is more(↑) ...aporation will be less and viceversa.

...rface area: A substance that has a ...ger surface area will evaporate (↑) ...ter as there are more surface molecule unit of volume that are potentially ...le to escape.

...ility of water: If water contains some ...solved solids then evaporation reduces. eg. salt content in water affects the ...te of evaporation. From experiment it seen that the evaporation of sea ...ter is less than that of fresh water.

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(h) Vapour pressure Difference: The rate at which the molecules leave the surface depends on the vapor pressure of the liquid. similarly, the rate at which molecules enter the water depends on the vapour pressure of the air. The rate of evaporation therefore depends on the difference between saturation vapour pressure at the water temperature and actual vapour pressure in air. Higher the difference, more the evaporation.

Estimation of Evaporation (2010)

→ Evaporation is usually estimated as the depth of water lost per day (or month or year) in centimeter. Generally three methods are used to estimate the evaporation. They are as follows:

- → Empirical method
- → Analytical method
- → Evaporation pan method (using Evaporimeter)

1) Empirical Method:

Empirical equations used for estimating evaporation are functions of saturation vapour pressure at the water temperature (e_s) and actual vapour pressure in the air (e_a).

A/c to Dalton's law, the rate of evaporation (E_r) in mm/day is given by

$$E_r = C(e_s - e_a)$$

Where,

C = constant, which depends on the barometric pressure, wind velocity

e_s = Saturation vapour pressure (mm of Hg)

e_a = actual vapour pressure (mm of Hg)

Meyer's formula:

$$E = C\left[1 + \frac{U}{16}\right](e_s - e_a)$$

Where,

E = evaporation in (mm/day)

C = coefficient [0.36 for lakes, 0.5 for small water depth]

U = Monthly mean wind velocity in km/hr measured at 9m above ground

Rhower's formula:

$$E = 0.771\left[1.465 - 0.000732P\right]\left[0.44 + 0.0733U\right](e_s - e_a)$$

Where,

P = Mean barometric reading in mm Hg

V = Mean wind velocity at 0.6m above ground in km/hr. $\left[V = cz^{0.143}\right]$

2) Analytical Method: (2007)

Analytical methods are used for the determination of lake evaporation which can be broadly classified into three categories. They are as follows:

- → Water-budget Method
- → Energy-budget Method
- → Mass transfer Method

Ravi

a) **Water-Budget Method:** The water-budget method is the simplest method among other analytical methods and is also least reliable. It depends upon water/Mass balance eqn.

Thus,

ΣInflow − ΣOutflow = Change in storage + Evaporation loss

or, $E = \Sigma I - \Sigma O \pm \Delta S$

Considering the daily average values for a lake, the continuity equation is written as

$E = (P + I_{sf} + I_{gf}) - (O_{sf} + O_{gf} + T) \pm \Delta S$

where,

P = precipitation

I_{sf} = surface inflow

I_{gf} = Ground water inflow

O_{sf} = surface water outflow

O_{gf} = Ground water outflow

T = Transpiration loss

ΔS = Change in storage.

Measurement of I_{gf}, O_{gf} and T is not possible, so these are only be estimated. This method gives satisfactory ~~result~~ result and is good for long term evaporation of a reservoir.

b) **Energy-budget Method:** Energy-budget method is based on conservation of energy. Hence, energy available for evaporation is determined by considering the incoming, outgoing and energy stored.

Thus,

Incoming energy = outgoing energy + stored energy

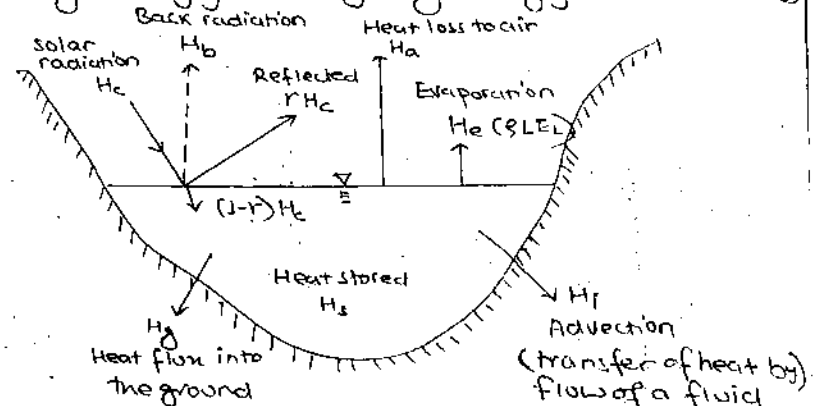

fig. Energy balance in a water Body

Consider a water body as shown in fig. The energy balance to evaporating surface in a period of one day is given by

$$H_n = H_a + H_e + H_g + H_s + H_i \longrightarrow (1)$$

where,

H_n = net heat energy by water surface

$= H_c(1-r) - H_b$

H_a = heat transfer from water surface to air.
H_e = heat energy used in evaporation
H_g = heat transfer to ground
H_s = heat stored in water body
H_i = Net heat conducted out of the system by water flow.
H_c = incoming solar radiation
rH_c = reflected radiation

For short period of time H_s and H_i can be neglected. All the terms expect H_i can either be measured or evaluated indirectly.
H_a is estimated using Bowen's ratio.

$$\beta = \frac{H_a}{H_e} = \frac{H_a}{\rho \cdot E_L \cdot L} \longrightarrow (ii)$$

where, β = Bowen ratio
ρ = Density of water
E_L = Evaporation loss
L = Latent heat of vaporization.

Adding 1 on both sides in eqn (ii)

$$1+\beta = 1 + \frac{H_a}{H_e}$$

or, $H_a + H_e = H_e(1+\beta)$

From eqn (i).

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

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

or, $H_n - H_g - H_i - H_s = (1+\beta)\,\rho \cdot L \cdot E_L$

or, $$E_L = \frac{H_n - H_g - H_i - H_s}{(1+\beta)\cdot \rho \cdot L} \longrightarrow (iii)$$

For short period of time, H_s and H_i can be neglected. ∴ eqn (iii) becomes

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

c) <u>Mass transfer Method:</u> This method is based on theories of turbulent mass transfer in boundary layer to calculate the mass water vapour transfer from the surface of surrounding.

3) <u>Evaporation Pan Methods</u>

Evaporation pan, also called Evaporimeter is shallow vessels containing water. These pans are placed in open to measure the loss of water by evaporation. The measurement result from the evaporimeter may be affected by sitting of the pan, color of pan and heat transfer through the side of the pan. Water is placed in the evaporation pan and the change in

depth of water due to evaporation is measured.
There are different types of evaporation pans used during the estimation of evaporation. They are

- Class A Evaporation pan
- ISI standard pan
- colorado sunken pan

- class A Evaporation pan

→ The pan is standard pan of 1210 mm diameter and 255 mm depth which is used by US weather Bureau. The pan is normally made of un-painted galvanized iron sheet.

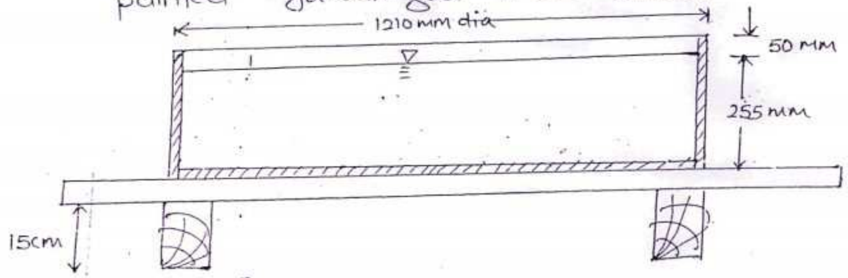

fig. class A Evaporation pan

The pan is placed 15cm above the ground surface in such a way that it gets free circulation of air. Readings are taken daily in the morning.

- ISI standard pan

→ This pan evaporimeter is also called modified class A pan. It consists of a pan 1220 mm in diameter with 255 mm of depth. The pan is made of copper sheet, tinned inside and painted white outside. The top of pan is covered due to which evaporation loss is less as compared to unscreen pan.

- Colorado sunken pan

→ The pan is burried into the ground such that the water level is at the ground level. It is 920mm square and 460mm deep made up of unpainted galvanised iron sheet.

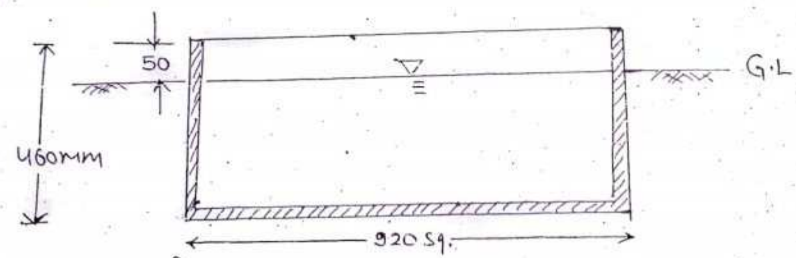

fig. Colorado sunken pan

The advantage of this type of pan is that radiation and aerodynamics characteristics are similar to those of lake.

Pan coefficient (2008)

→ Evaporation pans are not exact models of large reservoirs. So, the evaporation observed from a pan has to be corrected to get the evaporation from a lake under similar climatic and exposure conditions. Thus, a coeff is introduced as C_p which is given by

$$C_p = \frac{\text{lake evaporation}}{\text{pan evaporation}}$$

and is known as pan coefficient. Different value of C_p is used for different pans. Generally experimental value for pan coefficient ranges from 0.67 to 0.82.

(2010)

⊛ Methods to reduce evaporation losses

→ The different methods available for reduction of evaporation losses are as follows.

a) Reduction of surface area: The volume of water lost by evaporation is directly proportional to the surface area of the water body. hence the reduction of surface area also reduces the evaporation losses.

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b) Mechanical covers: permanent roofs over the reservoir, temporary roofs and floating roofs such as raft can be adopted to minimize the evaporation loss.

c) Chemical films: A thin chemical film can be sprayed on the water surface to reduce the evaporation loss. Certain chemicals such as cetyl alcohol can be used as an evaporation inhibitor.

d) prevent wind exposure: wind helps to carry moisture away as it evaporates and thus accelerates the rate of evaporation. So, to reduce water loss adjust the landscape around reservoir with walls and hedge that creates shelter from the wind.

e) Addition of soluble salts: when soluble salts are dissolved in water, the vapour pressure of the solution is less and hence causes reduction in the rate of evaporation.

f) Use of underground storage rather than use of surface storage.

g) plantation of trees to act as wind breakers.

⊛ Transpiration

→ Transpiration is the vaporization of liquid water contained in plant tissues and reaches to the atmosphere as water vapour. The water is taken up by the plant root system and escape through the leaves. Transpiration depends on day light hours and depends on the growth period of plant.

The factors affecting transpiration are discussed below.

- Temperature
- solar radiation
- Wind
- light intensity
- Soil Moisture
- Atmospheric pressure
- Characteristics of plant, root & leaf system.

⊛ Evapotranspiration

→ Evapotranspiration is defined as the total amount of water used by the plant in transpiration (building of plant tissues) and evaporation from adjacent soils or from plant leaves in any specified time.

The process by which water is transferred from the land to the atmosphere by evaporation from soil and other surface and by transpiration from plants.

- Potential Evapotranspiration (PET) (2006) (2012)

→ Potential evapotranspiration is the evapotranspiration from the short, green vegetation when the roots are supplied with unlimited water covering the soil. PET depends on climatological factors rather than characteristics of soil & vegetation.

- Actual Evapotranspiration (AET) (2011)

→ The real evapotranspiration occuring in a specific situation is called AET. It is largely affected by the characteristics of soil and vegetation. AET in a field can be measured by an instrument called a lysimeter.

(IMP) (2008) (2009)

Factors affecting Evapotranspiration

→ Evapotranspiration depends upon all those factors on which evaporation and transpiration depend.

a) Area of the water surface : The amount of evapotranspiration is directly proportional to the area of evaporation. If the exposed area is large, the evapotranspiration will be more and vice versa.

b) Depth of Water : The depth of water influences the evaporation. More depth reduces the summer evaporation and increase the winter evaporation.

c) Humidity: If the humidity is more the evapotranspiration will be less. This is because during the process of evaporation water vapour moves from the point of higher moisture content to the point of lower moisture content. But if the atmospheric air contain higher moisture then evaporation will be less.

d) Temperature : The process of evapotranspiration also depends upon the temperature. If the temp is high, the saturation vapour pressure increases and the evapotranspiration also increases. Thus, in summer the evapotranspiration is more as compared to that in winter season.

e) Wind velocity: The process of evapotranspiration also depend upon the turbulence in the air. Wind helps to carry moisture away as it evaporates and thus accelerates the rate of evapotranspiration.

f) Atmospheric pressure: Increase in atmospheric pressure decreases the rate of evapotranspiration and vice versa.

g) Quality of water: The quality of water in the water body also affects the rate of evapotranspiration. If the water contains dissolved solid, salts reduces the evapotranspiration rate.

⊛ Measurement of Evapotranspiration

→ The measurement of evapotranspiration can be carried out in following two ways:-

- → Lysimeter Method
- → Field Plots.

(2006)

a) Lysimeter Method :

A Lysimeter is a special watertight tank containing a block of soil and set in a field of growing plants. The tank is buried in ground such that its top is made like the surrounding ground surface. The plants grown in the lysimeter are the same as in the surrounding field. Water is applied to the lysimeter for the satisfactory growth of plant. Evapotranspiration is ~~measured~~ estimated in terms of the amount of water required to maintain constant moisture conditions within the tank measured either volometrically or gravimetrically. Lysimeter should be designed to accurately reproduce the soil condition, moisture content, type and size of vegetation of the surrounding area. Lysimeter studies are time-consuming and expensive.

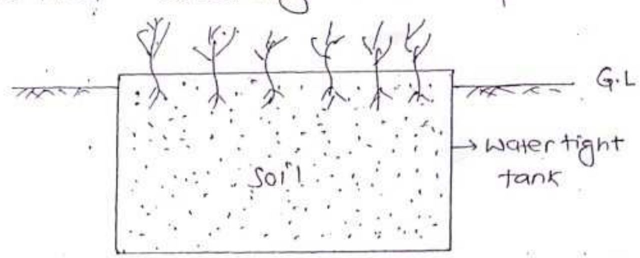


fig. Lysimeter

b) Field Plots:

In this method all the elements of the water budget in a known interval of time are measured in a special plot. Then the evapotranspiration are determined as

Evapotranspiration = [Precipitation + Irrigation in – runoff – ground water loss]

Ground water loss due to percolation is difficult to measure so it can be minimized by keeping the moisture condition of the plot at the field capacity. This methods provides fairly reliable results.

⊛ Evapotranspiration Equation:

→ The lack of reliable field data and the difficulties of obtaining reliable evapotranspiration data have given rise to no. of methods to predict PET by using climatological data.

✱ PENMAN'S Equation:

Penman's equation is based on sound theoretical reasoning and is obtained by a combination of the energy-balance and mass-transfer approach. It can be applicable where there is data of temperature, wind, radiation etc available and it gives most satisfactory result

It is given by

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

Where,

PET = daily potential evapotranspiration in mm/day.
A = Slope of saturation vapour pressure Vs temp. curve at the mean air temp in mm of Hg/°c.
H_n = Net radiation, mm/day
E_a = parameter including wind velocity and saturation deficit. (mm/day)
γ = psychrometric constant (mm Hg/°c) & can be taken as 0.49 mm of Hg/°c.

The net radiation is estimated by the following equation.

$$H_n = H_a(1-r)\left(a + b\frac{n}{N}\right) - \sigma T_a^4 (0.56 - 0.092\sqrt{e_a})\left(0.10 + 0.90\frac{n}{N}\right)$$

Where,

H_a = incident solar radiation outside the atmosphere (mm/day)
a = constant depending upon the latitude ϕ and given by $a = 0.29 \cos\phi$
b = constant with an average value of 0.52
n = actual duration of bright sunshine in hrs.
N = Max^m possible hours of sunshine.
r = reflection coefficient (albedo).
σ = Stefan-Boltzman constant $= 2.01 \times 10^{-9}$ mm/day
T_a = mean air temp in degree Kelvin = 273 + °c.
e_a = actual mean vapour pressure in air in mm of Hg.

The parameter E_a is estimated as

$$E_a = 0.35\left[1 + \frac{U_2}{160}\right](e_w - e_a)$$

Where, U_2 = Mean wind speed at 2m above ground in km/day
e_w = saturation vapour pressure at mean temp in mm of Hg
e_a = actual mean vapour pressure in air in mm of Hg.

<u>Numericals:</u>

1) Calculate evaporation rate from an open water source if the net radiation is 300 W/m² and the air temp is 30°C. Assume value of zero for sensible heat (H_a), ground heat flux (H_g), heat stored in water body (H_s) and advected energy (H_i). The density of water at 30°C is 996 kg/m³.

latent heat of vaporization is given by the relation: $(2501 - 2.37T)$ KJ/kg.

→ Given:

Temperature $(T) = 30°C$.

Net radiation $(H_n) = 300 W/m^2$

Density of water $(\rho) = 996 \ kg/m^3$

Latent heat of vaporization $(L) = (2501 - 2.37 \times 30)$

or, $L = 2429.9$ KJ/kg

From Energy budget Method

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

or, $E = \dfrac{300-0-0-0}{2429.9 \times 10^3 \times 996}$ [For $H_a = 0$, Bowen ratio $(\beta) = 0$]

or, $E = 1.24 \times 10^{-7}$ m/sec

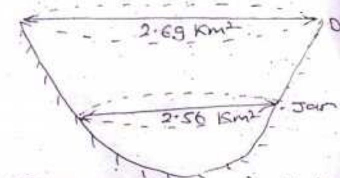
or, $E = 10.71$ mm/day.

2) The monthly evaporation (in mm) observed on a pan from january to December in 2006 is as follows:

111, 126, 127, 132, 141, 146, 148, 143, 138, 126, 118, 114. The water spread of reservoir in January 2006 and December 2006 were 2.56 and 2.69 Km^2 resp. Assuming a pan coefficient of 0.8. Find the total loss of water due to evaporation in the given year from reservoir. Neglect all losses.

→ Here, $A_1 = 2.56 \ km^2$, $A_2 = 2.69 \ km^2$

Mean area of the reservoir $(A) = \frac{1}{3}\left[A_1 + A_2 + \sqrt{A_1 A_2}\right]$



or, $A = \frac{1}{3}\left[2.56 + 2.69 + \sqrt{2.56 \times 2.69}\right]$

or, $A = 2.624 \ Km^2$.

