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CIVIL ENGINEERING

For

UPSC Engineering Services Examination, GATE,

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(BHEL, NTPC, NHPC, DRDO, SAIL, HAL, BSNL, BPCL, NPCL, etc.)

HYDROLOGY

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CHAPTER

1

Introduction

Hydrology is an earth science. It encompasses the occurance, distribution, movement and properties of water of earth.

Hydrological Cycle

- The hydrological cycle is a global sun-driven process whereby water is transported from the oceans to the atmosphere to the land and back to the sea.
- The hydrology cycle is usually described in terms of six major components: Precipitation (P), Infiltration (I), Evaporation (E), Transpiration (T). Surface Runoff (R), and Ground water flow (G). For computational purposes, evaporation and transpiration are sometimes lumped together as evapotranspiration (ET). Figure 1.1 define these components and illustrates the paths they define in the hydrological cycle.
- The processes constituting this cycle extend from an average depth of about 1 km in the lithosphere (the crust of the earth), to a height of about 15 km in the atmosphere. The hydrological cycle has no beginning or end.
- A convenient starting point to describe the cycle is in the oceans.
- Water in the oceans evaporate due to the heat energy provided by solar radiation. The water vapour moves upwards and forms clouds. While much of the clouds condense and fall back to the oceans as rain, part of the clouds is driven to the land areas by winds. There they condense and *precipitate* onto the land mass as rain, snow, hail, sleet, etc.

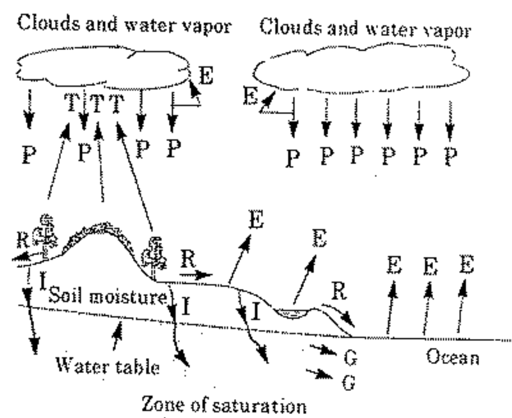


Fig. 1.1 The hydrologic cycle: (T), transpiration; (E), evaporation; (P), precipitation; (R), surface runoff; (G), groundwater flow and (I), infiltration.

- A part of the precipitation may evaporate back to the atmosphere even while falling.
- Another part may be intercepted by vegetation, structures and other such surface modifications from which it may be either evaporated back to atmosphere or move down to the ground surface.
- A portion of the water that reaches the ground, enters the earth's surface through infiltration, enhances the moisture content of the soil and reaches the groundwater body.

- Vegetation sends a portion of the water from under the ground surface back to the atmosphere through the process of transpiration.
- Some infiltrated water may emerge to surface-water bodies as interflow, while other portions may become groundwater flow.
- Groundwater may ultimately be discharged into streams or may emrege as springs.
- After an initial filling of depression storages and interception, overland flow (surface runoff) begins provided that the rate of precipitation exceeds that of infiltration.

The magnitude and duration of a precipitation event determine the relative importance of each component of the hydrological cycle during that event. During storm events, evaporation and transpiration may be of minor considerations, but during rain-free periods, Evapotranspiration becomes a dominant feature of the hydrological cycle.

Note:

- **Evaporation** is the transfer of water from a liquid state to a gaseous state, i.e., it is the conversion of liquid to the vapour phase.
- **Precipitation** is the deposition of water on the earth's surface in the form of rain, snow, hail, frost and so on.
- **Interception** is the short-term retention of rainfall by the foliage of vegetation.
- **Infiltration** is the movement of water into the soil of the earth's surface.
- **Transpiration** is the soil moisture taken up through the roots of a plant and discharged into the atmosphere through the foliage by evaporation.
- **Percolation** is the movement of water from one soil zone to a lower soil zone.
- **Storage** is the volume of water which gets stored in natural depressions of a basin.
- **Runoff** is the volume of water drained by a river at the outlet of a catchment.

Conversion of Precipitation into Stream Flow

In most of the hydrological analysis concerned with civil engineering design, we start with input as rainfall (i.e. hyetograph plot of rainfall intensity against time) and desire to obtain output as stream flow (i.e. hydrograph ppt of discharge against time).

The following chart indicates the conversion of rainfall to stream flow.

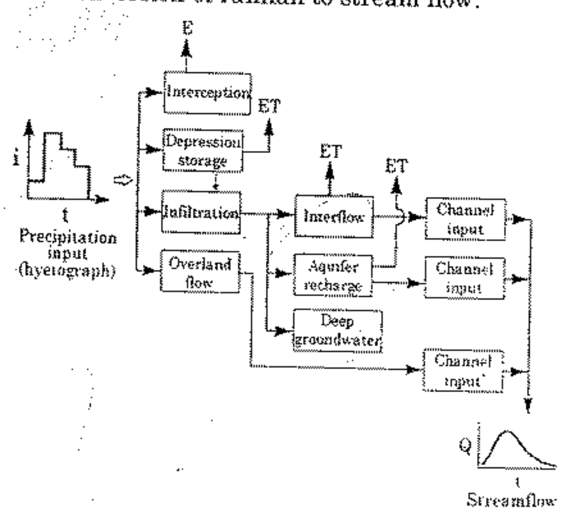


Fig. 1.2 Distribution of precipitation input.

Residence Time

Average duration of a particle of water to pass through a phase of the hydrological cycle is known as the residence time of that phase.

$$\text{Residence time} = \frac{\text{Volume of water in a phase}}{\text{Average flow rate in that phase}}$$

Average residence time of ocean is larger than that of global ground water.

Do You Know

- Runoff is min in Africa and Max in Europe/North America rainfall.
- Evaporation from ocean contributes 90% of the atmospheric moisture.
- Runoff/rainfall for India = 46% (long term estimate).
- In the ocean about 9% more water evaporates than that falls back as ppt.

Catchment Area and Watershed Divide

- A catchment area is an area of land where surface water from rain and melting snow or ice converges to a single point, usually the exit of the basin, where the waters join another waterbody, such as a river, lake, reservoir, sea, or ocean.

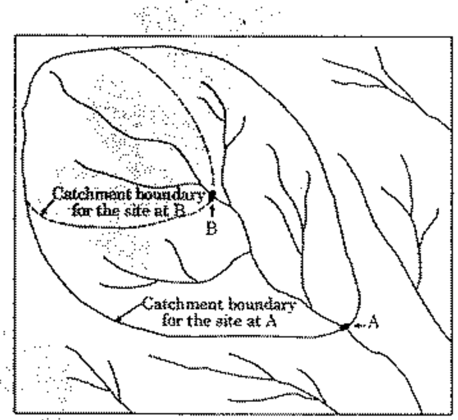


Fig. 1.3 Topographic map showing location of a divide.

- In closed catchment the water converges to a single point inside the basin, known as a Sink, which may be a permanent lake, dry lake, or a point where surface water is lost underground.
- The catchment acts as a funnel by collecting all the water within the area covered by the catchment and channelling it to a single point. Each catchment area is separated topographically from adjacent catchment area by a geographical barrier such as a ridge, hill or mountain.
- The line which divides the surface runoff between two adjacent river basins is called the *topographic water divde*, or the *watershed divide*, or simply the divide.
- The divide follows the ridge line around the catchment, crossing the stream only at the outlet point.
- It marks the highest points between the basins, but isolated peaks within a basin may reach higher elevations than any point on the divide.

Catchment Leakage

- Sometimes, owing to the underlying geology, the runoff measured at the outlet of a particular catchment may contain some contribution belonging to the precipitation fallen on a neighbouring catchment by way of subsurface runoff as shown in Figure 1.4. Thus the catchment leakage is said to occur.
- Catchment leakage also occurs when the *groundwater divide* (also known as *phreatic divide*) and the topographic divide are not coincident in plan as shown in Figure. However for large catchments, catchment leakage is neglected. i.e. ground water divide and topographical divide are assumed to match.

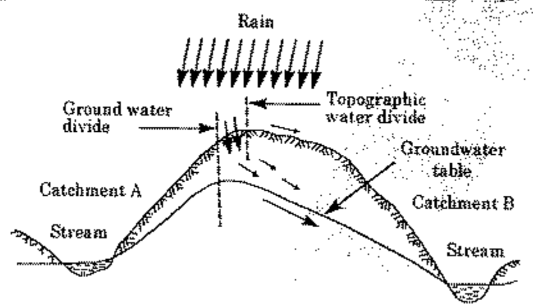


Fig. 1.4 Topographical and ground water divide.

- Other terms that are used to describe a catchment area are drainage basin, catchment, catchment basin, drainage area, river basin, water basin and watershed.

The Hydrological Budget

For a given catchment in a time interval Δt,

$$\text{Inflow} - \text{Outflow} = \text{Storage [continuity equation]}$$

This continuity equation expressed in terms of various phase of hydrological cycle is called water budget equation/hydrological budget equation.

For surface flow:

$$P + R_1 + R_g - R_2 - E_s - T_s - I = \Delta S_s \text{ (storage)} \quad \text{(i)}$$

P = ppt.
R_1 = Surface water inflow
R_g = Ground water appearing as surface water
R_2 = Surface water outflow
E_s = Evaporation
T_s = Transpiration
I = Infiltration

For underground flow:

$$I + G_1 - G_2 - R_g - E_g - T_g = \Delta S_g \quad \text{(storage)} \qquad \text{(ii)}$$

I = Infiltration
G_1 = Ground water inflow
G_2 = Ground water outflow
R_g = Ground water appearing as surface water

E_g = Evaporation
T_g = Transpiration

Combined hydrological budget (water budget eq.) is obtained by addition eq. (i) and (ii).

$$P - (R_2 - R_1) - (E_s + E_g) - (T_s + T_g) - (G_2 - G_1) = \Delta(S_s + S_g)$$

$$= \boxed{P - R - E - T - G = \Delta S} \text{ Water Budget Equation}$$

P = Precipitation
R = Net runoff
E = Net evaporation
T = Net transpiration
G = Net ground water flow
ΔS = Net storage increase

Note:

- Infilteration getting cancelled out.
- For large river basin ground water system boundary often follow surface divides in such case ($G = G_2 - G_1 = 0$).
- Over a long period of time (5 or more yr), seasonal excesses and deficit in storage tend to balance out in large catchments. Thus $\Delta S = 0$.
- Under above assumptions
  $P - R - ET = 0$ (Water Budget Equation)

Example 1

The drainage area of river Gandak is 11839 km². If the mean annual runoff is determined to be 144.4 m³/s and the average annual rainfall is 1.08 m, estimate the ET losses [evapotranspiration] for the area. How does it compare with the lake evaporation of 1m/yr measured at Muzaffarpur.

Sol: Runoff is converted from m³/s to m/year as follows:

$R = (144.4 \times 864000 \times 365)/(11839 \times 10^6) = 0.38$ m

Let $G = 0$ and $\Delta S = 0$

$\Rightarrow$ $ET = P - R = 1.08 - 0.38 = 0.7$ m

The ET losses over the catchment area are less then the measured lake ET losses. In lakes ET losses are due to E only.

Thus, ET rate is less for the vegetated drainage basin than for open water bodies.

CHAPTER

2

Hydrometeorology

Introduction

- Hydrometeorology is a special branch of hydrology that deals with the study of the atmospheric and land phases of hydrological cycles with the emphasis on the interrelationship involved between them.
- The processes of precipitation and evaporation, are the most important processes responsible for making water available on the land surface.
- Much of the water precipitated on the land surface is derived from moisture evaporated from oceans and transported long distances by atmospheric circulation.
- The two basic driving forces of atmospheric circulation result from the rotation of the earth and the transfer of heat energy between the equator and the poles.
- A general understanding of the properties of atmosphere, and of the main features of solar radiation is essential for considering the physics of evaporation and formation of precipitation.

Composition of the Atmosphere

- The atmosphere is a thick gaseous envelope which surrounds the earth from all sides and is attached to the earth's surface by gravitational force.
- It provides necessary gases for the sustenance of all life forms in the biosphere. It also filters the incoming solar radiation and thus prevents the ultraviolet solar radiation waves to reach the earth's surface and thus protects the earth from becoming too hot.
- Atmosphere is responsible for climatic and weather changes and hence gives rise to the distribution and circulation of water.
- Atmosphere is the mechanical mixture of (i) gases, (ii) water vapour and (iii) particulates.

(i) **Gases:** Nitrogen (78%) and oxygen (21%) are major gases which constitute 99% of the total gaseous composition of the atmosphere. The remaining one per cent is represented by argon (0.93%), carbon dioxide (0.03%), neon (0.0018%), helium (0.0005%), ozone (0.00006%), hydrogen (0.00005%).

Carbon dioxide is used by green plants for photosynthesis, it absorbs most of radiant energy from the earth and reradiates it back to the earth. Thus, carbon dioxide, a greenhouse gas, increases the temperature of the lower atmosphere and the earth's surface. Ozone gas absorbs most of the ultra-violet rays radiated from the sun and thus prevents the earth from becoming too hot.

(ii) **Water vapour:** The vapour content in the atmosphere ranges between zero and 5 per cent by volume.

- The atmospheric vapour is received through the evaporation of moisture and water from the water bodies (like seas and oceans, lakes, tanks and ponds, rivers etc.), vegetation and soil covers.
- Vapour depends on temperature and therefore it decreases from the equator poleward in response to decreasing temperature towards the poles.
- The content of vapour decreases upward. More than 90 per cent of the total atmospheric vapour is found upto the height of 5 km.
- The moisture content in the atmosphere creates several forms of condensation and precipitation e.g. clouds, fogs, dew, rainfall, frost, hailstorm, ice, snowfall etc.
- Vapour is almost transparent for incoming shortwave solar radiation so that the electromagnetic radiation waves reach the earth's surface without much obstacles but vapour is less transparent for outgoing long wave terrestrial radiation and therefore it helps in heating the earth's surface and lower portion of the atmosphere.

(iii) **Particulate matter:** The solid particles present in the atmosphere include dust particles, salt particles, pollen, smoke and soot, volcanic ashes etc.

- Most of the solid particles are kept in suspension in the atmosphere. These particulates help in the scattering of solar radiation.
- Salt particles become hygroscopic nuclei and thus help in the formation of water drops, clouds and various forms of condensation and precipitation.

Structure of the Atmosphere

(1) Troposphere

- The lowermost layer of the atmosphere is known as troposphere and is the most important layer because almost all of the weather phenomena (e.g. fog, cloud, dew, frost, rainfall, hailstorm, storms, cloud thunder, lightning etc.) occur in this layer.
- The average height of the troposphere is about 16 km over the equator and 6 km over the poles. Upper limit of the troposphere is called **tropopause** which is about 1.5 km thick.
- Temperature decreases upwards in troposphere at the rate of 6.5°C/km.

(2) Stratosphere

- The layer just above the troposphere is called stratosphere.
- On an average the upper limit of the stratosphere is taken to be 50 km. Mostly stratosphere is isothermal i.e. there is no change in temperature with increasing height.
- The lower portion of the stratosphere having maximum concentration of ozone is called *ozonosphere*, which is confined between the height of 15 km to 35 km from sea level.
- It acts as a protective cover for the biological communities in the biosphere because it absorbs almost all of the ultra-violet rays of solar radiation and thus protects the earth's surface from becoming too hot.
- There is gradual depletion of ozone gas in the atmosphere due to human activities.

  The main culprits of ozone destruction are halogenated gases called chlorofluorocarbons halogens and nitrogen oxides.
- Chlorofluorocarbons, released into the air are transported in the stratosphere by vertical atmospheric circulation.
- Chlorine when separated from chlorofluorocarbons reacts with water and thus depletes ozone (rather breaks ozone into O_2 and O).

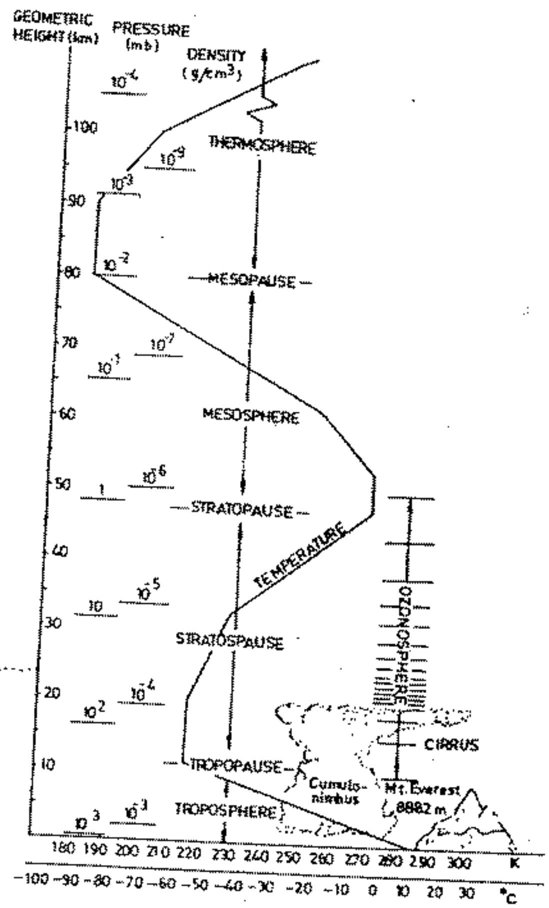
- Besides, nitrogen oxides released by supersonic jets which fly all the height of 18–22 km also depletes ozone.
- Depletion of ozone would result in the rise of temperature of the ground surface and lower atmosphere. This would cause global warming, acid rain, melting of continental glaciers and rise in sea level, skin cancer to white-skinned people, poisonous smogs, decrease in photosynthesis, ecological disaster and ecosystem instability.

(3) Mesosphere

Mesosphere extends between 50 km and 80 km. Temperature again decreases with increeasing height. At the uppermost limit of mesosphere (80 km) temperature becomes –80°C. This limit is called mesopause above which temperature increases with increasing height.

(4) Thermosphere

The part of the atmosphere beyond mesopause is known as thermosphere wherein temperature increases rapidly with increasing height. Temperature at its upper limit (height) becomes 1700°C.



Structure of the atmosphere

Insolation and Heat Budget

- Solar radiation is the most significant source of terrestrial heat energy. The radiant energy received from the sun, transmitted in a form analogous to shortwaves (wavelength equal to 1/250 to 1/6700 mm in length), and travelling at the rate of 1,86,000 miles a second, is called solar radiation or insolation.

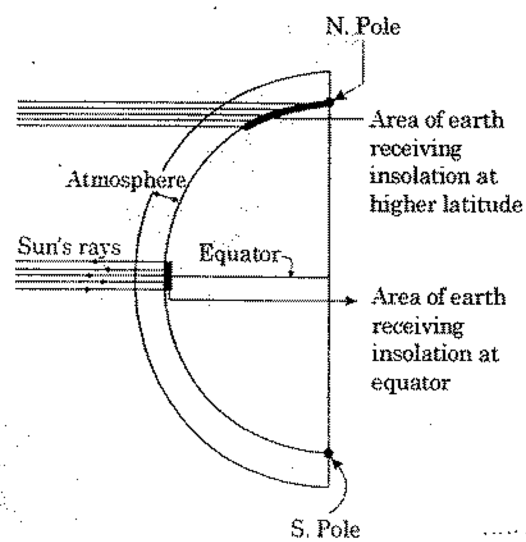
- Solar radiation heats the earth's surface and the atmosphere and thus is responsible for the movement of air and currents through changes in pressure gradients, drives the hydrological cycle through evaporation and precipitation.

Distribution of Insolation

- On an average, the amount of insolation received at the earth's surface decreases from equator foward the poles but there is temporal variation of insolation received at different latitudes at different times of the year. The insolation becomes so low at the poles that they receive about 40 per cent of the amount received at the equator.
- The amount of solar radiation reaching the outer limit of our atmosphere is significantly more at different latitudes than the amount of insolation received at the ground surface.
- This reveals the fact that a sizeable portion of incoming solar radiation is lost while passing through the atmosphere due to cloudiness, atmospheric trubidity (scattering).

Factors Affecting the Distribution of Insolation

- Distribution of insolation varies with (i) angle of the sun's rays, (ii) length of day, (iii) distance between the sun and the earth and (iv) effects of the atmosphere.



Effect of the angles of sun's rays on the distribution of insolation..

(i) Angle of the Sun's Rays

The sun's rays are more or less vertical at the equator and become more and more oblique poleward. In other words, the angle of the sun's rays decreases poleward.

- Verticle rays are spread over minimum area of the earth's surface and they heat the minimum possible area and thus the energy received per unit area increases. On the other hand, oblique rays are spread over larger area of the earth's surface and hence the amount of energy received per unit area decreases.
- Oblique rays have to pass through thicker portion of the atmosphere than the vertical rays. Consequently, the amount of solar energy lost due to reflection, scattering and absorption increases with increasing distance of travel path covered by the sun's rays through the atmosphere.

(ii) Length of Day

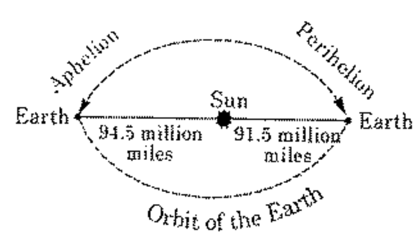
If all the other conditions are equal then longer duration of sunshine (or length of day) enable the ground surface to receive larger amount of insolation. On the other hand, shorter the duration of sunshine the lesser the amount of insolation received at the earth's surface.

- The length of day varies at all places except at the equator due to inclination of the earth's axis, its rotation and revolution.
- The length of day is always of 12 hours at the equator because the light circle always divides the equator into two equal halves.
- But the length of day increases toward north pole with northward March of the sun while it decreases in the southern hemisphere at the time of summer solstice (21 June). On the other hand, the length of day increases from the equator towards southpole in the southern hemisphere but it decreases in the northern hemisphere at the time of winter solstice (22 December) (southward march of the sun).
- The duration of day becomes of 6 months at the north pole from 21 March to 23 September while nights is of the duration of 6 months at the south pole during this period.
- The length of day becomes of 6 months at the south pole (23 September to 21 March) while the night becomes of 6 months at the north pole during this period.
- Inspite of increasing length of day from the equator towards the pole, amount of insolation received at the ground surface decreases considerably poleward because of decrease in the angle of rays of sun. Inspite of the longest length of day at the poles insolation becomes minimum because (i) the sun's rays become more or less parallel to the ground surface, and (ii) the ice cover reflects most of the solar radiation.

Note: *It is apparent that the angle of the sun's rays controls the amount of insolation received more effectively than the length of day.*

(iii) Distance between the Earth and the Sun

- The distance between the sun and the earth changes during course of a year because the earth revolves around the sun in elliptical orbit.
- At the time of perihelion on January 3 the earth is nearest to the sun and hence should receive max insolation, while at the time of aphelion on July 4 it is farthest from the sun and hence should receive min insolation.
- However, in the month of January, when the earth is nearest to the sun, there is winter season in the northern hemisphepere due to low amount of insolation received. On the other hand, in the month of July, when the earth is farthest from the sun, there is summer in the northern hemisphere due to high amount of insolation received.
- Thus it is understood that factors of the angle of the sun's rays and length of day play more dominant role in the distribution of insolation than the factor of varying distances between the earth and the sun.



Relative distance between the sun and the earth.

(iv) Effects of the Atmosphere

- Solar radiation has to pass through thick layer of the earth's atmosphere and hence it is partly absorbed, partly reflected and partly scattered by the atmosphere and partly transmitted to the earth's surface.
- **Absorption:** A part of total incoming radiation is absorbed by the atmospheric gases (e.g. oxygen and carbon dioxide to very limited extent), water vapour, haze etc.

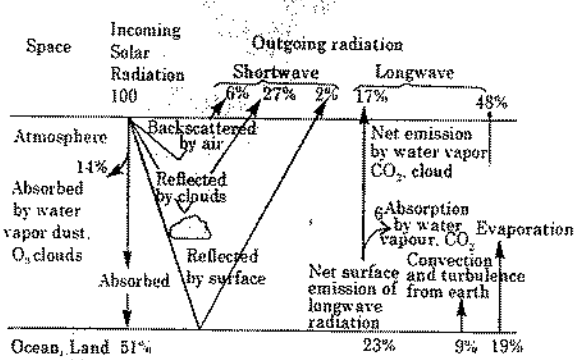
  Scattering: Some portion of the incoming solar radiation (23%) is scattered in the atmosphere by dust particles and haze. Six per cent of this scattered energy is sent back to space while 17 per cent reaches the earth's surface.

  Reflection: Reflection sends some portion of incoming solar energy back to space while some portion remains in the lower atmosphere.
- The portion of incident radiation energy reflected back from a surface is called **albedo**.

Heat Budget of the Earth and the Atmosphere

The total solar radiation reaching a horizontal surface on the ground is called global radiation. It comprises of direct shortwave radiation from the sun and the diffuse radiation scattered by the atmosphere.

- The solar energy received at the earth's surface is converted into heat energy which heats the outer surface of the earth. Thus, the earth after being heated also radiates energy in the form of longwave radiation.
- On an average, there exists heat balance between the amount of solar radiation received by the earth's surface and its atmosphere and the amount of heat lost by the outgoing terrestrial longwave radiation from the earth's surface and loss of heat from the atmosphere.



Radiation and heat balance in the atmosphere and at the earth's surface.

Note: *A total of 14% of heat is received by atmosphere from direct sunlight but a total of 34% (6 + 9 + 19) of heat is received by atmosphere from earth.*

Net Radiation

Net radiation received at the earth's surface is the difference between the radiation absorbed and that emitted back. It is this energy which is available at any point on the surface of earth for heating the ground and the lower layers of the atmosphere; and, it is a major energy input for the evaporation of water.

Net Radiation and Latitudinal Heat Balance

The net radiation from the whole globe is theoretically zero but if we look at the regional distribution of insolation, there are some locations where the receipt of solar energy is more than the energy lost because the solar energy comes at faster rate than the terrestrial energy goes out.

Similarly, in some locations the loss of energy through outgoing terrestrial radiation is faster than the gain of incoming solar radiation.

This mechanism results in the development of areas of energy **surplus** and **energy deficit**.

The energy surplus and energy deficit area may be identified and studied at two levels viz. (i) at the earth's surface, and (ii) in the atmosphere, latitudinal base being common in both the cases. The distribution of net radiation at the earth's surface from equator towards the poles shows that (a) there is large **energy surplus area** between the zones of 20°N and 20°S (b) net radiation rapidly decreases from the energy surplus areas of low latitudes towards midlatitude; (c) net radiation becomes practically zero near 70° latitude in both hemispheres, and (d) the polar areas are the zones of perennial **energy deficit**.

There exist a two-way heat transfer; from the earth's surface to the atmosphere and from the equator to the poles. The transport of heat from equatorial area towards the poles is called **'meridional transport of heat'**.

The meridional transport of heat energy is accomplished by the atmospheric circulation and ocean currents. The vertical transport of heat in the atmosphere is accomplished by ascending air in the form of sensible heat and latent heat.

Heating and Cooling of Atmosphere

The atmosphere receives very low amount of heat energy (14%) from the sun as it receives most of its energy from the longwave terrestrial radiation (34%). The heating and cooling of the atmosphere is accomplished through direct solar radiation and through transfer of energy from the earth through the processes of conduction, convection and radiation.

1. Heating of the Atmosphere by Direct Insolation

The atmosphere absorbs 14 per cent of incoming short wave solar radiation through ozone, water vapour etc. Seven per cent of this energy is spread in the lower atmosphere up to the height of 2 km. This amount is too low to heat the atmosphere significantly.

2. Conduction

- The transfer of heat through the molecules of matter in any body is called conduction. The heat moves from warmer body to the cooler body through molecular movement. The rate of transfer of heat through molecular movement depends on the heat conductivity of the substance.
- The earth's surface is heated during day-time after receiving solar radiation. The air coming in contact with the warmer ground surface is also heated because of transfer of heat (conduction of heat) from the ground surface through the molecules of the air. Since air is very poor conductor of heat hence the transfer of heat from the ground surface through conduction is effective only upto a few metres in the lower atmosphere.

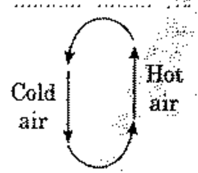
3. Terrestrial Radiation

- The process of transfer of heat from one body to the other body without the aid of a material medium (e.g. solid, liquid or gas) is called radiation.
- The earth's surface after receiving insolation from the sun through shortwave electromagnetic radiation gets heated and radiates heat to the atmosphere in the form of longwave of infrared radiation throughout 24 hours. The atmosphere is more or less transparent for incoming shortwave solar radiation but it absorbs large per cent of outgoing longwave terrestrial radiation through water vapour, carbon dixodie, ozone etc.
- Thus, the terrestrial radiation is the most important source of heating of the atmosphere.
- A part of this ground radiation after being absorbed by the atmosphere is radiated back to the earth's surface. This process of radiation of terrestrial heat energy from the atmosphere back to the earth's surface

is called **counter-radiation** which is affected mainly by water vapour and atmospheric carbon dioxide (green house effect).

4. Convection

- The transfer of heat energy through the movement of a mass of substance from one place to another place is called convection. The process of convection becomes effective only in fluids or gases because their internal mass motion activates convection of heat energy.
- The earth's surface gets heated due to insolation. Consequently, the air coming in contact with the warmer earth's surface also gets heated and expands in volume.
- Thus, warmer air becomes lighter and rises upward and a vertical circulation of air is set in. Conversely, the relatively colder air aloft becomes heavier and thus descends to reach the earth's surface. The descending air is warmed because of dry adiabatic rate and warm ground surface.



- This warm air again ascends because of increase in volume and decrease in density.
- The whole mechanism of ascent of warmer air and descent of colder air generates convection currents in the lower atmosphere.
- This convective mechanism transport heat from the ground surface to the atmosphere and thus helps in the heating of the lower atmosphere.
- Similarly, horizontal convection currents are also generated on the ground surface.
- Besides, atmosphere is also heated through latent heat of condensation and expansion and compression of air.

Temperature

- The earth's surface receives maximum energy at 12 noon but the maximum temperature never occurs at 12 noon because the transformation of solar energy into heat requires time. The energy received by the earth from solar radiation continues to exceed the energy lost by outoing longwave radiation from the earth's surface from 6 A.M. to 2-4 P.M. Thus, the curve of air temperature also rises upto 4 P.M. This is why maximum temperature is recorded between 2 P.M.-4 P.M.
- On the other hand, the loss of energy through longwave radiation from the earth's surface exceeds the energy received from the sun from 4 P.M. to the sunrise and hence there is gradual fall of the curve of temperature but the lowest (minimum) temperature is recorded between 4-5 A.M. instead of midnight.
- The lowest temperature within 24 hours is called **minimum daily temperature**. Thus, there is no coincidence between the time of maximum and minimum amount of insolation received from the sun and maximum and minimum amount of insolation received from the sun and maximum and minimum temperatures of the air. This is called **lag of temperature**.

Distribution of Temperature

The spatial and temporal distribution of temperatures is very significant because different types of weather, climates, vegetation zones, animals and human life etc. basically depend on the distribution of temperature, whether horizontal or vertical.