Total loss of water due to evaporation in a year on a pan $= (111 + 126 + 127 + 132 + 141 + 145 + 148 + 143 + 138 + 126 + 118 + 114)$

$= 1570$ mm

Now,

$$\text{Pan coefficient} = \frac{\text{lake / reservoir evaporation}}{\text{pan evaporation}}$$

∴ Reservoir evaporation $= 1570 \times 10^{-3} \times 2.624 \times 10^6 \times 0.8$

or, Reservoir evaporation $= 3.29 \times 10^6 \ m^3$.

3) Compute the daily evaporation from a Class-A Pan of the amount of water added to bring the level to the fixed point are as follows.

Day	1	2	3	4	5	6	7
Rainfall (mm)	14	6	12	8	0	5	6
Water added (mm)	-5	3	0	0	7	4	3

What is evaporation loss of water in this week from a lake if surface area of lake is 640 ha. Assume pan coefficient as 0.75.

→ Here,

Day	1	2	3	4	5	6	7
Evaporation (E_p)	9	9	12	8	7	9	9

Evaporation in a pan in that week $= \Sigma E_p$
$= 63\,mm$

Then,

lake evaporation = pan evaporation × pan coeff.
$= 63 \times 0.75$
$= 47.25\,mm$

water loss due to evaporation = Area of lake × lake evaporation
$= 47.25 \times 10^{-3} \times 640 \times 10^4$
$= 302400\,m^3$.

(2005)

4) A reservoir had an average surface area of $20\,km^2$, during june 1982. In that month the mean rate of flow was $10\,m^3/sec$. Total rainfall was 10 cm and change in storage was 16 M m^3. Assuming seepage losses to be 1.8 cm, estimate evaporation in that month.

→ Here,

Surface area of reservoir (A) = $20\,km^2$

Mean rate of inflow = $10\,m^3/sec$.
$= \frac{10 \times 60 \times 60 \times 24 \times 30}{20 \times 10^6}$
$= 1.296\,m$

Total rainfall (ie. precipitation) = 10 cm = 0.1 m

Change in storage (ΔS) = $16\,Mm^3 = \frac{16 \times 10^6}{20 \times 10^6}$

or, $\Delta S = 0.8\,m$

Seepage loss = 1.8 cm = 0.018 m

Evaporation = ?

∴ Evaporation = Inflow + precipitation − change in storage − seepage loss
$= (1.296 + 0.1 - 0.8 - 0.018)$
$= 0.578\,m/month$
$= 19.267\,mm/day$.

(2006)

5) What is the evaporation if 4.75 litre of water is removed from an evaporation pan of 1.22 m diameter and simultaneous rainfall measurement is 8.8 mm.

→ Here,

Diameter of pan (d) = 1.22 m

Rainfall measurement = 8.8 mm = 8.8×10^{-3} m.

Vol. of rainfall = pan area × rainfall

$= 8.8\times10^{-3} \times \left(\frac{\pi \times 1.22^2}{4}\right)$

$= 0.010287\ m^3$

Vol. of water removed = 4.75 litre

$= 4.75\times10^{-3}\ m^3$

∴ Evaporation = Vol. of rainfall − Vol. of water removed

$= 0.010287 - 4.75\times10^{-3}$

$= 5.537\times10^{-3}\ m^3$.

6) Determine the evapotranspiration and irrigation requirement for wheat, if the water application efficiency is 65% and consumptive use coefficient is 0.8 from following data.

Month	Mean temp (°c)	Monthly % of sunshine (hr)	Effective rainfall (cm)
Nov	18	7.20	2.6
Dec	15	7.15	2.8
Jan	13.5	7.30	3.5
Feb	14.5	7.10	2.0

⇒ Given:

Water application efficiency (η_a) = 65%.

consumptive use coeff. (k) = 0.8

Month	T (°c)	% of sunshine p (hr)	Effective rainfall (Pe)(cm)	$f = \frac{P(4.6T+81.3)}{100}$
Nov	18	7.2	2.6	11.82
Dec	15	7.15	2.8	10.74
Jan	13.5	7.3	3.5	10.48
Feb	14.5	7.1	2.0	10.50
			$\Sigma Pe = 10.9$	$\Sigma f = 43.54$

∴ seasonal consumptive use (U) = $k\Sigma f$

or, $U = 0.8 \times 43.54$

and, or, $U = 34.83$ cm

Irrigation requirement (F.I.R) = $\frac{U - \Sigma Pe}{\eta_a}$

or, $F.I.R = \frac{34.83 - 10.9}{0.65}$

or, F.I.R = 36.815 cm.

7) The following observations were taken from 8am to 6pm in a 1.5m dia. circular pan. Quantity water added to keep the water level in the pan constant = 5 litre, precipitation (P) = 10mm, leakage from pan = 1 litre. Estimate the rate of evaporation from the pan in mm/m²/hr.

→ Here,

Diameter of pan (d) = 1.5m

Area of pan (A) = $\frac{\pi}{4}\times1.5^2 = 1.767\ m^2$

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Quantity of water added (Q) = 5 l = $5 \times 10^{-3}\ m^3$

Vol. of precipitation (p) = 0.01×1.767

or, $p = 0.01767\ m^3$

leakage (L) = 1 l = $0.001\ m^3$

Evaporation rate (E_r) = ?

We know,

Evaporation (E) = Q + p − L

or, E = 0.005 + 0.01767 − 0.001

or, E = $0.02167\ m^3$

Time duration (t) = 10 hrs.

∴ Rate of Evaporation (E_r) = $\frac{E}{A \times t}$

or, $E_r = \frac{0.02167}{1.767 \times 10} \times 10^9$

or, $E_r = 1.226 \times 10^6\ mm^3/m^2/hr$

(2011)

8) Calculate the PET for an area over Ktm in the month of January by Penman method by using following data.

Mean monthly temp = 11.5°c

Mean relative humidity (RH) = 75%

Mean sunshine hrs = 9 hrs

Potential sunshine hrs = 11.6 hr

Wind velocity at 2m height = 100 km/day

Albedo = 0.15

Upper terestrial solar radiation (H_a) = 8 mm of Hg/day

Latitude = 26.5°, Longitude = 84.5°

Saturated vapour pressure at 11.5°C (e_w) = 10.4 mm of Hg

Slope of saturated vapour pressure curve (A) = 1.24 mm/°c

Psychrometric constant (γ) = 0.49 mm/°c

Boltzman constant (σ) = 2.01×10^{-9} mm/day.

→ Given:

$T_a = (11.5 + 273) = 284.5\,K$, RH = 75%

n = 9 hr, e_w = 10.4 mm of Hg

N = 11.6 hr, A = 1.24 mm/°c

u_2 = 100 km/day, γ = 0.49 mm/°c

r = 0.15 (reflection coeff), σ = 2.01×10^{-9} mm/day

H_a = 8 mm of Hg/day, PET = ?

φ = 26.5

We know that,

$a = 0.29 \cos\phi = 0.29 \cos 26.5 = 0.26$

$b = 0.52$

Actual vapour pressure (e_a) = RH × e_w

or, $e_a = 0.75 \times 10.4$

or, $e_a = 7.8$ mm of Hg

Net radiation (H_n) is calculated by

$$H_n = H_a(1-r)\left(a+b\frac{n}{N}\right)-\sigma T_a^4\left[0.56-0.092\sqrt{e_a}\right]\times(0.1+0.9\,n/N)$$

or $H_n = 8(1-0.15)\left(0.26+0.52\times\frac{9}{11.6}\right)-2.01\times10^{-9}\times284.5^4$
$(0.56-0.092\sqrt{7.8})\left(0.1+0.9\frac{9}{11.6}\right)$

or $H_n = 1.326$ mm/day

Aerodynamic contribution (E_a) is computed by $E_a = 0.35\left(1+\frac{u_2}{160}\right)(e_w-e_a)$

$= 0.35\left(1+\frac{100}{160}\right)(10.4-7.8)$

$= 1.479$ mm/day

A/c to penman eq^n

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

or $PET = \frac{1.24\times1.326+1.479\times0.49}{1.24+0.49}$

or $PET = 1.369$ mm/day.

9) At the begining of a certain week, the depth of water in an evaporation pan, 1.2m diameter was 7.75cm. During the week, the rainfall was 3.8cm and 2.5cm of water was removed from pan. At the end the rain guage indicated a depth of 8.32cm of water in the pan. Assume pan coefficient 0.7, estimate the evaporation during the week.

→ Here,

Initial depth of water in the pan = 7.75cm
Rainfall = 3.8cm
Water removed = 2.5cm
Net addition of water in the pan = 3.8 − 2.5 = 1.3cm
Depth of water at the end = 7.75 + 1.3 = 9.05cm
Actual depth at the end of week = 8.32cm
∴ Evaporation lost from pan = 9.05 − 8.32
= 0.73cm

Evaporation loss from reservoir = pan coeff. × evaporation lost from pan
= 0.7 × 0.73
= 0.511 cm.

10) A reservoir with a surface area of 300ha had the following average meterological value during a given week.
Water temp. = 30°c
Relative humidity (R.H) = 50%.
Wind velocity at 1m above ground = 1? km/h

Mean barometer reading = 750 mm of Hg
Estimate the daily evaporation from the lake reservoir & the vol. of water evaporated from the lake during this week.

→ Here,

From Meyer's formula:

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

where,

$K_m = 0.36$ for large deep water

e_s = saturation vapour pressure at 30°C
= 31.82 mm of Hg

We know,

$$R.H = \frac{e_a}{e_s}$$

or, $e_a = 50\% \times e_s$

or, $e_a = 0.5 \times 31.82 = 15.91$ mm of Hg

$V_9 = ?$ $\quad [V = CZ^{0.143}]$

$$\frac{V_9}{V_1} = \left[\frac{9}{1}\right]^{0.143} \Rightarrow V_9 = 16.43 \text{ km/hr}$$

Substituting these values in eqn ①

$$E = 0.36(31.82 - 15.91)\left[1 + \frac{16.43}{16}\right] = 11.61 \text{ mm/day}$$

∴ Total evaporation in a week

$$= \frac{11.61}{1000} \times 7 \times 300 \times 10^4$$

$= 223810 \text{ m}^3$.

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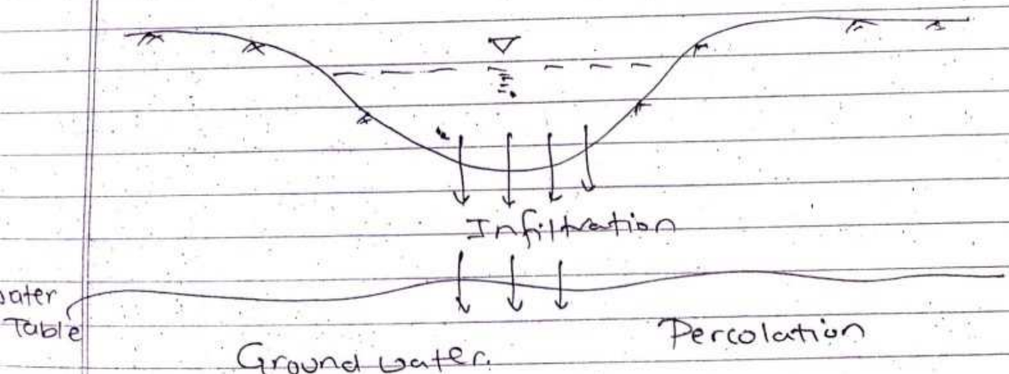
Infiltration: →

→ Infiltration is the process by which water enters the soil from the ground surface. It is one of the important component of hydrological phenomenon. It has great influence on rainfall, runoff, transpiration of plants and evaporation. Infiltration replenishes the soil moisture deficiency. The excess water then moves downward by the force of gravity. This downward movement under gravity is called percolation or seepage. It is responsible for subsurface and ground water flow. The infiltration rate is used for the computation of water losses due to infiltration for the determination of surface runoff.



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Infiltration capacity:-

It is the maximum rate at which water can enter the soil at a particular point under a given set of conditions.
It is expressed in mm/hr or cm/hr.

Factors affecting Infiltration:

→ The different factors affecting infiltration are discussed below.

- → soil characteristics
- → Intensity of rainfall
- → Temperature
- → soil Moisture
- → compaction due to rain
- → Urban areas
- → Vegetative cover.

⊙ **Soil characteristics:-** Textural and structural properties of soil affect infiltration. A light texture soil has greater rate of infiltration than the big size particle. The infiltration rate of sandy soil is greater than clayey soil.

⊙ Intensity of Rainfall: If the Intensity of rainfall is more, the impact of rain drop on the soil surface cause fine particles of soil to be displaced which may deposited in the voids of soil reducing the infiltration capacity.

⊙ Temperature:- Increase in temperature reduces the viscosity of water. so, water enters rapidly into soil. Hence, temp is directly proportional to infiltration.

⊙ Soil Moisture:- The amount of soil moisture effect the Infiltration capacity. In winter season the soil moisture is generally high and the infiltration is low. similarly, in summer season, the soil moisture content is low and so infiltration increases.

⊙ compaction due to Rain: When rain falls over the soil, the soil automatically undergoes mechanical compaction. This compaction reduces the voids in fine soil and hence, reduces their infiltration capacity.

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⊙ Urban areas: The most of the urban area are covered by pavements, buildings and lined drainage system. Rain falling on urban areas eventually flows as runoff through drains. Therefore infiltration in urban area is very low.

⊙ Vegetative Cover:- considerable increase in infiltration rate take place when the soil has full dense cover because runoff water is reduced by the vegetative cover.

⊕ Infiltration Indices:

The average constant value of infiltration rate is called infiltration index. It may be also defined as the avg rate, which is to be substracted from rainfall rate to get the surface runoff. It is useful in computation of surface runoff and flood discharge. There are two mostly used infiltration indices. They are

- φ-index
- W-index

⊙ φ-index: The average rainfall intensity above which the rainfall volume equals to the runoff volume is called φ-index. The φ-index is derived from the rainfall hyetograph with the knowlege of resulting runoff volume.

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The φ value is found by treating it as a constant infiltration capacity. If the rainfall intensity is less than φ, then infiltration rate is equal to the rainfall intensity and if the rainfall intensity is larger than φ, the difference between the rainfall and infiltration rate in an interval of time represents runoff volume.

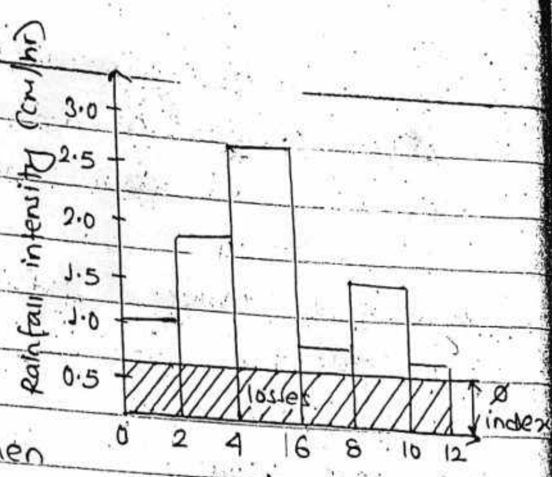

fig. φ-index

- The amount of rainfall in excess of the index is known as effective rainfall or excess rainfall.

- Procedure for calculation of φ-index:

→ consider a rainfall hyetograph and direct runoff. Now trail and error approach is adopted.

- Choose trail value of φ.
- compute rainfall excess :

  Rainfall excess = observed rainfall (R) − φ Δt

1cm 0.5cm

Δt = interval of rainfall data.

- compute total rainfall excess.
- compare total rainfall excess with direct runoff.
- If rainfall excess (Re) is not same to that of direct runoff (Q), take another value of φ.

  If Re > Q, increase φ

  If Re < Q, decrease φ.

- Repeat the procedure until excess rainfall equals to direct runoff.

- W-index: It is defined as the average rate of infiltration during the period when the rainfall intensity exceeds the infiltration rate. It is considered as an improvement over φ-index because initial losses (interception and surface storage are also considered.

$$W = \frac{P - R - I_a}{t_s}$$

where, W = defined average rate of infiltration (cm)

P = total storm precipitation (cm)

R = total storm runoff (cm)

I_a = Initial losses (cm)

t_s = time during which rainfall intensity exceeds infiltration rate (duration of rainfall excess)

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⊛ Measurement of infiltration

→ There are two methods for the measurement of infiltration. They are

- → Analytical Method
- → Experimental Method

a) Analytical Method:

Horton developed a equation for calculation of infiltration. The Horton's eqn states that "the infiltration begin at some rate f_o and exponentially decreases until it reaches a constant rate f_c. Horton's equation is expressed as

$$f(t) = f_c + (f_o - f_c)e^{-kt} \longrightarrow (I)$$

for $0 \le t \ge t_d$

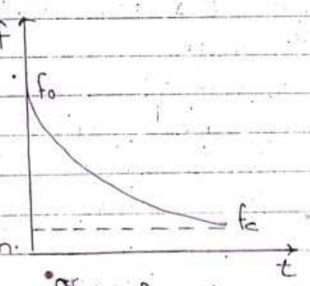


fig. Infiltration curve

where,

k = decay constant depending upon soil characteristics & vegetation.

f_o = Initial Infiltration capacity at t=0

f_c = final infiltration rate.

f (t) = Instantaneous infiltration capacity at any time 't' from the start of rainfall.

The eqn (I) also represent the eqn of st. line

$$f = f_c + (f_o - f_c)e^{-kt}$$

$$f - f_c = (f_o - f_c)e^{-kt}$$

Taking log on both sides,

$$\log(f - f_c) = \log(f_o - f_c) + \log(e^{-kt})$$

$$\text{or, } \log(f - f_c) = \log(f_o - f_c) - kt \longrightarrow (II)$$

let $\log(f - f_c) = y$ and

$\log(f_o - f_c) = c$

Then above eqn (II) becomes

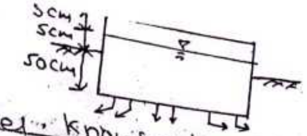
$$y = c - kt$$

or, $y = -kt + c$ which is in the form of $y = mx + c$ and proves Horton's eqn represent the equation of straight line.

b) Experimental Method:

Experiment are performed on small area to collect the information about the infiltration characteristics of soil. The experimental set up is called infiltrometer.

Ring infiltrometer is a metal ring that is driven into the soil. Water is poured to a certain depth and a pointer is set



to mark the water level. Knowing the vol. of water added at different time interval, the graph is plotted between infiltration capacity Vs time. The experiment is continued till a uniform rate of infiltration (f) is obtained.

(2012) (2008)

① The average rainfall over a basin of area 50ha during a storm was as follows:

Time (hr)	0	1	2	3	4	5	6	7
Rainfall (mm)	0	6	11	34	28	12	6	0

If the vol. of runoff from this storm was measured as 25×10^3 m³, determine the ϕ-index for the storm.

→ Here,

Area of basin = 50ha $= 50\times10^4$ m²

Vol. of Run off $= 25\times10^3$ m³

Depth of run off $= \frac{\text{Vol. of Runoff}}{\text{Area of basin}}$

$= \frac{25\times10^3}{50\times10^4}$

$= 0.05$ m

$= 50$ mm

Total rainfall $= \Sigma P_i = 97$ mm

Infiltration = Tot. Rainfall − Runoff

$= (97-50)$

$= 47$ mm.

First trial:

Take effective duration of rainfall (t_e) = 6 hrs.

ϕ-index $= \frac{47}{6} = 7.83$ mm/hr.

Then,

Rainfall excess $= (0-7.83) + (6-7.83) + (11-7.83) + (34-7.83) + (28-7.83) + (12-7.83) + (6-7.83) + (0-7.83)$

$= 0 + 0 + 3.17 + 26.17 + 20.17 + 4.17 + 0 + 0$

$= 53.68$ mm

which is greater than net runoff of 50mm, so ϕ-index is increase

Second trial

Take ϕ-index = 8.75 mm/hr.

Rainfall excess $= 0 + 0 + (11-8.75) + (34-8.75) + (28-8.75) + (12-8.75) + 0 + 0$

$= 50$ mm which is equal to net runoff of 50mm. Hence, ϕ-index = 8.75 mm/hr.

Ravi.

(2007) (2009)

② A storm with 10cm precipitation produced a direct runoff of 5.8cm. Given the time dist^n of storm as below, estimate φ-index.

Time from start (hr)	1	2	3	4	5	6	7	8
Incremental rainfall in each hr (cm)	0.4	0.9	1.5	2.3	1.8	1.6	1.0	0.5

Here,

→ Direct runoff = 5.8cm.

Total rainfall = 10cm

Infiltration = total rainfall − Direct runoff

= 10 − 5.8

= 4.2 cm

First trial

Assume effective duration of rainfall (t_e) = 8 hr.

ϕ-index = $\frac{4.2}{8}$ = ~~0.7cm/hr~~. 0.525 cm/hr

Rainfall excess = 0 + (0.9 − ~~0.7~~ 0.525) + (1.5 − ~~0.7~~ 0.525) + (2.3 − ~~0.7~~ 0.525) + (1.8 − ~~0.7~~ 0.525) + (1.6 − ~~0.7~~ 0.525) + (1 − ~~0.7~~ 0.525) + 0

= ~~4.9cm~~ 5.95. Which is ~~less~~ greater than direct runoff of 5.8 cm. so, ~~decrease~~ increase φ-index.

Sri.

2nd trial

Take φ-index = ~~0.55~~ 0.55 cm/hr.

Rainfall excess = 0 + (0.9 − 0.55) + (1.5 − 0.55) + (2.3 − 0.5 + (1.8 − 0.55) + (1.6 − 0.55) + (1 − 0.

= 5.8 cm Which is equal to dire runoff.

Hence, φ-index = 0.55 cm/hr.

(2009) ③ A 6hr storm with intensities of 5.4, 15.6, 24.3, 2 and 4 mm/hr in successive one hour interval produced the runoff of 3.7 million m³ over a basin of area 570 km². Determine φ-index basin.

→ Here, Total rainfall = 94.3 mm

Area of basin = 570 km²

Run off = 3.7 million m³ = 3.7×10^6 m³

Depth of runoff = $\frac{3.7 \times 10^6}{570 \times 10^6}$ = 0.0064 m = 6.4 mm

Infiltration = 94.3 − 6.4 = 87.9 mm.

First trial

Assume, effective duration of rainfall (t_e) = 6h

φ-index = $\frac{87.9}{6}$ = 14.65 mm/hr.

Rainfall excess = 0 + (15.6 − 14.65) + (24.3 − 14.65) + (28 − 14.65) + (17 − 14.65) + 0

= 26.3 mm.

which is greater than Depth of runoff. so increase ϕ-index.

2nd trial

Take ϕ-index = 22.95 mm/hr

Rainfall excess = 0 + 0 + (24.3 − 22.95) + (28 − 22.95) + 0 + 0

= 6.4 mm which is equal to Depth of runoff.

Hence, ϕ-index = 22.95 mm/hr.

④ precipitation falls on a 500 km² drainage basin according to the following schedule.

Time (30 min. period)	1	2	3	4
Rainfall intensity (cm/hr)	8	4	12	10

Determine the total storm rainfall in cm.

Determine also the ϕ-index for basin if the net storm runoff is 6 cm.

→ Here,

storm runoff = 6 cm.

Total rainfall = 8×0.5 + 4×0.5 + 12×0.5 + 10×0.5

= 17 cm.

Infiltration = 17 − 6 = 11 cm.

first trial

Take effective duration of rainfall (t_e) = 2 hr

ϕ-index = $\frac{11}{2}$ = 5.5 cm/hr.

Rainfall excess = (8 − 5.5)×0.5 + 0 + (12 − 5.5)×0.5 + (10 − 5.5)×0.5

= 6.75 cm which is greater than net runoff of 6 cm. Hence, increase ϕ-index.

2nd trial.

Take ϕ-index = 6 cm/hr.

Rainfall excess = (8 − 6)×0.5 + 0 + (12 − 6)×0.5 + (10 − 6)×0.5

= 6 cm. which is equal to net runoff of 6 cm.

Hence, ϕ-index = 6 cm/hr.

⑤ For a storm of 2hr duration, the rainfall rate are as follows.

Time period (min)	20	20	20	20	20	20
Rainfall rate (cm/hr)	2.5	2.5	10	7.5	5.1	1.25

If the ϕ-index is 3 cm/hr, estimate the surface runoff and W-index.

→ Here,

The surface runoff occurs when the intensity of rainfall exceeds the ϕ-index. so, runoff starts after 40 min and continues up to next 1 hour

∴ Surface runoff = $\left[(10-3) + (7.5-3) + (5.1-3)\right] \times \frac{20}{60}$

or, R ≑ 4.53 cm.

Total rainfall = $[2.5 + 2.5 + 10 + 7.5 + 5.1 + 1.25] \times \frac{20}{60}$

$\therefore P = 9.62$ cm.

Neglecting, depression and ~~inter~~ inception the W-index can be calculated from

$$W = \frac{P - R}{t_e}$$

$$or,\ W = \frac{9.62 - 4.53}{2}$$

$$or,\ W = 2.55\ cm/hr.$$

(2007) (6) An isolated 3hr storm occurred over a basin in the following fashion. Compute hourly distribution of avg. rainfall & total rainfall on basin

% of catchment	ϕ-index (cm/hr)	Rainfall 1st hr	2nd hr	3rd hr
10	1.0	0.8	2.3	1.5
30	0.75	0.7	2.1	1.0
30	0.50	1.0	2.5	0.8

Estimate the runoff from the catchment due to this storm.

→ Here,

Total % of catchment = 70%.

Average rainfall at time t = $\sum \frac{A_i}{A} \times (\text{Rainfall})$

Then,

Avg. rainfall in 1st hr = $\left[\frac{10}{70} \times 0.8 + \frac{30}{70} \times 0.7 + \frac{30}{70} \times 1\right]$

= 0.842 cm.

Avg. rainfall in 2nd hr = $\left[\frac{10}{70} \times 2.3 + \frac{30}{70} \times 2.1 + \frac{30}{70} \times \right.$

= 2.3 cm.

Average rainfall in 3rd hr = $\left[\frac{10}{70} \times 1.5 + \frac{30}{70} \times 1.0 + \frac{30}{70} \times \right.$

= 0.985 cm

Hence, hourly distribution of rainfall in 3hrs

= 0.842 cm, 2.3 cm, 0.985 cm.

Total rainfall = (0.842 + 2.3 + 0.985) cm = 4.127

Then,

total runoff = $\frac{10}{70}[0 + (2.3-1) + (1.5-1)] + \frac{30}{70}[0 + (2.1 - 0.75) + (1 - 0.75)] + \frac{30}{70}[(1-0.5) + (2.5-0.5) + (0.8-0.5)]$

or, total runoff = 2.142 cm.

(7) A storm with a uniform intensity of 1.6 cm/hr for period of 8 hrs occuring over a basin of area 650 produced a runoff estimated to be 57.2 million m. Find average infiltration rate during a storm

→ Here, Total rainfall = 1.6 × 8 = 12.8 cm.

Runoff depth = $\frac{\text{Runoff vol.}}{\text{Basin area}} = \frac{57.2 \times 10^6}{650 \times 10^6} = 8.8$

Infiltration = Rainfall − Runoff

= 12.8 − 8.8

= 4 cm. and Infiltration rate = 4/8 = 0.5 cm

(8) An isolated storm in a catchment produce a runoff of 3.5cm. The mass curve of the average rainfall depth over the catchment was as below.

Time from begining of storm (hr)	0	1	2	3	4	5	6
Accumulated avg. rainfall (cm)	0	0.5	1.65	3.55	5.65	6.8	7.75

⇒ Given,

Total rainfall = 7.75 cm (P)

Run off (Q) = 3.5 cm

Then,

Infiltration = P − Q
= 7.75 − 3.5
= 4.25 cm.

1st Trial

Effective duration of rainfall (te) = 6 hrs

ϕ-index = $\frac{4.25}{6}$ = 0.7 cm/hr

Time from begining of storm (hr)	0	1	2	3	4	5	6
Accumulated avg. rainfall (cm)	0	0.5	1.65	3.55	5.65	6.8	7.75
Incremental rainfall (cm)	–	0.5	1.15	1.90	2.1	1.15	0.95

Rainfall excess = (0.5 − 0.7) + (1.15 − 0.7) + (1.90 − 0.7) + (2.1 − 0.7) + (1.15 − 0.7) + (0.95 − 0.7)

= (0 + 0.45 + 1.2 + 1.4 + 0.45 + 0.25)

ie. Re = 3.75 cm > Q.

Since, Rainfall excess is not equal to runoff (Q). So assume next trial. Increase the value of ϕ, since Re > Q.

2nd Trial

Take ϕ = 0.75 cm/hr.

Rainfall excess (Re) = (0.50 − 0.75) + (1.15 − 0.75) + (1.9 − 0.75) + (2.1 − 0.75) + (1.15 − 0.75) + (0.95 − 0.75)

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Page

or, $R_e = 3.5cm.$

Here, $R_e = Q$ ie. rainfall excess is equal to runoff so,

ϕ-index $= 0.75 cm/hr.$

Chapter → 4

Surface Runoff and Flow Measurement

→ Surface runoff is one of the important component of hydrologic cycle. It is defined as the portion of precipitation which reaches the stream channel by a variety of paths above and below the surface of earth. Runoff includes all the water flowing in the stream channel at any given section.

⊛ Rainfall runoff correlation

→ There is very complex relationship between rainfall and resulting runoff.

The equation of the straight line regression between runoff (R) and rainfall (p) is given by

$$R = ap + b \longrightarrow (1)$$

where, a and b are coefficients & given by

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

and

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

N = no. of sites of observations.

The coefficient of correlation 'r' is calculated as

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

Where, r lies between 0 and 1. The value of r between 0.6 to 1 indicates good correlation.

For large catchments,

$$R = \alpha P^{\beta}$$

$$\ln R = \beta \ln P + \ln \alpha$$

The coeff. α and β are calculated using the regression analysis.

⊛ Stream gauging:

→ The process of measuring discharge of a stream is called stream guaging. A guaging station is established on a stream for the discharge measurement. The runoff from the catchment is determined after stream guaging.

The basic principle followed in all the discharge measurement method is the continuity equation.

$$Q = A \times V$$

Where, Q = Discharge in m^3/sec.

A = Area of cross section (m^2)

V = velocity of flow (m/s)

Ravi

The main purpose of stream guaging is to provide systematic records of stage (elevation) and discharge of stream.

The following factors should be considered for the selection of site for stream guaging.

- The stream should have well defined & regular cross-section.
- It should be located where great fluctuation in the stage occurs.
- It should be easily accessible.
- It should be at upstream of the desired site.
- The site should be free from any disturbance.
- Channel bed should be stable and regular.

(IMP) ⊛ Stream flow measurement

→ Stream flow may be measured using one or a combination of the following methods.

1) Direct determination of stream discharge
   - → Area velocity Method (Hydrometric Technique)
   - → salt dilution techniques
   - → Electro magnetic method
   - → ultrasonic method

2) Indirect determination of stream flow
   - → slope area Method
   - → Hydraulic structure such as weirs

(1) Area velocity Method:

→ In this method of discharge measurement the area of cross-section of river is measured at selected section and measuring the velocity of flow the cross-section area. At the selected site, the section line is marked off by permanent survey marking and cross-section is determined. Towards this, the depth at various locations are measured by sounding rods. When the stream depth is large and for accurate depth measurement echo-depth recorder is used.

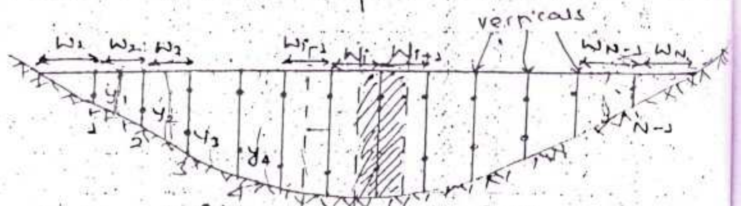


Figure shows fig. stream section for Area-velocity cross section of a river in which N-1 verticals are drawn. The velocity averaged over the vertical at each station is known. Considering total area to be divided into N-1 segments. Total discharge is calculated by

$$Q = \sum_{i=1}^{N-1} \Delta Q_i$$

where,

ΔQ_i = discharge in i^{th} segment
= (depth at i^{th} segment) × [$\frac{1}{2}$ width to the left + $\frac{1}{2}$ width to right) × avg. velocity at i^{th} vertical

or, $$\Delta Q_i = y_i \left[\frac{W_i}{2} + \frac{W_{i+1}}{2}\right] \times v_i$$

for $i = 2$ to $N-2$

For first and last section, segments are taken to have triangular area and area is calculated as

$$\Delta A_1 = \bar{W}_1 \cdot y_1$$

where,

$$\bar{W}_1 = \frac{(W_1 + W_2/2)^2}{2W_1}$$

$$\Delta A_N = \bar{W}_{N-1} \cdot y_{N-1}$$

where,

$$\bar{W}_{N-1} = \frac{(W_N + W_{N-1}/2)^2}{2W_N}$$

Then, $$\Delta Q_1 = \bar{v}_1 \cdot \Delta A_1$$

$$\Delta Q_{N-1} = \bar{v}_{N-1} \cdot \Delta A_{N-1}$$

b) Salt dilution techniques:

→ The salt dilution method is a simple and pratical technique for measuring the discharge of mountainous stream where turbulence is high. It can be also used as an accasional method for checking the calibration, stage-discharge curve etc. obtained by any other method. This technique is based on the principle that a given amount of salt is more diluted by a large amount of water than by a small amount. After injecting a known amount of salt into a stream, the concn of dissolved salt is measured at a downstream point where it has fully mixed with the stream water.

The discharge can be expressed as

$$Q = \frac{Q_t (C_1 - C_2)}{(C_2 - C_0)}$$

where,

C_0 = Initial concn salt.
C_1 = Amount of salt in dosing soln.
C_2 = concn of salt in the sample at downstream
Q_t = quantity of solution ejected.

Ravi.

c) Electromagnetic Method:

→ This method is based on Faraday's principle. In this method, a long conductor is burried at the bottom of the channel covering the entire width of the river which carries a current I, to produce a controlled vertical magnetic field. As the water flowing in the stream cuts its magnetic field, it produces electromagnetic ~~force~~ force which is related to discharge in the river, depth of flow and current in the conductor at the bottom of the river.

The discharge is given by

$$Q = K_1 \left[\frac{Ed}{I} + K_2 \right]^n$$

Where,

d = depth of flow
I = current in the coil
K_1, K_2, n = system constant
E = emf produced.

d) Ultrasonic Method:

→ In this method the velocity is measured by using ultrasonic signals. Two transducers are provided in the either side of the channel at same level 'h' above the bed which can receive and send ultrasonic signal.

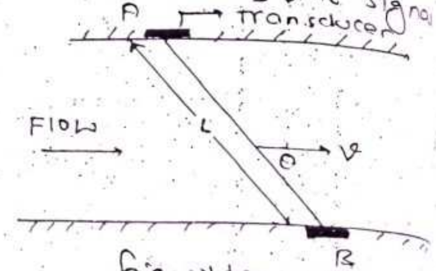


fig: ultrasonic Method

If t_1 is the time to travel signal from A to B and t_2 be the time to travel signal from B to A, then

$$\text{Velocity } (v) = \frac{L}{2\cos\theta} \left[\frac{1}{t_1} + \frac{1}{t_2} \right]$$

(Imp) e) Slope - area Method: (Hydraulic Method)

→ slope - area method is used to measure the flow indirectly when direct flow measurement not possible. eg: during floods because of excessive rate of change of discharge, excessive velocities etc. Hence, this method is suitable for flood estimation.

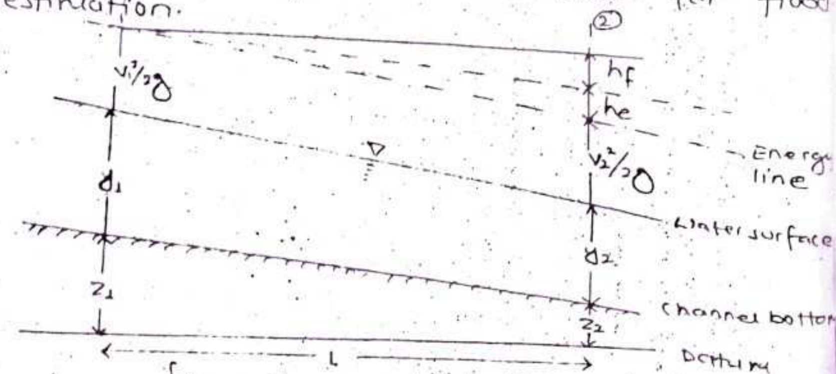


fig: slope - area method

Figure shows a longitudinal section of the flow of river between two sections ① and ②.

Applying Bernoulli's eqn for section ① & ②.

$$z_1 + y_1 + \frac{V_1^2}{2g} = z_2 + y_2 + \frac{V_2^2}{2g} + h_L$$

where, z_1, z_2 = datum head at section ① & ②

y_1, y_2 = water depth at section ① & ②

V_1, V_2 = velocities at section ① & ②

h_L = head loss $= h_f + h_e$

h_f = friction loss

h_e = eddy loss.

Denoting $z + y = h$ = water surface elevation above datum

Then,

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

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

From Manning's eqn

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

Where,

Q = discharge

n = roughness coeff

A = cross-sectional area

R = Hydraulic radius $= A/P$, P = wetted perimeter

S_f = slope of energy line between two points.