Distribution of temperature is controlled by:

1. Latitudes

The temperature of the atmosphere of a particular place near the ground surface depends on the amount of insolation received at that place. Since the amount of insolation received by the ground surface decreases poleward from the equator i.e. from low latitudes towards high latitudes because the sun's rays become more and more oblique (slanting) poleward and hence air temperature also decreases poleward. It may be noted that though sun's rays are almost vertical over the equator throughout the year but there is no maximum temperature on it. Rather, maximum temperature is recorded along 20°N latitude in July because major portion of insolation is reflected by clouds and sizeable amount of heat is lost in evaporation in the low latitude zone (equatorial zone).

2. Altitude

- The temperature decreases with increasing height from the earth's surface at an average rate of 6.5°C per kilometre because of the following reasons.

  (i) The major source of atmospheric heat is the earth's surface from where heat is transferred to the atmosphere through the processes of conduction, radiation and convection. Thus, the portion of the atmosphere coming in direct contact with the earth's surface gets more heat from the ground surface than the portion lying above because as we ascend higher in the atmosphere the amount of heat to be transported above decreases and hence temperature decreases aloft.

  (ii) The layers of air are denser near the earth's surface and become lighter with increasing altitudes. The lower layer of air contains more water vapour and dust particles than the layers above and hence it absorbs larger amount of heat radiated from the earth's surface than the upper air layers.

3. Distance from the Coast

The marine environment moderates the weather conditions of the coastal areas because there is more mixing of temperatures of the coastal seas and oceans and coastal lands due to daily rhythm of land and sea breezes. Thus, the daily range of temperature near the coastal environment is minimum but it increases as the distance from the sea coast increases.

4. Natnre of Land and Water

Land becomes warm and cold more quickly than the water body. This is why even after receiving equal amount of insolation the temperature of land becomes more than the temperature of the water body because:

(i) The sun's heat penetrates to a depth of only 3 feet in land because it is opaque but they penetrate to a greater depth of several metres in water because it is transparent to solar radiation. The thin layer or soils and rocks of land gets heated quickly because of greater concentration of insolation in much smaller mass of material of ground surface. Similarly, the thin ground layer emits heat quickly and becomes colder. On the other hand, the same amount of insolation falling on water surface has to heat larger volume of water because of the penetration of solar rays to greater depth and thus the temperature of ground surface becomes higher than that of the water surface though the amount of insolation received by both the surfaces may be equal.

(ii) There is more evaporation from the seas and the oceans and hence more heat is spent in this process with the result oceans get less insolation than the land surface. On the other hand, there is less evaporation from the land surface because of very limited amount of water.

(iii) The **specific heat** (the amount of heat needed to raise the temperature of one gram of a substance by 1°C) of water is much greater than the land because the relative density of water is much lower than that of land surface. This means that more heat is required to raise the temperature of water than of land.

(iv) The reflection (albedo) of incoming solar radiation from the oceanic water surface is far more than from the land surface and thus water receives less insolation than land.

6. Nature of Ground Slope

The ground slope facing the sun receives more insolation because the sun's rays reach the surface more or less straight and hence sun-facing ground surfaces record higher temperature than the leeward slopes where sun's rays reach more obliquely. In the northern hemisphere the southward facing slopes of East-West stretching mountains receive greater amount of insolation than the north ward facing slopes because of their exposure to the sun for longer duration. This is why most of the valleys situated on the southern slopes of the Alpine mountains have settlements and cultivation.

7. Prevailing Winds

The prevailing winds help is redistribution of temperature and in carrying moderating effects of the oceans to the adjacent coastal land areas. The winds blowing from low latitudes to high latitudes raise the temperature of the regions while winds blowing from high latitudes to low latitudes lower the temperature. The winds blowing from oceans to coastal lands bring in marine effects and thus lower the daily range of temperature. The winds coming from higher parts of the mountains lower the temperature in the valleys. The temperature rises at the time of arrival of temperate cyclones while it falls sharply after their passage.

Inversion of Temperature

Meaning of Temperature Inversion

- Temperature decreases with increasing altitudes in the troposphere at an average rate of 6.5°C per 1000 metres (normal lapse rate) but some times this normal trend of decrease of temperature with increasing heights is reversed under special circumstances i.e. temperature increases upward upto a few kilometres from the earth's surface. This is called **negative lapse rate.**
- Thus, warm air layer lies over cold air layer. This phenomenon meteorologically is called inversion of temperature.
- Such situation may occur near the earth's surface or at greater height in the troposphere but the inversion of temperature near the earth's surface is of very short duration because the radiation of heat from the earth's surface during daytime warms up the cold air layer which soon disappears and temperature inversion also disappears. On the other hand, upper air temperature inversion lasts for longer duration because the warming of cold air layer aloft through terrestrial radiation takes relatively longer period of hours.

Types of Temperature Inversion

Temperature inversion is classified into the following types on the basis of relative heights from the earth's surface at which it occurs and the type of air circulation.

(i) Non-advectional inversion

(ii) Advectional inversion

(iii) Mechanical inversion

Non-Advectional Inversion

(1) Ground surface inversion also called as radiation inversion occurs near the earth's surface. This inversion is also called non-advectional inversion because it occurs in static atmospheric condition characterized by no movement of air whether horizontal or vertical. Such inversion normally occurs during the long cold winter nights in the snow-covered regions of the middle and high latitudes. Inversion is caused due to excessive nocturnal cooling of the ground surface due to rapid rate of loss of heat from the ground through outgoing longwave terrestrial radiation. Thus, the air coming in contact with the cool ground surface also

becomes cold while the air layer lying above is relatively warm. Consequently, temperature inversion develops because of cold air layer below and warm air layer above.

- This inversion occurs in the low latitude areas (tropical and subtropical areas) during winter nights only and the inversion generally disappears with sunrise but some times it persists upto noon. The inversion occurs upto the height of 30-40 feet in the low latitudes, a few hundred feet in the middle latitudes and half a mile in the high latitudes. it is apparent that the duration and height of surface inversion increase poleward.
- This inversion promotes stability in the lower portion of the atmosphere and causes dense fogs.
- The **mechanical inversion** of temperature is caused at higher heights in the atmosphere due to subsidence of air and turbulence and convective mechanism. Such inversion occurs in a number of ways e.g. (a) Some times, warm air is suddenly transported upward (due to eddies formed by frictional forces) to the zone of cold air and thus cold air being denser lies under the warm air and inversion of temperature is caused. (b) The descending air is warmed at the dry adiabatic rate of 10°C per 1000 m because of compression. This situation causes the existence of warm air above the cold air. Such mechanical inversion caused by the subsidence of air currents is generally associated with the anticylonic conditions.
- Such inversion of temperature is of very common occurrence in the middle latitudes where high pressures are characterized by sinking air.

(3) **Advectional inversion of temperature** is also called as dynamic inversion because it is always caused due to either horizontal or vertical movements of air. Strong wind movement and unstable conditions of the atmosphere are prequisite conditions for advectional inversion of temperature. This type is further divided into 3 subtypes on the basis of the nature of air movements e.g. (i) frontal or cyclonic inversion, (ii) valley inversion due to vertical movement of air, and (iii) surface inversion due to horizontal movement of air.

(i) **Frontal or cyclonic inversion** is caused in the temperate zones due to temperate cyclones which are formed due to the convergence of warm westerlies and cold polar winds in the northern hemisphere. The warm air is pushed up by the cold polar air and thus the warm air overlies the cold air because it is lighter than the cold air. Thus, the existence of warm air above and cold air below reverses the normal lapse rate and inversion of temperature occurs.

(ii) **Valley inversion** generally occurs in the mountainous valleys due to radiation and vertical movement of air. This is also called vertical advectional inversion of temperature. The temperature of the upper parts of the valleys in mountainous areas becomes exceedingly low during winter nights because of rapid rate of loss of heat from the surface through terrestrial radiation. Consequently, the air coming in contact with the cool surface also becomes cool. On the other hand, the temperature of the valley floor does not fall considerably because of comparatively low rate of loss of heat through terrestrial radiation. Thus, the air remains warmer than the air aloft and hence the warm and light air of the valley floor is pushed upward by the descending cold and heavier air of the upper part of the valley. Thus, there is warm air aloft and cold air in the valley floor and inversion of temperature is caused.

- This situation is responsible for severe frost in the valley floors causing great damage to fruit orchards and vegetables and agricultural crops whereas the upper parts of the valleys are free from frost. This is why the valley floors are avoided for human settlements while the upper parts are inhabited in the mountainous valleys of middle latitudes.

Significance of Inversion of Temperature

Though inversion of temperature denotes local and temporary conditions of the atmosphere but there are several climatic effects of inversion which are of great significance to man and his economic activities.

- Fog is formed due to the situation of warm air above and cold air below because the warm air is cooled from below and resultant condensation causes the formation of tiny droplets around suspended dust particles and smokes during winter nights. The smokes coming out of houses and chimneys intensify fogs and become responsible for the occurrence of **urban smogs**.
- When smog is mixed with air pollutants such sulphur dioxide it becomes poisonous and deadly health hazard to human beings.
- Though generally fogs are unfaourable for many agricultural crops such as grams, peas, mustard plants, wheat etc. but some times they are also favourable for some crops such as coffee plants in Yemen hills of Arabia where fogs protect coffee plants from direct strong sun's rays.

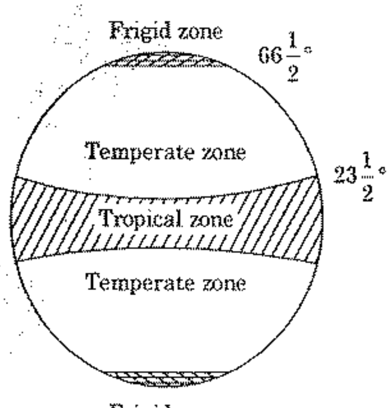
(2) Inversion of temperature causes frost when the condensation of warm air due to its cooling by cold air below occurs at temperature below freezing point.

(3) Inversion of temperature causes atmospheric stability which stops upward (ascent) and downward (descent) movements of air.

- The atmospheric stability discourages rainfall and favours dry condition.
- The inversion of temperature caused by the subsidence of air resulting into anticyclonic conditions increases aridity.
- This is why the western parts of the continents situated between 20°-30° latitudes and characterized by anticyclonic condition represent most widespread tropical deserts of the world.

Regional Distribution of Temperature

The globe is divided into three temperature zones on the basis of latitudes (1) tropical zone, (2) temperate zone, and (3) frigid zone.



Temperature zones

- **Tropical zone** extends between the tropics of Cancer (23.5°N) and Capricorn (23.5°S). The Sun's rays are more or less vertical on the equator through out the year. The remaining areas are also characterized by vertical sun's rays at least once every year.
- There is no winter around the equator because of high temperature prevailing throughout the year but as one approaches the tropics of Cancer and Capricorn summer and winter are clearly observed and differentiated.

(2) **Temperate zone** extends between 23.50 and 66.5° latitudes in both the hemispheres. Though the duration of day and night is longer in this zone but it is never more than 24 hours. There are marked seasonal contrasts with the northward and southward (summer and winter solstices) migration of the overhead sun and thus the range of temperature between summers and winters becomes exceptionally very high.

(3) **Frigid zone** extending between 66.5° latitude and the poles in both the hemispheres is characterized by more oblique sun's rays throughout the year resulting into exceptionally very low temperature. The length of day and night is more than 24 hours. Days and nights are of 6 months duration at the poles. Sun is never vertical and the ground is covered with snow as temperature more or less remains below freezing point.

Air Pressure and Atmospheric Circulation

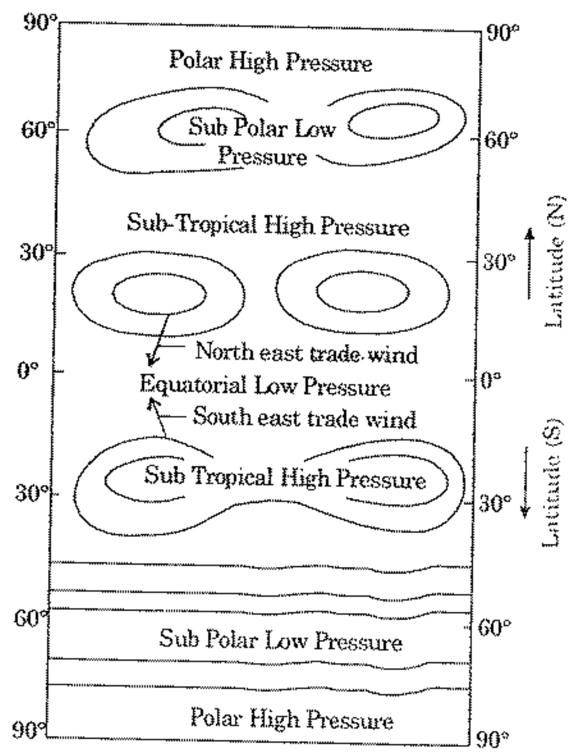
- The air exerts pressure through its weight.
- It is maximum at sea level.
- Air pressure is measured with the help of mercurial barometer (Fortin's barometer), aneroid barometer, altimeter (altitude barometer), barograph, microbarograph etc.
- The lines joining the places of equal pressure at sea level are called **isobars**. Air pressure decreases with increasing altitudes at the rate of 3.4 mb per 600 feet but this rate of decrease is confined to the altitude of a few thousand feet only.
- Normally, half of the total atmospheric pressure is confined to the altitude of 1800 feet.

Horizontal Distribution of Air Pressure and Pressure Belts

- The horizontal distribution of air pressure on the globe is studied on the basis of isobars. Air pressure is generaly divided in two types viz. (1) high pressure, also called as '**high**' or anticyclone, and (2) low pressure, also called as '**low**' or **cyclone or depression.**
- There is no definite trends of distribution of pressure from equator towards the pole.

If the air pressure would have been the function of air temperature alone there should have been regular increase of pressure poleward because temperature regularly decreases from the equator towards the poles but this is not the case.

- There is low pressure near the equator due to high mean annual temperature but the existence of high pressure belts near the tropics of Cancer and Capricorn cannot be explained on the basis of temperature because the tropics record very high temperature and hence there should have been low pressure if the temperature would have been the only controlling factor of air pressure.
- Pressure belts are not only induced by thermal factor but they are also induced by dynamic factors.
- On the basis of mode of genesis pressure belts are divided into two broad categories e.g. (1) thermally induced pressure belts (e.g. equatorial low pressure belt and polar high pressure belt), and (2) dynamically induced pressure belts (e.g. subtropical high pressure belt and subpolar low pressure.



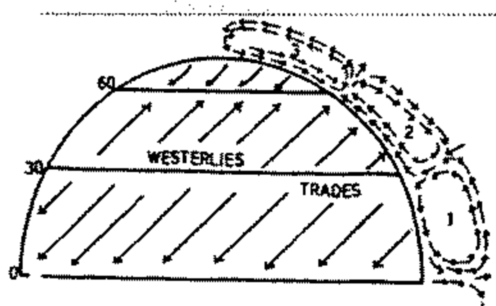
Generalized distribution of air pressure

(1) Equatorial Low Pressure Belt

- The equatorial low pressure belt is located on either side of the geographical equator in a zone extending between 5°N and 5°S latitudes but his zones is not stationary because there is seasonal shift of this belt with the northward (summer solstice) and southward (winter solstice) migration of the sun.
- During northern summer this belt extends upto 20°N in Africa and to the north of tropic of Cancer in Asia while during southern summer this low pressure belt shifts to 10° to 20S latitude. This equatorial low pressure belt is thermally induced because the ground surface is intensely heated during the day due to almost vertical sun's rays and thus the lowermost layers of air coming in contact with the heated ground surface also gets warmed. Thus, warmed air expands, becomes light, and consequently rises upward causing low pressure.
- The equatorial low pressure belt represents the zone of convergence of north-east and south-east trade winds. There are light, feeble and variable winds within this convergence belt. Because of frequent calm conditions this belt is called a **belt of calm** or **doldrum**.

(2) Sub-Tropical High Pressure Belt

- Sub-tropical high pressure belt extends between the latitudes of 25°-35° in both the hemispheres. This high pressure belt is not thermally induced because this zone, besides two to three winter months, receives fairly high temperature throughout the year.
- Thus, this belt owes its origin to the rotation of the earth and sinking and settling down of winds.
- It is, thus, apparent that the sub-tropical high pressure belt is dynamically induced.



Tricellular meridional circulation of the atmosphere (1) tropical Hadley cell, (2) Midlatitude cell, and (3) Polar cell.

- The convergence of winds at higher altitude above this zone results in the subsidence of air from higher altitudes. Thus, descent of winds results in the contraction of their volume and ultimately causes causes high pressure.
- This is why this zone is characterized by anticyclonic conditions which cause atmospheric stability and aridity.
- This is one of the reasons for the presence of hot deserts of the world in the western parts of the continents in a zone extending between 25°-35° in both the hemispheres. This zone of high pressure is called '**horse latitude**' because of prevalence of frequent calms.

Note: *In ancient times, the merchants carrying horses in their ships, had to throw out some of the horses while passing through this zone of calm in order to lighten their ships. This is why this zone is called horse latitude.*

(3) Sub-Polar Low Pressure Belt

- This belt of sub-polar low pressure is located between 60°-65° latitudes in both the hemispheres. The low pressure belt does not appear to be thermally induced because there is low temperature throughout the year and as such there should have been high pressure belt instead of low pressure belt.

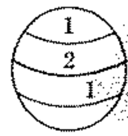
- It is, thus, obvious that this low pressure belt is dynamically produced.
- In fact, the surface air spreads outward from this zone due to rotation of the earth and low pressure is caused.

Pressure Gradient and Air Circulation

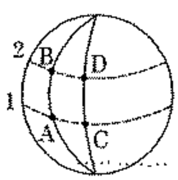
The difference of pressure between any two places is called pressure gradient. The air moves from high pressure to low pressure.

Wind Direction and Related Laws

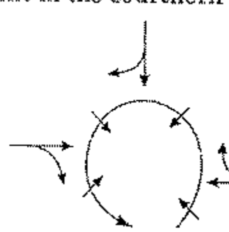
- The direction of surface winds is usually controlled by the pressure gradient and rotation of the earth. Because of rotation of the earth along its axis the winds are deflected. The force which deflects the direction of winds is called **deflection force**. This force is also called **coriolis force**. Becauseof coriolis force all the winds are deflected to the right in the northern hemisphere while they are deflected to the left in the southern hemisphere with respect to the rotating earth.



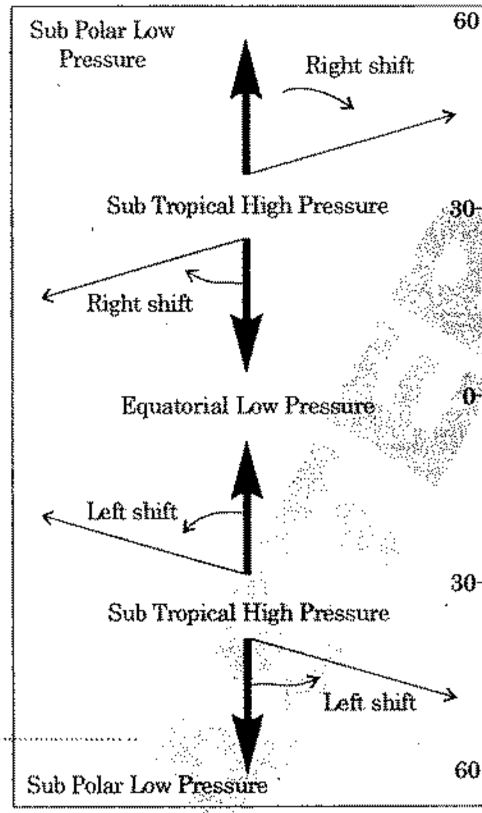
- Equatorial latitudinal circle is the largest one and the latitudinal circles decrease poleward wherein polar circle is the smallest one.
- The whole earth completes one rotation along its axis roughly in 24 hours. Thus, the linear speed of the earth is highest at the equator and decreases poleward.
- If earth did not rotate, a wind blowing from A towards B, will reach B. However due to rotation of earth. While wind is starting from A toward B, it has a velocity towards C also. This velocity is more than velocity of any point in latitude 2.



- Thus wind starting at A towards B will go a head of B when it reaches latitude 2.
- Hence wind is thought to be deflected towards right in northern hemisphere.
- The same logic applies to anypoint in the sourthern hemisphere where winds are deflected to the left.



- This is why winds blow counterclockwise around the centre of low pressure (to make cyclonic circulation) in the northern hemisphere while they blow clockwise in the southern hemisphere.



Deflective force and wind direction

Classification of Winds

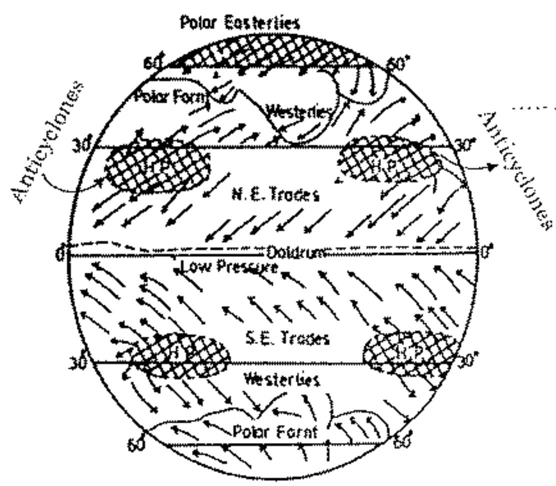
- The winds blowing almost in the same direction throughout the year are called prevailing or **permanent winds**. These are also called as **invariable** or **planetary winds** because they involve larger areas of the globe.
- On the other hand, winds with seasonal changes in their directions are called **seasonal winds**.

Permanent or Planetary Winds

- On an averages, the location of high and low pressure belts is considered to be stationary on the globe. Consequently, winds blow from high pressure belts to low pressure belts. The direction of such winds remains more or less the same throughout the year. Thus, such winds are called permanent winds. These winds include trade winds westerlies and polar winds.

(1) Winds in the Tropics

- Generally, the areas extending between 30°N and 30°S latitudes are included in tropical zone.



The generalized global pattern of planetary winds

- Trade winds blow from the subtropical high pressure belts to the equatorial low pressure belt. The north-east and south-east trades converge along the equator and there are upper air anti-trades blowing in the opposite directions of the surface trade winds. The weather conditions throughout the tropical zone remain more or less uniform. There is a belt of calm or doldrum characterized by feeble air circulation.

(i) **Doldrum and equatorial westerlies:** A belt of low pressure, popuarly known as equatorial trough of low pressure, extends along the equator within a zone of 5°N and 5°S latitudes. This belt is called the **belt of calm** or **doldrum** because of light and variable winds.

- This belt is subjected to seasonal and spatial variations due to northward and southward movement of the overhead sun (summer and winter solstices).
- In fact, the belt of doldrum shifts northward during summer solstice (when the sun is vertical).

  On an average, there is westerly air circulation (form west to east) in the doldrums or say in the intertroppical convergence.

  Note: *Intertropical convengence (ITC) represent the meeting ground of north-east and south-east trade winds. The northern and southern boudnaries of intertropical convergence are called north intertropical convergence (NITC) and south intertropical convergence (SITC) respectively. There is seasonal shifting in the NITC and SITC with the northward (summer solstice) and southward migration (winter solstice) of the sun.*

  These westerly winds have been called as **equatorial westerlies**.

- The equatorial westerlies are associated with strong atmospheric disturbances (cyclonic storms). South-western monsoons which causes major rainfall in India are, infact, equatorial westerlies because these winds are extended upto 30-35°N latitudes over Indian subcontinent due to northward shifting of NITC at the time of summer solstice.

(ii) **Trade winds:** there is more or less regular inflow of winds from subtropical high pressure belts to equatorial low pressure belt. These tropical winds have north-easterly direction in the northern hemisphere while they are south-easterly in the southern hemisphere. These winds are called trade winds because of the fact that they helped the sea merchants in sailing their ships as their (of trade winds) direction remains more or less constant and regular.

- Trade winds are deflected to the right in the northern hemisphere and to the left in the southern hemisphere.
- The poleward parts of the trade winds or eastern sides of the subtropical anticyclones are dry because of strong subsidence of air currents from above.
- Because of the dominance of anticyclonic conditions there is strong atmospheric stability, strong inversion of temperature and clear sky. On the other hand, the equatorward parts of the trade winds are humid because they are characterized by atmospheric instability and much precipitation as the trade winds while blowing over the oceans pick up moisture.

(2) Horse Latitudes and Westerlies

(i) **Horse latitudes:** The dynamically induced (due to subsidence of air currents) subtropical high pressure belt extends between 30°-35° (25°-35°) latitudes in both the hemispheres.

- Thus, this belt separates two wind systems viz. trade winds and westerlies.
- Subtropical high pressure belt is the source for the origin of trade winds (blowing towards equatorial low pressure belt) and westerlies (blowing towards subpolar low pressure belt) because winds always flow from high pressure to low pressure.

- The air after being heated near the equator ascends and after blowing in opposite direction to the surface trade winds descends in the latitudinal zone of 30°-35°.
- Thus, the descent of winds from above causes high pressure on the surface which in turn causes anticyclonic conditions. This is why the anticyclonic conditions cause atmospheric stability, dry condition and very weak air circulation.
- This zone (30°-35°) is characterized by weak and variable winds and calm.

  Note: *This belt of calm is very popularly known as horse latitudes because of the fact that in ancient times the merchants had to throw away some of the horses being carried in the ships inorder to lessen the weight so that the ships could be sailed through the calm conditions of these latitudes.*

- Anticyclones are produced due to subsidence of air current in the horse latitudes. these anticyclones are known as '**subtropical highs**' or subtropical anticyclones, the eastern and western parts of which are characterized by contrasting weather conditions. The eastern parts (spreading over the western parts of the continents) is marked by descent of air currents, inversion of temperature and consequent atmospheric stability and dry conditions. This is why hot and dry tropical deserts are found in the western parts of the continents within the latitudinal zones of 20°-30°in both the hemisphere (e.g. Sahara and Kalahari in Africa, Chile-Peru desert or Acataman in South America, Arabian and Thar deserts in Asia, deserts of S.W. USA, and Australian deserts).
- The western parts of subtropical anticyclones (covering the eastern parts of the continents and western parts of the oceans) are humid because some sort of atmospheric instability is caused due to weakening of air descept (e.g. in the areas of Caribbean Sea, Mexican Gulf and adjoining areas, eastern China, southern Japan, south-east Brazil and eastern Australia).

(ii) Westerlies: The permanent winds blowing from the subtropical high pressure belts (30°-35°) to the subpolar low pressure belts (60°-65°) in both the hemispheres are called westerlies. The general direction of the westerlies is S.W. to N.E. in the northern hemisphere and N.W. to S.E. in the southern hemisphere.

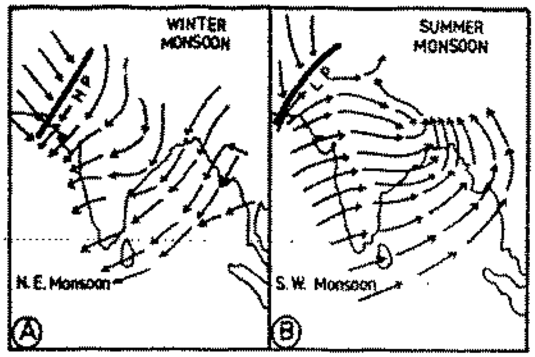
- There is much variation in the weather conditions in their poleward parts where there is convergence of cold and denser polar winds and warms and lighter westerlies. In fact, a cyclonic front, called as **polar front**, is formed due to two contrasting air masses as referred to above and thus temperate cyclones are originated.
- A low pressure belt, produced due to dynamic factor, lies within the latitudinal belt of 60°-65° in both the hemispheres.
- There is very high pressure over the poles because of **exceedingly low temperature**. **Thus**, winds blow from polar high pressure to subpolar low pressure cells. These are called polar winds which are north-easterly in the Northern hemisphere and south-easterly in the southern hemisphere.

Seasonal Shifting of Wind Belts and Their Climatic Significance (Monsoon)

Monsoon climate is the result of the shifting of pressure and wind belts.

- Due to northward migration of the sun in the northern hemisphere at the time of summer solstice the north intertropical convergence (NITC) is extended upto 30°N latitude over Indian subcontinent, south-east Asia and parts of Africa.
- Thus, the equatorial westerlies are also extended over the aforesaid regions. These equarotial west earlies are also extended over the aforesaid regions. These equatorial westerlies, in fact, become the south-west or summer monsoons. These south-west monoon winds bring much rains because they come from over the ocean and are associated with tropical atmospheric storms (cyclones).

- The NITC is withdrawn from over the Indian subcontinent and south-east Asia because of southward shifting of pressure and wind belts due to southward migration of the sun at the time of winter solstice.
- Thus, north-east trades are re-eastablished over the aforesaid areas. These north-east trades, in fact, are north-east or winter monsoons. Since they come from over the lands, and hence they are dry.



A-winter monsson, B-summer monsoon

Humidity and Precipitation

Water Vapour and Evaporation

- Humidity of the air refers to the content of water vapour present in the air at a particular time and place.
- The presence of water vapour in the atmosphere is a vital factor for weather conditions of a particular region. The nature and amount of precipitation, the amount of loss of heat through radiation from the earth's surface, surface temperature, latent heat of the atmosphere, stability and instability of air masses etc. depend on the amount of water vapour present in the atmosphere.
- The atmospheric water vapour is derived through evaporation of water from oceans and seas, terrestrial lakes, land water bodies (tanks, ponds), rivers etc.
- The process of transformation of liquid (water) into gaseous form is called **evaporation**. The amount and intensity of evaporation depend on aridity, temperature and velocity of winds.
- The higher the aridity, temperature and velocity of winds, the higher the rate and amount of evaporation because dry air with high temperature is capable of retaining more moisture (vapour) as dry air requires more time and moisture to become saturated.
- A stable air becomes saturated soon because there is no transfer of moisture while unstable air attains saturation quite late because there is much transfer of moisture.
- There is more evaporation from the oceans than from the lands, there is maximum evaporation from the lands between 10°N and 10°S latitudes whereas maximum evaporation occurs from the loceans between 10°-20° latitudes in both the hemispheres.
- The process of conversion of vapour into liquid (water) and solid form (ice, snow, frost) is called **condensation**. It is apparent that evaporation and condensation are opposite processes wherein the former involves conversion of liquid (water) into gaseous form (water vapour) while the latter refers to conversion of water vapour into liquid or solid form.
- Energy in the form of heat is required for the conversion of water into gaseous form (water vapour).
- The amount is never lost rather it is always associated with water vapour.

- Amount of energy required to convert water to water vapour is called latent heat of evaporation.
- Heat energy is released at the time of condensation (conversion of vapour into liquid or solid form). This energy, released after condensation, is called **latent heat of condensation**.

The amount of water vapour present in the air is indirectly expressed through what is known as the *vapour pressure*. The partial pressure exerted by the water vapour is called the vapour pressure. Consider certain amount of moist air in a closed container, it will be exerting a total pressure p on the container. Now, if all the water vapour is removed from the container without affecting the temperature, the pressure exerted by the dry air p' will be less than the original pressure exerted by the moist air, and the quantity $(p - p')$ is obviously the pressure contribution due to vapour and is therefore the vapour pressure. The more the vapour present in the air, the higher is the vapour pressure. Vapour pressure is usually denoted by e, and is expressed either in millibars or in mm of mercury.