Also, Manning's equation is expressed as

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

Where,

K = conveyance of channel, $K = \frac{1}{n} A R^{2/3}$

$$S_f = h_f/L$$

For two section average conveyance is

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

Where, $K_1 = \frac{1}{n_1} A_1 R_1^{2/3}$ and $K_2 = \frac{1}{n_2} A_2 R_2^{2/3}$

Eddy loss is given by

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

K_e = eddy loss coeff.

Now, the discharge is calculated by a trial and error procedure, using the following sequence of calculation.

- Assume $V_1 = V_2$. This leads to $h_f = h_1 - h_2$ = fall of water surface betwn two sections.
- Calculate Q using eqn $Q = K\sqrt{S_f} = K\sqrt{h_f/L}$

- compute $V_1 = \frac{Q}{A_1}$ and $V_2 = Q/A_2$
- Now calculate refined value of h_f.

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

- Take refined value of h_f for next iteration and repeat the above process until the difference between two successive value of h_f is negligible.
- compute Q using final value of h_f.

f) Hydraulic structure.

→ The different flow measuring structures such as V-notch weir, rectangular weir, broad crested weir can be used to measure the flow.

- V-notch weir

The discharge for a V-notch is given by

$$(Q) = C \, \frac{8}{15} \sqrt{2g} \, \tan(\theta/2) \, h^{5/2}$$

- Rectangular weir

$$Q = C \times 2.953 \, b \, h^{3/2}$$

- Broad-crested weir

$$Q = C_d L h \sqrt{2g(H-h)}$$

where, Q = discharge in m^3/sec.

C = discharge coeff.

g = acc^n due to gravity

θ = angle of the notch

h = head over weir measured from the vertex of notch.

b = width of weir in m.

(Imp) ⑨ Rating curve:

→ If the measured discharge is plotted against the corresponding stage a curve is obtained which is known as rating curve. This curve represents the integrated effect of a wide range of channel and flow parameters.

The rating curve has lots of application in river engineering. After plotting the curve it is very easy to find out the discharge in the stream simply by reading the river guage & reading out the corresponding discharge.

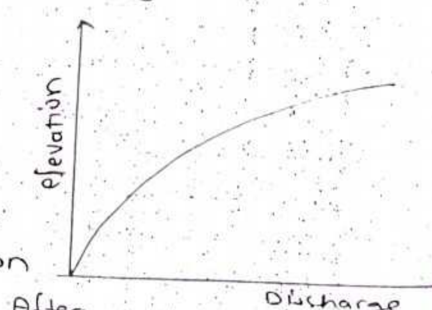


fig. Rating curve

such curve gives a relation between the stage of river at a given time and a corresponding discharge so, it is also Known as stage-discharge curve.

limitation

- Vegetation growth, weed growth near the guage site affect the rating curve.
- Variable back water effect indicate the different discharge at the same stage hence no unique rating curve is obtained.
- Aggradation and degradation of alluvial channel affect the stage of the stream.
- Unsteady flow effect the stage due to which for the same stage there will be low discharge during rising and viceversa.
- During high stage of flow, river become inaccessible and therefore extension of rating curve is necessary which induces some error in the extension part and true rating curve is not obtained.

⊛ Stage - Discharge Relationship

→ When a number of discharge observation along with the stage observation are taken at a guaging site, these observations are plotted on a simple graph, with discharge on the x-axis and stage on the y-axis. once a relation of stage and discharge is established it becomes easy to calculate the discharge flowing in a channel. such graph is known as stage discharge relation or also a rating curve. which is normally parabolic in nature.

The relation between stage and discharge is given by

$$Q = C_r (G-a)^{\beta} \longrightarrow (1)$$

where, Q = stream discharge

β, C_r = rating curve constant.

G = gauge ht.

a = gauge reading correspoing to zero discharge.

The value of C_r and β are obtained by the least - square method.

taking log on both side of eqn (1)

$$\log Q = \beta \log (G-a) + \log C_r$$

or, $Y = \beta X + b$.

where, $Y = \log Q$

$X = \log(G-a)$ and $b = \log C_r$

Then value of β and b are obtained from regression

$$\beta = \frac{N(\Sigma XY) - \Sigma X \cdot \Sigma Y}{N(\Sigma X^2) - (\Sigma X)^2}$$

and

$$b = \frac{\Sigma Y - \beta(\Sigma X)}{N}$$

Thus, stage discharge relation can be used for estimating discharge 'Q' of the stream for a given gauge reading G.

(IMP) *

Current Meters

→ Current Meter is the most commonly used instrument to measure the velocity at a point in a stream. It consists of a rotating element. Which rotates due to the reaction of stream current with an angular velocity proportional to the stream velocity. It is weighted down by lead weight called sounding weight to keep in stable position in flowing water.

There are two types of current Meter. They are

- → vertical axis meter (cup type)
- → Horizontal axis meter (propeller type)

a) vertical axis Meter

→ These instrument consists of a series of conical cups mounted around the vertical axis. The cup rotates in a horizontal plane and a cam attached to the vertical axial spindle records generated signals proportional to the revolution of the cub assembly. Then, the velocity is calculated by using the eqⁿ

$$V = aN + b$$

Where,

a, b = Consts of instrument

and a = 0.65

b = 0.03

N = No. of revolution per sec

V = velocity in m/s.

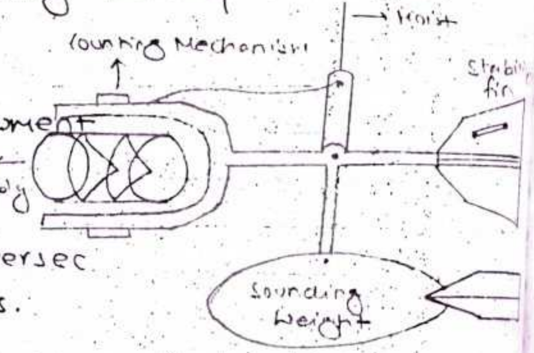


fig. vertical axis Meter

b) Horizontal axis Meter

→ These instrument consists of a propeller mounted at the end of horizontal shaft. The revolution of propeller for a certain time is recorded which rotates about a horizontal axis.

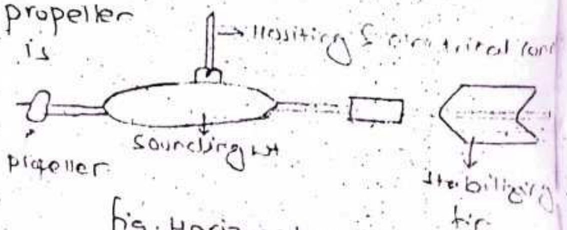


fig. Horizontal axis Current Meter

Then the velocity of stream is calculated by using eqn $V = aN + b$.

(IMP)

* Factors affecting surface Runoff

→ The various factor affecting the runoff can be divided into two groups. They are

- → characteristics of precipitation
- → characteristics of drainage basin.

① characteristics of precipitation

→ precipitation plays an important role in surface runoff. The characteristics of precipitation which affect surface runoff are discussed below.

a) Type of precipitation: precipitation generally occurs either in the form of rain or snow. Rain immediately produce runoff while snow produce runoff at a slow and steady rate.

b) Intensity of precipitation: If the intensity of precipitation goes on increasing, the runoff increases rapidly.

c) Duration of Rainfall: Infiltration capacity goes on reducing with longer duration of rainfall. If infiltration is less, surface runoff will be more. sometimes, if the rain is extended over a large period water table may rise quite high, it may sometime reach the ground in which case there will be no infiltration and flood may become more serious.

d) Rainfall Distrubtion: The rain fall either on the whole basin or on a small part of it. If rainfall occur at the U/s of the outlet, the runoff time to reach the outlet and peak runoff at outlet occurs after considerable time. on the other hand if rainfall occurs close to the outlet, runoff is quicker and higher.

e) Direction of storm movement: If the direction of storm is in the same direction of the movement of water runoff is quicker and higher in magnitude then storm moving in the opposite direction to the water movement.

<Ridhe>

f) Other climatic condition: Various climatic factors temperature, wind, humidity affect the loss of drainage basin and therefore affect the runoff. If losses are more, runoff will be less and vice versa.

② Characteristics of drainage basin

→ The different characteristics of drainage basin which effect the surface runoff are as follows.

a) Shape of basin: The time taken for the water to reach to outlet from the remote part depends upon the shape of basin. Fan shaped catchment give greater runoff because tributaries are nearly of same size and therefore time of flow is nearly the same and is smaller.

b) Size of the basin: The size of catchment has a definite effect on the runoff. If the basin area is large, more will be the runoff than in case of small basin.

Ravi

c) Slope of the catchment: Slope of catchment effect the surface runoff. The slope of catchment control velocity of flow.

d) Elevation: The elevation of basin affect the rainfall, it amount and its type and hence produce enough effect on the runoff.

e) Basin geology: The runoff from the basin also depend upon the lithological factors like composition, texture, sequence of rocks and structural factors like fault and folds for the basin.

f) Vegetative cover: If the catchment area is covered with grasses, weeds the runoff flow is reduced where as in case of barren land there will be high runoff.

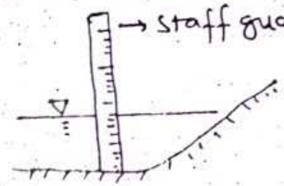
g) Orientation of basin: Orientation plays important role in relation to the direction of storm movement. The windward side of the mountains alway get more rain and so, runoff is high than leeward side.

रविशंकर

⊛ Stage Measurement

→ The stage of a river is the height of water surface above any arbitary datum which is expressed in meter (m).
The different types of stream guage used for stage measurement are
- → staff guage
- → wire guage
- → float guage recorder
- → Bubble guage.

a) **Staff guage:** This is the simplest method for stage measurement. The stage measurement can be made by noting the elevation of the water surface in contact with a fixed graduated staff.



b) **Wire guage:** It is a gauge used to measure the water-surface elevation. In this a weight is lowered by a reel to touch the water surface. A mechanical counter measures the rotation of the wheel which is proportional to the length of wire paid out.

c) **Float guage recorder:** It is common type of automatic stage recorder. A float is connected to one end of a wire which passes through a recorder and the other end of a rope is balanced by a suitable counter weight. Displacement of float due to the rising and lowering of the water surface elevation cause an angular displacement of the pully and hence of the input shaft of the recorder. Mechanical linkage convert this angular displacement to the linear displacement of a pen to record over a drum driven by clock work.

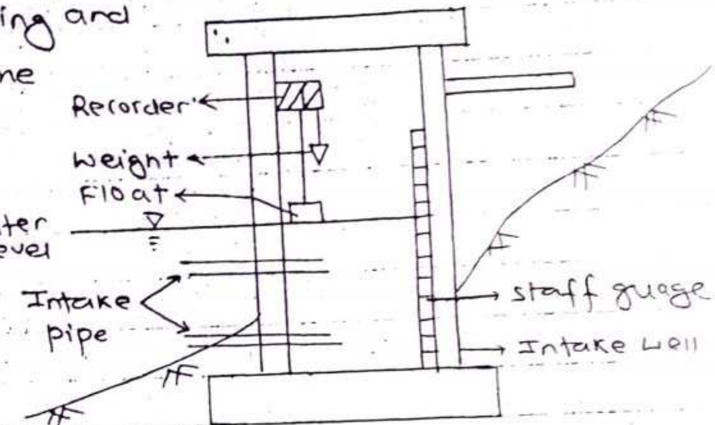


fig. Float gauge recorder

d) **Bubble guage:** In this guage compressed air or gas is made to bleed out at a very small rate through an outlet placed at the bottom of river. The pressure req. to continuously push the gas stream out beneath the water surface is a measure of depth of water over the nozzle of the bubble stream.

(2006) (2011) (2012)

① Taking the rating of current meter as $V = 0.05 + 0.8N$, where V is velocity in m/s. compute the stream flow corresponding to the following observation. All the velocities of the verticals are measured at 0.6 times the depth of flow from the free surface.

Distance from bank	0.6	1.2	2.0	3.0	3.8	4.5	5.0	5.6
Depth (m)	1	4	5.5	6.5	4.5	2.5	1.0	0
Revolution	15	39	50	56	39	35	20	0
Time	50	55	50	50	50	50	50	0

⇒ Here,

The eqn of velocity is given by

$$V = 0.05 + 0.8N \longrightarrow ①$$

For first and last sections,

overage width $\bar{W}_1 = \dfrac{\left(W_1 + \frac{W_2}{2}\right)^2}{2W_1}$

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

and for other section $(W_i) = \left(\dfrac{W_i + W_{i+1}}{2}\right)$

$\dfrac{\left(0.6 + \frac{0.6}{2}\right)^2}{2\omega}$

Distance from left Bank	Width of section (m)	Avg. Width (m)	Depth (m)	Cross-sectional Area (m²)	Revolution per sec (N_s)	Avg. velocity (m/s)	Disch. (m³/s)
0	—	—	—	—	—	—	—
0.6	0.6	0.675	1	0.675	0.3	0.29	0.19
1.2	0.6	0.70	4	2.8	0.709	0.617	1.72
2.0	0.8	0.9	5.5	4.95	1.0	0.85	4.20
3.0	1.0	0.9	6.5	5.85	1.12	0.946	5.53
3.8	0.8	0.75	4.5	3.375	0.78	0.674	2.27
4.5	0.7	0.6	2.5	1.5	0.70	0.61	0.91
5.0	0.5	0.602	1.0	0.602	0.4	0.37	0.22
5.6	0.6	—	0	—	0	sum.	15.07

Hence, discharge (Q) = 15.079 m³/sec.

(2007) (2008)

② During a high water surface elevations of a small stream were noted at two sections A and B, 10 km apart. The elevations and other salient hydraulic properties are given below.

Section	Elevation (cm)	Area (m²)	Hydraulic Radius (m)
A	104.771	73.293	2.733
B	104.50	93.375	3.089

The eddy loss coeff of 0.3 for gradual expansion, 0.1 for gradual contraction and $n = 0.02$. Estimate stream discharge.

→ Here,
Manning's coff (n) = 0.02

For section-A
cross-sectional area $(A_1) = 73.293 m^2$
Hydraulic radius $(R_1) = 2.733 m$.
conveyance $(K_1) = \frac{1}{n} A_1 R_1^{2/3}$

or, $K_1 = \frac{1}{0.02} \times 73.293 \times 2.733^{2/3}$

or, $K_1 = 7163.505\ m^3/sec$

For section-B
cross-sectional area $(A_2) = 93.375 m^2$
Hydraulic radius $(R_2) = 3.089 m$
conveyance $(K_2) = \frac{1}{n} A_2 R_2^{2/3}$

or, $K_2 = \frac{1}{0.02} \times 93.375 \times 3.089^{2/3}$

or, $K_2 = 9902.524\ m^3/sec$

Then,
average conveyance $(K) = \sqrt{K_1 \cdot K_2}$

or, $K = \sqrt{7163.505 \times 9902.524}$

or, $K = 8422.40\ m^3/s$

Head loss (h_f) is given by

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

Fall in water level $(h_1 - h_2) = 104.771 - 104.5$
$= 0.271 m$

length $(L) = 10 km = 10000 m$.

As $A_2 > A_1$, eddy loss coeff. of expansion is taken. i.e. $K_{ei} = 0.3$

Now,
Refined value of h_f is computed by

$$h_f = 0.271 + \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) - K_e\left[\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right]$$

Taking $h_f = 0.271$ for first trial.

Trial	h_f (trial)	$S_f = \frac{h_f}{L}$	$Q = K\sqrt{S_f}$	$V_1 = \frac{Q}{A_1}$	$V_2 = \frac{Q}{A_2}$	h_f (refined)
1	0.271	2.71×10^{-5}	43.65	0.596	0.47	0.2759
2	0.2759	2.76×10^{-5}	44.24	0.604	0.474	0.2760

The difference in h_f (refined) in two trials is negligible. so, Peak discharge of the stream = $44.24\ m^3/sec$.

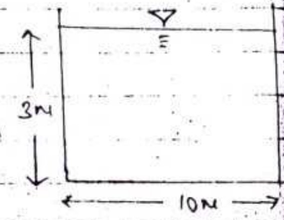
Javi.

(2007)

③ During the flood, the depth of water in the rectangular channel of 10m wide was found to decrease downstream from 3m to 2.94m over the reach of 90m. The bottom slope of channel was 0.01 and Manning's coeff = 0.0025. Estimate the flood discharge through the channel using slope area method.

→ Here,

At upstream



Area $(A_1) = 10 \times 3 = 30 m^2$

Perimeter $(P_1) = 10 + 2 \times 3 = 16m$

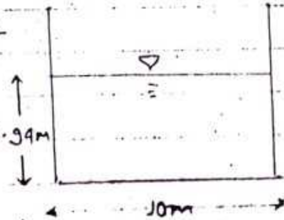
$R_1 = \frac{A_1}{P_1} = \frac{30}{16} = 1.875 m$.

Then,

$K_1 = \frac{1}{n} A_1 R_1^{2/3} = \frac{1}{0.0025} \times 30 \times (1.875)^{2/3}$

or, $K_1 = 18246.606 m^3/sec$.

At down stream



Area $(A_2) = 10 \times 2.94 = 29.4 m^2$

Perimeter $(P_2) = 10 + 2 \times 2.94$
$= 15.88m$

$R_2 = A_2/P_2 = \frac{29.4}{15.88} = 1.851$

$K_2 = \frac{1}{n} A_2 R_2^{2/3} = \frac{1}{0.0025} \times 29.4 \times (1.851)^{2/3}$

or, $K_2 = 17728.756 m^3/sec$.

Average conveyance $(K_{av}) = \sqrt{K_1 \cdot K_2}$

or, $K_{av} = \sqrt{18246.606 \times 17728.756}$

or, $K_{av} = 17985.817 m^3/sec$

Hence, Discharge $(Q) = K_{av} \cdot \sqrt{S}$

or, $Q = 17985.817 \times \sqrt{0.01}$

or, $Q = 1798.582 m^3/sec$.

(2008)

④ During a flow flow the depth of water in a 10m wide rectangular channel was found to be 3m 2.90m at two stations 250m apart. The drop of water elevation was found to be 0.15m. Assume Manning's coefficient to be 0.02 estimate flood discharge through the channel.

→ Here,

Manning coeff. $(n) = 0.02$

Distance betwn. two station $(L) = 250m$ apart.

Drop of water $(h_f) = 0.15m$.

At upstream

Area $(A_1) = 10 \times 3 = 30m^2$

Perimeter $(P_1) = 10 + 2 \times 3 = 16m$

$R_1 = A_1/P_1 = 30/16 = 1.875m$

Ravi

$K_1 = \frac{1}{n} A_1 \cdot R_1^{2/3}$

or, $K_1 = \frac{1}{0.02} \times 30 \times (1.875)^{2/3} = 2280.82\ m^3/sec.$

For downstream

Area $(A_2) = 10 \times 2.9 = 29\ m^2$

perimeter $(P_2) = 10 + 2 \times 2.9 = 15.8\ m$

$R_2 = A_2/P_2 = \frac{29}{15.8} = 1.835\ m$

$K_2 = \frac{1}{0.02} \times 29 \times (1.835)^{2/3} = 2173.32\ m^3/sec.$

Average conveyance $(K_{av}) = \sqrt{K_1 \cdot K_2}$

or, $K_{av} = \sqrt{2280.82 \times 2173.32}$

or, $K_{av} = 2226.42\ m^3/sec.$

slope $(S) = \frac{h_f}{L} = \frac{0.15}{250} = \frac{1}{1666.67}$

We know that,

$Q = K_{av} \sqrt{S}$

or, $Q = 2226.42 \times \sqrt{\frac{1}{1666.67}}$

or, $Q = 54.53\ m^3/sec.$

(2005)

⑤ An auxiliary guage was used downstream of a main guage in a river to provide correction to the guage discharge relationship due to backward effects. The following data were noted at a certain main guage reading.

Main guage (m above datum)	Auxiliary guage (m above datum)	Discharge (m^3/sec)
86.0	85.50	275
86.0	84.80	600

If the main guage reading is still 86.0 m and the auxiliary guage reads 85.30 m. Estimate the discharge in the river.

→ Here,

Fall $(F_1) = 86 - 85.5 = 0.5\ m.$, $Q_1 = 275\ m^3/s$

$F_2 = 86 - 84.8 = 1.2\ m.$ $Q_2 = 600\ m^3/s$

We know,

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

or, $\left(\frac{0.5}{1.2}\right)^m = \left(\frac{275}{600}\right)$

or, Taking log on both side

$m \log(0.417) = \log(0.458)$

or, $m = 0.893$

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When auxillary gauge reads 85.3 m

Fall (F) = 86 − 85.3 = 0.7 m

Then,

$$\left(\frac{F}{F_2}\right)^n = \left(\frac{Q}{Q_2}\right)$$

Or, $$\left(\frac{0.7}{1.2}\right)^{0.893} = \left(\frac{Q}{600}\right)$$

Or, $Q = 370.779\ m^3/sec.$

=) We know that,

Avg. width for 1st section $(\bar{W}_1) = \frac{(W_1 + W_2/2)^2}{2W_1}$

Avg. width for last section

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

Avg. width for other section

$$(\bar{W}_i) = \frac{W_i + W_{i+1}}{2}$$

Total discharge (Q) = ?

Distance from right bank (m)	Width of section (m)	Avg. width (m) $\bar{W}_i$	Depth (m) Y_i	Cross-sectional area (m^2) A_i	Avg. velocity V_i	Segment discharge Q_i (m^3/sec)
0	—	—	—	—	—	—
2	2	2.25	0.5	1.125	0.3	0.3375
4	2	2	0.9	1.8	0.45	0.81
6	2	2	1.1	2.2	0.5	1.1
8	2	2	0.8	1.6	0.32	0.512
10	2	2.25	0.6	1.35	0.27	0.3645
12	2	—	—	—	—	—

∴ total discharge (Q) = 3.129 m^3/sec.

Hydrograph Analysis

Hydrograph:

→ The graphical representation of the instantaneous rate of discharge (Q) of a stream plotted with respect to time (t) is known as hydrograph. The hydrograph is the response of a catchment to the rainfall input. The hydrograph may have a single peak or multiple depending upon the nature of characteristics of storm and catchment. Hydrograph is also known as flood hydrograph or storm hydrograph.

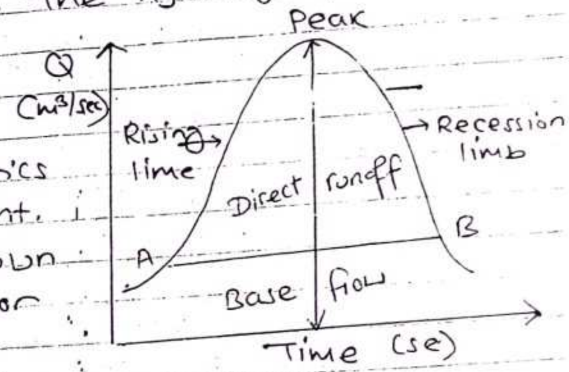

fig. Hydrograph

Why construct and analyze Hydrographs

- To find out discharge pattern of a particular drainage basin
- Help predict flooding events, therefore influence implementation of flood prevention measure.

Component of Hydrograph:

The component of a hydrograph are

→ The rising limb
→ The crest segment (peak)
→ Recession limb

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ⓐ The Rising Limb:

→ • It is the ascending portion of hydrograph
• It is influenced by storm and basin characteristics.
• The rising limb slowly rises in the early stage of flood but more rapidly toward the end portion because the losses are high at initial stage.

ⓑ Crest Segment (Peak)

→ • It is one of the important part of hydrograph as it contains peak flow.
• peak flow occurs when the runoff from various parts of the catchment contribute at the outlet simultaneously at the max^m rate.
• Peak may be sharp, flat depending upon the rainfall basin characteristic

ⓒ Recession Limb

→ • Recession limb represents the withdrawal of water from the storage.
• It entirely depends upon basin characteristics not on a storm.
• It extends from end of the crest to the begining of natural ground water flow.

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The equation of recession curve is given by

$$Q_t = Q_0 K_r^{\wedge t}$$

Where,

Q_t = discharge at a time interval of 't' days

Q_0 = initial discharge

K_r = recession constant.

Factors affecting Hydrograph

→ The various factors affecting the shape of flood hydrograph are discussed below.

- → Shape of basin
- → Size of basin
- → Slope of basin.
- → Drainage density
- → Land Use
- → Elevation of catchment
- → soil type & geological condition
- → Intensity of Rainfall
- → Duration of rainfall
- → Movement of storm.

① Shape of Basin: The shape of basin influences the time taken for water to arrive at the outlet. Thus the occurence of the peak and the shape

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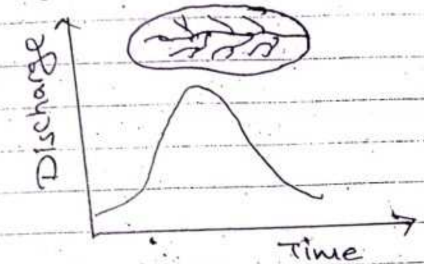
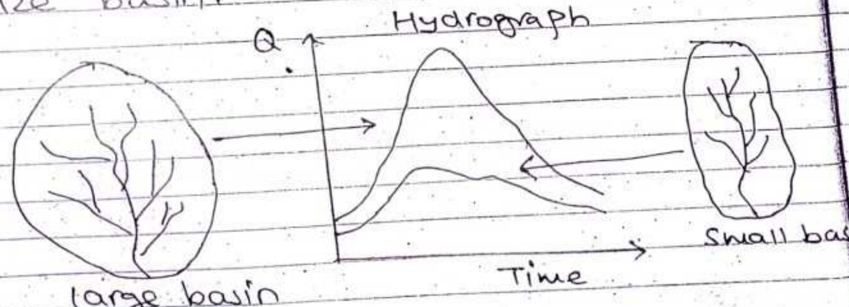
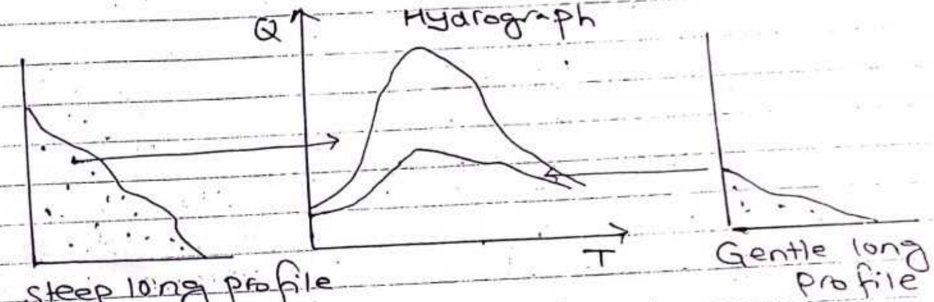
of hydrograph are affected by the basin shape.
For example:- For a fan shaped basin the hydrograph is shown below.



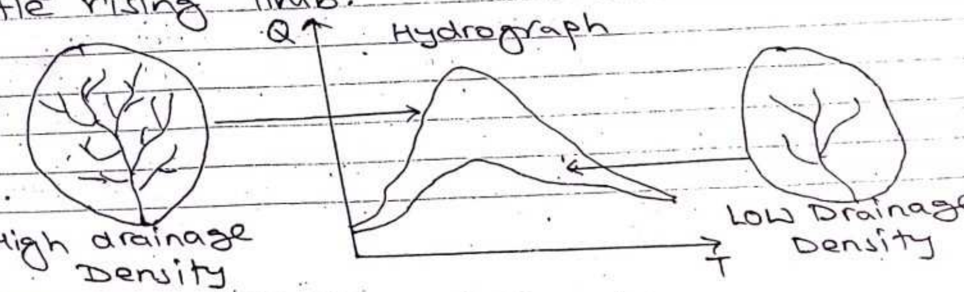
② **Size of Basin:** A larger river basin will collect more water than a smaller one, potentially leading to a higher peak flow as compared to the small size basin.



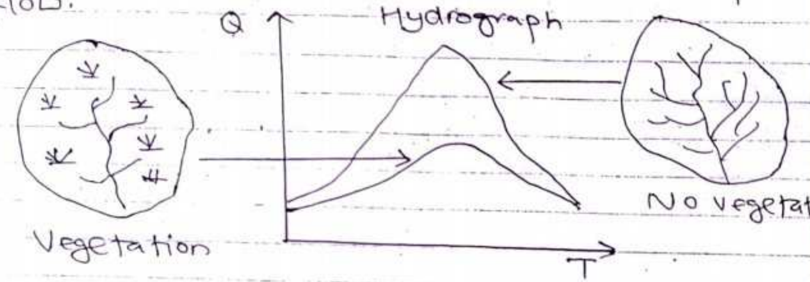
③ **Slope of Basin:** The steeper the slopes the faster the water flows overland into rivers, making the rising limb steeper. If the land is more gently sloping, water flows over it more slowly and is therefore more likely to infiltrate into the soil resulting rising limb will be less steeper.



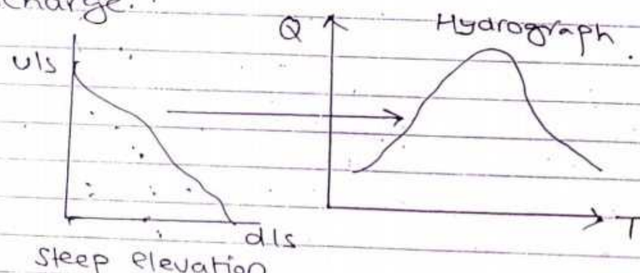
④ **Drainage Density:** A river basin with many tributaries (high drainage density) will offer many opportunities for overland flow, which flows more quickly and so the rising limb will be steeper. Conversely where there is a lower drainage density there will be more through flow, which is slower leading to a gentle rising limb.



⑤ Land use: Vegetation and forest increase the infiltration and storage capacities of the soils and thus reduce the peak flow.



⑥ Elevation of catchment: If the U/s elevation of catchment is higher than D/s, flow is quick and result in larger peak discharge.



⑦ Soil type and Geological condition:
→ The type of soil like sand absorbs quite a large part of water and so flood is less where as clay soil produce more flow.

Similarly geological condition also affects the runoff. If there are some cracks on rock a part of water flows quickly to underground and thus reduce the peak flow.

⑧ Intensity of Rainfall:
→ The peak and volume of rainfall is directly proportional to the intensity of rainfall. If rainfall intensity is high then peak and volume of rainfall is also high and viceversa.

⑨ Duration of Rainfall:
→ The duration of rainfall also has a direct proportional effect on the volume of runoff. The effect of duration is seen in the rising limb and peak flow.

⑩ Movement of Storm: If the storm moves from upstream of the catchment to the down-stream end, there will be a quicker concentration of flow at the basin outlet. The effect is seen in the rising limb & peak flow.

Some Termonologies:-

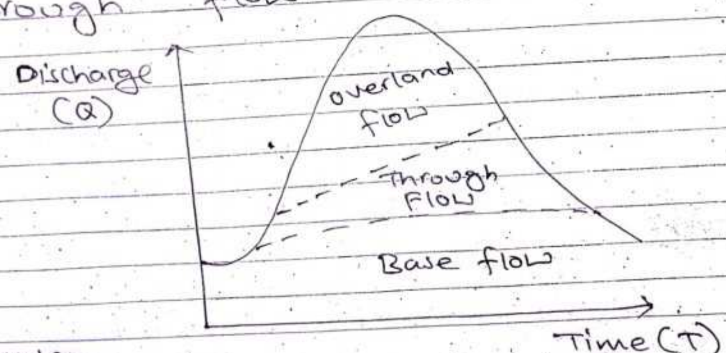
ⓐ Excess Rainfall: If the initial losses and infiltration losses are substracted from the total rainfall, the remaining portion of rainfall is called excess rainfall.

i.e. Excess Rainfall = [Total Rainfall − (Initial and Infiltration loss

(b) **Base flow:** The flow of water entering stream channels from groundwater sources is known as base flow. It is also known as groundwater recession, low flow or low-water flow.

(c) **Overland flow:** Volume of water reaching the river from surface runoff is known as overland flow.

(d) **Through flow:** Volume of water reaching the river through the soil and underlying rock layers is known as through flow.



Note:-
Storm flow = Overland flow + Through flow

(e) **Direct runoff:** It is the part of precipitation which appears quickly as flow in the river.

Direct runoff = surface runoff + subsurface runoff

(or)

Direct runoff = overland flow + Through flow

Base flow separation:

A hydrograph consists of mainly three parts

i.e.
- surface runoff (overland flow)
- Interflow (Through flow)
- Ground water flow (Base flow)

It is necessary to separate base flow from total storm hydrograph to obtain the surface flow hydrograph or direct runoff hydrograph (DRH).

There are basically two methods for separating base flow. They are
- → Straight Line Method
- → Variable Slope Method

(a) **Straight Line Method:**
→ Base flow separation is acheived by joining with a straight line, the begining of the surface runoff to a point on the recession limb representing the end of direct runoff.

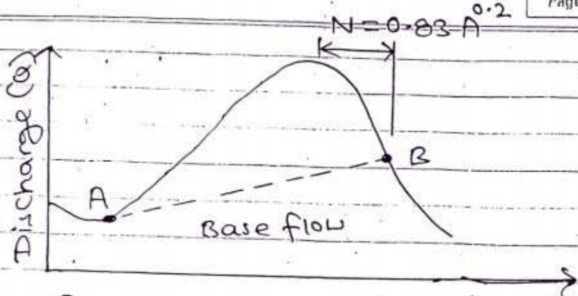


fig. Base flow separation

- The point 'A' represents the begining of direct runoff and is easy to find
- point B represents the end of direct runoff and is difficult to find.
- Thus, Point B is find by emperical equation as follows,

$$N = 0.83\,A^{0.2}$$

Where,

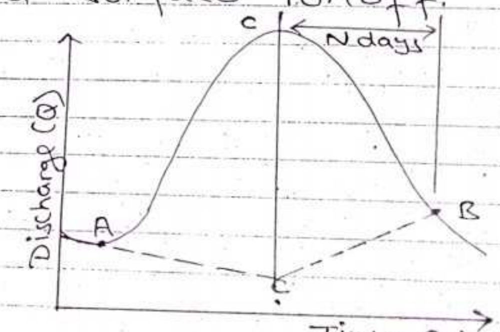
A = basin area in km^2

N = time interval from peak to the end of direct runoff (in days)

Finally point B is located in graph and point A and point B are joined which separates base flow from direct runoff.

(b) Variable Slope Method:

- Variable slope method is widely used for base flow separation. In this method the base flow curve is extended prior to the commencement of surface runoff till it intersects the ordinate draw at the peak point (ie. point C).
- The point is joined to point B by a straight line.
- Segment AC and BC separates the base flow and surface runoff.



Where,

$N = 0.83\,A^{0.2}$

fig. Variable slope Method.

(Imp)

Unit Hydrograph (UH):

A Unit hydrograph (UH) is defined as a direct runoff hydrograph (DRH) excluding the base flow resulting from one unit depth of rainfall excess occuring uniformly over the basin at constant rate.

- The term unit refers to a unit depth of rainfall excess which is 1cm in SI unit and 1 inch in FPS unit.
- Duration of unit hydrograph (D-hour UH) indicates the duration of rainfall excess
- Thus, unit hydrograph gives the response of the catchment due to 1cm of effective rainfall, which occurs due to a storm of unit duration.

Assumption for UH:

⇒ • There is a constant intensity of excess rainfall within effective duration.
- There is uniform distribution of excess rainfall over the basin.
- The direct runoff response to the rainfall is assumed to be linear.
- Given excess rainfall will always produce same DRH whatever may be the season.
- The DRH doesnot depend upon the time when storm occurs.
- The base period of the direct runoff hydrograph remains the same whatever may be the magnitude of effective rainfall.

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Uses of Unit Hydrograph (UH):

→ • Unit hydrograph can be used to develop the flood hydrograph for extreme rainfall magnitude for the design of hydraulic structure
- Used to extend the flood flow records based on the rainfall records.
- This can be also used for flood forecasting and warning system.
- It can be used for the watershed simulation models.
- Once a unit hydrograph is prepared for a duration D hr. of a basin, the storm hydrograph for that basin for any other storm of different intensities but same duration can be developed.

Limitation of Unit Hydrograph (UH):

→ • The unit hydrograph can be adopted for very large catchment area more than 5000 km^2 and for a basin having less area than 2 km^2.
- It is not suitable for long basin.
- It is not applicable for basin having high variation of rainfall intensity.
- Precipitation must be form rainfall only. Snow melt runoff cannot be satisfactorily represented.