HUMIDITY

- Humidity refers to the content of water vapour present in tbe air in gaseous form at a particular time and place.
- The atmospheric humidity is obtained through various processes of evaporation from the land and water surfaces of the earth. The atmospheric humidity of vital climatic significance because different forms of precipitation (dew, fog, rainfall, frost, snowfall, hailstorm etc.), atmospheric storms (cyclones) and turbulence etc. depend on humidity.
- The atmospheric humidity is expressed in a number of ways e.g. absolute humidity, specific humidity, relative humidity etc.

(1) Humidity Capacity

- The moisture retaining capacity or humidity capacity refers to the capacity of an air of certain volume at certain temperature to retain maximum amount of moisture content.
- The moisture content (humidity) of the air is measured in gram per cubic centimetre.
- Evaporation is the main mechanism through which water is converted into humidity (vapour). Since temperature and evaporation are directly positively related (evaporation increases with increasing temperature) and hence humidity and temperature are also directly positively related.
- Humidity capacity is directly positively related with temperature, higher the temperature, higher the humidity capacity and lower the temperature, lesser the humidity capacity.
- In other words, as air temperature increases, humidity also increases and vice-versa.
- Humidity capacity becomes higher during summer months than during winter months and during daytime than nights.
- The extent of land and water and wind velocity also influence humidity capacity. Oceanic and coastal areas record higher humidity capacity of air than the remote areas of the continents.
- Humidity capacity decreases from equator poleward. the air having moisture content equal to its humidity capacity is called saturated air.
- Humidity capacity is expressed in terms of saturated vapour pressure.
- The partial pressure exerted by the water vapour under saturated condition is known as the *saturation vapour pressure* and is denoted by e_s. The saturation vapour pressure increases with increase in temperature. The variation of e_s with temperature can be approximated by the simple equation given below.

$$\boxed{e_s = 6.11 \exp.\left(\frac{17.27\,T}{237.3 + T}\right)}$$

where e_s is in millibars and T is in °C.

Absolute Humidity

- The total wt of moisture content (water vapour) per unit volume of air at a definite temperature is called absolute humidity.
- Absolute humidity is indirectly expressed as vapour pressure.

Relative Humidity

Relative humidity is defined as a ratio of the amount of water vapour actually present in the air having definite volume and temperature (i.e., absolute humidity) to the maximum amount the air can hold (i.e., humidity capacity).

- Relative humidity is generally expressed as percentage.

$$\text{Relative humidity} = \frac{\text{Absolute humidity}}{\text{Humiditycapacity}} \times 100$$
$$= e/e_s \times 100$$

There is inverse relationship between air temperature and relative humidity i.e., relative humidity decreases with increasing temperature while it increases with decreasing temperature.

- When the humidity capacity and absolute humidity of the air are the same, the air is said to be saturated and the relative humidity becomes 100 per cent. Relative humidity changes in two ways viz., (i) if the absolute humidity increases due to additional evaporation or (ii) if the temperature of the air decreases so that humidity capacity also decreases.
- **Importance of relative humidity:** Relative humidity has a great climatic significance because the possibiliity of precipitation depends on it. High and low relative humidity is indicative of the possibility of wet (precipitation) and dry conditions respectively. The amount of evaporation also depends on relative humidity. Evaporation decreases with high relative humidity while it increases with low relative humidity.
- **Distribution of relative humidity**: The horizontal distribution of relative humidity on the earth's surface is zonal in character.
- Equatorial regions are characterized by highest relative humidity. It gradually decreases towards subtropical high pressure belts where it becomes minimum (between 25°-35° latitudes).
- It further increases poleward. The zones of high and low relative humidity shift northward and southward with northward and southward migration of the sun respectively.
- Seasonal distribution of relative humidity is largely controlled by latitudes. Maximum relative humidity is found during summer season between 30°N and 30°S latitudes while high latitudes record relative humidity more than average value during winters. There is maximum relative humidity in the morning whereas lowest value is recorded in the evening.

Condensation and Associated Forms

- The transformation of gaseous form of water (i.e., water vapour) into solid form (ice) and liquid form (water) is called condensation.

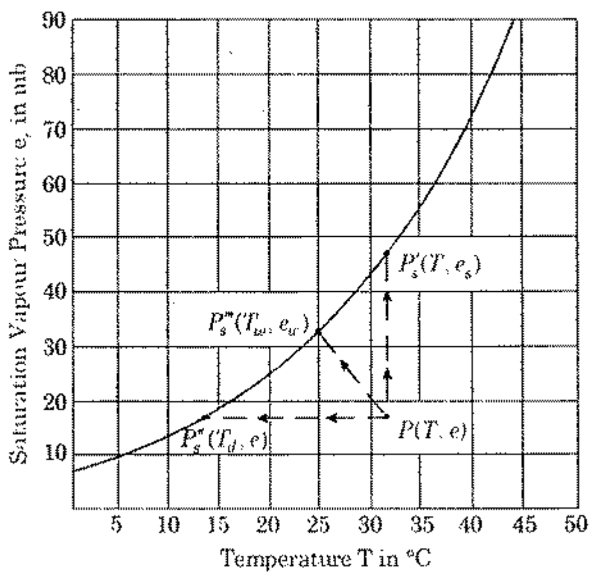
- The process and mechanism of condensation depends on the amount of relative humidity present in the air. The air having 100 per cent relative humidity is called **saturated air**.
- An air may become saturated in two ways e.g. either (i) the absolute humidity at a given temperature is raised to equal the humidity retaining capacity of the air or (ii) the temperature of the air is reduced to such an extent that the humidity capacity becomes equal to its absolute humidity.
- Air becomes saturated as relative humidity becomes 100 per cent, and hence condensation begins.

The equation of state for any gas is given by the equation

$$\boxed{\rho = \frac{p}{RT}}$$

where p is the absolute pressure, ρ is the density, T is the temperature.

Since the gas constant for water vapour is 1.6 times the gas constant for dry air, for a given pressure and temperature water vapour has lesser density than dry air. In SI units the gas constant for dry air has a value of 287 N-m/kg-K.



Variation of saturation vapour pressure with temperature

Let the moist air exist with a pressure p and temperature T and let the partial pressure by water vapour be e. Then the pressure exerted by the dry air would be $(p - e)$. If R denotes the gas constant of dry air, the density of water vapour ρ_v and the density of dry air ρ_d may be written as

$$\boxed{\rho_v = \frac{e}{1.6\, RT}}$$

$$\boxed{\rho_d = \frac{(p-e)}{RT}}$$

The density of the moist air ρ is given by

$$\boxed{\rho_v + \rho_d = \rho = \frac{p}{RT}\left(1 - 0.375\right)\frac{e}{p}}$$

Consider an atmosphere with vapour pressure e and at temperature T, which can be denoted by the point $P(T, e)$ in Fig. above. Since $e < e_s$, this point lies below the curve.

- If the vapour pressure of this atmosphere is increased by adding water vapour to it, keeping the temperature constant, then the point in figure moves vertically upwards till it meets the curve at P_s' with coordinates (T, e_s). At this condition the air is saturated and any further addition of water vapour results in condensation. The quantity $(e_s - e)$ is known as the *saturation deficit*. On the other hand, if the atmosphere is cooled without changing its pressure and vapour pressure the point P moves to the left horizontally till it meets the curve at P_s'' with coordinates (T_d, e). Here again the air is saturated and further cooling results in condensation. The temperature T_d is called the *dew point temperature*. It is the temperature at which the saturation vapour pressure is equal to the existing vapour pressure. Suppose the evaporation is allowed freely into the air without controlling the temperature. Then as more and more water vapour is added to the air it reduces the temperature of the air and consequently the saturation vapour pressure of the air also decreases. In this case the point P moves neither vertically nor horizontally but follow rather a diagonal path till it meets the curve at P_s''' with coordinated (T_w, e_w). This temperature T_w is called the *wet bulb temperature* and it is the temperature to which the original air can be cooled by evaporating water into it and is found by the wet bulb thermometer.
- It may be pointed out that condensation will begin only when the air is supresaturated i.e., if the relative humidity exceeds 100 per cent, and this can be achieved only when the air is further cooled.
- If T_w is above freezing point (32°F), condensation will occur in liquid form (.e., dew, fog, rainfall etc.) but if it is below freezing point, condensation occurs in solid form (e.g., frost, ice, snow, hailstorm etc.).
- It is apparent that condensation depends on (i) the percentage of relative humidity of the air and (ii) the degree of cooling of the air. The air becomes cool when it rises while it gets heated when it descends.
- Thus, the ascending air may bring moist weather while descending air causes dry condition. Much cooling of the air is required in hot arid regions before T_w is reached. On the other hand, very little cooling causes condensation in humid regions.
- The heat released at the time of condensation is called latent heat of condensation.

Cooling of Air and Adiabatic Change of Temperature

- Temperature decreases with increasing height at the rate of 6.5°C per 1000 m. This rate of decrease of temperature with increasing height is called **normal lapse rate**.
- A definite ascending air with given volume and temperature expands due to decrease in pressure and thus cools.
- On the other hand, a descending air contracts and thus its volume decreases but its temperature increases.
- It is apparent that there is change in temperature of air due to ascent or descent but without addition or substraction of heat.
- Such type of change of temperature of air due to contraction or expansion of air is called **adiabatic change of temperature**.
- Adiabatic change of temperature is of two types viz. (i) **dry adiabatic change** and (ii) **moist adiabatic change**.
- The temperature of unsaturated ascending air decreases with increasing height at the rate of 10°C per 1000 m. This cending or descending air is called dry adiabatic rate.
- It may be pointed out that if an air descends its temperature increases at the above mentioned rate.

- The rate of decrease of temperature of an ascending air beyond condensation level is lowered due to addition of latent heat of condensation to the air.
- This is called moist adiabatic rate wherein temperature of an ascending air beyond condensation level decreases (and hence the air cools) at the rate of 6°C per 1000 m. This is also called as retarded adiabatic rate.

Stability and Instability of the Atmospbere

- Different forms of precipitation (dew, fog, rainfall, frost, snowfall, hailstorm etc.) depend on stability and instability of the atmosphere.
- The air without vertical movement is called **stable air** while **unstable air** undergoes vertical movement (both upward and downward).
- An airmass ascends and becomes unstable when it becomes warmer than the surrounding airmass while descending airmass becomes stable.
- The stability and instability depend on the relationships between '**normal lapse rate**' and '**adiabatic change of temperature**'.
- Adiabatic rate is always constant whereas normal lapse rate of air temperature changes.
- When the normal lapse rate is higher than dry adiabatic rate, the air being warmer rises and becomes unstable.
- On the other hand, when the normal lapse rate of temperature is lower than dry adiabatic rate, the air being cold descends and becomes stable.
- **Stability:** When dry adiabatic lapse rate of an ascending dry air is higher than the normal lapse rate and if it is not saturated and does not attain dew point it becomes colder than surrounding air at certain height with the result it becomes heavier and descends. This process causes stability of atmospheric circulation due to which vertical circulation of air is resisted. For example, at ground surface if the temperature of a parcel of air is 40°C, the dry adiabatic lapse rate and normal (environmental) lapse rate are 10°C per 1000m and 6.5°C per 1000 m respectively, then at the height of one kilometre (or 1000 m) from the ground surface the temperature of the ascending air would be 30°C (40° – 10° = 30°C) while the temperature of surrounding air at that height would be 33.5°C (40° – 6.5° = 33.5°C). Thus, the ascending air being colder than surrounding air would descend and atmospheric stability is caused.
- **Instability:** When normal lapse rate is greater than dry adiabatic lapse rate of ascending parcel of air the rising air continues to rise upward and expand and thus becomes **unstable** and is in **unstable equilibrium**.
- In other words, atmospheric instability is caused when the rate of cooling of rising air (dry adiabatic lapse rate) is lower than the normal lapse rate. For example, if the temperature of a certain parcel of air at ground surface is 40°C, the dry adiabatic and normal lapse rates are 10°C and 11°C per 1000 m respectively, then the temperature of ascending air at the height of 1000 m (one kilometre) would be 30°C (40° – 10° = 30°C) while the temperature of the atmosphere at that height would be 29°C (40°C – 11°C = 29°C). Thus, the rising air being warmer (30°C) than the surrounding air (29°C) continues to rise and expand to cause atmospheric instability.
- If the wet adiabatic lapse rate is also less than normal lapse rate, the rising air further continues to rise upward. Such state of continued upward movement of air is called **absolute instability**.
- When the ascending parcel of air reaches such height that its temperature equals the temperature of surrounding air, its further upward movement is stopped. Such air is said to be in the state of **neutral equilibrium**.

- **Conditional instability:** When a parcel of air is forced to move upward, it cools at dry adiabatic lapse rate (10°C per 1000 m) whereas normal lapse rate is 6.5°C per 1000 m. After rising to certain height the air becomes saturated and latent heat of condensation is added to the rising air so the rising air cools at wet adiabatic lapse rate (5°C per 1000 m) whereas normal lapse rate (6.5°C per 1000) is greater than it. Consequently, the air becomes warmer than the surrounding air and hence rises upward automatically. This is called conditional instability because the air is initially forced to move upward but rises automatically due to its own properties after condensation point is reached.
- For example, if a parcel of air with 35°C temperature is initially forced to rise upto the height of 1000 m, its temperature decreases to 25°C (35°C – 10°C, dry adiabatic rate) whereas the temperature of surrounding layers of air at the height of 1000 m would be 28.5°C (35°C – 6.5°C, normal lapse rate) and thus the rising air becomes colder by 3.5°C than the surrounding air. If the rising air gets saturated at this temperature (25°C), the latent heat of condensation returns back to the rising air and hence it cools at the wet adiabatic lapse rate (5°C per 1000 m). Thus, the rising air becomes warmer and unstable. Normal lapse rate ranges between dry adiabatic and wet adiabatic lapse rates. In other words, conditional instability occurs when normal lapse rate is greater than dry adiabatic lapse rate but less than wet abatic lapse rate.

Clouds

Clouds are defined as aggregates of innumerable tiny water droplets, ice particles or mixture of both in the air generally above the ground surface. Clouds are formed due to condensation of water vapour around hygroscopic nuclei caused by cooling due to lifting of air generally known as adiabatic cooling. Meteorologically clouds are very significant because all forms of precipitation occur from them. it may be mentioned that not all clouds yield precipitation but ~~no precipitation is possible without cloud.~~

Rainfall

Origin of Rainfall

- The presence of warm, moist and unstable air and sufficient number of hydroscopic nuclei are prerequisite conditions for rainfall.
- The warm and moist air after being lifted upward becomes saturated and clouds are formed after condensation of water vapour around hygroscopic nuclei (salt and dust particles) but still there may not be rainfall unless the air is supersaturated.
- The process of condensation begins only when the relative humidity of ascending air becomes 100 per cent and air is further cooled through dry adiabatic lapse rate but first condensation occurs around larger hygroscopic nuclei only and droplets are formed.
- Such droplets are called **cloud droplets.**
- The aggregation of large number of cloud droplets forms clouds. These cloud droplets are so microscopic in size that they remain suspended in the air.
- Rainfall does not occur unless these cloud droplets become so large due to coalescence that the air becomes unable to hold them. This is why, some times the sky is overcast by thick couds but there is no rainfall. If by chance these cloud droplets fall downward they are evaporated before they reach the ground surface. Rainfall occurs only when cloud droplets change to raindrops. There are two possible processes of change of cloud droplets into raindrops.

1. If warm and moist air ascends to such a height that condensation begins below freezing point, then both, water droplets and ice droplets, are formed. The water droplets are evaporated because of difference of vapour pressure between them and ice droplets and there is condensation of evaporated water around ice crystals which go on increasing in size. If they become sufficiently large in size, they cannot be held

in suspension by the air and consequently they begin to fall down. If the temperature above the ground is high they fall in the form of raindrops.

2. The suspended cloud droplets in the clouds are of different sizes. These cloud droplets collide among themselves at varying rates due to difference in their sizes and thus form large droplets. In the process several cloud droplets are coalesced to form raindrops. When they become so large in size that ascending air becomes unable to hold them, they fall down as rainfall.

- The diameter of a raindrop is upto 5 mm and one raindrop contains about 8,000,000 cloud droplets. When raindrops become very large and fall down at greater speed (more than 30 kilometres per hour), they are split in the transit but give heavy downpour.
- When the air ascends slowly, the process of condensation is also very slow and hence small raindrops are formed and the resultant rainfall is drizzle.
- But if the air ascends huriedly with greater speed. Very large raindrops are formed and resultant rainfall is heavy downpour.
- When condensation occurs below freezing point, the resultant precipitation is in solid form and is called snowfall.

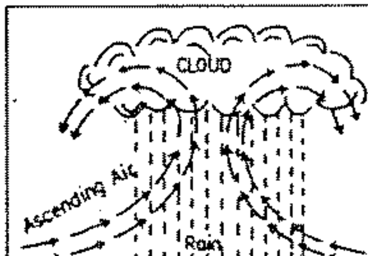
Types of Rainfall

Rain is the most common form of precipitation. For rainfall, it is necessary that moist air must ascend, saturate (relative humidity 100 per cent) and condense. Adiabatic cooling due to upward movement of air is by far the most important mechanism of condensation and related precipitation including rainfall. It is apparent that upward movement is a prerequistite condition for cloud formation and rain fall. Thus, precipitation and rainfall are classified on the basis of conditions and mechanisms of upward movement of air. There are three ways in which air is forced to move upward and thus cools according to adiabatic lapse rate e.g. (1) due to heating of ground surface the air being heated expands and rises upward in the form of convection currents, the mechanism is known as thermal convection, (2) ascent of air over an orographic barrier, and (3) uplift of air associated with low pressure system, known as cyclonic or frontal ascent. It may be pointed out that it is not necessary that all these three factors work independently in relation to the ascent of air. Some times, more than one factor are operative. In such situation, the form of precipitation is determined on the basis of dominant factor. Thus, precipitation and rainfall are classified into the following three types:

1. **Convectional rainfall**, occurring due to thermal convection currents caused due to insolational heating of ground surface.
2. **Orographic rainfall**, occuring due to ascent of air forced by mountain barrier, and
3. **Cyclonic or frontal rainfall**, occuring due to upward movement of air caused by convergence of extensive air masses.

1. Conventional Rainfall

The principal motivating force behind the ascent of warm and moist air is thermal convection caused by heating of the ground surface through insolation. Two conditions are necessary to cause convectional precipitation and rainfall e.g. (i) abundant supply of moisture through evaporation to the air so that relative humidity because high, and (ii) intense heating of ground surface through incoming shortwave electromagnetic solar radiation. The mechanism of convectional rainfall may be explained in the following manner.



Conventional rainfall

The ground surface is intensely heated due to enormous amount of heat received through solar radiation during daytime, with the result the air coming in contact with warm ground surface also gets heated, becomes warm, expands, and ultimately rises upward. The ascending warm and moist air cools according to **dry adiabatic lapse rate** (decrease of temperature at the rate of 10°C per 1000 metres). The cooling of ascending air increases its relative humidity. The moist air becomes saturated soon (relative humidity becomes 100 per cent) and further ascent of air beyond saturation level causes condensation and cloud formation (cumulo-nimbus clouds) and thus rainfall starts. The air still continues to rise and in the process further cools but at **moist adiabatic lapse rate** or **retarded adiabatic rate** (decrease of temperature at the rate of 5°C per 1000 metres) due to addition of **latent heat of condensation** to the ascending air released after condensation of atmospheric vapour. When the ascending air reaches such height where its temperature matches with the temperature of surrounding air, the process of condensation is more activated and hence cumulo-nimbus clouds are formed and there begins instantaneous heavy rainfall (Figure).

Since the ascending moist air (convectively motivated) cools soon after rising to very little height, causing immediate saturation and condensation, the convectional rainfall occurs in the form of heavy downpour. It is also apparent from the above description that convectional rainfall is a warm weather phenomenon and is associated with lighting and cloud thunder. Conventional rainfall mainly occurs in equatorial regions of low latitudes where daily heating of ground surface upto noon causes convection currents. Consequently, the sky becomes overcast by 2-3 P.M. daily causing pitch darkness and heavy rains and the sky becomes clear by 4 P.M. Thus, the convectional rainfall in the equatorial region is a daily regular feature.

Convectional rainfall also occurs in the tropical, subtropical and temperate regions in summer months and in the warmer parts of the day. The following are characteristic features of convectional rainfall:

(i) It occurs daily in the afternoon in the equatorial regions.

(ii) It is of very short duration but occurs in the form of heavy showers (heavy downpour).

(iii) It occurs through thick dark and extensive cumulo-nimbus clouds.

(iv) It is accompanied by cloud thunder and lightning.

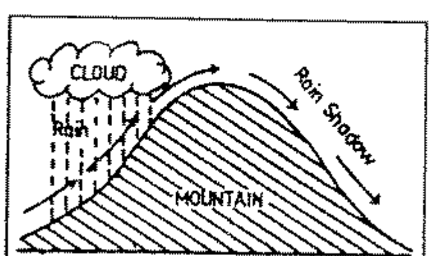
(v) Out side equatorial regions convective rainfall is of little significance to crop growth because most of rainwater is drained to the streams through surface runoff which causes severe rill and gully erosion resulting into enormous loss of loose soils.

(vi) Convective rainfall supports luxurious evergreen rainforests in the equatorial regions.

(vii) Convective rainfall in the temperate regions is not in the form of heavy showers rather it is slow and of longer duration so that most of rainwater infiltrates into soils. here rains are always in summers.

(viii) Convective rainfall in hot deserts is not regular but is irregular and sudden.

2. Orographic Rainfall



Orographic rainfall

Orographic rainfall occurs due to ascent of air forced by mountain barriers. The mountain barriers lying across the direction of air flow force the moisture laden air to rise along the mountain slope and thus lifted air mass cools according to dry adiabatic lapse rate (decrease of temperature at the rate of 10°C per 1000 metres) which increases the relative humidity of the air. The ascending air becomes saturated after reaching certain height and condensation begins around by hygroscopic nuclei. The addition of latent heat of condensation to the air causes it to move further upward and cool at moist adiabatic lapse rate (decrease of temperature with increasing height at the rate of 5°C per 1000 metres). Thus, ascending air continues to yield precipitation with increasing height. It is apparent that mountain barriers produce trigger effect for the moist air to ascend, cool and become unstable. The slope of the mountain facing the wind is called **windward slope** or onward slope and receives max. Precipitation while the opposite slope is called leaward slope or **rainshadow region** because the ascending air after crossing over the mountain barrier descends along the leeward slope and thus is warmed at dry adiabatic lapse rate (increase in temperature with decreasing height at the rate of 10°C per 1000 metres). Consequently, the humidity capacity of the descending air increases resulting into substantial decrease in relative humidity. Secondly, the moisture present in the air is already precipitated on the windward slope and thus there is very little precipitation on the leeward slope. Most of the world precipitation occurs through orographic rainfall.

The following conditions are necessary for the occurrence of orographic rainfall.

(i) There should be mountain barier across the wind direction, so that the moist air is forced on obstruction to move upward. If the mountain barriers are parallel to the wind direction, the air is not obstructed and no rainfall occurs. For example, Aravallis ranges running in southwest-northeast direction are parallel to the Arabian Sea Branch of south-west Indian monsoon and hence Rajasthan receives very low amount of rainfall.

(ii) If the mountains are very close and parallel to the sea coasts, they become effective barriers because the moisture laden winds coming from over the oceans are obstructed and forced to ascend and soon become saturated. For example, the situation of the Western Ghats in India presents ideal conditions for orographic rainfall.

(iii) The height of mountains also affects the form and amount of orographic raiufall. If the mountains are very close to the sea coast, even low height can be effective barrier and can yield sufficient rainfall because the moist air becomes saturated at very low height. On the other hand, the inland mountains should be of higher height because the air after covering long distances loses much of its moisture content.

(iv) There should be sufficient amount of moisture content in the air.

The following are the characteristic features of orographic rainfall:

(a) The windward slope, also called as rain slope, receives maximum amount of rainfall whereas leeward side of the mountain gets very low rainfall. For example, Mangalore located on the western slope (windward slope) of the Western Ghats receives mean annual rainfall of above 2000 mm whereas Bangalore situated in the rain shadow region gets only 500 mm of mean annual rainfall. The southern slopes of the Himalayas receive mean annual rainfall of more than 2000 mm whereas the northern slope receives only 50 mm of mean annual rainfall.

(b) There is maximum rainfall near the mountain slopes and it decreases away from the foothills. For example, the cities and towns located at the southern slopes of the Himalayas receive more rainfall e.g. Simla 1520 mm, Nainital 2000 mm and Drazeeling 3150 mm whereas the places away from the Himalayan foothills receive relatively low rainfall e.g. Patna 1000 mm, Allahabad 1050 mm and Delhi 650 mm.

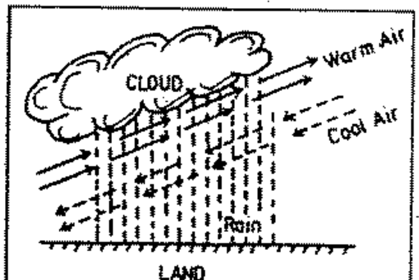
(c) If the mountains are of moderate height, the maximum rainfall does not occur at their tops rather it occurs on the other side.

(d) The height of the mountains beyond which the amount of rainfall decreases upward is called **maximum rainfall line** which varies spatially depending on the location of mountains, their distance from the sea, moisture content in the air, mountain slope season etc.

(e) Orographic rainfall may occur in any season. Unlike other types of rainfall it is more wide-spread and of long duration.

(f) It may be pointed out that orographic rainfall is induced not only because of lifting of moist air due to mountain barrier but convective and cyclonic mechanisms also help in the process of orographic rainfall. For example, in warm regions valleys are heated during daytime and hence winds are also heated and ascend along the hillslopes in the form of convection currents and yield rainfall after being saturated. Some times, forward moving cyclones are also forced to ascend along the hillslope due to obstructions offered by mountain barriers.

3. Cyclonic or Frontal Rainfall



Cyclonic or Frontal rainfall

- Cyclonic or frontal rainfall occurs due to ascending of moist air and adiabatic cooling caused by convergence of two extensive air masses.
- The mechanism of cyclonic precipitation is of two types on the basis of two types of cyclones viz. temperate cyclones and tropical cyclones.
- Rainfall associated with temperate cyclones occurs when two extensive air masses of entirely different physical properties (warm and cold air masses) converge. When two contrasting air masses (cold polar air mass and warm westerly air mass) coming from opposite directions converge along a line, a front is formed. The warm wind is lifted upward along this front where as cold air being heavier settles downward. Such cyclonic fronts are created in temperate regions where cold polar winds and warm westerlies converge. The warm air lying over cold air is cooled and gets saturated and condensation begins around hygroscopic nuclei. It may be pointed out that lifting of warm air along cyclonic front is not vertical like convective currents rather it is oblique.
- Since the lifting of warm air along the warm front of temperate cyclone is slow and gradual and hence the process of condensation is also slow and gradual, with the result precipitation occurs in the form of drizzles but continues for longer duration.
- Thus, the precipitation associated with warm front is widespread and of long duration.
- On the other hand, the precipitation associated with cold fronts is always in the form of thunder showers but is of very short duration. Sometimes, the precipitation occurs in the form of snowfall and hailstorms. This is because of the fact that lifting of warm air along cold front occurs quickly as cold air pushes warm air upward with great force.
- Most of the rains of temperate regions are received through cyclones.

In tropical regions two extensive air masses of similar physical properties converge to form tropical cyclones wherein lifting of air is almost vertical and is very often associated with convection. Tropical cyclones, regionally called as typhoons hurricanes, tornadoes etc. yield heavy downpour in China, Japan, South East Asia, Bangladesh, India, USA etc.

Other Forms of Precipitation

1. **Snowfall:** Th fall of larger snowflakes from the clouds on the ground surface is called snowfall. In fact, snowfall is 'precipitation of white and opaque grains of ice'. The snowfall occurs when the freezing level is so close to the ground surface (less than 300 m from the surface) that aggregations of ice crystals reach the ground without being melted in a solid form of precipitation as snow.
2. **Sleet** refers to a mixture of snow and rain but in American terminology sleet means falling of small pellets of transparent of translucent ice having a diameter of 5 mm or less.
3. **Hail** consists of large pellets or spheres of ice. In fact, hail is a form of solid precipitation wherein small balls or pieces of ice, known as hailstones, having a diameter of 5 to 50 mm fall downward known as hailstorms. Hails are very destructive and dreaded form of solid precipitation because they destroy agricultural crops and claim human and animal lives.
4. **Drizzle:** The fall of numerous uniform minute droplets of water having diameter of less than 0.5 mm is called drizzle. Drizzles fall continuously but the total amount of water received on the ground surface is significantly low.

CHAPTER

3

Measurement of Precipitation

- The total amount of precipitation on a given area is expressed as the depth over the horizontal projection of the area. Thus 1 cm of rainfall over a catchment area of 1 km^2 represents a volume of water equal to 10^4 m^3.
- Any part of this precipitation, if falling as snow or ice, is to be accounted for in its melted form.
- Since it is not physically possible to catch all the rainfall or snowfall over a drainage basin, it is only sampled by rain gauges whose **catch**, in a perfect exposure, represents the precipitation falling on their respective surrounding areas.
- Terms such as pluviometer, ombrometer and hyetometer are also sometimes used to designate a raingauge.

Types of Gauges

The various types of precipitation gauges used are broadly classified as:

(a) non-recording gauges, and

(b) recording gauges.

Non-Recording Gauges

- The nonrecording guage extensively used in India is the Symons' raingauge. It is installed in an open area on a concrete foundation. The distance of the rain gauge from the nearest object should be at least twice the height of the object. It should never be on a terrace or under a tree. The gauge may be fenced with a gate to prevent animals and unauthorized persons from entering the premises.
- Measurements are to be made at a fixed time, normally at 08:30 hrs. In case of heavy rainfall areas, measurements are made as often as possible.

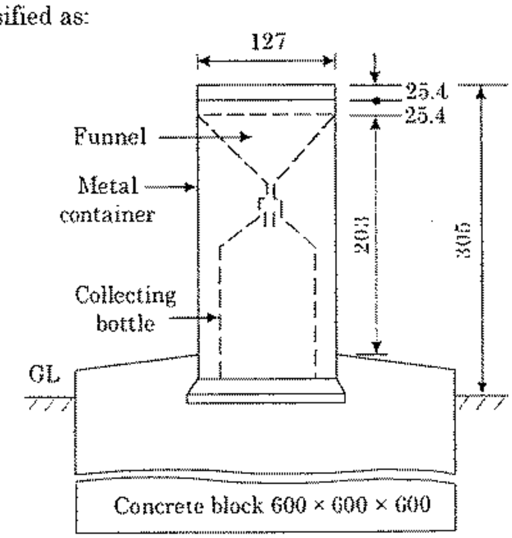


Fig. 3.1: Non-recording raingauge.

Recording Gauges

The recording gauges produce a continuous plot of rainfall against time and provide valuable short duration data on intensity and duration of rainfall for hydrological analysis of storms. The commonly used recording gauges are:

(a) Tipping bucket type

(b) Weighing type, and

(c) Natural syphon type

The weighing type is suitable for measuring all kinds of precipitation (rain, sleet etc.).