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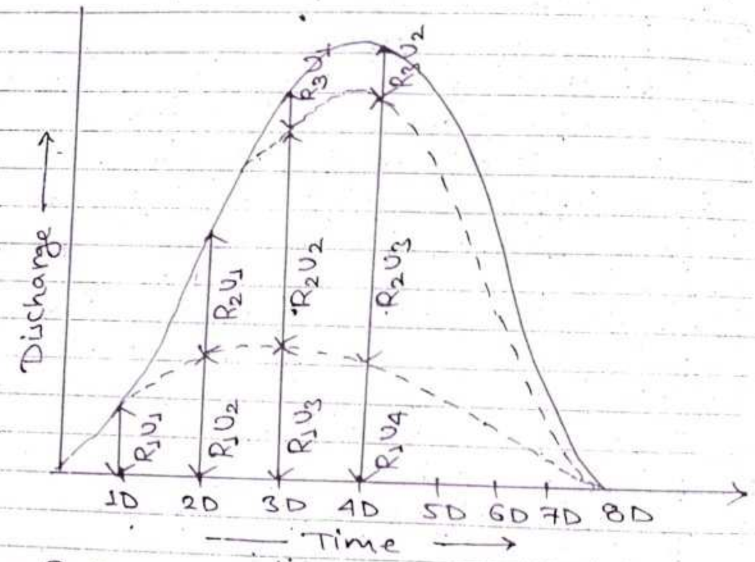
- The Unit hydrograph theory is not very accurate. The accuracy obtained is 10%.
- Unit hydrograph is applicable for short duration.
- The base period of the ~~catchment~~ direct runoff is not exactly same for all the storm of the same duration but different intensities.

Derivation of UH from Flood Hydrograph

- When suitable simple isolated storms are not available, data from complex storm of long duration will have to be used to derive the unit hydrograph.
- The problem is to decompose a measured composite flood hydrograph into its components DRHs and base flow.
- let $U_1, U_2, U_3 \ldots U_n$ be the ordinate of unit hydrograph.
- consider a rainfall excess made up of three consecutive durations of D hours and ER (effective rainfall) values of $R_1, R_2, R_3 \ldots$

- From storm hydrograph, deduct base flow (Q_b) to calculate the direct runoff hydrograph (DRH).
- let $Q_1, Q_2, \ldots Q_n$ be the ordinate of Direct runoff hydrograph (DRH).



Then,

$Q_1 = R_1 U_1$

$Q_2 = R_1 U_2 + R_2 U_1$

$Q_3 = R_1 U_3 + R_2 U_2 + R_3 U_1$

$Q_4 = R_1 U_4 + R_2 U_3 + R_3 U_2$ ..... and so on

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(#) Unit hydrograph of Different Duration:

→ The Unit hydrograph of various durations should be derived by the analysis of the storm hydrograph of the same duration as far as possible. But if the storm hydrograph for the storms of required duration are not available, the unit hydrograph of other duration can be used to develop the unit hydrograph of the required duration.

There are two methods available for this purpose. They are

- Method of superposition
- Summation curve (S-curve)

① Method of superposition:

→ If a D-h Unit hydrograph is available and it is desired to develop a unit hydrograph of nD-h, where 'n' is integer, it is easily accomplished by superposing n-unit hydrograph with each graph separated from the previous one by D-h. If the two unit hydrograph are added graphically or analytically, the combined hydrograph curve is obtained.

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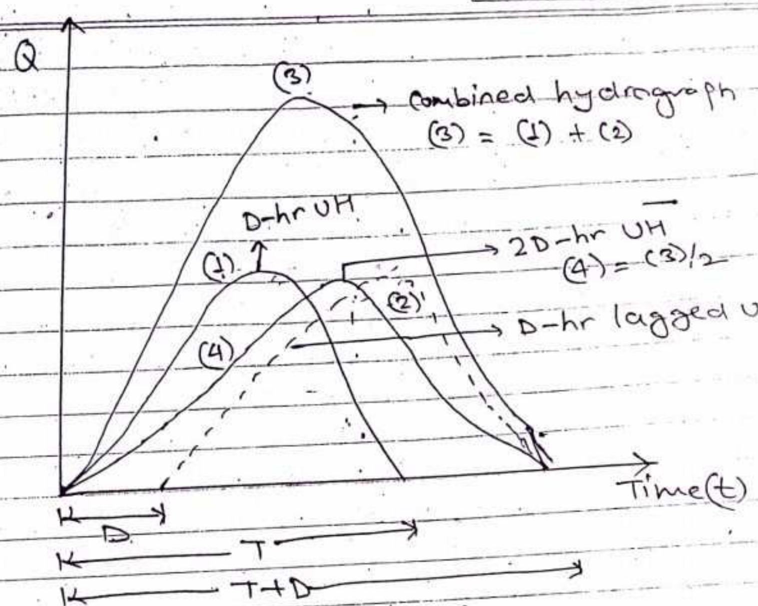

fig: Method of superposition

steps:

- lag the UH ordinate by D, 2D, ..(n-1)D
- Add the ordinates of all UHs (U_1)
- ordinate of D'-hour UH = U_1/n

If the ordinate of combined hydrograph are divided by n, we obtain 2hr unit hydrograph.

② Summation curve (S-curve):

If it is desired to develop a unit hydrograph of duration mD, where 'm' is a

fraction, the method of superposition can not be used. A different technique known as S-curve is adopted.
This method is applicable for rational value of 'm'. The hydrograph produced by a continuous effective rainfall at a constant rate for an infinite period is S-curve. This curve is obtained by summation of an infinite series of D-h unit hydrograph spaced D-apart.

Note: lag the ordinate of S_1 curve by duration D'.

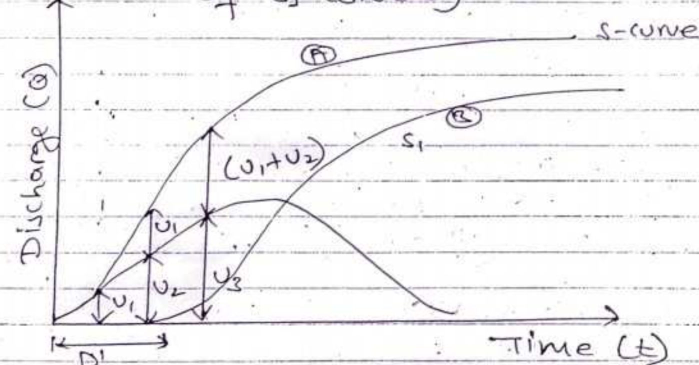

fig. S-curve method

Note:

Direct runoff (DRH) = UH ordinate × Re

also, UH ordinate = $\frac{DRH}{Re}$

Numericals:

① The peak of a flood hydrograph due to a storm is 470 m³/sec. The mean depth of rainfall is 8 cm. Assume the avg. infiltration loss of 0.25 cm/hr and a constant base flow of 15 m³/sec. Estimate the peak discharge of a 6hr-unit hydrograph for this catchment.

→ Given:

Peak discharge (Q) = 470 m³/sec

Base flow (Q_b) = 15 m³/sec

∴ peak of DRH (Q_d) = 470 − 15 = 455 m³/sec.

Rainfall (P) = 8 cm

loss in 6hr (due to infiltration) = 0.25 × 6

(I) = 1.5 cm

∴ Effective rainfall (Re) = P − I = 8 − 1.5 = 6.5 cm

∴ Peak of 6-hr UH = $\frac{\text{Peak of DRH}}{Re} = \frac{455}{6.5}$ = 70 m³/s

② Given below are the ordinate of a 6-hr unit hydrograph for a catchment. Calculate the ordinate of DRH due to rainfall excess of 3.5 cm occuring in 6hr.

Time (hr)	0	3	6	9	12	15	18	24	30	36	42	48	54	60	69
UH ordinate (m³/sec)	0	25	50	85	125	160	185	160	110	60	36	25	16	8	0

Given:

Rainfall excess $(R_e) = 3.5\text{cm}$

⇒ Then,

we have

DRH ordinate = UH ordinate × R_e

∴ DRH ordinate = (UH ordinate × 3.5)

Time (hr)	Ordinate of 6-hr UH	Ordinate of DRH (m³/sec)
0	0	0
3	25	87.5
6	50	175
9	85	297.5
12	125	437.5
15	160	560
18	185	647.5
24	160	560
30	110	385
36	60	210
42	36	126
48	25	87.5
54	16	56
60	18	28
69	0	0

(2011)

③ The following are the ordinate of the hydrograph of the flow from a catchment area of 770 km² due to 6-hr rainfall. Derive the ordinate of 6hr-unit hydrograph. Make suitable assumption regarding base flow.

Time (hr)	0	6	12	18	24	30	36	42	48	54	60	66	72
Discharge (m³/sec)	40	65	215	360	400	350	270	205	145	100	70	50	42

→ Given:

Area of catchment $(A) = 770\ \text{km}^2 = 770 \times 10^6\ \text{m}^2$

Assume base flow $(Q_b) = 40\ \text{m}^3/\text{sec}$.

∴ Direct runoff $(Q_d) = Q - Q_b$

Time (hr)	Q (m³/sec)	Base flow (Q_b)	DRH ordinate $Q_d = Q - Q_b$	UH ordinate (m³/sec)
0	40	40	0	0
6	65	40	25	5
12	215	40	175	35
18	360	40	320	64
24	400	40	360	72
30	350	40	310	62
36	270	40	230	46
42	205	40	165	33
48	145	40	105	21
54	100	40	60	12
60	70	40	30	6
66	50	40	10	2
72	42	40	2	0.4
		Sum	$\Sigma Q_d = 1792$	

$\therefore UH = \frac{DRH}{R_e}$

∴ DR in 6hrs $= 1792 \times 6 \times 60 \times 60 = 38707200\ \text{m}^3$

Depth of runoff $(R_e) = \frac{38707200}{770 \times 10^6} = 0.05\ \text{m} = 5\text{cm}$

(4) The ordinate of 5hr unit hydrograph are given below

Time (hr)	0	5	10	15	20	25	30	35	40	45	50	55
UH ordinate (m^3/sec)	0	20	60	150	120	90	66	50	32	20	10	0

If the storm of 1.2cm effective rainfall with 5 hr - duration occurs in the catchment; calculate the resulting hydrograph of flow. Assume base flow to be uniform at 10 m^3/sec.

⇒ Given.

Rainfall excess (Re) = 1.2cm

Base flow (Q_b) = 10 m^3/sec.

Direct runoff (DRH) = Q_d = UH ordinate × R_e

and

Total discharge = $Q_d + Q_b$

Time (hr)	UH ordinate	DRH (Q_d) ordinate	Base flow Q_b	$Q = Q_d + Q_b$
0	0	0	10	10
5	20	24	10	34
10	60	72	10	82
15	150	180	10	190
20	120	144	10	154
25	90	108	10	118
30	66	79.2	10	89.2
35	50	60	10	70
40	32	38.4	10	48.4
45	20	24	10	34
50	10	12	10	22
55	0	0	10	10

(5) Given the following data about a catchment of area 1000km². Determine the peak discharge corresponding to a storm of 5cm in 1hr.

Time (hr)	0	1	2	3	4	5
Rain fall (cm)	0	2.5	0	0	0	0
Run off (m^3/sec)	300	300	1200	450	300	300

⇒ Given:

Total runoff = (300 + 300 + 1200 + 450 + 300 + 300)

= 2850 m^3/sec.

In 1-hrs

Runoff = 2850 × 1 × 60 × 60 = 10260000 m^3

Area of catchment (A) = 1000 km² = 1000×10^6 m²

∴ Rainfall excess (Re) = $\frac{10260000}{1000 \times 10^6}$ = 1.03 cm

Time (hr)	Runoff ordinate	UH ordinate
0	300	291.26
1	300	291.26
2	1200	1165.04
3	450	436.89
4	300	291.26
5	300	291.26

∴ From table, Peak of UH = 1165.04 m^3/sec.

For 5cm of rainfall

Peak of DRH = 5 × 1165.04

= 5825.20 m^3/sec.

(2006) (6) The ordinate of a 4-hr in unit hydrograph of a stream catchment are given. Using this derive the ordinate of 12-h unit hydrograph for the same catchment. Assume other data if necessary.

Time (hr)	0	4	8	12	16	20	24	28	32	36	40	44
ordinate of 4h UH (m^3/sec)	0	20	80	130	150	130	90	52	27	15	5	0

→ Here, Req. duration of UH (D') = 12 hr

Given duration (D) = 4 hr

$\therefore n = D'/D = 12/4 = 3$ (integer)

∴ let us lag the UH ordinate by (n−1) D i.e. 2D

i.e. 4hr, 8hr.

$\therefore$ 12h-UH ordinate $= \frac{\text{DRH in 12-hr}}{n}$

Time (hr)	Ordinate of 4h-UH (a)	lagged by 4hr (b)	lagged by 8hr (c)	DRH of 3cm in 12-h (m^3/sec) (d)	Ordinate of 12-h UH (m^3/sec)
0	0			0	0
4	20	0		20	6.67
8	80	20	0	100	33.33
12	130	80	20	230	76.7
16	150	130	80	360	120
20	130	150	130	410	136.7
24	90	130	150	370	123.3
28	52	90	130	272	90.7
32	27	52	90	169	56.3
36	15	27	52	94	31.3
40	5	15	27	47	15.7
44	0	5	15	20	6.7
48	–	0	5	5	1.7
52	–	–	0	0	0

Note: d = a+b+c.

(2011) (7) A 3hr duration unit hydrograph has following ordinates.

Time (hr)	0	3	6	9	12	15	18	21	24	27	30
Discharge (m^3/sec)	0	3	4.9	8.6	9.8	7.4	4.94	3.7	2.4	1.23	0

Determine the unit hydrograph of 6hr.

→ Given:

Given duration (D) = 3hr

Required duration (D') = 6hr

$\therefore n = D'/D = 6/3 = 2$ (ie. integer)

∴ Superposition method can be used if n is integer.

Although it can be solved by summation curve (S-curve) method.

Here let us solve this problem by S-curve method.

~~In S-curve, ordinate of curve is lagged by D' hr~~ ie. 6hr.

$0 = (t-D) \quad 3-3 = 0 \quad 0 = (t-D) = (6-3) = 3$

$(t-D) = 6$

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(C) = (A)+(B)

D(E) = (C) − (D)

Time (hr)	(A) ordinate of 3-hr UH (m³/sec)	(B) S-curve addition (m³/sec)	(C) = (A)+(B) S-curve ordinate (m³/s)	(D) S-curve lagged by 6hr	DRH of 2cm in 6hr	6-hr UH ordinate
0	0	—	0	—	0	0
3	3	0	3	—	3	1.5
6	4.9	3	7.9	0	7.9	3.95
9	8.6	7.9	16.5	3	13.5	6.75
12	9.8	16.5	26.3	7.9	18.4	9.2
15	7.4	26.3	33.7	16.5	17.2	8.6
18	4.94	33.7	38.64	26.3	12.34	6.17
21	3.7	38.64	42.34	33.7	8.64	4.32
24	2.4	42.34	44.74	38.64	6.1	3.05
27	1.23	44.74	45.97	42.34	3.63	1.815
30	0	45.97	45.97	44.74	1.23	0.615
		45.97	45.97	45.97	0	0
				45.97	0	0
				45.97	0	0

(2013) (2009)

(8) The ordinate of a flood hydrograph resulting from two sucessive storm of 1cm each rainfall excess and 6hr duration are tabulated below. Find a resulting hydrograph. Assume base flow to be uniform at 10 m³/sec.

Time (hr)	0	6	12	18	24	30	36	42	48	54	60	66	72
Flood (m³/sec) (UH)	10	30	90	220	280	220	166	126	92	62	40	20	10

→ Here, Base flow (Q_b) = 10 m³/sec.

Rainfall excess are $R_1 = 1cm$, $R_2 = 1cm$

$DRH_1 = UH \times R_1$

$DRH_2 = UH \times R_2$ (laged by 6hr)

$\therefore DRH = DRH_1 + DRH_2$

and $Q = DRH + Q_b$

Time (hr)	UH	DRH_1	DRH_2	DRH	Q_b	$Q = DRH + Q_b$
0	10	10	—	10	10	20
6	30	30	10	40	10	50
12	90	90	30	120	10	130
18	220	220	90	310	10	320
24	280	280	220	500	10	510
30	220	220	280	500	10	510
36	166	166	220	386	10	396
42	126	126	166	290	10	300
48	92	92	126	218	10	228
54	62	62	92	154	10	164
60	40	40	62	102	10	112
66	20	20	40	60	10	70
72	10	10	20	30	10	40
78	—	—	10	10	10	20

(2007)

(9) The ordinate of 6hr unit hydrograph are given below.

Time (hr)	0	3	6	9	12	18	24	30	36	42	48	54	60	66
6-hr UH ordinate (m³/sec)	0	150	250	450	600	800	700	600	450	320	200	100	50	0

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A storm had three successive 6-hr intervals of rainfall magnitude of 3, 5 and 4 cm resp. Assuming a ϕ-index of 0.2 cm/hr and a base flow of 30 m^3/sec, Determine the resulting hydrograph of flow.

⇒ Here,
ϕ-index (infiltration loss) = 0.2 cm/hr
For 6-hr, loss (L) = 0.2 × 6 = 1.2 cm
Rainfall value of 6hr interval
R_1 = 3 cm, R_2 = 5 cm & R_3 = 4 cm

Then,
Rainfall excess (Re_1) = $R_1 - L$ = 3 − 1.2 = 1.8 cm
Rainfall excess (Re_2) = $R_2 - L$ = 5 − 1.2 = 3.8 cm
Rainfall excess (Re_3) = $R_3 - L$ = 4 − 1.2 = 2.8 cm
Base flow (Q_b) = 30 m^3/sec.

∴ DRH_1 = UH × Re_1

DRH_2 = UH × Re_2 (lagged by 6hr)
DRH_3 = UH × Re_3 (lagged by 12 hr)

DRH = (DRH_1 + DRH_2 + DRH_3)

∴ Total runoff (Q) = DRH + Q_b

Computation of flood hydrograph are calculated below.

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Time (hr)	UH	DRH_1	DRH_2	DRH_3	DRH	Q_b	Q
0	0	0	–	–	0	30	30
3	150	270	–	–	270	30	300
6	250	450	0	–	450	30	480
9	450	810	570	–	1380	30	1410
12	600	1080	950	0	2030	30	2060
18	800	1440	1710	420	3570	30	3600
24	700	1260	2280	700	4240	30	4270
30	600	1080	3040	1260	5380	30	5410
36	450	810	2660	1680	5150	30	5180
42	320	576	2280	2240	5096	30	5126
48	200	360	1710	1960	4030	30	4060
54	100	180	1216	1680	3076	30	3106
60	50	90	760	1260	2110	30	2140
66	0	0	380	896	1276	30	1306
72	–	–	190	560	750	30	780
78	–	–	0	280	280	30	310
84	–	–	–	140	140	30	170
90	–	–	–	0	0	30	30

(10) Given below is the ordinate of 24-hr unit hydrograph. Derive 12-hr UH ordinate.

Time (hr)	0	12	24	36	48	60	72	84	96	108	120
UH (m^3/s)	0	103	275	165	78	36	20	11	5	3	0

⇒ Given:

Required duration of UH (D') = 12 hr

Given duration (D) = 24 hr

$\therefore n = \frac{D'}{D} = \frac{12}{24} = 0.5$ (real)

So, using summation curve method (S-curve) method.

Time (hr)	Ordinate of 24hr UH	S-curve addition	S-curve ordinate	S-curve lagged by 12 hr	DRH of 0.5cm in 12-hr	Ordinate of 12-hr UH
0	0	—	0	—	0	0
12	103	—	103	0	103	206
24	279	0	279	103	176	352
36	165	103	268	279	(-11)	(-22)
48	78	382	460	268	192	384
60	36	547	583	460	123	246
72	20	625	645	583	62	124
84	11	661	672	645	27	54
96	5	681	686	672	14	28
108	3	692	695	686	9	18
120	0	697	697	695	2	4
		700	700	697	3	6
		700	700	700	0	0
				700	(-700)	(-1400)

(2010) (2008) (2012)

(11) The ordinate of 4-hr unit hydrograph of a catchment area of 25 km² are given below.

Time (hr)	0	4	8	12	16	20	24	28	32	36	40	44	48
UH (m³/s)	0	30	55	90	130	170	180	160	110	60	35	20	0

Calculate the following:

a) The 4-hr DRH of rainfall of 3.25cm with ϕ-index of 0.25cm/hr

b) The 12-hr unit hydrograph using superposition

c) The 12-hr unit hydrograph using S-curve method

⇒ Here,

(a) ⇒ Rainfall (P) = 3.25cm

ϕ-index = 0.25 cm/hr (Infiltration capacity)

For 4-hr, ϕ-index = (I) = 0.25 × 4 = 1 cm

∴ Rainfall excess (R_e) = 3.25 − 1 = 2.25cm

∴ DRH ordinate = UH ordinate × Re

Time (hr)	UH (m³/s)	DRH = UH × Re
0	0	0
4	30	67.5
8	55	123.75
12	90	202.5
16	130	292.5
20	170	382.5
24	180	405.0
28	160	360.0
32	110	247.5
36	60	135.0
40	35	78.75
44	20	45.0
48	0	0

b) Given duration of UH (D) = 4 hr
Req. duration of UH (D') = 12 hr

$\therefore n = D'/D = 12/4 = 3$ (integer)

So, superposition method is applicable.

Time (hr)	Ordinate of 4hr-UH	lagged by 4hr	lagged by 8hr	DRH of 3cm in 12-hr	Ordinate of 12-hr UH
0	0	–	–	0	0
4	30	0	–	30	10
8	55	30	0	85	28.33
12	90	55	30	175	58.33
16	130	90	55	275	91.67
20	170	130	90	390	130
24	180	170	130	480	160
28	160	180	170	510	170
32	110	160	180	450	150
36	60	110	160	330	110
40	35	60	110	205	68.33
44	20	35	60	115	38.33
48	0	20	35	55	18.33
		0	20	20	6.67
			0	0	0

c) ⇒ Here,

Req. Duration of UH (D') = 12 hr
Given duration of UH (D) = 4 hr

$\therefore n = D'/D = 12/4 = 3$ (integer)

So, summation method can be also applied.

Time (hr)	Ordinate of 4hr-UH	S-Curve addition	S-Curve ordinate	S-Curve lagged by 12-hr	DRH of 3cm in 12hr	12-hr UH ordinate
0	0	–	0	–	0	0
4	30	0	30	–	30	10
8	55	30	85	–	85	28.33
12	90	85	175	0	175	58.33
16	130	175	305	30	275	91.67
20	170	305	475	85	390	130
24	180	475	655	175	480	160
28	160	655	815	305	510	170
32	110	815	925	475	450	150
36	60	925	985	655	330	110
40	35	985	1020	815	205	68.33
44	20	1020	1040	925	115	38.33
48	0	1040	1040	985	55	18.33
		1040	1040	1020	20	6.67
				1040	0	0

- Method 3:

→ In this method the base flow recession curve after the depletion of the flood water is extended backward till it intersects the ordinate at the point of inflection (line EF). points A and F are joined by arbitary smooth curve.

Engineering Application

Frequency and probability concepts:

→ When all the causative factors are known the outcome can be predicted uniquely. such approach is known as determinist approach. If all the causative factors are unknown, then out come cannot be predicted uniquely. such approach is known as probabilistic approach. The out come of experiment is governed by some probability law.

If an experiment conducted N times or if the outcome of a process is observed 'N' times and if a particular attribute 'A' occurs n times, then n/N is the probability of event A. Frequency or reoccurance interval or return period 'T' is the reciprocal of probability.

i.e. $T = \frac{1}{P}$

Where,

T = return period in years

p = probability.

If the probability of an event occuring is 'p', the probability of not occuring event is $q = (1-p)$.

$\therefore \boxed{p+q = 1}$

Return period / Recurrence interval

→ Return period is defined as the average interval of time 'T' within which an event of given magnitude will be equalled or exceeded at least once.

For example: A coin is tossed once a year. on average, it's head will appear once in two years.

$$\text{ie. } T = \frac{1}{P} = \frac{1}{0.5} = 2.$$

This reciprocal of the probability of occurrence is termed as return period or recurrence interval T. The return period is widely used in hydrologic frequency analysis.

Flood:

→ A flood is an unusually high stage in a river, normally the level at which the river overflows its banks and inundates the adjoining area. In other words, it is an overflow of water that submerges land which is usually dry.

Causes of flood

→ Flood are caused by the natural phenomena but may be increased by human activities.

⊙ **Natural causes**

- continuous rainfall and cloudbrust.
- Blocking of rivers by landslide.
- sudden release of huge amount of water stored in the dam due to failure.
- Earthquake in sea which may rise the river bed (ie. tsunami)
- Melting of snow and glacior.
- Brusting of glacior lake.
- sea storm.
- synchronisation of peak flow of rivers.

⊙ **Human intervention**

- Land-use change, eg. deforestation, urbanization
- structural failure. eg. dam, embankment failure
- Drainage congestion caused by uncoordinate development activities.

Effects of flood:

→ The effects of flood are as follows:

- Loss of life and property.
- Destruction of physical infrastructure.
- Damage to agriculture.
- Damage to reservoirs and dams
- Disruption of social and economic development.

- Damage to hydraulic structure like bridge, embankment.
- contamination of water sources.

Flood control / mitigation Measures

→ Flood causes huge loss of life, property and economic loss due to disruption of economic activity. Thousands of crores of rupees are spent every year in flood control and flood casting. The different measures for flood control are as follows.

- Storage reservoir: Flood water is stored in the reservoir and released in controlled manner over an extended time.

- Detention reservoir: It is an obstruction in river in the form of small structure. It is used for storing water temporarily and restricting the outflow rate.

- Flood embankment: It is an earthern bank constructed parallel to river course to confine it to fixed course and limit cross-sectional width. It is the most common method for flood protection work.

- Soil conservation: soil conservation measures increases infiltration and evaporation and thus reduces soil erosion and also runoff.

- Flood way: They are natural or manmade channel to divert flood.

- Channel improvement: Widening or deepening channel, reduction of roughness helps to minimize flood.

- Flood forecasting and warning: Flood forecasting provides warning for people to evacuate areas threatened by floods and also to help water management personnel to operate flood control structures such as spillways or reservoirs.

- Flood plain zoning: Flood plain zoning is a map which shows the location and extent of areas likely to be affected due to flood. Development plans of these area are prepared in such a way that the resulting damage due to flood are within acceptable limits of risk.

Factors affecting floods

→ The different factors affecting flood are described below:

- Precipitation: More the intensity, duration and coverage of storm more will be the flood.

- Size and shape of basin: Larger the size of the catchment more will be the flood. On the other hand fan shaped catchment has more peak discharge than elongated shape due to less concentration time.

- Movement of storm: If the storm moves along the flow direction, the time of the concentration will be less and the base period will also be less and hence peak discharge will be more.

- Geographical factors: The geographical factors like nature of soil, permeability, rock outcrop, faults, fissure, cracks, vegetation also affect the flood.

- Antecedent Precipitation Index (API): When API is high, infiltration rate of the soil will be low resulting in high value of runoff. This results to higher value of flood discharge.

- Physical factors: The physical factors like slope of catchment, drainage pattern, drainage density, Manning's coefficient affects the flood.

Methods of Estimation of flood:

→ The different methods available to estimate the magnitude of flood are as follows. They are

- → Rational Method
- → Empirical formulas
- → Probable Maxm precipitation Method
- → Unit-hydrograph technique
- → Physical Indication of past records
- → Flood frequency analysis

→ WECS/DHM Method

① Rational Method:

→ Rational method is commonly used method for computing peak discharge for small basins. up to 50 sq km. A/c to this method a rainfall of intensity 'i' begins instantaneously & continues indefinitely, the rate of runoff will increase until the time of concentration (t_c). After (t_c), runoff becomes constant for the period of rainfall excess $t - t_c$.

The peak discharge is given by

$$Q = \frac{C i A}{360}$$

where,

i = intensity of rainfall (mm/hr)

A = Area of catchment (hectares)

C = runoff coefficient

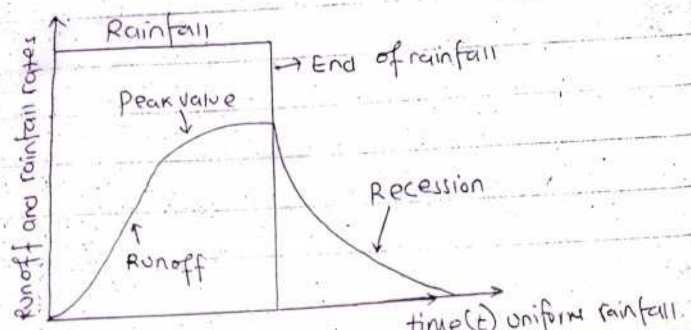

fig. Runoff hydrograph due to a uniform rainfall.

Assumptions:

- Rainfall intensity is constant throughout the storm duration.
- The computed peak rate of runoff at the outlet point is a function of the average rainfall rate during t_c.
- The runoff will become constant beyond t_c. i.e. time of concentration.
- Runoff rate gradually increases from zero to a constant value.
- t_c employed is the time for runoff to become established and flow from the remote part of the basin to the outlet.

Limitation:

- Applicable for small basin (up to 50 km²)
- Duration of rainfall intensity > t_c.
- It gives only peak discharge and does not give complete hydrograph.
- 'C' assumed to be same for all storms.
- Rainfall intensity must be constant over the entire basin during t_c.

Application: It is used for design of storm sewers, channels and other drainage structure.

② Empirical formula:

→ Empirical formula can be used to estimate flood when a more accurate method for flood prediction cannot be applied because of lack of data. The empirical formula are based on statistical correlation of the observed peak and important catchment properties. The different empirical formula are

- Dicken's formula:

$$Q = C A^{3/4}$$

where,

A = Basin area in km^2

C = Dicken's constant (6-30).

- Ryves formula:

$$Q = C \cdot A^{2/3}$$

where,

A = catchment area in km^2

C = Ryves constant (6-40).

- Inglis formula:

$$Q = \frac{124 A}{\sqrt{A + 10.4}}$$

where,

A = catchment area in km^2

③ probable Max^m precipitation Method

→ This method is generally adopted in case of hydraulic structures like Dam, weir, spillways. The PMP is defined as the greatest or extreme rainfall of given duration that is physically possible over basin. It is calculated by meterological method or statistical study. statistically,

$$PMP = \bar{P} + K\sigma$$

where,

$\bar{P}$ = Avg. rainfall

K = freq. factor

σ = standard deviation.

limitations:

- Applicable for huge hydraulic structure.
- It gives over estimation for medium and small catchment.

④ Unit - hydrograph technique:

→ If the rainfall data, infiltration characteristics of the catchment and appropriate unit hydrograph are available then flood hydrograph can be computed using the following equations.

$$Q_n = \sum_{M=1}^{n \le M} P_m U_{n-m+1}$$

Ravi

where,

n = no. of runoff ordinates

m = no. of pulses of rainfall excess

Q = Direct runoff

P_m = Excess rainfall

U_{n-m+1} = Unit hydrograph ordinate

Physical Indication of past records:

→ The value of HFL can be taken from the past records. After knowing HFL the cross section of river can be plotted. Suitable value of 'n' is selected based on bed roughness. Bed slope and hydraulic radius is computed to calculate the velocity.

Then, Discharge is given by

$$Q = A \cdot V$$

- This method gives only rough estimate.
- River cross-section doesnot remain costant due to sedimentation, erosion & weed growth.
- The manning's coeff. 'n' also changes with the river bed source.
- Alignment of river may change from the time of past records.

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⑥ WECS/ DHM Method:

→ The mean monthly flow and flow duration curve can be determined from this method. It is developed for predicting river flows for catchment areas larger than 100 km² of ungauged rivers based on hydrological theories, empirical equations and statistics. In the context of Nepal, Water and Energy commission secretariat (WECS) / Department of Hydrology and Meterology (DHM) developed empirical relationships for analyzing flood of different frequencies.

For 2-year return period:

$$Q_2 = 1.8767\,(A_{3000} + 1)^{0.8783}$$

For 100-year return period:

$$Q_{100} = 14.63\,(A_{3000} + 1)^{0.7342}$$

where,

A_{3000} = Basin area (km²) below 3000m elevation.

For other return period

$$Q_T = \exp(\ln Q_2 + S\sigma)$$

where,

Q_T = flood of T year return period (m³/s)

S = standard normal variate

ϵ = parameter which is computed as

$$\sigma = \ln(Q_{100}/Q_2)/2.326.$$

(7) Flood frequency Analysis:

→ It can be used to estimate the peak discharge of known frequency directly with the runoff data. This analysis can be used to estimate flood magnitudes for return periods less than the periods of the observed record where the estimation can be performed by interpolation.

The general equation of hydrologic frequency analysis is

$$x_T = \bar{x} + K\sigma$$

where,

x_T = value of the variate x of a random hydrologic series with a return period T.

$\bar{x}$ = Mean of the variate

σ = standard deviation of the variate

K = freq. factor which depends upon return period T.

The some of the frequency distribution function commonly used for the predication of extreme flood discharge are as follows.

- Gumbel's extreme-value distribution
- Log pearson Type III distribution
- Log normal distribution.

(a) Gumbel's extreme-value distribution: (2005)

→ It is one of the most widely used probability distribution functions for extreme values in hydrologic and meteorologic studies for prediction of flood peaks, max^m rainfalls, max^m wind speed etc. Gumbel defined a flood as the largest of the 365 daily flows and the annual series of flood flows constitute a series of largest values of flows. A/c to his theory of extreme events, the probability of occurrence of an event equal to or larger than the value x_0 is

$$P[X \geq x_0] = 1 - e^{-e^{-y}}$$

where,

y is a dimensionless variable

$$y = \alpha(x - a)$$

$$a = \bar{x} - 0.45005\,\sigma_x$$

$$\alpha = \frac{1.2825}{\sigma_x}$$

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Thus,

$$y = \frac{1.2825(x-\bar{x})}{\sigma_x} + 0.577$$

In practise,

$$y_p = -\ln[-\ln(1-p)]$$

we know, return period $(T) = 1/p$

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

or, $$y_T = -\left[-0.834 + 2.303 \log\log \frac{T}{T-1}\right]$$

Now re arranging the value of the variate x with a return period T is

$$x_T = \bar{x} + K\sigma_x$$

where,

$$K = \frac{(y_T - 0.577)}{1.2825}$$

Gumbel probability paper (2007)

→ The Gumbel probability paper is the graphical representation of Gumbel distn. The Gumbel distribution will plot as a straight line on a Gumbel probability paper. On the x-axis return period 'T' is plotted where as on y-axis the discharge is plotted. This graph gives a st. line for the relationship of return period and discharge. From this we can calculate the discharge for any return period within the graph.

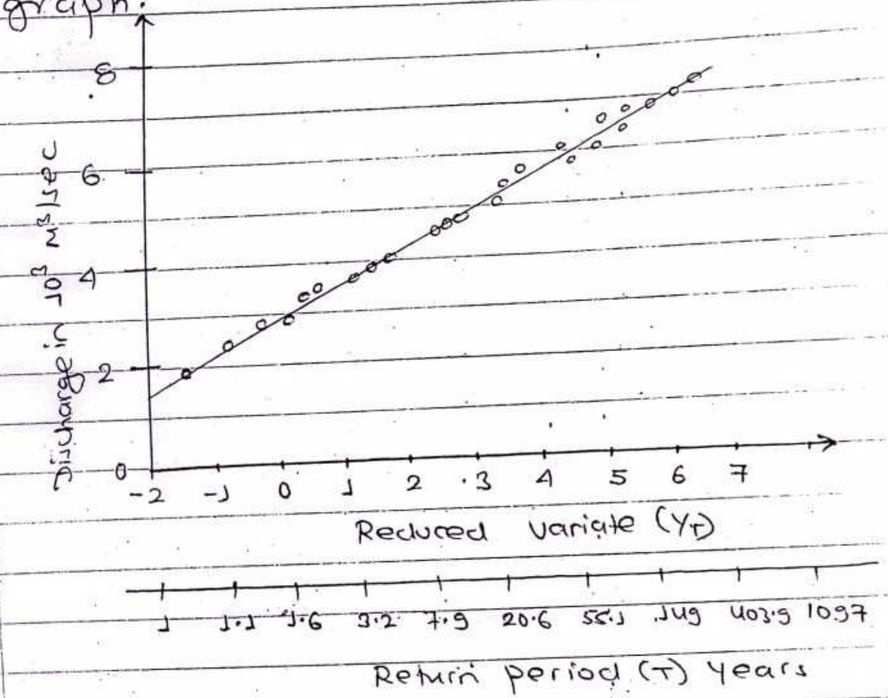

fig. Flood probability analysis by Gumbel's Distribution.

Procedure:

- construct an arithmetic scale of Y_T values from -2 to 7 as shown in fig.

The relationship between given variables such as discharge and the reduced variate (Y_T) is linear. For constructing Gumbel probability paper return period T can be computed for different values of reduced variate Y_T as shown in table below.

Y_T	-2	-1	0	1	2	3	4	5	6	7
T	1	1.1	1.6	3.2	7.9	20.6	55.1	149	403.9	1097

- Find value of Y_T by equation $Y_T = \ln(\ln(T/(T-1)))$ for selected values of T.

First mark value of discharge on y-axis and value of Y_T (say from -2 to 7) on x-axis. For corresponding value of Y_T return period, T is marked on the x-axis below Y_T

- Mark off those positions on the abscissa. prepare a table for plotting position and plot the data on the graph. Fit st. line & extrapolate it for finding flood at different frequencies.

b) **Log Pearson Type III distribution:** (2006)

→ Log pearson III distribution is extensively used in USA for frequency analysis of annual maximum floods. In this method, the variate is first transfered into logarithmic form (base 10) and then transformed data is analysed.