Tipping–Bucket Type

The catch from the funnel falls onto one of a pair of small buckets. These buckets are so balanced that when 0.25 mm of rainfall collects in one bucket, it tips and brings the other one in position. The tipping actuates an electrically driven pen to trace a record on clockwork-driven chart. The record from tipping bucket gives data on the **intensity of rainfall**. The main advantage of this type of instrument is that it gives an electronic pulse output that can be recorded at a distance from the raingauge.

Weighing–Bucket Type

The catch from the funnel empties into a bucket mounted on a weighing scale. The weight of the bucket and its contents are recorded on a clockwork-driven chart. This instrument gives a plot of the **accumulated rainfall** against the elapsed time, i.e. the **mass curve of rainfall** (accumulated precipitation against time).

Natural Syphon Type

This type of recording rain-gauge is also known as **float type gauge**. Here the rainfall collected by a funnel shaped collector is led into a float chamber causing a float to rise. As the float rises, a pen attached to the float through a lever system records the elevation of the float on a rotating drum driven by a clockwork mechanism. A syphon arrangement empties the float chamber when the float has reached a pre-set maximum level which resets the pen to its zero level. This type of raingauge is adopted as the **standard recording type rain gauge in India**.

This type of gauge gives a plot of the **mass curve** of rainfall.

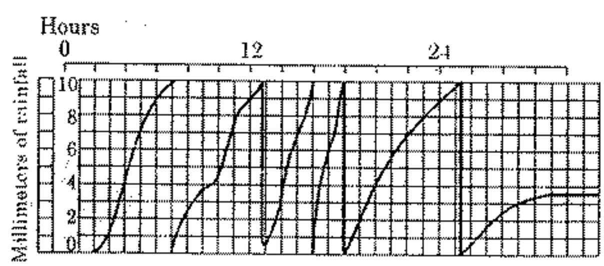


Fig. 3.2. (Record of float gauge).

Other Precipitation Measurement Method

Storage Gauges

In sparsely populated or remote regions such as in a desert or a mountainous terrain, storage gauges are used to measure total seasonal precipitation. These gauges are read monthly, seasonally, or whenever it is possible to inspect the stations.

Telemetering Rain Gauges

These rain gauges are of the recording type and contain electronic units to transmit the data on rainfall to a base station both at regular intervals and on interrogation. The tipping-bucket type raingauge, being ideally suited. Telemetering gauges are of utmost use in gathering rainfall data from mountainous and generally inaccessible places.

Radar Measurement of Rainfall

Radar permits the observation of the location and movement of areas of precipitation in the atmosphere; and certain types of radar equipment can yield estimates of rainfall rates over areas within the range of the radar.

Observations by Satellites

Satellite images can be used for estimating precipitation over areas ranging from the global to the very local scale in real or near-real time. This complements the conventional precipitation measurements in areas of sparse rain gauge networks and can improve the accuracy of estimating precipitation for short time periods (several hours).

Snowfall Water Equivalent

- The *water equivalent* of a given snowfall is the amount of liquid precipitation contained in that snowfall.
- The water equivalent of the snowfall is determined either by weighing or melting.
- Snow collected in a non-recording rain-gauge is melted immediately and measured by means of an ordinary measuring cylinder graduated for rainfall measurements.
- The weighing-type recording gauge is also used to determine the water content of snowfall.
- During periods of snowfall, the funnels of the gauges are removed so that all precipitation can fall directly into the receiver.

Rain Gauge Network

- For proper assessment of water resources, a good network of rain gauges is a must. More is the variability of rainfall, denser should be the rain gauge network.
- As per the IS: 4987-1968, the recommended rain gauge network density is as follows:

In plains	One in every 520 km^2,
Moderately elevated area (av. elevation up to 1000 m)	One in 260 to 390 km^2
Hilly areas	One in 130 km^2

As far as possible, 10 percent of the rain gauge stations should be equipped with automatic (self recording) rain gauges.

Adequacy of Raingauge Stations

Number of raingauge stations for an area to give necessary average rainfall with certain percentage of error can be obtained as follows:

Step 1: Calculate mean rainfall, P_m

$$P_m = \frac{(P_1 + P_2 + ... + P_n)}{n}$$

P_1 is precipitation magnitude of the 1st station and so on and n is the total number of raingauge stations in the catchment.

Step 2: Calculate the standard deviation, σ

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

Step 3: Calculate coefficient of variation, C_V

$$C_V = \sigma/P_m$$

Step 4: Optimal number of stations, N

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

where, ϵ is the allowable degree of error in the mean rainfall (in fraction).

Step 5: Additional number of raingauge station required = $N - n$

In routine hydrological investigation, error of estimate should not exceed 10% i.e. $\varepsilon = 0.1$

Example 3.1

The average normal rainfall of 5 raingauges in the base stations are 89, 54, 45, 41 and 55 cm. If the error in the estimation of rainfall should not exceed 10%, how many additional gauges may be required?

Sol: The mean rainfall is obtained as:

$$P_m = \frac{89+54+45+41+55}{5} = 56.8 \text{ m}$$

Now,

$$\sigma^2 = \frac{(89-56.8)^2+(54-56.8)^2+(45-56.8)^2+(41-56.8)^2+(55-56.8)^2}{5-1}$$

or $\sigma^2 = 359.2$

$\therefore$ $\sigma = 19.95$

The coefficient of variation is calculated as:

$$\therefore \quad C_V = \frac{19.95}{56.8} = 0.33367$$

$$\therefore \quad N = \left(\frac{C_V}{0.10}\right)^{10} = \left(\frac{0.33367}{0.1}\right)^2 = 11.13$$

$$\approx 12$$

Thus additional no. required = (12 – 5) = 7.

Normal Precipitation

The normal rainfall is the average value of rainfall of a particular date, month or year over a specified 30-year period (like normal rainfall of 5th March or normal rainfall of Jaunary or yearly rainfall). The 30-year normals are recomputed every decade to account for change in environment and land use, because these factor may affect the amount of rainfall on that area.

Normal rainfall is used to find out the missing data of certain raingauges.

Preparation of Data

Before using the rainfall records of a station it is necessary to 1st check the data for ***continuity*** and ***consistency***. Continuity means availability of continuous record of previous rainfall and consistency means that rainfall data of previous years should be consistent with the present environmental and land use conditions (like if there is a jungle in a particular area which did not exist 15 years ago then previous records will not be consistant with current record).

Estimation of Missing Data

Sometimes a station has a break in record due to absence of observer or failure of the instrument. It is then necessary to estimate that missing data. To estimate the data, three or more stations close to this station are selected. Following are the different ways of calculating the missing data.

Arithmetic Mean Method

If the normal precipitation at each of these selected stations is within 10% of that for the station with missing data, then simple arithmetical mean of the precipitation of those stations will give the value of the missing station.

$$P_x = \frac{P_1 + P_2 + P_3 + \ldots + P_m}{m}$$

Normal Ratio Method

If normal precipitation at any of these selected stations is above 10% of that for station with missing data then,

$$\frac{P_x}{N_x} = \frac{1}{m}\left(\frac{P_1}{N_1} + \frac{P_2}{N_2} + \ldots + \frac{P_m}{N_m}\right)$$

where P_1 = Precipitation of 1st station

N_1 = Normal precipitation of the 1st station

m = No. of additional station chosen

P_x = Precipitation (missing data)

N_x = Normal precipitation of the station at which data is missing

Example 3.2

The normal annual rainfall of stations *A, B, C* and *D* in a catchment is 80 mm, 91 mm, 85 mm and 87 mm respectively. In the year 2007, the station *D* was inoperative when stations *A, B* and *C* recorded annual rainfall of 91.11, 72.23 and 79.89 mm. Estimate the missing rainfall at station *D* in the year 2007.

Sol: Normal precipitation of all the station A, B and C are within 10% of that at station D. Hence simple arithmetic average will be used.

$$\Rightarrow \quad P_x = \frac{91.11 + 72.23 + 79.89}{3} = 81.08 \text{ mm}$$

Example 3.3

Find the missing rainfall at station x.

Raingauge	Normal	Actual
A	1125	875
B	910	1021
C	765	915
x	830	?

As the normal ppt of other stations A, B and C are not within in 10% of normal ppt at station x. Hence

$$\frac{P_x}{N_x} = \frac{1}{3}\left[\frac{P_A}{N_A} + \frac{P_B}{N_B} + \frac{P_C}{N_C}\right]$$

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

$$\Rightarrow \boxed{P_x = 856.5}$$

Inconsistency of Records

Some of the common causes for inconsistency of the records are:

- Shifting of raingauge station to a new location
- Neighborhood of the station undergoing a marked change
- Change in the ecosystem due to calamities such as forest fires, landslides etc.
- Occurrence of observational error from a certain date

Inconsistency of record is corrected by using double mass curve technique. Thus on correction, the previous record becomes consistent with the present day environmental and land use condition.

Double Mass Curve Technique

- To draw this curve, a group of stations (say 10) is taken as base station in the neighbourhood of the problamatic station X.
- The accumulated rainfall of station $X(\Sigma P_x)$ and accumulated values of average of group of base stations (ΣP_{AV}) are calculated starting from the latest record.
- The values of ΣP_x as ordinate and ΣP_{AV} as abscissa are plotted for available data of rainfall.
- In the plot, a break in the slope is observed. It indicates a change in precipitation of station X.
- The values precipitation at X beyond the break point are corrected based on the slope of both the lines.
- A change in slope is normally taken as significant only where it persists for more than 5-yrs.

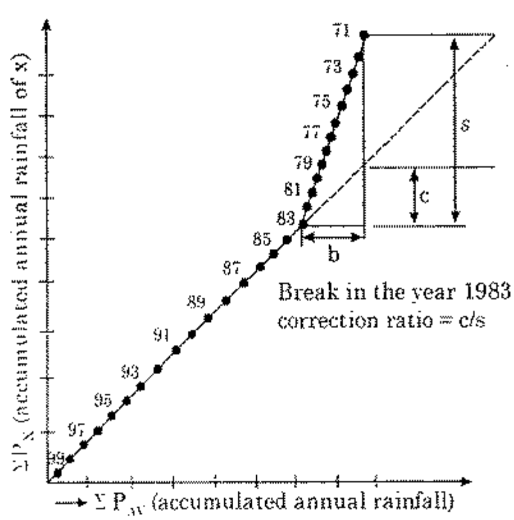


Fig. 3.3. Double mass curve analysis to adjust the error.

In the above figure data for year 1983 and beyond (i.e. 1982, 1981.....) is to be multiplied by correction ratio c/s to make it consistant with the data before the year 1983 (i.e. 1984, 1985 .....).

Example 3.4

The annual rainfall at station X and the average annual rainfall at 18 surrounding stations during 1952 to 1970 are as follows:

Annual rainfall in cm at X:

30.5, 38.9, 43.7, 32.2, 27.4, 32.0, 49.3, 28.4, 24.6, 21.8, 28.2, 17.3, 22.3, 28.4, 24.1, 26.9, 20.6, 29.5 and 28.4.

18 stations average annual rainfall in cm:

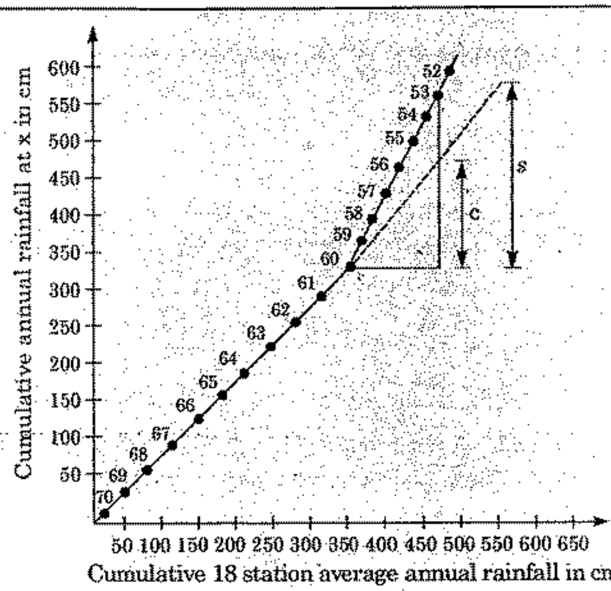
22.8, 35.0, 30.2, 27.4, 25.2, 28.2, 36.1, 18.4, 25.1, 23.6, 33.3, 23.4, 36.0, 31.2, 23.1, 23.4, 23.1, 33.2 and 26.4.

Explain how the consistency of the record at station X can be verified and how to determine the year in which a change in regime has occurred.

Sol: The given data is arranged in a reverse chronological order and their cumulative values are worked out as shown in the table below:

Year	Annual rainfall at X in cm	Cumulative annual rainfall at X in cm	18 stn. avg. annual rainfall in cm	18 stn. cum. avg. annual rainfall in cm
1970	28.4	28.4	26.4	26.4
1969	29.5	57.9	33.2	59.6
1968	20.6	78.5	23.1	82.7
1967	26.9	105.4	23.4	106.1
1666	24.1	129.5	23.1	129.2
1965	28.4	157.9	31.2	160.4
1964	22.3	180.2	36.0	196.4
1963	17.3	197.5	23.4	219.8
1962	28.2	225.7	33.3	253.1
1961	21.8	247.5	23.6	276.7
1960	24.6	272.1	25.1	301.8
1959	28.4	300.5	18.4	320.2
1958	49.3	349.8	36.1	356.3
1957	32.0	381.8	28.2	384.5
1956	27.4	409.2	25.2	409.7
1955	32.2	441.2	27.4	437.1
1954	43.7	485.1	30.2	467.3
1953	38.9	524.0	35.0	502.3
1952	30.5	554.5	22.8	525.1

A graph is now plotted with cumulative annual rainfall at y on y-axis and cumulative average annual runoff of 18 stations on x-axis.



This graph shows that inconsistency has occurred in 1960. Hence the present data, since 1960 to 1970 will be treated as correct and the data prior to the year 1960 will be corrected.

$$\text{Correction factor} = \frac{c}{s} = \frac{173}{248} = 0.6976$$

The yearly rainfall values of station X between the years 1959 to 1952 are thus corrected by multiplying the original values by 0.6976 as shown in table below.

Year (1)	Annual rainfall at station X in cm (2)	Corrected annual rainfall at station X = 0.6976 × col.(2) (3)
1959	28.4	19.8
1958	49.3	34.4
1957	32.0	22.3
1956	27.4	19.1
1955	32.2	22.5
1954	43.7	30.5
1953	38.9	27.1
1952	30.5	21.3

Year	Corrected annual rainfall values at station X in cm
1952	21.3
1953	27.1
1954	30.5
1955	22.5
1956	19.1
1957	22.3
1958	34.4
1959	19.8
1960	24.6
1961	21.8
1962	28.2
1963	17.3
1964	22.3
1965	28.4
1966	24.1
1967	26.9
1968	20.6
1969	29.5
1970	28.4
	Σ = 469.1

The final adjusted values of annual rainfall at station X can be tabulated as shown above.

Presentation of Rainfall Data

Rainfall data is presented in the form of:

(a) Mass curve

(b) Hyetograph

(c) Moving average

Mass Curve of Rainfall

The mass curve of rainfall is a plot of the accumulated precipitation against time, plotted in chronological order. Records of float type and weighing bucket type gauges are of this form. Mass curves of rainfall are very useful in extracting the information on the duration and magnitude of a storm. Intensities at various time intervals in a storm can be obtained from the slope of the curve.

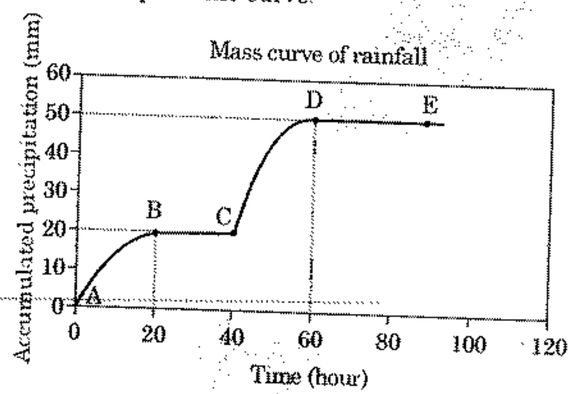


Fig. 3.4

Average intensity in 1st storm (A-B) = $\frac{20}{20} = 1$ mm/hr

No rainfall during BC

Average intensity in 2nd storm (C-D) = $\frac{30}{20} = 1.5$ mm/hr

No rainfall during DE

Hyetograph

A hyetograph is a plot of the intensity of rainfall against the time interval. The hyetograph is derived from the mass curve and is usually represented as a bar chart. The area under a hyetograph represents the total precipitation received in the period.

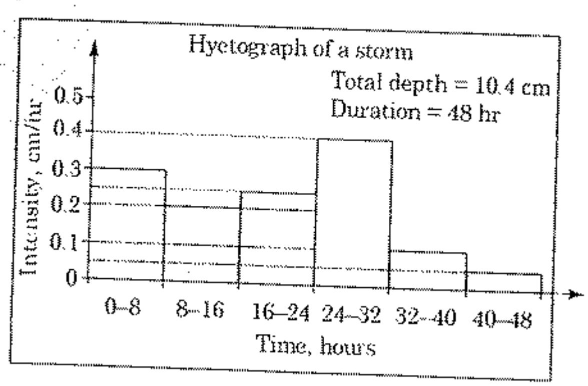


Fig. 3.5

Moving Average

If we plot point rainfall (rainfall collected at raingauge station) with time in chronological order the fluctuations will be large in the time series of rainfall. From this it will be difficult to determine the trend of the rainfall. Thus a moving average plot is made which smoothens out the fluctations in time series and **help in determining the trend of rainfall**.

To find out moving average, for say 3 yrs., average of rainfall of 1st, 2nd and 3rd yrs is plotted against 2nd yrs. average of 2nd, 3rd and 4th yr is plotted against 3rd yr. and so on.

Similarly for 5 yr moving average, av. of rainfall of 1st, 2nd, 3rd, 4th and 5th yr is plotted against 3rd yr, av. of 2nd, 3rd, 4th, 5th and 6th yr is plotted against 4th yr and so on.

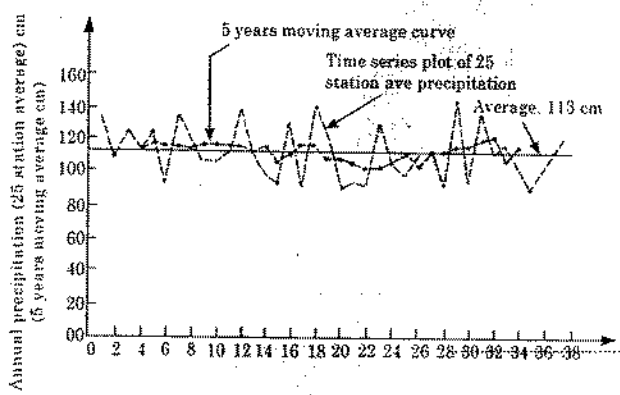


Fig. 3.6

Example 3.5

The record of annual rainfall at a place is given for 25 years. Find the 5-yr and 3-yr moving average of the rainfall data.

Year

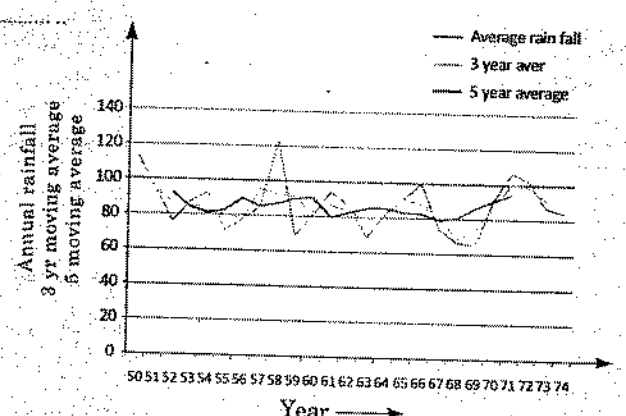
1950	1951	1952	1953	1954	1955
1956	1957	1958	1959	1960	1961
1962	1963	1964	1965	1966	1967
1968	1969	1970	1971	1972	1973
1974					

Annual rainfall (cm)

113.0	94.5	76.0	87.5	92.7	71.3
77.3	85.1	122.8	69.4	81.0	94.5
86.3	68.6	82.5	90.7	99.8	74.4
66.6	65.0	91.0	106.8	102.2	87.0
84					

Sol:

Year	Annual Rainfall (cm)	3 yr Total (cm)	3 yr moving average (cm)	5 yr Total	5 yr moving average (cm)
1950	113				
1951	94.5	283.5	94.50		
1952	76	258	86.00	463.7	92.74
1953	87.5	256.2	85.40	422	84.4
1954	92.7	251.5	83.83	405	81
1955	71.3	241.5	80.50	414.1	82.82
1956	77.5	233.9	77.97	449.4	89.88
1957	85.1	285.4	95.13	426.1	85.22
1958	122.8	277.3	92.43	435.8	87.16
1959	69.4	273.2	91.07	452.8	90.56
1960	81	244.9	81.63	454	90.8
1961	94.5	261.8	87.27	399.8	79.96
1962	86.3	249.4	83.13	412.9	82.58
1963	68.6	237.4	79.13	422.6	84.52
1964	82.5	241.8	80.60	427.9	85.58
1965	90.7	273	91.00	416	83.2
1966	99.8	264.9	88.30	414	82.8
1967	74.4	240.8	80.27	396.5	79.3
1968	66.6	206	68.67	396.8	79.36
1969	65	222.6	74.20	403.8	80.76
1970	91	262.8	87.60	431.6	86.32
1971	106.8	300	100.00	452	90.4
1972	102.2	296	98.67	471	94.2
1973	87	273.2	91.07		
1974	84				



Calculation of Average Depth of Precipitation over a Catchment

The precipitation over a catchment is actually measured as point values at a finite number of precipitation stations (Raingauge station).

However, hydrological analysis requires a knowledge not of point rainfall but of the rainfall over an area, such as over a catchment.

To convert the point rainfall values at various stations into an average value over catchment, several methods are available.

(i) Arithmetical-mean method,

(ii) Thiessen-polygon method

(iii) Isohyetal method

Arithmetic Mean Method

- The arithmetic mean method gives equal weights to all the rain gauges. Apart from being quick and easy, it yields fairly accurate results if the rain gauges are uniformly distributed and are under homogeneous climate. Under normal situation this method is **least accurate method**.
- This method doesn't take into account the raingauges located outside the catchment.

  As per this method:

$$P_m = \frac{(P_1 + P_2 + ... + P_n)}{n}$$

P_m is the average rainfall in the catchment

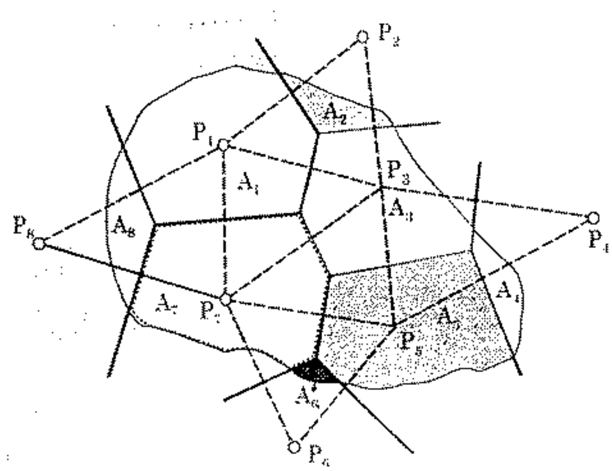
P_i is the rainfall magnitude at the i[th] station inside catchment

n is the number of raingauges in the catchment.

Thiessen Polygon Method

This method considers the representative area for each rain gauges representative area can also be thought of as the areas of influence of each raingauge. These areas are found out using the following steps:

1. Joining the rain gauge station locations by straight lines to form triangles.
2. Bisecting the edges of the triangles to form the so-called "Thiessen polygons".
3. Calculate the area enclosed ~~around each rain gauge stations bounded~~ by the polygon edges (and the catchment boundary, wherever appropriate) to find out the area of influence corresponding to the rain gauge.



$$P_m = \frac{\sum_{i=1}^{n} P_i A_i}{\sum_{i=1}^{n} A_i} = \sum_{i=1}^{n} P_i \frac{A_i}{A} = \sum_{i=1}^{n} P_i W_i$$

P_m is the average rainfall in the catchment.

P_i is the rainfall magnitude at the i[th] station

A_i is the area of influence for the i[th] station

A is the total area of the catchment.

W_i is weight of station i

- Weighted average rainfall is calculated by this method.
- Polygon needs to be calculated only once for a given distribution of raingauge network. Thus once the weight of a station is fixed thereafter calculation is easy.
- The new polygon is required to be redrawn when, due to addition or deletion of rain gauges to the network, weight of each station changes.
- This method takes care of non uniform distribution of raingauge.
- However the variability in rain fall due to elevation differences is not taken care of (i.e. topographical influence are not taken care of).
- Hence polygon method is reliable only for plain areas and isohytel method is used for both plain and hilly areas.
- This method is more accurate than arithmetic method and takes care of raingauges located out side the catchment also.

Example 3.6

Calculate the average depth of rainfall for the catchment with an area of 200 km^2 using arithmetic mean method and Thiessen polygon method. The rainfall amounts are indicated in mm at the respective gauge sites. Also compute the volume of rain water in m^3 received by the catchment area in both cases.

Station no. (i)	Rainfall in (mm) (P_i)	Thiessen polygon area with in catchment boundary (km^2)	Description inside catchment = I outside catchment = 0
1	40	7.2	0
2	25	10.4	0
3	37	49.8	I
4	49	35.8	I
5	55	6.6	0
6	38	47.2	I
7	48	41.5	I
8	40	1.5	0

Sol: The average depth of rainfall is determined as given below.

(i) *Arithmetic mean method*

$$P = \frac{P_1 + P_2 + ...+ P_n}{n}$$

There are only 4 raingauge stations within the catchment area.

Volume of rain water = Area × depth of rainfall = A × P

= 200 × 43 km^2-mm

$$= (200 \times 1000 \times 1000) \times \left(\frac{43}{1000}\right) = 86 \times 10^5 \, m^3$$

(ii) *Thiessen Polygon method*

Computations for Thiessen Polygon Method

Station no. (i)	Rainfall in mm (P_i)	Thiessen polygon area within catchment boundary km^2 (A_i)	Rainfall volume in km^2-mm $A_i \times P_i$	Description
1	40	7.2	288.0	O
2	25	10.4	260.0	O
3	37	49.8	1842.6	I
4	49	35.8	1754.2	I
5	55	6.6	363.0	O
6	38	47.2	1793.6	I
7	48	41.5	1992.0	I
8	40	1.5	60.0	O
	Totals	200.0	8353.4	

$$\text{Average depth of rainfall } P = \frac{\sum P_i A_i}{A}$$

$$= \frac{8353.4}{200} = 41.77 \text{ mm}$$

$$\text{Volume of rainwater} = SP_iA_i = 8353.4 \text{ km}^2\text{-mm}$$

$$= 8353.4 \times 10^6 \times 10^3 = 83.534 \times 10^5 \text{ m}^3.$$

Isohyetal Method

- Isohyetal is a line joining points of equal rainfall magnitude. The area between two adjacent isohytes is determined by a planimeter.
- This is the most accurate method.
- Topographic influences are taken into account.
- New isohyets have to be made for each rainfall event.

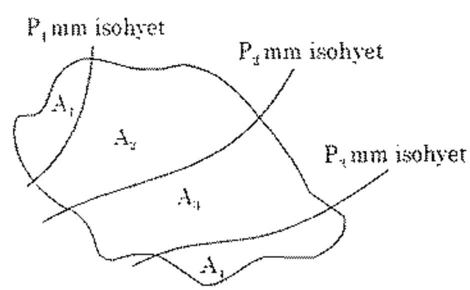
  $P_m = (\Sigma P_{ij}A_{ij})/A$

  P_m is the average rainfall in the catchment

  $P_{ij} = (P_i + P_j)/2$

  A_{ij} is the area between two successive isohytes P_i and P_j

  A is the total area of the catchment.



Area A_2 and A_3 falls between two isohyets each. Hence, these areas may be thought of as corresponding to rainfall depth of $(P_1 + P_2)/2$ and $(P_2 + P_3)/2$ respectively. For Area A_1 we would expect rainfall to be different from P_1, but if no record of rainfall is available beyond the catchment, A_1 may be assume to correspond to P_1 only. If however, rainfall record of area beyond catchment is available we can find the rainfall detph for area, A_1 by interpolation. Similar logic applies for area A_4.

Example 3.7

The isohyets for annual rainfall over a catchment basin were drawn. The areas of strips between the isohyets are given. Determine the average precipitation over the basin.

Isohyets	Area in sq km
75 – 85	1600
85 – 95	3000
95 – 105	2800
105 – 115	1000
115 – 135	900
135 – 155	700

Sol: Average precipitation is given as:

$$P = \frac{A_1\left(\frac{P_1+P_2}{2}\right) + A_2\left(\frac{P_2+P_3}{2}\right) + \dots + A_{n-1}\left(\frac{P_{n-1}+P_n}{2}\right)}{A_1 + A_2 + A_3 + \dots + A_n}$$

$$= \frac{1600\left(\frac{75+85}{2}\right) + 3000\left(\frac{85+95}{2}\right) + 2800\left(\frac{95+105}{2}\right) + 1000\left(\frac{105+115}{2}\right) + 700\left(\frac{135+155}{2}\right)}{1600+3000+2800+1000+900+700}$$

$$= \frac{(1600\times 80 + 3000\times 90 + 2800\times 100 + 1000\times 110 + 900\times 125 + 700\times 145)}{10{,}000}$$

$= 100.2$ cm

Depth Area Duration Relationship

- Depth of rainfall at a raingauge station is called point rainfall. To convert the point rainfall datas to areal rainfall data, [i.e. to find out how much of rainfall will occur over various areas] Depth area duration curve is used.
- To find out the depth area duration curve for a rainfall of particular duration in a catchment. rainfall of that duration is selected and followings steps are taken:

Step 1: From the data of rainfall at different places of a catchment for a particular storm, isohyetal map is prepared.

Step 2: Area between the isohyets are determined using planimeter.

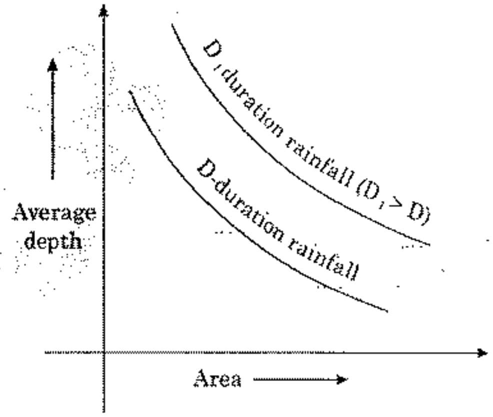
Step 3: Area between two isohyets is multiplied by the average of the corresponding isohyets and hence volume of rainfall in that area is found.

Step 4: Cumulative volume and cumulative area are calculated.

Step 5: Cumulative volume is divided by cumulative area to know the average depth of rainfall over the cumulative area for the chosen duration of rainfall. The same procedure is repeated for other durations also.

Step 6: Average depth for a particular duration is plotted against the cumulative area to obtain the DAD curve for that duration of rainfall.

Thus average depth of rainfall for a particular area can be calculated from DAD curve.



For a given duration of rainfalls (say 6-hr rainfall) column (1), (2), (3) will be known and from this (4), (5), (6), (7) are calculated. Thus curve is plotted with (7) as ordinate and (5) as abscissa.

DAD curve can also be obtained from emperical equation like

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

$\bar{P}$ = Average depth in catchment over an area A (km^2)

P_0 = Highest amount of rainfall in catchment at the storm centre

K, n = constant for a given region. They can be obtained by regression analysis.

But it is unlikely that the storm centre coincides with the raingauge station. Hence exact determination of P_0 is not possible.

However, P_0 can be calculated from the assumption that height rainfall over a raingauge in the catchment is the average depth over an area of 25 km^2. Hence

$$\boxed{P_{max} = P_0 e^{-(K \times 25^n)}}$$

From this P_0 can be calculated.

In a typical depth area curve, depth decrease with increase in area. Similarly analysis of rainfall of larger duration for a given area indicates that depth of rainfall increases as the duration increases.

Calculation of DAD curve is done as follows:

Limits of isohytes (mm) for a rainfall of particular duration	Average of isohytes (mm)	Area (km^2)	Cumulative area (km^2)	Volume of rainfall (km^2)	Cumulative volume ($mmkm^2$)	Average depth (mm)
1	2	3	4	5 = 2 × 3	6	7 = 6/4
90-100	95	0.02	0.02	1.9	1.9	95.00
80-90	85	3.8	3.82	323	324.9	85.05
70-80	75	21.1	24.92	1582.5	1907.4	76.54
60-70	65	149.66	174.58	9727.9	11635.3	66.65
50-60	55	1184.18	1358.76	65129.9	76765.2	56.50
40-50	45	3175.47	4534.23	142896.15	219661.35	48.45
30-40	35	3221.91	7756.14	112766.85	332428.2	42.86
20-30	25	10676.7	18432.84	266917.5	599345.7	32.52
20-10	15	9731.51	28164.35	145972.65	745318.35	26.46
0-10	5	73.57	28237.92	367.85	745686.2	26.41

Now average depth is plotted against cumulative area to get **depth area curve** for a particular duration.

Maximum Depth Area Duration Curve

In the design of hydraulic structures, we need to determine the design flood. The design flood will be determined from the design storm. To find out the design storm we need to know the maximum rainfall of a particular duration over a particular area. This is found from Max. DAD curve.

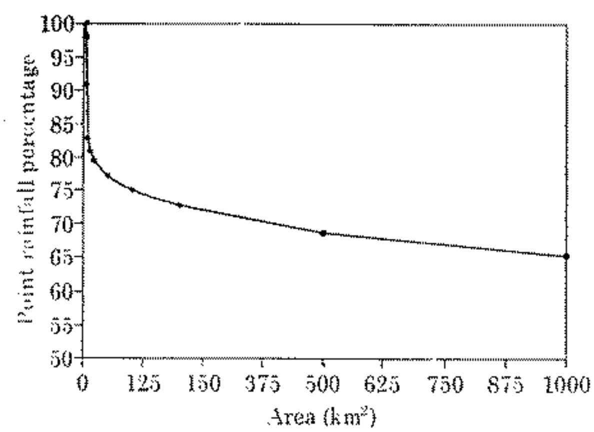
Due to various storms of a particular duration (i.e. say various storms of 24 hr duration) detph area curve is found. Them area is plotted w.r. to Max depth of rainfall corresponding to the various storms of 24 hr duration. This is max DAD curve.

The amount of 24 hours rainfall in mm for different area (i.e. DAD curve of rainfall of various days)

Area (km^2)	2/11/1993 (mm)	23/11/1994 (mm)	13/04/1996 (mm)	17/03/1997 (mm)	18/03/1997 (mm)	Maximum rainfall
10	81.30	46.20	109.20	95.72	72.70	109.20
20	78.80	45.99	107.27	95.19	70.22	107.27
50	75.05	45.58	104.20	94.18	66.65	104.20
100	71.84	45.13	101.42	93.08	63.72	101.42
200	68.20	44.49	98.15	91.58	60.58	98.15
500	62.78	43.23	92.95	88.73	56.12	92.95
1000	58.14	41.83	88.23	85.65	51.52	88.23
2000	52.98	39.90	82.68	81.48	48.69	82.68
3000	49.70	38.45	79.01	78.38	46.37	79.01
4000	47.25	37.24	76.19	75.84	44.69	76.19
5000	45.28	36.19	73.87	73.65	43.36	73.87

When the above maximum depth of rainfall is plotted w.r.t area we get maximum DAD curve. Max. DAD curve can also be plotted in terms of point rainfall percentage with area.

Point rainfall percentage means average depth over an area is how much percentage of point rainfall recorded by rainfall.



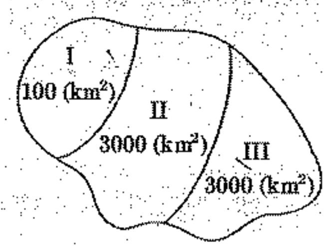
Example 3.8

A catchment is divided into three zones – zone I, zone II and zone III with area of each zone as 100 km^2, 3000 km^2 and 3000 km^2 respectively. Cumulative average rainfall (in mm) in different zones is given as:

Time (hrs)	Zone I	Zone II	Zone III
2	8	5	2
4	14	11	7
6	23	18	12
8	35	28	21
10	48	40	30

Draw the maximum average depth vs. area plot for the duration of 2 hours, 4 hours and 6 hours.

Sol:



5

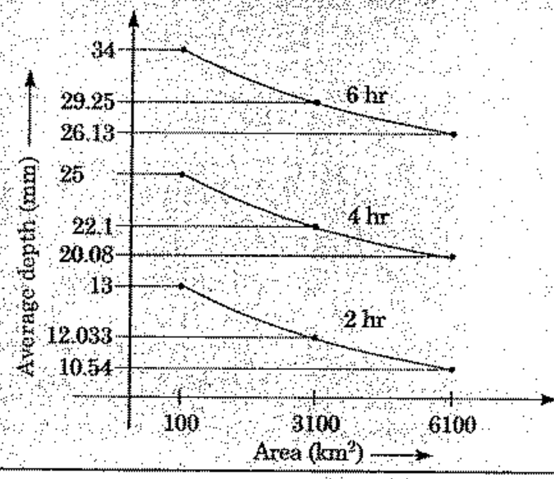
Cumulative average rainfall of accumulated areas in mm

Time	Zone I	Zone II + Zone I	Zone II + Zone III
2	$\frac{8\times100}{100}=8$	$\frac{8\times100+5\times3000}{3100}=5.1$	$\frac{8\times100+5\times3000+2\times3000}{6100}=3.574$
4	14	11.01	9.08
6	23	18.16	15.13
8	35	28.255	24.67
10	48	40.258	35.21

Maximum depth of rainfall in any of the 2 hr, 4 hr and 6 hr rainfall is

	100 km^2	3100 km^2	6100 km^2
2 hr	13 $\begin{bmatrix} \text{Max. of} \\ 14-8=6 \\ 23-14=9 \\ 35-23=12 \\ 48-35=13 \end{bmatrix}$	12.033 $\begin{bmatrix} \text{Max. of} \\ 11.01-5.1=5.91 \\ 18.16-11.01=7.15 \\ 28.225-18.16=10.065 \\ 40.258-28.225=12.033 \end{bmatrix}$	10.54
4 hr	25	22.098	20.08
6 hr	34	29.248	26.13

Maximum DAD Curve for various durations are as shown below.



Frequency of Point Rainfall

In many hydraulic design, we need to know what is the chance of recurrence of a particular rainfall or what is the probability that a particular rainfall will not be exceeded during the design life of the structure.

- Such information is obtained from the frequency analysis of the point rainfall data.
- Rainfall data when arranged in chronological order constitutes a time-series. One may prepare time series like annual series or monthly series etc. of extreme value of an event (like extreme value of 24-hr rainfall). For example if data for maximum magnitude of 24 hr rainfall in a year is collected year over year, we get annual series of extreme value of 24 hr rainfall.
- The purpose of the frequency analysis of an annual series is to obtain a relation between the magnitude of the event and its probability of exceedence.
- If the annual extreme series is arranged in decending order of magnitude and each position given a number 1 to N, 1 being given to the 1st i.e. largest value and N-given to least or last value. Then probability P of a rainfall at position 'm' being equalled or exceeded is given by

$$\boxed{p = \frac{m}{N+1}}$$ (Plotting position approach)

where N = no. allotted to the last position i.e. no. of yr. of record.

Recurrence interval or return period is given by

$$\boxed{T = \frac{1}{p} = \frac{N+1}{m}}$$

Once the probability of exceedence has been determined one can also find out certain other probabilities like:

Method	P
Weibull	m/(N+1)
California	m/N
Hazen	(m − 0.5)/N
Chegodayev	(m−0.3)/(N + 0.4)
Blom	(m−0.44)/(N+0.12)
Gringorten	(m−3/8)/(N+1/4)

Various plotting position formula

(a) Probability of non occurrence = $(1 - P) = q$

(b) Probability of occurrence of event, r, times in n successive yrs = $nC_r P^r q^{n-r}$

$$= \frac{\lfloor n}{\lfloor n-r} P^r q^{n-r}$$

(c) Probability of the event not occuring at all in all n-successive yrs = ${}^nC_0 P^0 q^n$

$$= q^n = (1 - P)^n$$

(d) Probability of the event occuring at least once in n-successive yrs = $1 - q^n = 1 - (1 - P)^n$.

Example 3.9

For a station *A*, the recorded annual 24 h maximum rainfall are given below. (a) Estimate the 24 h maximum rainfall with return periods of 13 and 50 years. (b) What would be the probability of a rainfall of magnitude equal to or exceeding 10 cm occurring in 24 h at station A.

Year

1950	1951	1952	1953	1954	1955	1956	1957	1958	1959	1960	1961
1962	1963	1964	1965	1966	1967	1968	1969	1970	1971		

Year rainfall (cm)

13.0	12.0	7.6	14.3	16.0	9.6	8.0	12.5	11.2	8.9	8.9	7.8
9.0	10.2	8.5	7.5	6.0	8.4	10.8	10.6	8.3	9.5		

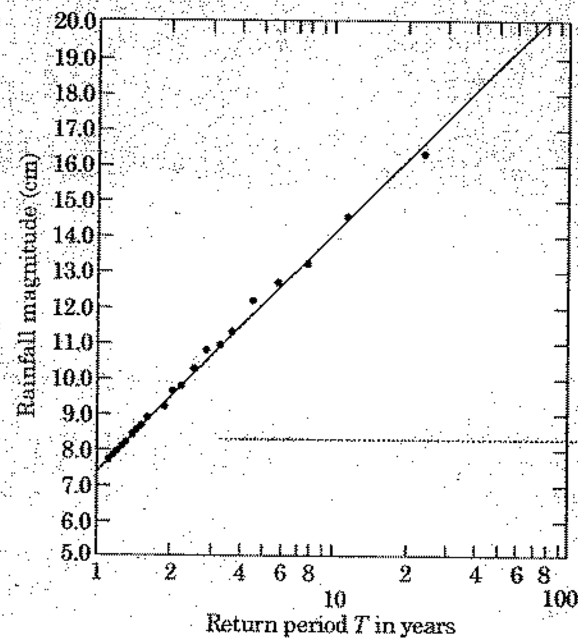
Sol: The data are arranged in descending order and the probability and recurrence intervals of various events are calculated as indicated in Table.

It may be noted that when two or more magnitudes are same (as m = 13 and 14 in Table) *P* is calculated for the largest m value of the set.

m	Rainfall (cm)	Probability $P = \frac{m}{N+1}$	Return period $T = 1/P$ (years)
1	16.0	0.043	23.00
2	14.3	0.087	11.50
3	13.0	0.130	7.67
4	12.5	0.174	5.75
5	12.0	0.217	4.60
6	11.2	0.261	3.83
7	10.8	0.304	3.29
8	10.6	0.348	2.88
9	10.2	0.391	2.56
10	9.6	0.435	2.30
11	9.5	0.478	2.09
12	9.0	0.522	1.92
13	8.9	—	—
14	8.9	$0.609\left(\frac{14}{23}\right)$	1.64
15	8.5	0.652	1.53
16	8.4	0.696	1.44
17	8.3	0.739	1.35
18	8.0	0.783	1.28
19	7.8	0.826	1.21
20	7.6	0.870	1.15
21	7.5	0.913	1.10
22	6.0	0.957	1.05

These value can be plotted with rainfall magnitude and return period and data for 13 yr and 50 yr return period can be obtained by interpolation and extrapolation respectively.

Return period T years	Rainfall magnitude (cm)
13	14.55
50	18.00



For rainfall = 10 cm, T = 2.4 years and P = 0.417. (As obtained from graph or by interpolation)

Example 3.10

Annual precipitation values at a place having 70 years of record can be tabulated as follows:

Range (cm)	Number of years
<60.0	6
60.0–79.9	6
80.0–99.9	22
100.0–119.9	25
120.0–139.9	8
>140.0	3

Calculate the probability of having:

(i) An annual rainfall equal to or larger than 120 cm.

(ii) Two successive years in which the annual rainfall is equal to or greater than 140 cm.

(iii) An annual rainfall less than 60 cm.

Sol: Rearranging the data in descending order

Rainfall (cm)	No. of years	Plotting position (m)
> 140	3	3
120 – 139.9	8	11
100 – 119.9	25	36
80 – 99.9	22	58
60 – 79.9	6	64
< 60	6	70

(i) Probability of annual rainfall $\geq$ 140 cm

$$= \frac{11}{70+1} = 0.155$$

(ii) Probability that rainfall exceeds 140 cm $= \frac{3}{71} = 0.042$

Probability that rainfall > 140 cm occurs twice in two successive yrs $= {}^2C_2\, P^2\, q^0$

$$= P^2 = (0.042)^2 = 0.0018$$

(iii) Probability that rainfall is less than 60 cm

$$= \frac{70}{71} \times 100 = 0.986$$

Example 3.11

Using 23 yr of annual precipitation depth for Darbhanga, estimate the exceedance frequencies and recurrence intervals of the highest ten values using plotting position approach.

yr	Annual rainfall (cm)
2000	9
1999	$\boxed{19}$
1998	$\boxed{19}$
1997	9
1996	8
1995	6
1994	15
1993	$\boxed{20}$
1992	11
1991	9
1990	$\boxed{18}$
1989	8
1988	$\boxed{23}$
1987	17
1986	$\boxed{23}$
1985	17

1984	10
1983	$\boxed{18}$
1982	5
1981	$\boxed{24}$
1980	$\boxed{19}$
1979	15
1978	$\boxed{21}$

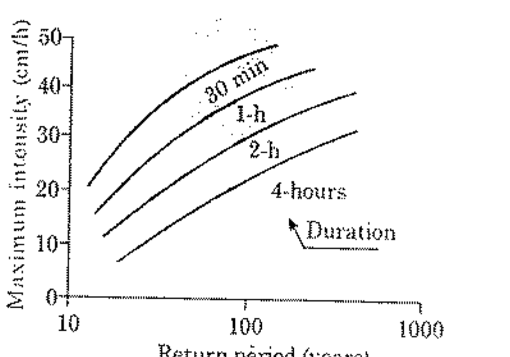
Sol: Ten highest flow rates are tabulated below. By ranking them from the highest to lowest, the value of P becomes exceedance probability

yr	Rain depth (cm)	Rank (m)	$P = \frac{m}{N+1} = \frac{m}{23+1}$	$T_r(\text{yr}) = \frac{1}{P}$
1981	24	1	$\frac{1}{24} = 0.042$	24
1986	23	3	—	—
1988	23	3	0.125	8
1978	21	4	0.167	6
1993	20	5	0.208	4.8
~~1999~~	~~19~~	~~8~~	—	—
1998	19	8	—	—
1980	19	8	0.333	3
1990	18	10	—	—
1983	18	10	0.417	2.4

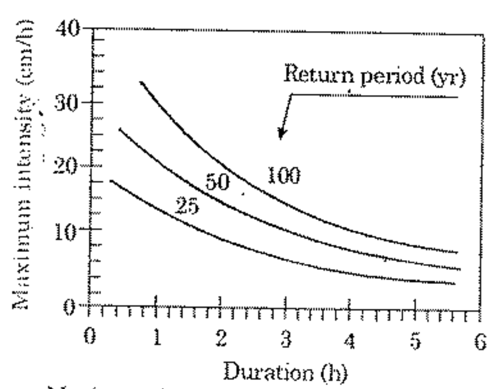
Intensity Duration Frequency Curves

Intensity duration frequency curve estimates the rainfall intensities of different durations and recurrence interval. These curves are used by engineers for risk assessment of dams, bridges, water drainage system, storm sewers, runoff canals etc.

- They can also be used as a prediction tool to identify when a certain rainfall rate or a specific volume of flow will recur in the future that will creat flooding havoc in an area.



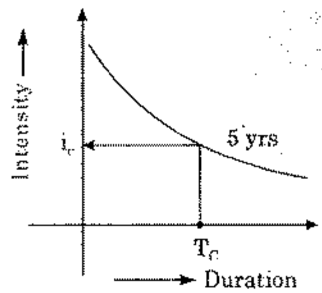
Maximum intensity return period duration curves



Maximum intensity-duration-frequency curves

- IDF curve is most often used to express the severity of a single rainfall event.
- Curves are used in design with the assumption that past rainfall statistics continue to represent rainfall statistics into the future.
- A simple use of IDF curve is illustrated as follows. Suppose one has to determine the design discharge of a storm sewer with a consideration that its return period is not less than 5 yrs. i.e. the risk of storm sewer getting overflooded is acceptable once in 5 yrs.
- As per rational formula, runoff is maximum when the rainfall has duration equal to or more than the time of concentration of the catchment i.e. the time when entire catchment starts contributing to the discharge at outlet.

Hence critical rainfall intensity is determined from IDF curve corresponding to duration equal to time of concentration and frequency or return period equal to 5 yrs.



From this intensity (i_c), using rational formula $Q_{design} = CiA$ [C = runoff coefficient = $\frac{\text{runoff}}{\text{rainfall}}$, A = Area of catchment], design discharge is calculated.

IDF curve is commonly expressed as:

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

where i = intensity, D = duration, T = return period

K, a, x, n are constants for a given catchment.

Example 3.12

Time since start (in min)	Cumulative rainfall (in mm)
0	0
30	6
60	18
90	21
120	36
150	43
180	49
210	52
240	53
270	54

Plot the maximum intensity vs. duration curve and maximum depth vs. duration curve.

Sol:

Time since start (min)	Cumulative rainfall	Rainfall (mm) in any possible time interval equal to					
		30 min	60 min	90 min	120 min	150 min	180 min
0	0						
30	6	6					
60	18	12	18				
90	21	3	15	21			
120	36	$\boxed{15}$	18	$\boxed{30}$	36		
150	43	7	$\boxed{22}$	25	$\boxed{37}$	$\boxed{43}$	
180	49	6	13	28	31	43	$\boxed{49}$
210	52	3	9	16	31	34	46
240	53	1	4	10	17	32	35
270	54	1	2	5	11	18	33

Max intensity for 30 min duration $= \dfrac{15}{0.5} = 30$ mm/hr

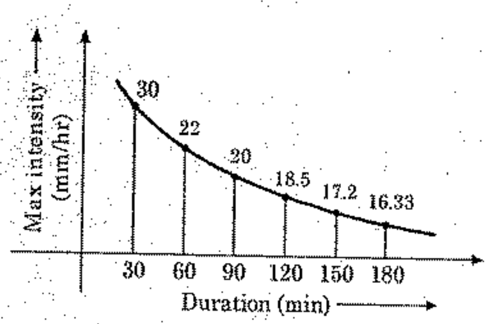
Max intensity for 60 min duration $= \dfrac{22}{1} = 22$ mm/hr

Max intensity for 90 min duration $= \dfrac{30}{1.5} = 20$ mm/hr

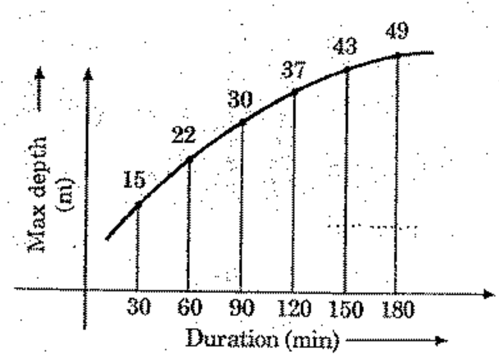
Max intensity for 120 min duration $= \dfrac{37}{2} = 18.5$ mm/hr

Max intensity for 150 min duration $= \dfrac{43}{2.5} = 17.2$ mm/hr

Max intensity for 180 min duration $= \dfrac{49}{3} = 16.33$ mm/hr



Max intensity duration curve



Max depth duration curve

Probable Maximum Precipitation

The probable maximum precipitation (PMP) is defined as the greatest or extreme rainfall of a given duration that is physically possible over a station or basin.

DAD of PMP is usually derived by taking the results of maximum depth area duration analysis and adjusting them for most favourable hydrometerological conditions that is possible to maximise the rainfall. From this DAD of PMP is obtained.

PMP be can be statistically estimated as:

$$\boxed{PMP = \bar{P} + K\sigma}$$

$\bar{P}$ = Mean annual rainfall series

K = Frequency factor

σ = Standard deviation of series

PMP is used for design of large hydraulic structures like spillway of large dams such that there is virtually no probability of failure.

Standard Project Storm (SPS)

For design of major and intermediate structure SPS is used. SPS is the greatest storm that may reasonably be expected without modifying the rainfall data for favourable hydrometerological conditions as was done in PMP.

Max Rainfall Observed (in any part of the world)

$$P_m = 42.16\ D^{0.475}$$

P_m = extreme rainfall depth (cm)

D = Duration in (hr)

□□□□

OBJECTIVE QUESTIONS

1. An accurate estimate of average rainfall in a particular catchment area can be obtained by
   (a) arithmetic mean method (b) isohyetal method
   (c) normal ratio method (d) THIESSEN method

2. The percentage standard error of precipitation averages is often expressed functionally or graphically in terms of (i) precipitating gauge network density expressed as area per gauge, and (ii) total area of catchment. The percentage standard error
   (a) increases with area per gauge as well as with total area
   (b) decreases with area per gauge as well as with total area
   (c) increases with area per gauge but decreases with total area
   (d) decreases with area per gauge but increases with total area

3. If 'p' is the precipitation, 'a' is the area represented by a rain gauge, and 'n' is the number of rain gauges in a catchment area, then the weighted mean rainfall is

   (a) $\frac{\Sigma ap^3}{\Sigma a^2}$ (b) $\frac{\Sigma ap}{n}$

   (c) $\frac{\Sigma ap}{\Sigma a}$ (d) $\frac{\Sigma ap^5}{\Sigma a^3}$

4. Depth-Area-Duration curves of precipitation are drawn as
   (a) minimizing envelopes through the appropriate data points
   (a) maximizing envelopes through the appropriate data points
   (c) best fit mean curves through the appropriate data points
   (d) best fit mean straight lines through the appropriate data points

5. Mean precipitation over an area is best obtained from gauged amounts by
   (a) arithmetic mean method
   (b) Thiessen method
   (c) linearly interpolated isohyetal method
   (d) orographically weighted isohyetal method

6. Depth-Area-Duration curves would seem to resemble
   (a) arcs of circle concave upwards with duration increasing outward
   (b) first quadrant limbs of hyperbolae with duration increasing outward
   (c) third quadrant limbs of hyperbolae with duration decreasing outward
   (d) first quadrant limbs of hyperbolae with duration decreasing outward

7. The following rainfall data refers to station and B which are equidistant from station X:

	Station A	Station X	Station B
Long-term normal annual rainfall in mm	200	250	300
Annual rainfall in mm for the year 1940	140	P	270

   The value of P will be
   (a) 250 (b) 220
   (c) 205 (d) 200

8. In a water-shed, four raingauges I, II, III and IV are installed. The depths of normal annual rainfall at these stations are 60, 75, 80 and 100 cm respectively. The rain gauge at station III went out of order during a particular year. The annual rainfall for that year, recorded at the remaining three stations was 90, 60 and 70 cm. The rainfall at station III can be considered as
   (a) 60 cm (b) 70 cm
   (c) 80 cm (d) 120 cm

9. The moving average of annual precipitation record is carried out to determine
   (a) Trend
   (b) Annual mean
   (c) Extreme annual variation
   (d) Extreme seasonal variation

10. Match List-I (Hydrological terms) with List-II (Relationship/Nature of curve) and select the correct answer using the codes given below the lists:

List-I

A. Theissen Polygon
B. Mass curve
C. Hyetograph
D. DAD curve

List-II

1. Average depth of rainfall over an area
2. Relationship of rainfall intensity and time
3. Relationship of accumulated rainfall and time
4. Relationship of river run-off and time
5. Always a falling curve

Codes:

	A	B	C	D
(a)	1	3	2	5
(b)	1	5	3	2
(c)	4	3	2	5
(d)	4	5	3	2

11. Match **List-I** (Type of precipitation) with **List-II** (Principal causes) and select the correct answer using the codes given below the lists:

List-I

A. Convective
B. Mountain barrier
C. Frontal
D. Orographic

List-II

1. Atmospheric disturbance
2. Mountain barrier
3. Pressure difference
4. Temperature difference
5. Warm and cold air masses

Codes:

	A	B	C	D
(a)	1	4	5	2
(b)	4	3	5	2
(c)	1	4	2	5
(d)	4	3	2	5

12. A catchment area of 90 hectares has a run-off coefficient of 0.4. A storm of duration larger than the time of concentration of the catchment and of intensity 4.5 cm/hr creates a peak discharge rate of

(a) 11.3 m^3/s (b) 0.45 m^3/s
(c) 450 m^3/s (d) 4.5 m^3/s

13. The area between the two isohyetes 45 cm and 55 cm is 100 km^2, and that between 55 cm is 100 km^2, and that between 55 cm and 65 is 150 km^2. What is the average depth of annual precipitation over the basin of 250 km^2?

(a) 50 cm (b) 52 cm
(c) 56 cm (d) 60 cm

14. For which one of the following purposes is the double mass curve used?
    (a) Checking on the consistency of precipitation records.
    (b) Prediction of annaul precipitation.
    (c) Defining which periods of storm should be analyzed to obtain the maximum useful information from storm rainfall records.
    (d) For estimating the capacity of a reservoir.

15. Which one of the following is not a major type of storm precipitation?
    (a) Frontal storm (b) Air mass storm
    (c) Orographic storm (d) Continental storm

16. Double mass curves are used
    (a) to ckeck on the consistency of precipitation records
    (b) as basis for storm rainfall analysis
    (c) to determine average rainfall over an area
    (d) to indicate rainfall distribution

17. What is the Probable Maximum Precipitation (PMP)?
    (a) Projected precipitation for a 100 year return period
    (b) Maximum precipitation for all past recorded storms
    (c) Upper limit of rainfall, which is justified climatologically
    (d) Effective precipitable water

18. The rainfall hyetograph shows the variation of which one of the following?
    (a) Cumulative depth of rainfall with time
    (b) Rainfall depth with area
    (c) Rainfall intensity with time
    (d) Rainfall intensity with cumulative depth of rainfall

19. The quantitative statement of the balance between water gains and losses in a certain basin during a specified period of time is known as which one of the following?
    1. Water budget
    2. Hydrologic budget
    3. Groundwater budget

    Select the correct answer using the codes given below:
    (a) 1 only (b) 2 only
    (c) 3 only (d) none of these

20. From the analysis of rainfall data at a particular station, it was found that a rainfall of 400 mm had a return period of 20 years. What is the probability of rainfall equal to or greater than 400 mm occurring at least once in 10 successive years?
    (a) $(0.95)^{10}$ (b) $1-(0.95)^{10}$
    (c) $1-(0.05)^{10}$ (d) $(0.05)^{10}$

21. Inconsistency of rainfall data can be checked by which one of the following?
    (a) Normal ratio method
    (b) Mass curve method
    (c) Double-mass curve method
    (d) Depth duration frequency curve

22. A 4 hr storm had 4 cm of rainfall and the resulting direct runoff was 2.0 cm. If the ϕ index remains at the same value, the runoff due to 10 cm of rainfall in 8 hr in the catchment is

(a) 6.0 cm (b) 7.5 cm
(c) 2.3 cm (d) 2.8 cm

23. What is 'Hydrological Cycle'?
(a) Processes involved in the transfer of moisture from sea to land
(b) Processes involved in the transfer of moisture from sea back to sea again
(c) Processes involved in the transfer of water from snowmelt in mountains to sea
(d) Processes involved in the transfer of moisture from sea to land and back to sea again

24. Consider the following with respect to 'double mass curve':
1. Plot of accumulated rainfall with respect to two chronological orders
2. Plot for estimating multiple missing rainfall data
3. Plot for checking the consistency of the rainfall data
4. Plot of accumulated annual rainfall of a station vs. accumulated rainfall of a group of stations

Which of these statements are correct?
(a) 1 and 3 (b) 2 and 3
(c) 3 and 4 (d) 1 and 4

25. Generally to estimate PMP, $P_m = 42.16D^{0.475}$ is used (P_m is maximum depth of precipitation, D = duration). What are the units of P_m and D in the equation?
(a) mm, sec (b) cm, sec
(c) mm, hr (d) cm, hr

26. Ombrometer (pluviometer) is used to measure
(a) soil moisture stress of a plant
(b) rainfall depth
(c) leaf area
(d) root zone depth

27. The coefficient of variation of the rainfall for six rain gauge stations in catchments was found to be 29.54%. The optimum number of stations in the catchments for an admissible 10% error in the estimation of the mean rainfall will be
(a) 3 (b) 6
(c) 9 (d) 12

28. The intensity of rainfall and time interval of a typical storm are:

Time interval (minutes)	Intensity of rainfall (mm/minute)
0-10	0.7
10-20	1.1
20-30	2.2
30-40	1.5
40-50	1.2
50-60	1.3
60-70	0.9
70-80	0.4

The maximum intensity of rainfall for 20 minutes duration of the storm is

(a) 1.5 mm/minute (b) 1.85 mm/minute

(c) 2.2 mm/minute (d) 3.7 mm/minute

ANSWERS

1. (b)	9. (a)	17. (c)	25. (d)
2. (a)	10. (a)	18. (c)	26. (b)
3. (c)	11. ~~(a)~~ (b)	19. (a)	27. (c)
4. (b)	12. (b)	20. (b)	28. (b)
5. (d)	13. (d)	21. (c)	
6. (b)	14. (c)	22. (a)	
7. (d)	15. (a)	23. (d)	
8. (c)	16. (d)	24. (c)	

CHAPTER

4

Abstractions from Precipitation

Introduction

- In engineering hydrology, runoff is the prime subject of study. Before the rainfall reaches the outlet of a basin in the form of runoff, a part of the rainfall is lost through various processes, such as, Evaporation (E), Transpiration (T), Interception (I), Depression Storage (DS) and Infiltration (IL).
- These processes E, T, I, DS and IL are termed as abstraction from precipitation.
- Generally, transpiration is studied in conjunction with evaporation from an area, hence the term *evapotranspiration* is more commonly used.