If x is a variate of a random hydrologic series, then the series of z variate where $z = \log x$ are first obtained.

For this z series, for any recurrence interval T

$$Z_T = \bar{z} + K_z \sigma_z \longrightarrow (1)$$

where,

K_z = a freq. factor. (function of recurrence interval T & C_s)

σ_z = standard deviation of z variate sample

$$= \sqrt{\frac{\Sigma (z-\bar{z})^2}{N-1}}$$

Now, coeff. of skewness (C_s) is given by

$$C_s = \frac{N \Sigma (z-\bar{z})^3}{(N-1)(N-2)(\sigma_z)^3}$$

where, $\bar{z}$ = mean of the z values

N = no. of years of record = sample size.

After finding z_T from eqn (1) the corresponding value of x_T is obtained by

$$x_T = \text{antilog}(z_T).$$

When the skew is zero (ie. $C_s = 0$) the log-pearson type III distn reduces to log normal distn.

Risk:

→ Any hydraulic structure is designed for a discharge having probability of P. This hydraulic structure is expected to have useful life of N-year. There is a probability that discharge is more than that may occur in the life time of the structure. Thus, there is some sort of risk involved in this.

The risk (R) represents probability of failure of a structure.

$$R = 1 - (1-P)^N$$

$$\text{or, } R = 1 - \left(1 - \frac{1}{T}\right)^N$$

In percentage,

$$R = \left[1 - \left(1 - \frac{1}{T}\right)^N\right] \times 100\%$$

(2007) Numericals:

(1) A bridge has a expected life 25 years and designed for a flood magnitude of return period 100 years.

a) What is risk of this hydraulic design?

b) If 10% risk is acceptable, what return period will have to be adopted?

→ Here,

a) Expected life (N) = 25 years

Return period (T) = 100 years.

Risk = R = ?

We know that,

$$R = \left[1 - \left(1 - \frac{1}{T}\right)^N\right] \times 100\%$$

$$\text{or, } R = \left[1 - \left(1 - \frac{1}{100}\right)^{25}\right] \times 100\%$$

$$\text{or, } R = 22.22\%$$

Hence, the risk of hydraulic design is 22.22%.

b) Risk (R) = 10%.

Expected life (N) = 25 years

Return period (T) = ?

We have, $R = \left[1 - \left(1 - \frac{1}{T}\right)^N\right] \times 100\%$

$$\text{or, } \frac{10}{100} = \left[1 - \left(1 - \frac{1}{T}\right)^{25}\right]$$

$$\text{or, } T = 237.78 \simeq 238 \text{ years.}$$

Hence, to get 10% acceptable risk the bridge will have to be designed for a flood of return period T = 238 years.

(2010) (2008)

2) What return period you would adopt in the design of a bridge on a river if you are allowed to accept 5% of flooding risk in the 30 years of expected life of the bridge.

→ Here,

Expected life (N) = 30 years.

Risk (R) = 5%

Return period (T) = ?

We have,

$$R = \left[1 - \left(1 - \frac{1}{T}\right)^N\right] \times 100\%$$

or, $$\frac{5}{100} = \left[1 - \left(1 - \frac{1}{T}\right)^{30}\right]$$

or, T = 585.37 years.

(2007)

3) Assume that the Gopi krishna Hall is located on the edge of the 100 year flood plain in Dhobi Khola. If the design life of cinema hall is 30 years, what is the reliability that it will not be flooded during its design life?

→ Here,

Return period (T) = 100 years

Design life (n) = 30 years.

Reliability (Re) = ? $[Re = 1 - R]$

We know,

probability $(P) = \frac{1}{T} = \frac{1}{100} = 0.01$

Then,

Reliability $(Re) = \left[(1-P)^N\right] \times 100\%$

or, $Re = (1-0.01)^{30} \times 100\%$

or, Re = 73.97%

(2009)

4) The observed annual instantaneous flood values in m^3/s of Kaboli River at Panchami station is given in the following table. Estimate the flood discharge of return period 100 and 1000 years. The reduced mean and reduced standard deviation for sample size of 10 are 0.4952 and 0.9496 resp.

Year	1975	1976	1977	1978	1979	1980	1981	1982	1983	1984
Flood flow	1200	950	1000	1058	700	650	759	509	400	1300

→ Here,

N = 10 years, $y_n = 0.4952$, $S_n = 0.9496$

Year	Q	$Q-\bar{Q}$	$(Q-\bar{Q})^2$
1975	1200	347.4	120686.76
1976	950	97.4	9486.76
1977	1000	147.4	21726.76
1978	1058	205.4	42189.16
1979	700	-152.6	23286.76
1980	650	-202.6	41046.76
1981	759	-93.6	8760.96
1982	509	-343.6	118060.96
1983	400	-452.6	204846.76
1984	1300	447.4	200166.76
	$\Sigma Q = 8526$		$\Sigma(Q-\bar{Q})^2 = 790258.4$

$\bar{Q} = \frac{\Sigma Q}{N} = \frac{8526}{10} = 852.6 \ m^3/sec.$

Standard deviation $(\sigma) = \sqrt{\frac{\Sigma(Q-\bar{Q})^2}{N-1}}$

or, $\sigma = \sqrt{\frac{790258.4}{10-1}}$

or, $\sigma = 296.322$

i) For return period (T) = 100 years

$Y_T = -\ln\left[\ln \frac{T}{T-1}\right]$

or, $Y_T = -\ln\left[\ln \frac{100}{99}\right] = 4.6$

$K_{100} = \frac{Y_T - Y_n}{S_n} = \frac{4.6 - 0.4952}{0.9496} = 4.323$

$\therefore Q_{100} = \bar{Q} + K_{100} \times \sigma$

or, $Q_{100} = 852.6 + 4.323 \times 296.322 = 2133.6 \ m^3/s$

ii) For return period (T) = 1000 years

$Y_T = -\ln\left[\ln \frac{T}{T-1}\right]$

or, $Y_T = -\ln\left[\ln \frac{1000}{999}\right] = 6.907$

$K_{1000} = \frac{Y_T - Y_n}{S_n} = \frac{6.907 - 0.4952}{0.9496} = 6.752$

$\therefore Q_{1000} = \bar{Q} + K_{1000} * \sigma$

or, $Q_{1000} = 852.6 + 6.752 \times 296.322$

or, $Q_{1000} = 2853.36 \ m^3/s$

(2011)

5) The mean annual flood of a river is $800 m^3/s$ & the standard deviation of the annual flood time series is $180 m^3/s$. What is the prob. of a flood of magnitude $2000 m^3/s$ occuring in the river within 10 years? Use Gumbel's method & assume the sample size to be very large.

→ Here,

Mean annual flood ($\bar{x}$) = 800 m³/sec.

σ = 180 m³/s.

x_T = 2000 m³/sec.

we have,

$$x_T = \bar{x} + K \cdot \sigma$$

or, $K = \frac{2000 - 800}{180} = 6.67$

But

$$K = \frac{y_T - y_n}{S_n}$$

For very large sample (ie. $N \to \infty$)

$y_n = 0.577$

$S_n = 1.2825$

$\therefore 6.67 = \frac{y_T - 0.577}{1.2825}$

or, $y_T = 9.131$

Also,

$$y_T = -\ln\left[\ln \frac{T}{T-1}\right]$$

or, $9.131 = -\ln\left[\ln \frac{T}{T-1}\right]$

or, $T = 9236.75$ years.

probability of occuring flood of magnitude 2000 m³/sec (p) $= \frac{1}{T} = \frac{1}{9236.75} = 0.00011$.

∴ prob. of a flood of magnitude 2000 m³/sec occuring at least once in 10 year

$$P_1 = [1 - (1-p)^N]$$

or, $P_1 = [1 - (1 - 0.00011)^{10}]$

or, $P_1 = 0.11\%$.

5) Flood freq. computations for the river Chambal at Gandhisagar dam by using Gumbel's method, yielded following results.

Return period T (years)	Peak flood (m³/sec)
50	40,809
100	46,300

Estimate the flood magnitude in this river with return period of 500 years.

→ Here,

$x_{50} = \bar{x} + K_{50}\, \sigma_n$ ⟶ (I)

$x_{100} = \bar{x} + K_{100}\, \sigma_n$ ⟶ (II)

substracting eqⁿ (I) from (II)

$$x_{100} - x_{50} = (K_{100} - K_{50})\, \sigma_n$$

or, $(K_{100} - K_{50})\,\sigma_n = (46300 - 40809)$

or, $(K_{100} - K_{50})\,\sigma_n = 5491$

But
$$K_T = \frac{Y_T - Y_n}{S_n}$$

where, Y_n and S_n are constant for the given data series.

$$\therefore\ K_{50} = \frac{Y_{50} - Y_n}{S_n} \longrightarrow \text{(iii)}$$

$$K_{100} = \frac{Y_{100} - Y_n}{S_n} \longrightarrow \text{(iv)}$$

Substracting eq^n (iii) from eq^n (iv)

or, $K_{100} - K_{50} = \dfrac{Y_{100} - Y_{50}}{S_n}$

or, $\dfrac{5491}{\sigma_n} = \dfrac{Y_{100} - Y_{50}}{S_n}$

or, $(Y_{100} - Y_{50})\dfrac{\sigma_n}{S_n} = 5491 \longrightarrow \text{(v)}$

But, $Y_T = -\ln\left[\ln \frac{T}{T-1}\right]$

For T = 50 years

$$Y_{50} = -\ln\left[\ln \frac{50}{49}\right] = 3.9019$$

$$Y_{100} = -\ln\left[\ln \frac{100}{99}\right] = 4.6001$$

$\therefore$ eq^n (v) becomes,

$$(4.6001 - 3.9019)\frac{\sigma_n}{S_n} = 5491$$

or, $\dfrac{\sigma_n}{S_n} = 7864.508$

For return period T = 500 years,

$$Y_{500} = -\ln\left[\ln 500/499\right] = 6.2136$$

Again,

$$(Y_{500} - Y_{100})\frac{\sigma_n}{S_n} = (X_{500} - X_{100})$$

or, $(6.2136 - 4.6001) \times 7864.508 = X_{500} - 46300$

(2008) or, $X_{500} = 58989.38\ m^3/sec.$

7) Annual flood data of the river Narmada at Garudesh war covering the period 1948 to 1979 yield for the annual flood discharges a mean of 29,600 m^3/s & a standard deviation of 14,860 m^3/sec. For a proposed bridge on this river near this site it is decided

to have an acceptable risk of 10% in its expected life of 50 years. a) Estimate the flood discharge by Gumbel's Method for use in the design of this structure. b) If the actual flood value adopted in the design is 125,000 m³/s. What are the safety factor and safety margin relating to maxᵐ flood discharge?

→ Here,

Risk = 10%.

life period of structure (n) = 50 years

But

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

$$\text{or, } 0.10 = 1 - \left[1 - \frac{1}{T}\right]^{50}$$

or, T = 475 years

∴ Return period (T) = 475 years.

Record length (N) = 1948 to 1979 = 32 years

For N = 32 years

$Y_n = 0.5380$ and $S_n = 1.1193$ [from table]

$$\therefore Y_T = -\ln\left[\ln \frac{T}{T-1}\right]$$

$$\text{or, } Y_T = -\ln\left[\ln \frac{475}{474}\right] = 6.16226$$

$$K_{475} = \frac{Y_T - Y_n}{S_n} = \frac{6.16226 - 0.5380}{1.1193}$$

or, $K_{475} = 5.0248$.

$$\therefore X_{475} = \bar{x} + K_{475} \cdot \sigma$$

or, $X_{475} = 29600 + 5.0248 \times 14860 = 104268 \text{ m}^3/\text{s}$.

Actual flood magnitude adopted in project = 125,000 m³/sec.

$$\text{safety factor } (SF)_{flood} = \frac{125000}{104268} = 1.19$$

saftey margin for flood magnitude

= 125000 − 104268

= 20732 m³/sec.

(2010)

8) A hydraulic structure on a stream has to be designed for discharge of 350 m³/s. The available flood data on the stream is for 20 years. Mean & standard deviation for annual flood series are 121 m³/s and 60 m³/s resp. Calculate the return period for design flood by using Gumbel's Method. The value of Y_n & S_n for 20 years is 0.5236 and 1.0628 resp. Also compute 80% and 90% confidence limits for above.

flood $f(c) = 1.282$ and 1.645 for 80% and 90% confidence level resp.

Here,

Discharge $(X) = 350\, m^3/s$

Mean of flood $(\bar{x}) = 121\, m^3/s$

Standard deviation of flood $(\sigma) = 60\, m^3/s$

expected life $(n) = 20$ years.

For $n = 20$ years

$$\bar{y}_n = 0.5236, \quad S_n = 1.0628$$

Return period $(T) = ?$

We know that

$$X_T = \bar{X} + K_T \cdot \sigma$$

or, $350 = 121 + K_T \times 60$

or, $K_T = 3.816$

But,

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

or, $3.816 = \dfrac{Y_T - 0.5236}{1.0628}$

or, $Y_T = 4.579$

Also,

$$Y_T = -\ln\left[\ln \frac{T}{T-1}\right]$$

or, $4.579 = -\ln\left[\ln \dfrac{T}{T-1}\right]$

or, $T = 101$ years.

For 80% confidence level

$f(c) = 1.282$

$K_T = 3.816$

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

or, $b_T = \sqrt{1 + 1.3 \times 3.816 + 1.1 \times 3.816^2} = 4.688$

probable error $(S_e) = b_T \dfrac{\sigma_n}{\sqrt{N}}$

or, $S_e = 4.688 \times \dfrac{60}{\sqrt{20}} = 62.89$

∴ For 80% confidence level

$$X_{1/2} = X_T \pm f(c)\, S_e$$

or, $X_{1/2} = 350 \pm 1.282 \times 62.89$

∴ $x_1 = 430.62\, m^3/sec$, and $x_2 = 269.37\, m^3/sec$

∴ The estimated discharge of $350\, m^3/sec$. has a 80% probability of lying between $430.62\, m^3/sec$ and $269.37\, m^3/sec$.

For 90% confidence level

$f(c) = 1.645$

$K_T = 3.816$

$b_T = 4.688$

$S_e = 62.89$

$\therefore x_{1/2} = x_T \pm f(c) \cdot S_e$

or, $x_{1/2} = 350 \pm 1.645 \times 62.89$

or, $x_1 = 453.454\ m^3/sec$

and $x_2 = 246.545\ m^3/sec$.

Hence, the discharge of $350 m^3/sec$ has a 90% probability of lying between $453.454\ m^3/s$ and $246.545\ m^3/sec$.

(2008) (2010)

9) For the annual flood series data given in the table, estimate the flood discharge for a return period of i) 50 years. ii) 100 yrs and iii) 1000 years by using log-pearson Type-III distribution.

Years	1966	1967	1968	1969	1970	1971	1972	1973	1974	1975
Discharge (m^3/s)	3300	2480	1780	1860	4130	3110	2320	2480	3405	1820

→ Here,

Years	Discharge (x)	$z = \log x$	$(z-\bar{z})$	$(z-\bar{z})^2$	$(z-\bar{z})^3$
1966	3300	3.519	0.110	0.012	0.0013
1967	2480	3.394	-0.015	0.0002	-0.000003
1968	1780	3.250	-0.159	0.0252	-0.00402
1969	1860	3.270	-0.139	0.0193	-0.00268
1970	4130	3.616	0.207	0.0428	0.0088
1971	3110	3.493	0.084	0.0070	0.00059
1972	2320	3.365	-0.044	0.0019	-0.000085
1973	2480	3.394	-0.015	0.0002	-0.000003
1974	3405	3.532	0.123	0.0151	0.0086
1975	1820	3.260	-0.149	0.0222	-0.0033
		$\Sigma z = 34.093$		$\Sigma(z-\bar{z})^2 = 0.1459$	$\Sigma(z-\bar{z})^3 = 0.00245$

$$\bar{z} = \frac{\Sigma z}{n} = \frac{34.093}{10} = 3.409$$

Standard deviation $(\sigma_z) = \sqrt{\frac{\Sigma(z-\bar{z})^2}{N-1}} = \sqrt{\frac{0.1459}{10-1}}$

or, $\sigma_z = 0.1273$

Coefficient of skew $(C_s) = \frac{n \cdot \Sigma(z-\bar{z})^3}{(n-1)(n-2)(\sigma_z)^3}$

or, $C_s = \frac{10 \times 0.00245}{(10-1)(10-2)(0.1273)^3}$

or, $C_s = 0.164$

Now,

For $C_s = 0.164$, $T = 50$ yrs, $K_z = 2.1402$

For $C_s = 0.164$, $T = 100$ yrs, $K_z = 2.4460$

For $C_s = 0.164$, $T = 1000$ yrs, $K_z = 3.3278$

Then,

$z_{50} = \bar{z} + K_z \sigma_z$

or, $z_{50} = 3.409 + 2.1402 \times 0.1273 = 3.6814$

$\therefore x_{50} = \text{antilog}(z_{50})$

or, $x_{50} = \text{antilog}(3.6814)$

or, $x_{50} = 4801.755$ m³/sec.

Again,

$z_{100} = \bar{z} + K_z \sigma_z$

or, $z_{100} = 3.409 + 2.4460 \times 0.1273 = 3.7203$

$X_{50} = \text{antilog}(z_{100}) = \text{antilog}(3.7203)$

$\therefore X_{50} = 5251.701$ m³/sec.

And,

$z_{1000} = \bar{z} + K_z \cdot \sigma_z$

or, $z_{1000} = 3.409 + 3.3278 \times 0.1273 = 3.8326$

$X_{1000} = \text{antilog}(z_{1000})$

or, $X_{1000} = \text{antilog}(3.8326)$

$\therefore X_{1000} = 6801.426$ m³/sec.

(10) Analysis of annual flood series of a river yielded a sample mean of 1000 m³/s and standard deviation of 500 m³/s. Estimate the design flood of a structure on this river to provide 90% assurance that the structure will not fail in the next 50 yrs. Use Gumbel's method and assume the same size very large.

→ Here, $\bar{x} = 1000$ m³/s and $\sigma_n = 500$ m³/s

Reliability $(R_e) = 90\%$

We know, $R_e = [1 - P]^n = \left[1 - \frac{1}{T}\right]^n$

or, $0.9 = \left[1 - \frac{1}{T}\right]^{50}$

or, $T = 475$ yrs.

Then, $Y_T = -\ln\left[\ln \frac{T}{T-1}\right] = -\ln\left[\ln \frac{475}{474}\right] = 6.16226$

$K_T = \frac{Y_T - 0.5577}{1.2825} = 4.355$

$\therefore X_T = \bar{x} + K_T \sigma_n = 1000 + 4.355 \times 500 = 3177$ m³/s