Evaporation

- The process of transformation of liquid water into gaseous form is called evaporation.
- The rate of evaporation is dependent on (i) the vapour pressures at the water surface and air above, (ii) air and water temperatures, (iii) wind speed, (iv) atmospheric pressure, (v) quality of water, (vi) depth of water body and (vii) shape and size of water body.

Vapour Pressure

The rate of evaporation is proportional to the difference between the saturation vapour pressure (e_w) at the existing water temperature, and the existing actual vapour pressure in the air, e_a.

The relationship is given by:

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

where, E_L = rate of evaporation (mm/day), C = a constant; and e_w and e_a are in mm of mercury. Evaporation continues till $e_w = e_a$; and if $e_w < e_a$, condensation takes place.

Temperature

The other factor remaining same, the rate of evaporation increases with an increase in the water temperature.

Wind

- Wind aids in removing the evaporated water vapour from the zone of evaporation hence increase in wind speed increases the scope of evaporation.
- However, if the wind velocity is large enough to remove all the evaporated water vapour (critical wind speed), any further increase in wind velocity does not influence the evaporation.

- This critical wind-speed value is a function of the size of the water surface. For large water bodies high speed turbulent winds are needed to cause maximum rate of evaporation.

Atmospheric Pressure

Other factors like heat input remaining same, a decrease in the barometric pressure, as in high altitudes, increases evaporation.

Water Quality

The rate of evaporation from water surfaces exposed to identical climatic conditions may vary according to the quality of water. For example, evaporation decreases by about 1 per cent for every 1 per cent increase in salinity, so that evaporation from sea water with an average salinity of about 3.5 per cent is some 2 to 3 per cent less than evaporation from fresh water at the same temperature.

Depth of Water Body

- For shallow water body seasonal temperature of water matches with that of air above. This means that maximum rates of evaporation from a shallow water body will be experienced during the summer months.

  In the case of a large deep water body, however, water temperatures commonly lag behind the temperatures of the overlying air.
- During the spring and early summer months considerable depths of water are slowly and gradually warmed up by a part of the incoming solar energy which would otherwise be available for evaporation. Subsequently the slow release of this stored heat, by the deep water body during the autumn and winter months, means that a supply of heat energy in excess of that received directly from the sun is made available for evaporation at that time of the year.
- Hence highest rates of evaporation from deep water bodies occurs during the winter. Furthermore, during winters, water vapour-laden air will be rapidly lifted away from the underlying water surface as a result of convectional activity, encouraged by the temperature gradient, whereas during the summer, the colder water will tend to cool and stabilize the air immediately above it and so inhibit the removal of vapour laden air.
- However, the effect of heat storage is essentially to change the seasonal evaporation rates and the annual evaporation rate is seldom affected.

Size and Shape of Water Surface

Air moving across a large lake has a low water vapour content at the upwind edge and evaporation from the lake surface will gradually increase the water vapour content. Thus a vapour blanket is created over the lake, the thickness of which increases in windward direction. There will be decrease in the rate of evaporation as the vapour blanket in contact with water surface increases in thickness. Thus, the larger the lake, the greater will be the total reduction in evaporation.

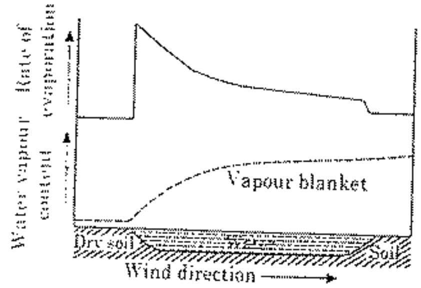


Fig. 4.1. Increasing humidity and decreasing evaporation as air moves across a large water surface.

In the case of a continuous water surface, e.g., the oceans, evaporation will be uniform over much larger areas

But a small evaporating surfaces such as evaporimeters and pans exert little influence on the humidity of the overlying air. The small amount of water vapour which leaves the surface, even with higher rates of evaporation, is quickly dissipated as more dry air moves in and, in this way, a continuous high rate of evaporation is maintained.

Evaporation Measurement

The amount of water evaporated from a water surface is estimated by the following methods:

(i) Using evaporimeter data,

(ii) Using empirical evaporation equations, and

(iii) Analytical methods.

Evaporimeters are water-containing **pans** which are exposed to the atmosphere, and the loss of water by evaporation in them is measured at regular intervals.

Types of Evaporimeter

- Class A Evaporation pan (US Weather Bureau)
- ISI Standard Pan (Used in India)
- Colorado Sunken Pan (Pan is sunk below ground such that water level in Pan is at ground level)
- US Geological Survey Floating Pan (Simulates the characterization of large water body. The evaporimeter is kept floating in lake).

Pan Coefficient, C_p

Evaporation pans are not exact models of large reservoirs.

In view of the above, the evaporation observed from a pan has to be corrected to obtain the value of evaporation from a lake under similar climatic and exposure conditions. Thus, a coefficient (C_p) is introduced as shown below:

$$\boxed{\text{Lake evaporation} = C_p \times \text{pan evaporation}}$$

in which, C_p = pan coefficient. The values of C_p in use for different pans are given in Table.

Sl.No.	Types of plan	Average value	Range
1	Class A Land Pan	0.70	0.60-0.80
2	ISI Pan (Modified Class A)	0.80	0.64-1.10
3	Colorado Sunken Pan	0.78	0.75-0.86
4	US GS Floating Pan	0.80	0.70-0.82

Evaporation Stations

It is usual to install evaporation pans at such locations where other meterological data are also simultaneously being collected. As per WMO recommendation.

1. Arid zones — Min of one station for every 30,000 km^2,
2. Humid temperate climates — Min of one station for every 50,000 km^2, and
3. Cold regions — Min of one station for every 100,000 km^2

1. Estimation of Evaporation

Methods of estimation of evaporation may be grouped into two categories: (1) empirical formulae, and (2) analytical methods (water budget method, energy balance method, mass-transfer method).

2. Empirical Formulae

$$E_L = K\, f(u)\, (e_w - e_a)$$

where, E_L = lake evaporation in mm/day; e_w = saturated vapour pressure at the water-surface temperature, in mm of mercury; e_a = actual vapour pressure of overlying air at a specified height, in mm of mercury; $f(u)$ = wind speed correction function; and K is a coefficient. The term e_a is measured at the same height at which wind speed is measured. The commonly used empirical evaporation formulae is:

3. Meyer's Formula (1915)

$$E_L = K_M (e_w - e_a)\left(1 + \frac{u_9}{16}\right)$$

in which, E_L, e_w, e_a are as defined above; u_9 = monthly mean wind velocity in km/h at about 9 m above the ground; K_M = 0.36 for large deep and 0.50 for small, shallow waters.

Often, the wind-velocity data would be available at an elevation other than that needed in the particular equation. However, it is known that in the lower part of the atmosphere, up to a height of about 500 m above the ground level, the wind velocity can be assumed to follow the 1/7 power law as

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

where u_h = wind velocity at a height h (in meter) above the ground in km/hr and C = constant. This equation can be used to determine the velocity at any desired level if u_h is known.

Example 4.1

A reservoir with surface area of 250 hectares has saturation vapour pressure at water surface = 17.54 mm of Hg and actual vapour pressure of air = 7.02 mm. Wind velocity at 1 m above ground surface = 16 km/h. Estimate the average daily evaporation from the lake using Meyer's formula.

Sol:

$$e_w = 17.54 \text{ mm of Hg}$$
$$e_a = 7.02 \text{ mm}$$

Wind speed of 9 m above ground = u_9

$$\Rightarrow \frac{u_1}{u_9} = \frac{c(1)^{1/7}}{c(9)^{1/7}}$$

$$\Rightarrow u_9 = (9)^{1/7} \times u_1$$

$$= (9)^{1/7} \times 16 = 21.9 \text{ km/hr}$$

hence using Mayer's formula

$$E_L = 0.36\,(17.54 - 7.02)\left(1 + \frac{21.9}{16}\right)$$

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

2. Analytical Methods

The analytical methods for the determination of lake evaporation can be broadly classified into three categories as:

(i) Water-budget method

(ii) Energy balance method and

(iii) Mass transfer method

(i) Water-Budget Method

The water budget method is the simplest of the three analytical methods and is also the least reliable. In this method we write the hydrological continuity equation for the lake and determine the evaporation from the knowledge or estimation of other variables.

$$\boxed{P + V_{is} + V_{ig} = V_{os} + V_{og} + E_L + \Delta S + T_L}$$

where P = daily precipitation

V_{is} = daily surface inflow into the lake

V_{ig} = daily groundwater inflow

V_{os} = daily surface outflow from the lake

V_{og} = daily seepage outflow

E_L = daily lake evaporation

ΔS = increase in lake storage in a day

T_L = daily transpiration loss

(ii) Energy-Balance Method

The energy budget method uses the law of conservation of energy. The energy available for evaporation is determined by considering the incoming energy, outgoing energy, and energy stored in the water body over a known time interval.

From the energy available for evaporation, the value of evaporation rate is calculated.

$$\boxed{H_n = H_a + H_e + H_g + H_s + H_i}$$

where H_n = net heat energy received by the water surface.

$H_n = H_c\,(1 - r) - H_b$

$H_c(1 - r)$ = incoming solar radiation into a surface

H_b = back radiation

r = reflection coefficient

H_a = sensible heat transfer from water surface to air,

H_g = heat flux into the ground.

H_s = heat stored in water body.

H_i = net heat conducted out of the system by water flow (advected energy), and

H_e = heat energy used up in evaporation,

$\approx \rho L E_L$

where, ρ = density of water

L = latent heat of evaporation, and

E_L = evaporation in mm

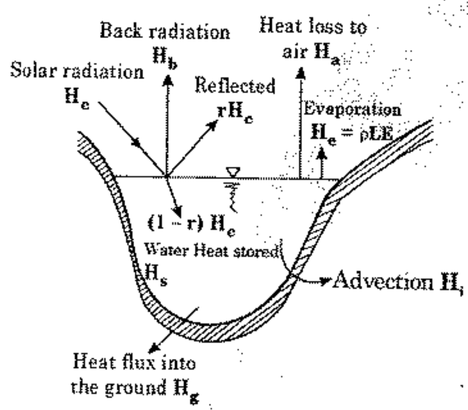


Fig. 4.2. Energy balance in a water body.

All the energy terms are in calories per square mm per day. If the time periods are short, the terms H_s and H_i can be neglected as negligibly small. All the terms except H_a can either be measured or evaluated indirectly. The sensible heat term H_a, which cannot be readily measured is estimated using Bowen's ratio β given by the following expression:

$$\boxed{\beta = \frac{H_a}{\rho L E_L} = 6.1 \times 10^{-4} \times p_a \frac{(T_w - T_a)}{(e_w - e_a)}}$$

where p_a = atmospheric pressure in mm of mercury, e_w = saturated vapour pressure in mm of mercury, e_a = actual vapour pressure of air in mm of mercury, T_w = temperature of water surface in °C, and T_a = temperature of air in °C. Hence E_L can be evaluated or:

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

Example 4.2

Calculate by energy balance method, the evaporation rate from an open water surface, if the net radiation is 200 W/m^2 and the temperature is 25°C, assuming no sensible heat or ground heat flux.

At 25°C, density of water is 997 kg/m^3, latent heat of vaporisation = 2441/kj/kg.

Sol: It is assumed that the temperature of water is constant in time, and the only change in the heat that is stored in a volume of water is the change in the internal energy of water that is evaporated.

Since all the terms are zero or negligibly small for short time periods i.e., H_s, H_i, H_a and H_g except H_n and noting that H_e is equal to $\rho L\ E_L$, we have:

$$E_L = \frac{H_n}{L \times \rho}$$

$$E_L = \frac{200\ (\text{W/m}^2)}{2441 \times 10^3\ (\text{J/kg}) \times 997\ (\text{kg/m}^3)}$$

$$= 8.22 \times 10^{-8}\ \text{m/s}\quad (\text{J} = \text{N}\times\text{m, and W} = \frac{\text{N}-\text{m}}{\text{s}})$$

$$= 8.22 \times 10^{-8} \times 1000 \times 86400\ \text{mm/day}$$

$$= 7.10\ \text{mm/day}$$

(iii) Mass-Transfer Method

This method, based on theories of turbulent mass transfer in a boundary layer, allows to calculate the transfer of mass of water vapour from the surface to the surrounding.

Lake Evaporation and its Reduction

The volume of water lost due to evaporation from a reservoir in a month is calculated by the formula:

$$V_E = AE_{pm}\ C_p$$

V_E = volume of water lost in evaporation during a months (m^3),

A = average reservoir area (i.e., water spread) during the month (m^2),

E_{pm} = pan evaporation loss in metres in a month,

= (E_L in mm/day) × (No of days in the month × 10^3), and

C_p = relevant pan coefficient.

The various methods available for reduction of evaporation losses can be considered under three categories, such as:

(i) Reduction of Surface Area

Since the volume of water lost by evaporation is directly proportional to the surface area of the water body. Deep reservoirs in place of wider ones will reduce evaporation losses.

(ii) Mechanical Covers

Permanent roof/temporary roof reduces lake evaporation.

(iii) Chemical Films

Certain chemicals such as cetyle alcohol (hexadecan) and stearyle alcohol (octadecanol) form layers on a water surface. These layers act as evaporation inhibitors by preventing the water molecules to escape past them. Application of a thin chemical film on the water surface reduces evaporation.

Evapotranspiration

Transpiration has been defined as the process by which water vapour escapes from the living plant, principally through its leaves, into the atmosphere.

In an area covered with vegation it is difficult and also unnecessary from practical view point to separately evaluate evaporation and transpiration. It is more convenient to estimate the evapotranspiration directly. Only over those areas of earth's surface where no vegetation is present will purely evaporation occur. Evapotranspiration represents the most important aspect of water loss in the hydrologic cycle.

The term *consumptive use* is also used to denote this loss by evapotranspiration. If sufficient moisture is always available to completely meet the needs of vegetation fully covering the area, the resulting evapotranspiration is called *potential evapotranspiration* (PET). Potential evapotranspiration does not depend on soil and plant factors but depends essentially on climatic factors. The real evapotrans-piration occurring in a specific situation is called *actual evapotranspiration* (AET).

Field capacity is the maximum quantity of water that the soil can retain against the force of gravity. Permanent wilting point is the moisture content of a soil at which the plant wilts and does not recover in a humid climate. At this stage, even though the soil contains some moisture, it will be so held by the soil grains that the roots of the plants are not able to extract it in sufficient quantities to sustain the plants and consequently the plants wilt. The field capacity and permanent wilting point depend upon the soil characteristics. The difference between these two moisture contents is called *available water* (the moisture available for plant growth).

- If the soil moisture is at the field capacity AET = PET. If the water supply is less than PET, the soil dries out and the ratio AET/PET would then be less than unity.
- The decrease of the ratio AET/PET with available moisture depends upon the type of soil and rate of drying. Generally, for clayey soils, AET/PET ≈ 1.0 for nearly 50% drop in the available moisture. When the soil moisture reaches the permanent wilting point, the AET reduces to zero.

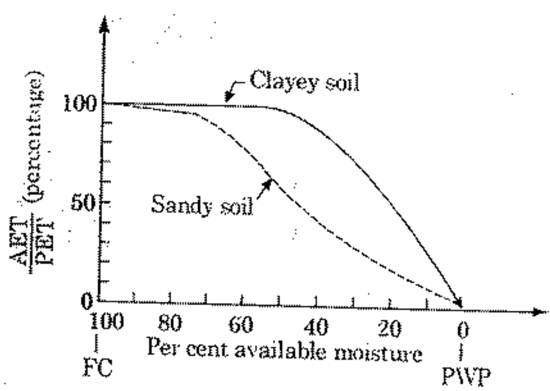


Fig. 4.3. Variation of AET.

Measurement of Actual Evapotranspiration

The measurement of evapotranspiration for a given vegetation type can be carried out in two ways: either by using lysimeters or by the use of field plots.

Lysimeter

A lysimeter (also known as *evapotranspirometer*) consists of a circular tank filled with soil and individual crops or natural vegetation, for which the evapotranspiration is required. It is buried so that its top is flush with the surrounding ground surface. The sides of the lysimeter are impervious whereas the bottom is pervious. Water passing through the soil column is collected at the bottom and conducted through a small tube to a measuring gauge in an adjacent pit. Evapotranspiration is calculated as:

$$\text{AET} = W_{Si} + W_{ad} - W_c - W_{sf}$$

W_{Si} = original wt of container + soil + plant + water moisture

W_{ad} = water added

W_c = water collected at bottom

W_{sf} = final wt of container content

Field Experimental Plots

- In this method an irrigation plot is chosen and the amounts of water added to the irrigation plot by way of precipitation and irrigation are measured along with runoff.
- The moisture content in various layers of the soil within the root zone depth are measured both at beginning and end of the crop season. Then the evapotranspiration is computed as:

$$ET = I - Q - \Delta S$$

where I is the total inflow in mm including precipitation and irrigation water, Q is the total surface runoff in mm and ΔS is the increase in soil moisture storage in mm.

Estimation of Potential Evapotranspiration

Potential evapotranspiration is found out using Penman's equation and some emperical formulae:

Penman's equation is, based on combination of the energy balance and mass-transfer approaches.

$$\boxed{PET = \frac{A(H_n) + E_a\gamma}{A + \gamma}}$$

PET = daily potential evapotranspiration, in mm per day,

A = slope of the saturation vapour pressure vs temperature curve at the mean air temperature, in mm of mercury per °C

H_n = net radiation, in mm, of evaporable water per day,

E_a = parameter including wind velocity and saturation deficit, and

γ = psychometric constant = 0.49 mm of mercury/°C.

Empirical Formulae

Blaney-Criddle formula

This is a purely empirical formula based on data from arid western United States. This formula assumes that the PET is related to the hours of sunshine and temperautre, which are taken as a measure of solar radiation on a given area. The potential evapotranspiration in a crop-growing season is given by:

$$E_t = 2.54K\,\Sigma\left\{P_h \times T_f/100\right\}$$

E_r = PET in a crop season, in cm,

K = an empirical coefficient, depending on the type of the crop, month and locality

Σ = sum of monthly consumptive use factors for the period,

P_h = monthly percent of annual day-time hours, depending on the latitude of the place and

T_f = mean monthly temperature, in °F.

Example 4.3

Estimate the PET of an area for the season November to February in which wheat is grown. The area is in North India at a latitude of 30°N with mean monthly temperatures and monthly percentage of annual day-time (hr) as below:

Month	Nov.	Dec.	Jan.	Feb.
Temperature (°C)	16.5	13.0	11.0	14.5
Monthly percentage of annual day time (hr)	7.19	7.15	7.30	7.03

Take K = 0.65.

Use the Blaney–Criddle formula.

Sol: The temperatures are converted to Fahrenheit and the calculations are performed in the following table

Month	$\bar{T}_f$	P_h	$P_h\bar{T}_f/100$
Nov.	61.7	7.19	4.44
Dec.	55.4	7.15	3.96
Jan.	51.8	7.30	3.78
Feb.	58.1	7.03	4.08
		$\Sigma P_h\bar{T}_f/100$ =	16.26

By Blaney–Criddle formula

$$E_r = 2.54 \times 16.26 \times 0.65 = 26.85 \text{ cm}$$

***Isopleths**—The lines on a map through places having equal depths of evapotranspiration.

Interception Depression Storage and Infilteration

Interception is that portion of total precipitation which, while falling on the surface of the earth, is intercepted by the surfaces of buildings vegetation cover on the ground, roads and pavement etc., and subsequently lost by evaporation.

The three main components of interception by vegetal cover are defined below:

Interception Loss: Water which is retained on a surface, as mentioned above, and which is later evaporated away.

Through Fall: Water which drips through comes down from the leaves, etc. onto the ground surface.

Stem Flow: Water which trickles along the branches and finally down the main trunk onto the ground surface.

Thus, it is only the interception loss that does not reach the ground surface; and it may be regarded as a primary water loss.

It is found that coniferous trees have more interception loss than deciduous ones. Also, dense grasses have nearly same interception losses as full-growtn trees and can account for nearly 20% of the total rainfall in the season. Agricultural crops in their growing season also contribute high interception losses.

Depression Storage

When the precipitation of a storm reaches the ground, it must first fill up all depressions before it can flow over the surface. The volume of water trapped in these depressions is called *depression storage*.

Rainfall held in these depressions does not contribute to surface runoff unless these are filled to capacity.

This amount is eventually lost through processes of infiltration and evaporation and thus form a part of the initial loss.

Infiltration

Infiltration is that process by which precipitation moves downward through the surface of the earth and replenishes soil moisture, recharges aquifers, and ultimately supports streamflows during dry periods.

A distinction is to be made between the terms infiltration and **percolation**, the latter being used to describe the downward flow of water through the zone of aeration towards the water table, the former being restricted to the entry of water through the surface layers of the soil.

Along with interception, depression storage, and storm period evaporation (evaporation during rainfall), infiltration determines the availability, if any, of the precipitation input for generating overland flows.

$$\text{(Infilteration + Depression storage + Interception) = (Rainfall − Runoff)}$$

During a major storm, capable of producing a flood, evapotranspiration loss is generally negligible and losses by interception and depression storage are small compared to infiltration.

$$\text{Hence infiltration} + 0 + 0 = \text{Rainfall} - \text{Runoff}$$

Infiltration continues as long as there is a supply of water at the soil surface either by direct precipitation or by a flowing sheet of water.

If the intensity of rainfall is less, all the water infilterating into the soil gets stored as soil moisture and doesnot contribute to the ground water flow. If however the rainfall intensity is more, the soil gets saturated and there after contribution to ground water flow starts.

Factors Affecting Infiltration

Infiltration capacity (f_c) is defined as the maximum rate at which rain can be absorbed by a soil in a given condition.

The infiltration process is affected by a large number of factors as discussed below.

Rainfall Characteristics

The actual rate of infiltration, f, at a given time can be expressed as:

$$f = f_c, \text{ when } i > f_c$$

$$f = i, \text{ when } i < f_c$$

where, i is intensity of rainfall and f_c is the infiltration capacity at a given time; i, f and f_c are expressed in cm/hr or mm/minute. The infiltration capacity (f_c) of a soil is high at the beginning of a storm and has an exponential decay as the time elapses.

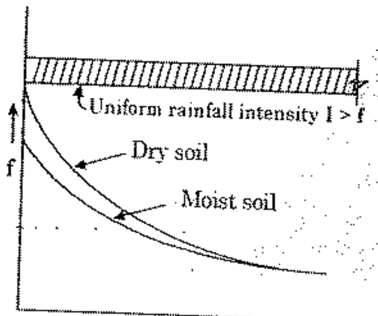


Fig. 4.4 Variation of infiltration capacity with time.

Increase in rainfall intensity is reflected in an increase in the size of raindrops, and consequently in an increase in their compacting force while the drops strike the ground surface. Thus, an inverse relationship between infiltration and rainfall intensity may occur under such conditions.

Characteristics of Soil

A loose permeable sandy soil will have a larger infiltration capacity than a light clayey soil. Clayey soils can be rendered virtually impermeable due to raindrop compaction, whereas clean sandy soils are much less susceptible to rain compaction.

A dry soil can absorb more water than the soil whose pores are already full of water.

Surface Cover

A vegetation cover tends to increase infiltration by:

(i) retarding surface flow, and thus allowing more time for water to enter the soil,

(ii) shielding the soil surface from direct impact of rain drops, thereby reducing surface compaction.

Spread of buildings and paved surfaces in urban areas effetively reduces the infiltration capacities, of various patches of ground to zero and thus contributes significantly to the frequency of flood peaks in such areas.

Characteristics of Infiltrating Water

Viscosity of water and, therefore, the ease with which it may move through soil pore spaces, varies with water temperature. it is therefore, expected that temperature will tend to exert some influence on the rate of infiltration.

Measurement of Infiltration

Infiltration characteristics of soil, at a given location, can be obtained by conducting controlled experiments on small areas. The experimental set-up is called an **infiltrometer**. There are two kinds of infiltrometers:

(i) Flooding type infiltrometers, and

(ii) Rainfall simulator

Note: Infilteration can also be found from hydrograph analysis.

Flooding Type Infiltrometer

This is a simple instrument consisting essentially of a metal cylinder. This cylinder is driven into the ground to a depth of 50 cm.

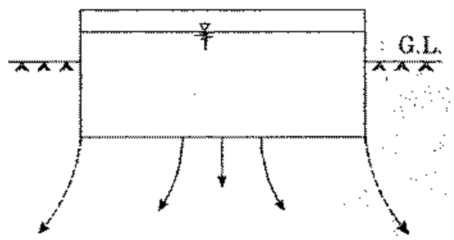


Fig. 4.5. Flooding type infiltrometer.

- Water is poured into the top part to a depth of 5 cm and a pointer is set to mark the water level. As infiltration proceeds, the volume is made up by adding water from a burette to keep the water level at the tip of the pointer. Knowing the volume of water added at different time intervals, the plot of the infiltration capacity *vs* time is obtained.

- A major objection to this simple infiltrometer is that the unfiltered water spreads at the outlet from the tube (as shown by dotted lines in the figure), and as such the tube area is not representative of the area into which infiltration takes palce.

- To overcome this lacuna a ring infiltrometer consisting of a set of two concentric rings is used.

- In this infiltrometer two concentric rings are inserted into the ground and the water depth is maintained on the soil surface as discussed above, in both the rings, to a common fixed level. The outer ring provides a water jacket to the infiltering water of the inner ring, and hence prevent, to a large extent, the spreading out of the infiltering water with respect to the inner tube. The measurement of water volume is done for the inner ring only.

The main disadvantages of a flooding type infiltrometer are:

(i) the raindrop-impact effect is not simulated,

(ii) the driving of the tube or rings disturbs the soil structure, and

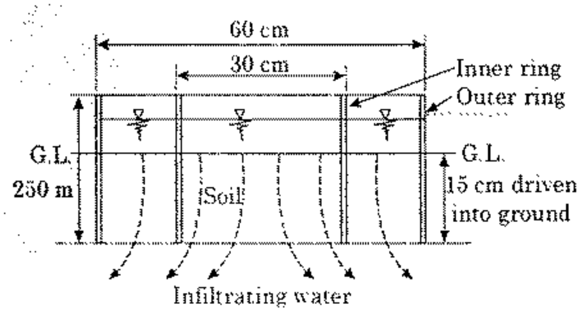


Fig. 4.6. Ring infiltrometer.

Rainfall Simulator

- Here, a small plot of land, of about 2 m × 4 m size is provided with a series of nozzles on the longer side, with arrangements to collect and measure the surface runoff rate.

- The specially designed nozzles produce raindrops falling from a height of 2 m, and are also capable of producing various intensities of rainfall.

- Experiments are conducted under controlled conditions with various combinations of intensity and duration, and the consequent surface runoff is measured in each case.
- Using the water-budget equation involving the volume of rainfall, infiltration and runoff, the infiltration rate and its variation with time is calculated.
- If the rainfall intensity is higher than the infiltration rate, infiltration capacity values are obtained.
- Rainfall simulator type infiltrometers give lower values than the flooding type infiltrometers. This is due to the effect of rainfall impact and turbidity of the surface water present in the former.

Empirical Infiltration Equations

Under given soil type and antecedent moisture conditions, there will be an initial infiltration rate, f_0. This rate will decrease as more water gets infiltrated, finally achieving a constant rate, f_c, i.e., *ultimate infiltration capacity*. This infiltration capacity rate prevails when the soils is saturated. The parameters f_0, f_c and the decay of infiltration capacity are functions of the soil moisture conditions, vegetation, rainfall, intensity and soil surface conditions.

Several empirical equations incorporating the above mentioned factors, affecting the behaviour of the soil have been proposed.

(a) Green-Ampt Model

(b) Horton Infiltration Equation

(c) Huggins-Monka Equation

(d) Soil Conservation Service Practice

(e) Antecedent Precipitation Method

Horton infiltration equation. as an example, is discussed herein. It takes the form

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

where, in practice, f_o, f_c and α (a constant) are parameters to be estimated from the given data; e is the napierian base and α is a constant and t is the time from the beginning of rainfall.

The equation is applicable only when rainfall less by retention is greater than or equal to f.

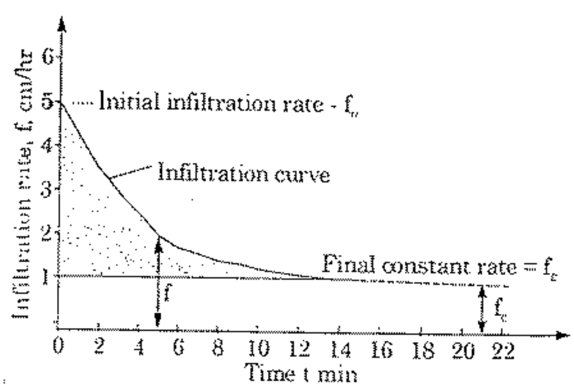


Fig. 4.7. Infiltration capacity vs time.

Relation of Infiltration to Runoff

- During a major storm, capable of producing flood conditions in a river basin, evapotranspiration losses are negligible and losses by interception and depression storage are small compared to the amount of infiltration.
- Too low a rate of infiltration causes high runoff and too high a rate of infiltration results in low runoff.
- Infiltration reduces floods and soil erosion and furnishes stream flow during the periods of dry weather, and also provides water for the growth of plants as well as recharges the ground water reservoir.
- In hydrological calculations involving floods, it is found convenient to use a constant value of infiltration rate for the duration of the storm. The average infiltration rate is called *infiltration index* and two types of indices, in this regards are in common use one such index is ϕ-index. The ϕ-index is the average rainfall above which the rainfall volume is equal to the runoff volume.
- The φ-index is dervied from the rainfall hyetograph with the knowledge of the resulting runoff volume. The initial loss is also considered as a part of infiltration.
- The φ value is found by treating this value as a constant infiltration capacity.
- If the rainfall intensity is less than φ then the infiltration rate is equal to the rainfall intensity.
- The amount of rainfall in excess of the index is called *rainfall excess*.
- The φ index thus accounts for the total abstraction and enables runoff magnitudes to be estimated for a given rainfall *hyetograph*.

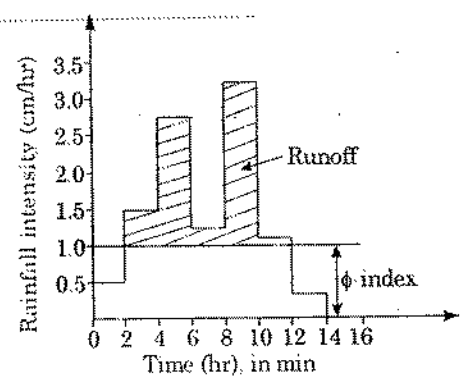


Fig. 4.8. Infiltration loss by φ-index.

In estimating the maximum floods for design purposes, in the absence of any other data, a φ-index value of 0.10 cm/h can be assumed.

W-Index

It is the average infilteration rate or the infilteration capacity averaged over the whole storm period.

$$W_{index} = \frac{P - Q}{t_r}$$

where P = Total precipitation

Q = Total runoff

t_r = Total duration of rainfall

W-index is always less than ϕ-index.

Example 4.4

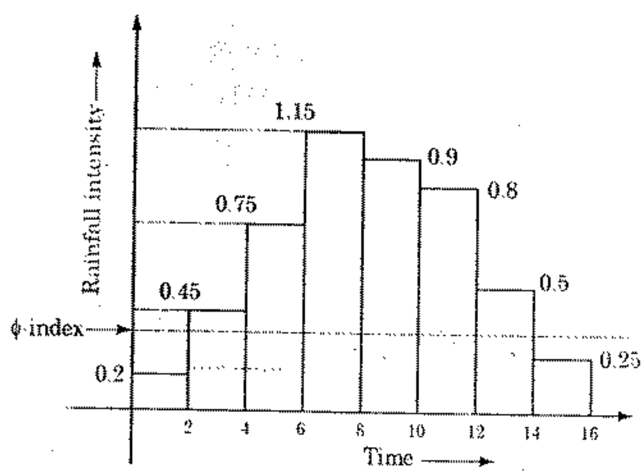
A storm with 10 cm of precipitation produced a direct runoff of 5.8 cm. The duration of the rainfall was 16 hours and its time distribution is given as below:

Time from start (hr)	0	2	4	6	8	10	12	14	16
Cumulative rainfall (cm)	0	0.4	1.3	2.8	5.1	6.9	8.5	9.5	10.0

Estimate the ϕ-index of the storm.

Sol:

Time from start (hr)	Cumulative rainfall (cm)	Incremental rainfall rainfall (cm)	Rainfall intensity (cm/hr)
0	0	0	0
2	0.4	0.4	0.2
4	1.3	0.9	0.45
6	2.8	1.5	0.75
8	5.1	2.3	1.15
10	6.9	1.8	0.90
12	8.5	1.6	0.80
14	9.5	1.0	0.50
16	10.0	0.5	0.25



Rainfall hyetograph

Total volume of rainfall = 10 cm

Total runoff = 5.8 cm

$\Rightarrow$ Infilteration = 10 − 5.8 = 4.2 cm

Time of rainfall = 16 hr

If infilteration is assumed constant, the infilteration rate will be $\frac{4.2}{16} = 0.2625$ cm/hr. But this infilteration rate can not be possible throughout the time period of 16 hrs because from 0-2 hr and 14-16 hr, rainfall rate is lesser and infilteration rate can never be more than rainfall rate.

Thus, ϕ-index will be some where between 0.25 cm/hr to 0.45 cm/hr.

$\Rightarrow$ Time of rainfall excess = 12 hr.

$$\phi\text{-index} = \frac{4.2 - 0.2 \times 2 - 0.25 \times 2}{12}$$

$= 0.275$ cm/hr

Example 4.5

An isolated 3-h storm occured over a basin in the following fashion:

% of catchment area	ϕ-index (cm/h)
20	1.00
30	0.75
50	0.50

Rainfall (cm)		
1st hour	**2nd hour**	**3rd hour**
0.8	2.3	1.5
0.7	2.1	1.0
1.0	2.5	0.8

Estimate the runoff from the catchment due to storm.

Sol: Volume of runoff = $[(2.3 - 1) \times 1 + (1.5 - 1) \times 1] \times 0.2A + [(2.1 - 0.75) \times 1 + (1 - 0.75) \times 1] \times 0.3A + [(1 - 0.5) \times 1 + (2.5 - 0.5) \times 1] \times 0.5A + (0.8 - 0.5) \times 1$

Volume of runoff = 2.24A

where A = Area of catchment.

$$\text{Runoff averaged over the catchment in depth} = \frac{\text{Volume of runoff}}{\text{Total catchment area}}$$

$= 2.24$ cm

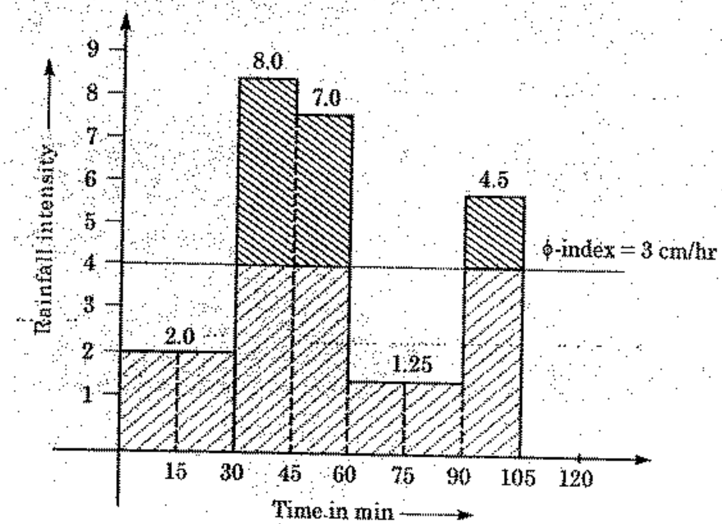
Example 4.6

The following is the set of observed data for successive 15 minutes period of 105 minutes storm in a catchment:

Duration (min)	15	30	45	60	75	90	105
Rainfall (cm/hr)	2.0	2.0	0.8	7.0	1.25	1.25	4.5

If the value of ϕ-index is 3 cm/hr, estimate the net runoff, the total rainfall and the value of W-index.

Sol: The rainfall intensity pattern (rainfall hyetograph) can be plotted from the given rainfall rates as shown below:



The hatched portion shows the total runoff and the dotted portion shows the total infiltration.

$$\therefore \text{ Total runoff} = (8-3)\times\frac{15}{60}+(7-3)\times\frac{15}{60}+(4.5-3)\times\frac{15}{60}$$

$$= 5\times\frac{15}{60}+4\times\frac{15}{60}+1.5\times\frac{15}{60}$$

$$= 1.25 + 1.0 + 0.375$$

$$= 2.625 \text{ cm}$$

Also, Total precipitation $= 2\times\frac{30}{60}+8\times\frac{15}{60}+7\times\frac{15}{60}+1.25\times\frac{30}{60}+4.5\times\frac{15}{60}$

$$= 1 + 2 + 1.75 + 0.625 + 1.125$$

$$= 6.5 \text{ cm}$$

$$\text{W-index} = \frac{\text{Precipitation} - \text{Runoff}}{\text{Duration of rainfall in hr}} = \frac{6.5-2.625}{105/60} = 2.21 \text{ cm/hr}$$

OBJECTIVE QUESTIONS

1. Match **List-I** with **List-II** and select the correct answer using the codes given below the lists:

List-I	List-II
A. Anemometer	1. Humidity
B. Rain simulator	2. Evapotranspiration

C. Lysimeter 3. Infiltration
D. Hygrometer 4. Wind speed

Codes:

	A	B	C	D
(a)	4	3	1	2
(b)	3	4	1	2
(c)	4	3	2	1
(d)	3	4	2	1

2. A 3-hour storm on a small drainage basin produced rainfall intensities of 3.5 cm/hr, 4.2 cm/hr and 2.9 cm/hr in successive hours. If the surface runoff due to the storm is 3 cm, then the value of ϕ-index will be
(a) 2.212 cm/hr (b) 2.331 cm/hr
(c) 2.412 cm/hr (d) 2.533 cm/hr

3. Match **List-I** with **List-II** and select the correct answer using the codes given below the lists:

List-I
A. f index
B. Lysimeter
C. Dilution technique
D. Snyder's equation]

List-II
1. Used for measurement of evapotranspiration for given vegetation
2. Used for flow measurement
3. Average rainfall above which the rainfall volume is equal to the runoff volume
4. Relates the basin lag to the basin characteristics

Codes:

	A	B	C	D
(a)	3	1	2	4
(b)	4	2	1	3
(c)	3	2	1	4
(d)	4	1	2	3

4. A 6-hour rainstorm with hourly intensities of 7, 18, 25, 17, 11 and 3 mm/hour produced a runoff of 39 mm. Then, the ϕ-index is
(a) 3 mm/hour (b) 7 mm/hour
(c) 8 mm/hour (d) 10 mm/hour

5. The Penman's evapo-transpiration equation is based on
(a) water budget method
(b) energy balance method
(c) mass transfer method
(d) energy balance and mass transfer approach

6. A storm with 14 cm precipitation produced a direct runoff of 8 cm. The time distribution of the storm is as shown in the table below.

Time from start (h)	Incremental rainfall in cm
1	1.0
2	2.0
3	2.8
4	3.3
5	2.5
6	1.8
7	0.6

What is the value of ϕ-index of the storm?

(a) 0.5 cm/h (b) 0.7 cm/h
(c) 0.8 cm/h (d) 0.9 cm/h

7. A 6 hour storm has 6 cm of rainfall and the resulting runoff was 3 cm. If ϕ-index remains at the same value, which one of the following is the runoff due to 12 cm of rainfall in 9 hours in the catchment?

(a) 4.5 cm (b) 6.0 cm
(c) 7.5 cm (d) 9.0

8. Penman's equation is based on

(a) energy dubgeting only
(b) Energy budgeting and water budgeting
(c) Energy budgeting and mass transfer
(d) Water budgeting and mass transfer

9. A standard, ground-based evaporation pan, corresponding to Indian Standards, is installed at the banks of a surface reservoir. The water surface area on a particular day is 100 hectares. The recorded evaporation loss from the pan, on a certain day, is nearly 4.0 cm. What is the anticipated evaporation loss from the reservoir for that day?

(a) (1.8 to 2) $\times 10^4$ m^3 per day
(b) (2.5 to 2.75) $\times 10^4$ m^3 per day
(c) (3.0 to 3.25) $\times 10^4$ m^3 per day
(d) (3.8 to 4.05) $\times 10^4$ m^3 per day

10. The total observed runoff volume during a 4 hour storm with a uniform intensity of 2.8 cm/hr is 25.2×10^6 m^3 from a basin of 280 km^2 area. What is the average infiltration rate for the basin?

(a) 3.6 mm/hr (b) 4.8 mm/hr
(c) 5.2 mm/hr (d) 5.5 mm/hr

11. Consider the following chemical emulsions:

1. Methyl alcohol
2. Cetyl alcohol
3. Stearyl alcohol
4. Kerosene

Which of the above chemical emulsions is/are used to minimize the loss of water through the process of evaporation?

(a) 1 only (b) 1 and 4
(c) 2 and 4 (d) 2 and 3

12. The vertical hydraulic conductivity of the top soil at certain stage is 0.2 cm/hr. A storm of intensity 0.5 cm/hr occurs over the soil for an indefinite period. Assuming the surface drainage to be adequate, the infiltration rate after the storm has lasted for a very long time, shall be
    (a) smaller than 0.2 cm/hr
    (b) 0.2 cm/hr
    (c) between 0.2 and 0.5 cm/hr
    (d) 0.5 cm/hr

13. The plan area of a reservoir is 1 km^2. The water level in the reservoir is observed to decline by 20 cm in a certain period. During this period the reservoir receives a surface inflow of 10 hectare-meters, and 20 hectare-meters are abstracted from the reservoir for irrigation and power. The pan evaporation and rainfall recorded during the same period at a nearby meteorological station are 12 cm and 3 cm respectively. The calibrated pan factor is 0.7. The seepage loss from the reservoir during this period in hectare-meters is
    (a) 0.0 (b) 1.0
    (c) 2.4 (d) 4.6

14. During a 3 hour storm event, it was observated that all abstractions other than infiltration are negligible. The rainfall was idealized as 3 one hour storms of intensity 10 mm/hr, 20 mm/hr and 10 mm/hr respectively and the infiltration was idealized as a Horton curve, $f = 6.8 + 8.7 \exp(-t)$(f in mm/hr and t in hr). What is the effective rainfall?
    (a) 10.00 mm (b) 11.33 mm
    (c) 12.43 mm (d) 13.63 mm

ANSWERS

1. (c)	5. (d)	9. (c)	13. (d)
2. (d)	6. (d)	10. (d)	14. (d)
3. (a)	7. (c)	11. (d)	
4. (c)	8. (c)	12. ~~(b)~~ (a)	

CHAPTER

5

Surface Water Hydrology (Runoff)

Runoff

Runoff may be referred to as stream flow, river discharge or catchment yield. It is normally expressed as volume per unit time.

The total runoff from a typical catchment area may be conveniently divided into four component parts: Direct precipitation on the stream channels, Surface Runoff, Interflow, Groundwater Flow.

Direct Precipitation

Direct precipitation onto the water surface and into the stream channels will normally represent only a small percentage of total volume of water flowing in the streams. This component is usually ignored in runoff calculations.

Surface Runoff

It has two components:

(a) Overland flow (flow of water over land before joining any open channel)

(b) Open channel flow

Over land flows are small and the flow is taken to be in laminar regime. Length of overland flow is generally small.

Open channel flow are in turbulent regime.

Interflow

- Water which infiltrates the soil surface and then moves laterally through the upper soil horizons towards the stream channels above the main groundwater table is known as the *interflow*. It is also known as *subsurface runoff, subsurface storm flow, storm seepage* and *secondary base flow*.
- If the lateral hydraulic conductivity of the surface layers are substantially greater than the overall vertical hydraulic conductivity, it is a favourable condition for the generation of interflow. Generally interflow moves more slowly than surface runoff.
- Depending upon the time delay between infiltration and its outflow from the upper crusts of the soil, the interflow is sometimes classified into prompt interflow and delayed interflow.

Ground Water Flow

- The infiltrated water which percolates deeply becomes groundwater and when groundwater table rises and intersects the stream channels of the basin it discharges into streams as the groundwater runoff.
- Ground water flow is sometimes referred to as base flow, dry weather flow, effluent seepage.

  For the practical purpose of analysis total runoff in stream channels is generally classified as *direct runoff* and *base flow*.

Direct Runoff

It is that part of runoff which enters the stream immediately after the precipitation. It includes surface runoff, prompt interflow and precipitation on the channel surface.

It is sometimes terms such as *direct storm runoff* or *storm runoff*.

Base Flow

The delayed flow that reaches a stream essentially as groundwater flow is called *base flow*. Many times delayed interflow is also included under this category.

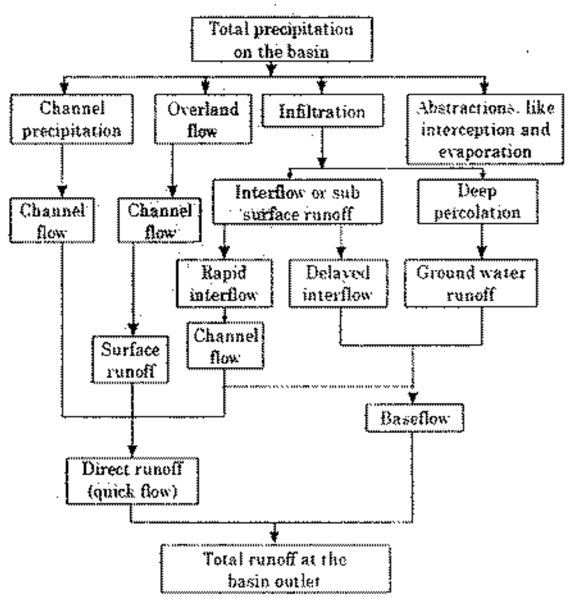


Fig. 5.1. Schematic representation of runoff process.

Factor Affecting the Distribution of Runoff in Time

Type of Precipitation

It occurs either as rainfall or as snowfall. Precipitation falling as rain contributes directly to runoff. Hail and sleet which melt rapidly after contact with the ground may also contribute immediately. Precipitation falling as snow, in winter months, will not contribute to runoff until melting occurs.

Rainfall Intensity

Heavy rain falling in excess of the infiltration capacity of the soil will largely contribute to surface runoff and will therefore tend to reach the stream channels very rapidly while the rain falling at lower intensities will cause delayed runoff.

Rainfall Duration

If the rainfall duration is equal to or greater than time of concentration (T_c i.e. the time in which the entire catchment starts contributing to runoff) the potential runoff is at a maximum. On the other hand, if the duration of rainfall is less than T_c, then the potential runoff will be lower than the maximum because only part of the catchment will be contributing to runoff before the rainfall ceases.

Also, since infiltration capacity is maximum at the beginning of rainfall and tends to decrease through the period of rainfall until it attains a constant value, the longer the rain continues, the smaller will the infiltration capacity become, and the greater the amount of surface runoff is likely to be.

Rainfall Distribution

The time relationship between rainfall and runoff may be greately affected by the distribution of rainfall over the catchment area. A uniformly distributed rainfall volume may lead to delayed runoff then the same volume falling over a localised part of catchment.

The first type of rainfall distribution will tend to result in an increased groundwater flow, and consequently a longterm increase in streamflow, while the latter type of distribution will tend to give large volumes of surface runoff and thus, a more sudden, sort-lived increase in streamflow.

Catchment Factors

Factors such as shape, topography and soil type remain constant over long periods while others such as land use etc. may change radically and so profoundly influence runoff.

- Shape of the catchment area influences the time of concentration of the catchment.
- Catchment shape is quantitatively expressed as form factor and compactness factor.
- The form factor is derived by dividing the average width of the catchment by its axial length measured from the outlet to the most remote point.

A long, narrow catchment will yield a low value of the **form factor**, and generally a lower-peak runoff total than a similar sized area with a high form factor.

The **compactness coefficient** is derived by dividing the periphery of the watershed by the circumference of the form circle whose area equals that of the catchment, and thus has a theoretical minimum value of unity for a completely circular catchment. The lower the value of the coefficient, the more rapidly is water likely to be discharged from the catchment area via the main streams.

Slope of the Catchment Area

If slope of catchment area is large, water moves faster over the surface and channel towards the stream channel. Hence runoff peak comes early.

Geology

Coarse textured, sandy soils will tend to give rise to little runoff. On the other hand fine grained closely compacted clay soils will tend to yield more surface runoff. Moisture content of soil also affects runoff through its effect on infiltration capacity.

Vegetation

The most important effect of the vegetal cover is to slow down the movement of water over the surface after rainfall, thus, allowing more time for infiltration to take place. The timing of runoff after rainfall may be considerably modified and, furthermore, peak stream flows will tend to be much lower.

Drainage Network

Closely spaced streams allow efficient drainage of precipitated water; and in such case overland flow will be short and surface runoff will rapidly reach the main streams.

Direction of Storm Movement

- The runoff from a basin is influenced by the direction of storm movement.
- If a storm striking a long and narrow basin is moving in an upstream direction the runoff contributed by the lower tributaries would have been already drained out by the time the runoff from the middle and upper tributaries reaches the basin outlet and therefore less peak discharge would be observed in this case.
- When, on the other hand, the storm moves downstream the runoff peaks from the individual tributaries are more likely to arrive at the basin outlet at approximately the same time with the result the runoff peak will be many times more than that in the case of storm moving upstream.
- A storm crossing the basin in the transverse direction would produce a peak discharge which would be somewhere between the peaks produced by the two cases described above.

Antecedent Precipitation

If due to previous rain, soil is already saturated runoff will be more due to next rainfall. The previous rain is called Antecedent precipitation.

The *antecedent precipitation index* (API) is taken as a measure of the soil moisture conditions existing on the day of storm under consideration.

$$\boxed{I_t = KI_{t-1}}$$

where I_t is the API of any day t, K is a constant known as the *recession factor*, and I_{t-1} is the API of the day $(t-1)$. If rain occurs on any day, the amount of rain is added to the index and this value is used in the computation of API of the next day. If there is no rainfall in the previous t days, then

$$I_t = K^t I_0$$

where I_0 is the initial value of API.

Example 5.1

Rainfall of 12, 30, 40, 44 and 17 mm were recorded on 3rd, 9th, 10th, 16th and 17th days of a particular month. Compute the antecedent precipitation index for the first 20 days of the month and sketch its variation with time. Assume that API of the last day in the previous month is 85 mm and the value of the recession factor K is 0.90.

The API of any day 't', denoted by I_t. I_t is obtained from equation $I_t = K.I_{t-1} + P_t$, where P_t is the precipitation of t^{th} day. I_0 = 85 and K = 0.9. The tabular calculation is shown below.

Day t	Precipitation in mm P_t	I_{t-1}	$K \cdot I_{t-1}$	$I_t = K \cdot I_{t-1} + P_t$
1	0	85.00	76.50	76.50
2	0	76.50	68.85	68.85
3	12	68.85	61.97	73.97
4	0	73.97	66.57	66.57
5	0	66.57	59.91	59.91
6	0	59.91	53.92	53.92
7	0	53.92	48.53	48.53
8	0	48.53	43.68	43.68
9	30	43.68	39.31	69.31
10	40	69.31	62.38	102.38
11	0	102.38	92.14	92.14
12	0	92.14	82.93	82.93
13	0	82.93	74.64	74.64
14	0	74.64	67.18	67.18
15	0	67.18	60.46	60.46
16	44	60.46	54.41	98.41
17	17	98.41	88.57	105.57
18	0	105.57	95.01	95.01
19	0	95.01	85.51	85.51
20	0	85.51	76.96	76.96

Hydrograph

A plot of the discharge in a stream plotted against time chronologically is called a hydrograph. Depending upon the unit of time involved, we have

1. Annual hydrograph → it shows variation of daily weakly or mean flow of any no. of successive days over a year.
2. Monthly hydrographs → it shows the variation of daily mean flow over a month.
3. Seasonal hydrographs → it shows the variation of the discharge in a particular season such, as the monsoon season or dry season.
4. Flood hydrographs or hydrographs due to a storm → it shows stream flow due to a storm over a catchment.

Hydrographs 1, 2 and 3 as above are called long term hydrographs and are used for long term studies like.

(a) Calculating the surface water potential of stream.

(b) Reservoir studies and

(c) Drought studies.

Flood hydrograph on the other hand is used to study the flooding characteristics of a stream. It is a short term study.

Beginning of Water Year

The time when precipitation exceeds the average evapotranspiration losses is called the beginning of water year. 1st June in India is considered the beginning of water year. Beginning of water year is decided such that flood season is not divided between successive year.

Runoff Characteristic of Stream

On the basis of hydrograph studies a stream can be classified as:

(a) Perennial (b) Intermittent (c) Ephemeral

Perennial Stream

- A perennial stream is one which always carries some flow throughout the year. Even during dry seasons the water table will be above the bed of the stream. Thus, considerable amount of ground water flow occurs during non precipitation period.

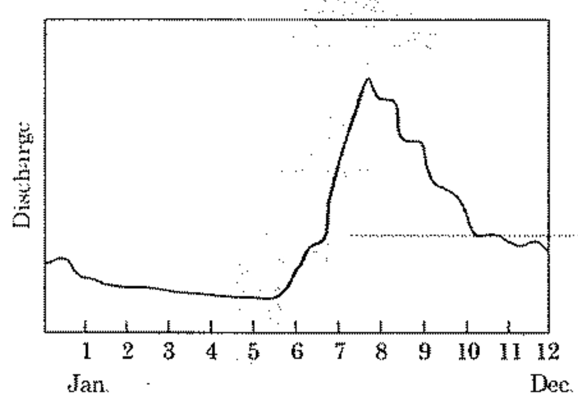


Fig. 5.2. Perennial stream.

Intermittent Stream

- Intermittent stream has limited contribution from the groundwater.
- Stream remains dry for most part of dry month.
- Base flow (ground water flow) occurs significantly during wet season.
- During dry seasons the water table drops to a level lower than that of the stream bed.

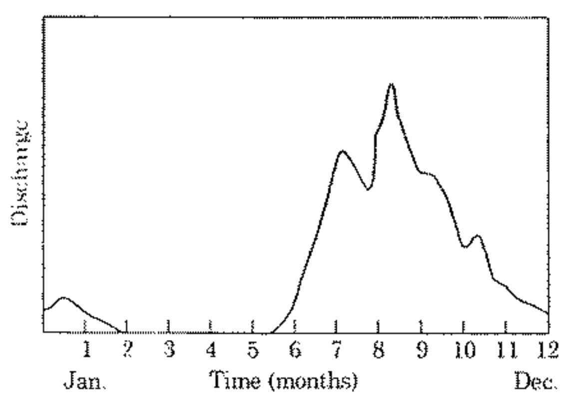


Fig. 5.3. Intermittent stream.

Ephemeral Stream

- An ephemeral stream is one which does not have any base-flow contribution.
- Annual hydrograph shows series of short-duration spikes marking flash flow in response to storms.
- The stream becomes dry soon after the end of the storm flow.
- An ephemeral stream does not have any well defined channel.
- Most river in arid zones are of the ephemeral kind.

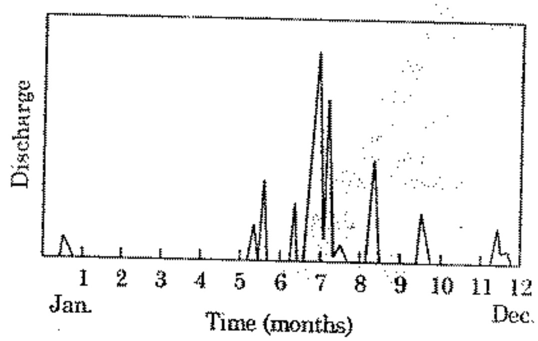


Fig. 5.4 Ephemeral stream.

Basin Yield

Basin yield refers to the quantity of water available from a stream at a given point over a specified duration of time. The duration of time in the definition of yield would normally be a month or longer. It is expressed in terms of volume not discharge.

Thus, $\int_{t_1}^{t_2} Q \cdot dt$ provides the estimate of basin yield.

Therefore yield from a basin is the summation of the continuous hydrograph of flow at its outlet over the specified time period.

When continuous flow measurements are not available the yield may be determined as below:

1. Correlation of streamflow and rainfall
2. Empirical equations, and
3. Watershed simulation.

Rainfall Runoff Relationships

In hydrological analysis and design, it is often necessary to develop relations between precipitation and runoff. Such relations are useful for extrapolation or interpolation of runoff records from the precipitation records which are generally available for longer periods. Through these relations the estimates of the runoff of ungauged catchments may be obtained.

One of the most common methods is to fit a linear regression line between R and P and to accept the result if the correlation coefficient is nearer to unity. The equation for straight line regression between runoff R and rainfall P is

$$\boxed{R = CP + d}$$

and the values of the coefficients a and b are given by

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

and
$$\boxed{d = \frac{\Sigma R - C\Sigma P}{N}}$$

in which N = number of observation sets R and P. The coefficient of correlation r can be calculated as

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

The values of r lies between 0 to + 1 as R can have only positive correlation with P. A value of $0.6 < r < 1.0$ indicates good correlation.

- For large catchments, it is found advantageous to have an exponential relationship

$$R = \beta P^m \qquad \text{(A)}$$

In that case Eq. (A) is reduced to a linear form by logarithmic transformation as

$$\boxed{\ln R = m \ln P + \ln \beta}$$

and the coefficients m and $\ln \beta$ determined by using the method indicated earlier.

Example 5.2

Given below are the monthly rainfall P and the corresponding runoff R values covering a period of 18 months for a catchment. Develop a correlation equation between R and P.

Month	P	R
1	5	0.5
2	35	10.5
3	40	138
4	30	8.2
5	15	3.1
6	10	3.2
7	5	0.1
8	31	12.0
9	36	16.0
10	30	8.0
11	10	2.3
12	8	1.6
13	2	0.0
14	22	6.5
15	30	9.4
16	25	7.6
17	8	1.5
18	6	0.5

Sol: For the given data

$\Sigma P = 348 \quad \Sigma R = 104.3 \quad N = 18$

$\Sigma P^2 = 9534 \quad \Sigma R^2 = 1040.51 \quad \Sigma PR = 3083.3$

$(\Sigma P)^2 = 121104 \quad (\Sigma R)^2 = 10878.49$

For the correlation equation $R = CP + d$, by Eq.

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

$$= \frac{(18 \times 3083.3) - (348 \times 104.3)}{(18 \times 9534) - 121104} = 0.380$$

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

$$= \frac{104.3 - (0.380 \times 348)}{18} = -1.55$$

Hence $R = 0.38\,P - 1.55$.

In this equation both R and P are in cm and $R \geq 0$. Correlation coefficient r given by is

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

$$r = \frac{(18 \times 3083.3) - (348 \times 104.3)}{\sqrt{[(18 \times 9534) - 121104] \times [(18 \times 1040.51) - 10878.49]}}$$

$$= 0.964$$

Empirical Equations

With a keen sense of observation in the region of their activity many engineers of the past have developed empirical runoff estimation formulae. There are:

(a) **Binnie's Percentages**—Developed for small catchment near Nagpur.

(b) **Barlow's Tables**—Developed for small catchments (area $\approx$ 130 km^2) in Uttar Pradesh.

(c) **Strange's Tables**—Developed for border areas of present-day Maharashtra and Karnataka.

(d) **Inglis and DeSouza Formula**—Developed for Western India.

(e) **Khosla's Formula**—Developed for a time period of a month.

His relationship for monthly runoff is

$$R_m = P_m - L_m \quad (\text{if } L_m \text{ is} > P_m,\ R_m = 0)$$

$$L_m = 0.48\ T_m \text{ for } T_m > 4.5°\text{C}$$

R_m = monthly runoff in cm and $R_m \geq 0$

P_m = monthly rainfall in cm

L_m = monthly losses in cm

T_m = mean monthly temperature of the catchment in °C

For $T_m \leq 4.5$°C, the loss L_m may provisionally be assumed as:

T° C	4.5	–1	–6.5	–12	–18
L_m (cm)	2.17	1.78	1.52	1.25	1.0

Annual runoff = ΣR_m.

Khosla's formula is indirectly based on the water-balance concept and the mean monthly catchment temperature is used to reflect the losses due to evapotranspiration.

Example 5.3

For a catchment in UP, India, the mean monthly rainfall and temperatures are given. Calculate the annaual runoff coefficient by Khosla's formula.

Month	Temperature (°C)	Rainfall (cm)
January	12	4
February	16	4
March	21	2
April	27	0
May	31	2
June	34	12
July	31	32
August	29	29
Sepember	28	16
October	29	2
November	19	1
December	14	2

Solution: In Khosla's formula,

$$R_m = P_m - L_m$$

If the loss L_m is higher than P_m then R_m is taken to be zero. The values of R_m calculated by eq. are

Month	Monthly loss (cm) $L_m = 0.48\ T_m$	Monthly rainfall (cm)	Runoff (cm)
January	5.7	4	0
February	7.68	4	0
March	10.08	2	0
April	12.96	0	0
May	14.88	2	0
June	16.32	12	0
July	14.68	32	17.1
August	13.92	29	15.1
Septemper	13.44	16	2.6
October	13.92	2	0
November	9.12	1	0
December	6.72	1	0
Σ		116	34.8

Annual runoff Σ = 34.8 cm

$$\text{Annual runoff coefficient} = \frac{\text{Annual runoff}}{\text{Annual rainfall}}$$

$$= \frac{348}{116.0} = 0.30$$

Watershed Simulation

The hydrologic water-budget equation for the determination of runoff for a given period is written as

$$\int Qdt = R = R_s + G = P - E - \Delta S$$

in which R_s = surface runoff, P = precipitation, E = actual evapotranspiration, G = net groundwater outflow and ΔS = change in the soil moisture storage.

With judicious selection of the time period, like a year, the magnitude of the term ΔS may be made insignificant. By estimating other terms runoff (R) may be calculated.

Flow-Duration Curve and Flow Mass Curve

When the runoff rate observed at the catchment outlet is continuously measured, it can be plotted as the ordinate against time on abscissa to provide the runoff hydrograph. The variability in runoff is expressed is terms of the fluctuations of the runoff rate in the hydrograph. There are two other ways of evaluating the variability in the runoff. They are the *flow-duration curve* and the *flow mass curve.*

Flow Duration Curve

- A *flow-duration* curve of a stream is a plot of discharge against the percent of time the flow was equalled or exceeded. This curve is also known as *discharge-frequency curve.*

- To determine the flow duration curve for a particular location, it is necessary to obtain daily flow data for a certain period of time, either 1 year or a number of years. The length of the record indicates the total numbers of days in the series.
- The daily flows are then arranged in descending order of magnitude, from the highest to the lowest flow value, with each flow value being assigned a rank. The highest flow would get a rank of 1, the next highest flow a rank of 2, and so on, and the lowest flow would get a rank n where n is the total number of days in the record.
- For each flow value, the percent of time is computed as $\frac{m}{n+1} \times 100$, where m is the rank assigned to the flow. The flow duration curve is obtained by plotting percent of time as the abscissa and the flow value as the ordinate. The flow duration curve can also be constructed using weekly, ten daily and monthly flow values.
- The amount of work involved in preparing a flow duration curve can be reduced by dividing the flow data into class intervals instead of handling each individual observation. Thus, for example if in a record length of 365 days the daily flow is between 40 m³/s and 50 m³/s for 73 days, then the mid value of the class interval, i.e. 45 m³/s, is plotted against percent of time given by $\frac{73}{365} \times 100 = 20\%$, on the flow duration curve.
- A flow duration curve constructed based on daily flows will be steeper than the flow duration curve prepared from the monthly flow data for the same period of record. This is because the larger interval data will smoothen out the variation in the shorter interval data.

Use of Flow Duration Curve

1. The flow duration curve is highly useful in the planning and design of water resources projects. In particular, for hydropower studies, the flow duration curve serves to determine the potential for *firm power* generation. In the case of a **run-of-the-river plant**, with no storage facilities, the firm power is usually computed on the basis of flow available **90 to 97** per cent of the time.

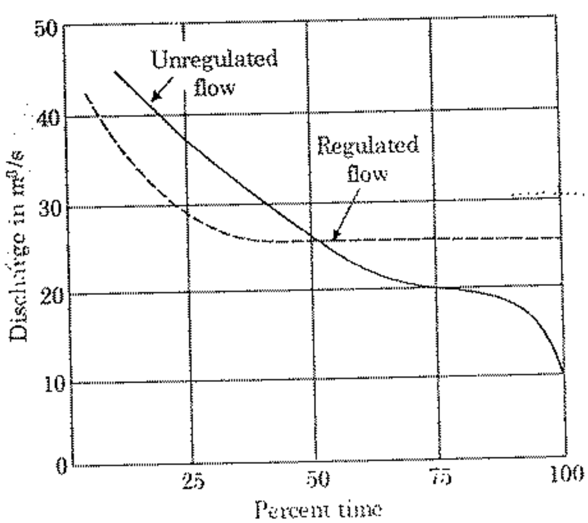


Fig. 10.4. Flow duration curve.

2. If a sediment rating curve is available for the given stream, the flow duration curve can be converted into cumulative sediment transport curve by multiplying each flow rate by its rate of sediment transport. The area under this curve represents the total amount of sediment transported.

3. The flow duration curve also finds use in the design of drainage systems and in flood control studies.

4. A flow duration curve plotted on a log-log paper provides a qualitative description of the runoff variability in the stream. If the curve is having steep slope throughout, it indicates a stream with highly variable discharge. This is typical of the conditions where the flow is mainly from surface runoff. A flat slope indicates small variability which is a characteristic of the streams receiving both surface runoff and groundwater runoff.

5. Flow duration curve can be used to estimate what minimum level of flow can be maintained through a reservoir so that minimum demand at all times can be met. Thus, if we have an unregulated flow in to a stream which gives variable discharge then we can make a sotrage reservoir and by controlled release of water a min necessary discharge can be ensured at all times.

6. The shape of the flow duration curve may change with the length of record. This aspect of the flow duration curve can be utilised for extrapolation of short records. Suppose there is a catchment A with a long record (say 1941 to 1980) which also covers the period of the short record of catchment B (say 1971 to 1980) in the same region. Then it is assumed that relationship which existed during the concurrent period (1971 to 1980) is also valid for the long period of catchment A for which catchment B has no record. [catchment B has no records for period (1941-1970)].

- First the flow duration curves are prepared for both the catchment for the concurrent period of record (1971 to 1980).
- Next the flow duration curve of A is constructed for the long period (1941 to 1980). Now the ordinates of flow duration curve of catchment B for the long period are obtained from the following equation

$$Q_B = Q_A \frac{q_B}{q_A}$$

where, for any given per cent of time, q denotes the ordinate of flow duration curve based on short record.

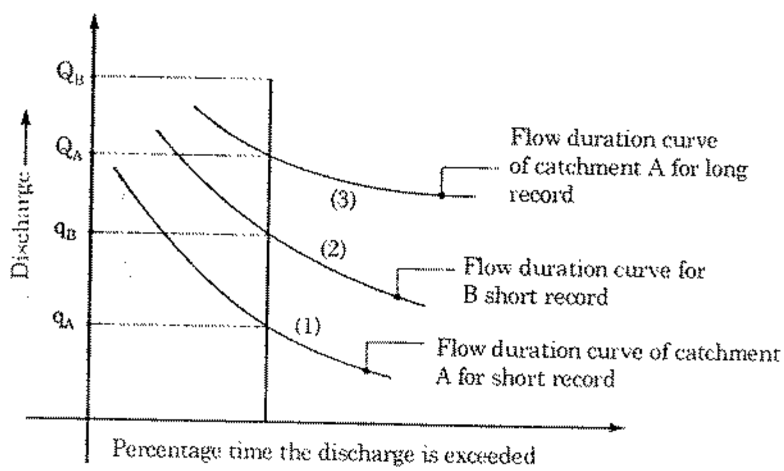


Fig. 5.5. Curve (1), (2) and (3) are known and our aim is to make curve 4.

Q denotes the ordinate of flow duration curve based on long record and suffixes A and B stand for the catchments A and B respectively.

Example 5.4

The observed mean monthly flows of a stream for a one year period from June to May in m^3/s are 18, 20, 46, 42, 37, 30, 33, 23, 26, 21, 19 and 8.

(a) Determine the flow which can be expected 80 per cent of time.

(b) What is the dependability of the flow of magnitude 40 m^3/s?

(c) If the run-of-the-river plant is proposed at the catchment of outlet what firm power can be produced assuming an average available head of 5 m and efficiency of power production = 80% for 90% dependeblity.

Sol: The computations required for the construction of the flow duration curve are shown in Table.

Month	Observed flow m^3/s	flow aranged in descending order m^3/s	Rank m	Percent of time flow equalled or exceeded $= \frac{m}{n+1} \times 100$ (%)
June	18	46	1	7.69
July	20	42	2	15.38
Aug.	46	37	3	23.07
Sept.	42	33	4	30.77
Oct.	37	30	5	38.46
Nov.	30	26	6	46.15
Dec.	33	23	7	53.84
Jan.	23	21	8	61.54
Feb.	26	20	9	69.23
Mar.	21	19	10	76.92
Apr.	19	18	11	84.61
May	8	8	12	92.3

From the table by using inter polation.

(a) Flow expected 80 per cent of the time = 18.56 m^3/s

(b) Dependability of flow of magnitude 40 m^3/s = 18.456%.

(c) From flow duration curve the magnitude of the flow available 90% of the time is 11 m^3/s. Available head is 5 m.

$$\Rightarrow \quad P = \eta \, \gamma Q H$$

$= 0.80 \times 9810 \times 11 \times 5$

$= 431.67$ kW

Example 5.5

The daily flows of a river for three consecutive years are shown in Table. For convenience the discharges are shown in class intervals and the number of days the flow belonged to the class is shown. Calculate the 50 and 75% dependable flows for the river.

Sol: The data are arranged in descending order of class value. In Table column 5 shows the total number of days in each class. Column 6 shows the cumulative total of column 5, i.e. the number of days the flow is equal to or greater than the class interval. This gives the value of m. The percentage probability P_p i.e. the probability of flow in the class interval being equalled or exceeded is given by eq.

$$\frac{m}{(N+1)} \times 100\%$$

Table 5.3: Calculation of flow duration curve from daily flow data:

Daily mean discharge (m^3/s)	No. of days flow in each class interval			Total of columns 2, 3, 4	Cumulative	$P_p = \left(\frac{m}{N+1}\right) \times 100\%$ % dependibility i.e. % chance that the discharge will be equalled or exceeded
	1961-62	1962-63	1963-64	1961-64	Total m	
1	2	3	4	5	6	7
140-120.1	0	1	5	6	6	0.55
120-100.1	2	7	10	19	25	2.28
100-80.1	12	18	15	45	70	6.38
80-60.1	15	32	15	62	132	12.03
60-50.1	30	29	45	104	236	21.51
50-40.1	70	60	64	194	430	39.19
40-30.1	84	75	76	235	665	60.62
30-25.1	61	50	61	172	837	76.30
25-20.1	43	45	38	126	963	87.78
20-15.1	28	30	25	83	1046	95.35
15-10.1	15	18	12	45	1091	99.45
10-5.1	5	—	—	5	1096	99.91
Total	365	365	366	N = 1096		

Note that in the above table if discharge for a particular rank no 'm' is asked, we should refer to the smallest value of discharge in the corresponding discharge interval rather than the mid value of the discharge interval. This approach will lead to safer estimation of dependable flow.

We will plot a graph between the smallest value of discharge in a disinterval against P_p and from the graph discharge for 50% and 75% dependibility can be found out.

or we can interpolate the data based on P_p and min discharge in the dam interval.

By interpolation

The intp. is to be done in lower limits.

$$50\% \text{ dependable discharge} = 50 - \frac{(50-40)\times(50-39.19)}{(60.62-39.19)}$$

$= 44.956\ m^3/s$

$$75\% \text{ dependable discharge} = 40 - \frac{(40-30)(75-60.62)}{(76.3-60.62)}$$

$= 30.65\ m^3/s$

Flow Mass Curve

A graph of the cumulative value of any hydrologic quantity, generally as the ordinate, plotted against the time as abscissa is known as a *mass curve*. When cumulative volume of runoff is taken as the hydrologic quantity, the mass curve may be called a *flow mass curve* or simply a mass curve. It is the integration of the hydrograph and therefore represents the area under the hydrograph from one time to another. Mathematically, the mass curve, is expressed by

$$V(t) = \int_0^t Q(t) \cdot dt$$

where $V(t)$ = cumulative volume of flow up to time t from the start of the record, that is, the ordinate of the mass curve at any time t.

$Q(t)$ = discharge as a function of time, *i.e.*, the ordinate of the hydrograph

When it is desired to use water at uniform or nearly uniform rate greater than the minimum discharge in the stream, it is necessary to provide storage where water is impounded during periods of high flow for use during periods of low flows. Mass curve serves as a very useful tool to determine the required storage capacity for any uniform rate of demand.

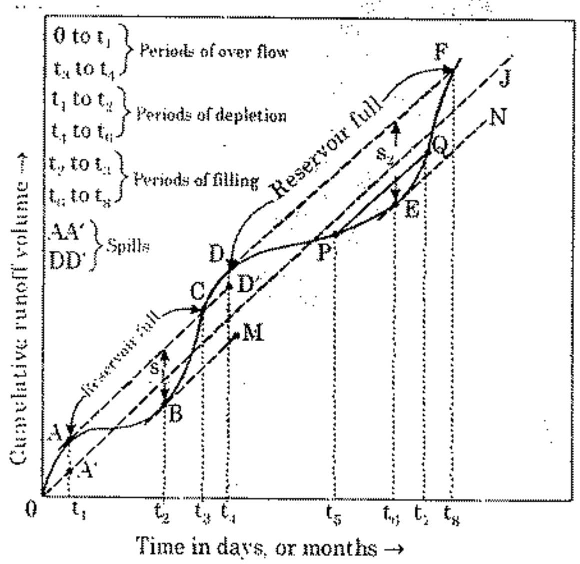


Fig. 10.5. Flow mass curve.

Storage Capacity Determination

- The difference in the ordinates of any two points on the curve is the summation of the flow during the intervening period of time. Thus, if two points such as P and Q in figure are joined by a straight line, the slope of the line PQ would give the average rate of flow during the period t_5 to t_7.
- The rate of flow at any time is indicated by the slope of the tangent drawn to the mass curve at the corresponding time.
- If the mass curve is horizontal, i.e., having zero slope, for a particular period of time it means that there is no discharge in the stream during that period.
- The rate of demand is the rate at which water is required for any use. A straight line having the slope equal to this rate is called a *demand line* or *draft line* or *use line*.
- Where the mass curve is steeper than the draft line, the flow in the stream is at a higher rate than the rate of draft and consequently some of the streamflow is available for storage, or it will spill if the reservoir is already full.
- Where the mass curve has flatter slope than the draft line, water will be drawn from storage in order to maintain the required rate of draft.

The storage required to meet any given draft rate may be found out from mass curve as follows.

- Let the line OJ represent the cumulative demand the slope of which is equal to the rate of draft. Let us assume that the reservoir is full at the beginning.
- From O to A the mass curve is steeper than OJ, indicating that the flow rate into the reservoir is more than the draft rate.
- Since the reservoir is already, full, no water can be stored though there is surplus water. hence, the reservoir will be overflowing during the period from 0 to t_1.
- The ordinate AA' indicates the total volume of the spill in this period.
- From A onwards the mass curve is flatter than OJ. So withdrawls from the reservoir start from the time t_1.
- As the amount of water indicated by AA' is already wasted, it cannot be included further in the analysis for the purpose of cumulative inflow. So all the inflows into the reservoir and the withdrawls from it are to be reckoned from A. In other words the origin will be shifted from O to A.
- So a line drawn parallel to OJ and tangential to mass curve at A, that is AC, will now become the cumulative demand.
- From A to B the mass curve is flatter than OJ. The reservoir is continuously depleted from t_1 to t_2 and S_1, the vertical intercept between the line AC and B, would indicate the maximum withdrawl from it.
- Thus S_1 determines the storage requirement for the dry period t_1 to t_2.
- From B to D the mass curve is again steeper than OJ and the reservoir starts filling from t_2 and it will be full at t_3. S_1 may be more easily measured as the vertical intercept between the two tangent lines AC and BM which are parallel to OJ and drawn to the mass curve at *peak* (*or the ridge*) point A and the *trough* (*or the valley*) point B.
- Again the reservoir will be overflowing from t_3 to t_4 and DD' indicates the amount of spill during this period.
- At D, the situation is identical to what it was at A.
- Therefore the vertical intercept S_2 between the two tangents DF and EN gives the storage requirement for the dry period t_4 to t_6.
- If the flow data for a large time period is available, the tangent lines are drawn at other ridge points and the storages are determined. The largest of these storages is then taken to be the actual storage requirement. That is,

$$S = \text{Maximum of } (S_1, S_2, \ldots\ldots)$$

Determination of Maintainable Max Demand Rate

- Mass curve can also be used to solve the reverse problem of determining the maximum demand rate that can be maintained by a given storage volume. However, it is a trial and error procedure.
- Straight lines tangential to the first peak point of the mass curve are drawn with different trial slopes such that all of them intersect the mass curve at or below the next higher peak point.
- The slope of the line for which the vertical intercept between itself and the trough point is equal to the given storage defines the possible maximum draft rate that can be maintained.
- Similar straight lines are drawn at other ridge points across the subsequent valleys and the possible demand rates are obtained.
- The smallest of the various demand rates thus found gives the maximum demand rate that can be maintained by a given storage.

Residual Mass Curve

In the residual mass curve, cumulative net inflow into the reservoir, i.e. the difference between the cumulative runoff and the cumulative demand, is plotted against time as shown in figure. In such a curve the inclined draft line OJ of previous figure is transformed into the horizontal line OJ of as shown in figure below and so are the tangent lines AC, BM and DF and EN. The storage required for the first dry period is now given as the difference between the ordinates of the residual mass curve at A and B and for the next dry period as that between the ordinates of D and E.

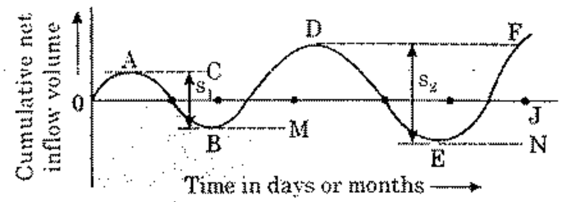


Fig. 10.6. Residual mass curve.

Example 5.6

Obtain the storage requirement of reservoir without the use of the mass curve.

Sol: When the runoff volume in any time period is more than the demand in the same time period, there is *surplus* water. This surplus water either goes to build up the storage or it will be spilled if the reservoir is already full. Therefore nothing is drawn from the reservoir during the surplus period. When the runoff in the stream is less than the demand, then there is a *deficit*. Water is drawn from the storage to meet the deficit. Starting from the beginning of the dry period, if the deficit in each time period is found out and if these deficits are added, the cumulative deficit at the end of the dry period denotes the maximum amount of water drawn from the storage and the capacity of the reservoir must at least be equal to this.

Similarly, if the cumulative surplus is found out for the following surplus period it must exceed the cumulative deficit to ensure that the reservoir will be full again and we can also find out when this happens. The table below shows the calculations to determine the storage capacity by this method.

Month	*I*	*Runoff volume* V_i	*Demand* D_i	$(V_i - D_i)$	*Cumulative deficit*	*Cumulative surplus*
June	1	3	8.33	–5.33	–5.33	
July	2	6	8.33	–2.33	–7.66	
Aug.	3	16	8.33	+7.67		+7.67
Sept.	4	30	8.33	+21.67		+29.34
Oct.	5	18	8.33	+9.67		+39.01
Nov.	6	15	8.33	+6.67		+45.68
Dec.	7	10	8.33	+1.67		+47.35
Jan.	8	8	8.33	–0.33	–0.33	
Feb.	9	6	8.33	–2.33	–2.66	
Mar.	10	4	8.33	–4.33	–6.99	
April	11	3	8.33	–5.33	–12.32	
May	12	1	8.33	–7.33	–19.65	
June	13	2	8.33	–6.33	–25.98	
July	14	5	8.33	–3.33	$\boxed{-29.31}$	
Aug.	15	17	8.33	+8.67		+8.67
Sept.	16	28	8.33	+19.67		+28.34
Oct.	17	20	8.33	+11.67		+40.01
Nov.	18	15	8.33	+6.67		+46.68
Dec.	19	12	8.33	+3.67		50.35
Jan.	20	7	8.33	–1.33	–1.33	
Feb.	21	5	8.33	–3.33	–4.66	
Mar.	22	4	8.33	–4.33	–8.99	
April	23	3	8.33	–5.33	–14.32	
May	24	2	8.33	–6.33	–20.65	

The Table above shows that the maximum cumulative deficit is 29.31 million m^3. Therefore the reservoir should have a capacity of 29.31 million m^3.

If the storage capacity of the reservoir is fixed as 29.31 million m^3 and if it is full at the beginning of the record, it will be depleted by 7.66 units by the end of July, full again by end of August, overflowing during September to December, empty by end of July, full again sometime in October and so on.

Example 5.7

The following table gives the average monthly runoff rates at a proposed reservoir site, the pan evaporation and the rainfall recorded at a nearby meteorological observations, and the estimated monthly demands. The downstream riparian rights require the release of natural flow or 18.5 million m^3 in each month whichever is less. The pan coefficient may be taken as 0.75. The average area of submergence at the reservoir site is 7.5 km^2 for which the runoff coefficient may be taken as 0.35. Determine the storage of the reservoir to meet the demands.

Month	*Average monthly runoff rate m^3/s*	*Evaporation mm*	*Precipitation mm*	*Monthly demand M m^3*
Jan.	18.8	120	18	19.6
Feb.	15.0	125	22	21.5
Mar.	11.0	130	9	23.4
April	7.5	190	11	27.0
May	6.0	200	5	24.0
June	19.0	160	135	26.0
July	75.0	120	225	20.0
Aug.	81.2	120	185	20.0
Sep.	60.0	110	205	18.5
Oct.	30.0	100	10	18.5
Nov.	26.4	90	45	16.0
Dec.	22.0	110	30	19.0

Sol: The runoff volume in any month in **Mm³** is given by

$$V = \frac{Q \times N \times 24 \times 60 \times 60}{10^6} = 0.0864\ QN$$

where Q is the mean monthly runoff rate in m^3/s and N is the number of days in the month. Therefore the runoff volume in May is equal to $0.0864 \times 6 \times 31 = 16.07$ M m^3 and in February, it is equal to $0.0864 \times 10 \times 28 = 36.29$ M m^3. The runoff volumes in M m^3 thus computed area shown in col. (2).

The volume of water evaporated from the reservoir is obtained from the expression $\left(A \times 10^6 \times \frac{E}{1000} \times C\right) m^3$, where A is the waterspread area in km^2, E is the evaporation in mm and C is the pan coefficient. Taking A as 75 C as 0.75, the volume is M m^3, is given by $0.005625\ E$. These evaporation losses are given in col. (4).

Except in the month of May, the natural flow is more than 18.5 M m^3 in all the months. Therefore the riparian release will be equal to 16.07 M m^3 in May, (which is the natural flow) and 18.5 M m^3 in all other months. These are shown in col. (5).

- The runoff coefficient is 0.35. In the absence of the reservoir only 35% of rainfall on the waterspread area becomes runoff which is already included in the monthly runoff volumes. But, when the reservoir is built 100% rainfall on the water spread area becomes runoff. Thus the additional volume of water added to the storage when the reservoir is built has to be estimated as $(1 - 0.35) \times A \times P$, and taking A is km² and P in mm, it is equal to $\underline{0.004875\ P}$ million m³. These volumes due to rainfall are given in col. (6).
- The monthly demands are shown in col. (3). Thus [col. (3) + col. (4) + col. (5) – col. (6)], which is entered in col. (7) indicates the net demand in each month. The difference between runoff volume V and the net demand D is computed for each month and entered in col. (8). When $(V - D)$ is positive, it is surplus and when it is negative, it is deficit. The cumulative deficit and the cumulative surplus are given in col. (9) and col. (10) respectively.

The storage required is equal to the maximum cumulative deficit, which in the present case is 72.309 M m³.

Month	*Monthly flow V M m³*	*Demand M m³*	*Evaporation M m³*	*Riparian release M m³*	*Rainfall M m³*	*Net demand D M m³*	*(V – D) M m³*	*Cumulative deficit M m³*	*Cumulative surplus M m³*
(1)	(2)	(3)	(4)	(5)	(6)	(7) = (3) + (4) + (5) – (6)	(8) = (2) – (7)	(9)	(10)
Jan.	50.35	19.6	0.675	18.5	0.089	36.686	+13.664	—	+13.664
Feb.	36.29	21.5	0.703	18.5	0.107	40.596	–4.306	–4.306	—
Mar.	29.46	23.4	0.731	18.5	0.044	45.287	–15.827	–20.133	—
April	19.44	27.0	1.069	18.5	0.054	46.515	–27.075	–47.208	—
May	16.07	24.0	1.125	16.07	0.024	41.171	–25.101	$\boxed{-72.309}$	—
June	49.25	26.0	0.900	18.5	0.658	44.742	+4.508	—	+4.508
July	200.88	20.0	0.675	18.5	1.097	38.078	+162.802	—	+167.310
Aug.	217.49	20.0	0.675	18.5	0.902	38.273	+179.217	—	+346.527
Sept.	155.52	18.5	0.619	18.5	0.999	36.620	+118.900	—	+465.427
Oct.	80.35	18.5	0.563	18.5	0.049	37.514	+42.836	—	+508.263
Nov.	68.43	16.0	0.506	18.5	0.219	34.787	+33.643	—	+541.906
Dec.	58.93	19.0	0.619	18.5	0.146	37.973	+20.957	—	+562.863

Droughts

- The extremes of the stream flow as reflected in floods and droughts.
- Drought is a climatic anomaly characterized by deficit supply of moisture. This may result from subnormal rainfall over large regions causing below normal natural availability of water over long periods of time.
- Unlike floods the droughts are of the creeping kind; they develop in a region over a length of time.
- During droughts the quality of available water will be highly degraded resulting in serious environmental and health problems.
- Drought can be classified as:

(a) Meteorological Drought

It is a situation where there is more than 25% decrease in precipitation from normal over an area.

(b) Hydrological Drought

Meteorological drought, if prolonged results in hydrological drought with marked depletion of surface water and groundwater. The consequences are the drying up tanks, reservoirs, streams and rivers, cessation of springs and fall in the ground water level.

(c) Agricultural Drought

This occurs when the soil moisture and rainfall are inadequate during the growing season to support healthy crop growth to maturity. There will be extreme crop stress and wilt conditions.

As per Indian meterological department

Decrease from normal precipitation	*Classification*
< 25	No drought effect
26-50%	Moderate
> 50%	Severe

- If the drought occurs in an area with a probability of $0.2 \le P \le 0.4$ the area is classified as a *drought prone area.*
- If the probability of occurrence of drought at an area is greater than 0.40, such an area is called as *chronically drought prone area.*

Agricultural drought is classified on the basis of aridity index.

An *aridity index* (AI) is defined as

$$\boxed{AI = \frac{PET - AET}{PET} \times 100}$$

where PET = Potential evapotranspiration, AET = actual evapotranspiration. The departure of AI from its corresponding normal value known as *AI anomaly*, represents moisture shortage. Based on AI anomaly, the intensity of agricultural drought is classified as follows:

AI anomaly	*Severity class*
1-25	Mild arid
26-50	Moderate arid
> 50	Severe arid

In addition to AI index, there are other indices such as Palmer Index (PI) and Moisture Availability Index (MAI) which are used to characterize agricultural drought.

□□□□

OBJECTIVE QUESTIONS

1. In a flow mass curve study, the damnd line drawn from a ridge does not intersect the mass curve again. This implies that
   (a) the reservoir is not full at the beginning
   (b) the storage is not adequate
   (c) the demand cannot be met by the inflow as the reservoir will not refill
   (d) the reservoir is wasting water by spill

2. The trap efficiency of a reservoir is a function of
   (a) inflow into the reservoir
   (b) ratio of inflow to storage capacity
   (c) ratio of reservoir capacity to inflow
   (d) reservoir capacity

3. In a tangent drawn prallel to the demand line from a ridge point of a mass curve does not intersect the mass curve again it can be inferred that the

(a) frequency of the flood entering into the reservoir is less
(b) inflow into the reservoir cannot meet the demand
(c) reservoir is overflowing resulting in wastage
(d) reservoir can meet higher demand

4. The life of a reservoir is determined by its capacity (C), volume of annual inflow into the reservoir (I) and concentration of sediment in the incoming flow (C_s). Life will be more if

(a) C, I and C_s are high
(b) C and I are high but C_s is low
(c) C is high but I and C_s are low
(d) C, I and C_s are low

5. Consider the following statements:

1. An ephemeral stream is one which has a base-flow contribution.
2. Flow characteristics of a stream depend upon rainfall and catchment characteristics and also the climatic factors which influence evapo-transpiration.
3. Sequent Peak Algorithm is used for estimating run off from rainfall.

Which of these statements is/are correct?

(a) 1, 2 and 3 (b) 1 and 3
(c) 2 and 3 (d) 2 alone

6. Which one of the following defines Aridity Index (AI)?

(a) $AI = \frac{PET - AET}{PET} \times 100$

(b) $AI = \frac{PET}{AET} \times 100$

(c) $AI = \frac{AET}{PET} \times 100$

(d) $AI = \frac{AET - PET}{AET} \times 100$

(where AET = Actual Evapotranspiration and PET = Potential Evapotranspiration)

7. Match List-I with List-II and select the correct answer using the code given below the lists:

List-I	List-II
A. Location	1. Perennial
B. Stability	2. Degrading
C. Variation of discharge	3. Tidal
D. Plan form	4. Braided

Codes:

	A	B	C	D
(a)	4	2	1	3
(b)	3	1	2	4
(c)	4	1	2	3
(d)	3	2	1	4

8. Maximum possible discharge from a small catchment corresponding to a particular rainfall intensity is independent of which one of the following?

   (a) Soil moisture condition
   (b) Drainage characteristics of catchment
   (c) Area of the catchment
   (d) Duration of the rainstorm

9. The land use of an area and the corresponding runoff coefficients are as follows:

SI No. No.	Land use	Area (ha)	Runoff coefficient
1.	Roads	10	0.70
2.	Lawn	20	0.10
3.	Residential area	50	0.30
4.	Industrial area	20	0.80

   What is the equivalent runoff coefficient?

   (a) 0.15 (b) 0.36
   (c) 0.40 (d) 0.51

10. A catchment consists of 30% area with runoff coefficient 0.40 with the remaining 70% area with runoff coefficient 0.60. The equivalent runoff coefficient will be

   (a) 0.48 (b) 0.54
   (c) 0.63 (d) 0.76

11. While applying the Rational formula for computing the design discharge, the rainfall duration is stipulated as the time of concentration because

   (a) this leads to the largest possible rainfall intensity
   (b) this leads to the smallest possible rainfall intensity
   (c) the time of concentration is the smallest rainfall duration for which the rational formula is applicable
   (d) the time of concentration is the largest rainfall duration for which the rational formula is applicable

12. The rainfall during three successive 2 hour periods are 0.5, 2.8 and 1.6 cm. The surface runoff resulting from this storm is 3.2 cm. The ϕ index value of this storm is

   (a) 0.20 cm/hr (b) 0.28 cm/hr
   (c) 0.30 cm/hr (d) 0.80 cm/hr

13. An isolated 4-hour storm occurred over a catchment as follows:

Time	1^{st} hr	2^{nd} hr	3^{rd} hr	4^{th} hr
Rainfall (mm)	9	28	12	7

The ϕ index for the catchment is 10 mm/h. The estimated runoff depth from the catchment due to the above storm is

(a) 10 mm (b) 16 mm
(c) 20 mm (d) 23 mm

ANSWERS

1. (c)	**5. (c)**	**9. (c)**	**13. (c)**
2. (c)	**6. (a)**	**10. (b)**	
3. (b)	**7. (d)**	**11. (c)**	
4. (d)	**8. (d)**	**12. (c)**	

CHAPTER

6

Stream Flow Measurement

- Water flowing in a stream is called stream flow.
- If the stream flow is unaffected by the artificial diversion, storage etc. then it is called runoff (virgin stream flow). Thus stream flow represents the runoff phase of hydrological cycle.
- Out of the various processes in hydrological cycle, like evaporation, precipitation, evapotranspiration etc. stream flow is amenable to fairly accurate measurement.
- Stream flow is measured in units of discharge (m^3/s)
- It is rather difficult to measure the discharge of flow in the natural streams directly as it is done in the case of flow in pipes or laboratory flumes using the flow meters. But it is very easy to make a direct and continuous measurement of *stage* in the river which is nothing but the height of the water surface in the river above some arbitrary datum. The higher the stage in the river, the higher is the discharge.

Hence two step procedure is followed for discharge measurement in a stream.

(a) Discharge in a given stream is related to elevation of the water surface (stage) through a series of careful measurements.

(b) Stage of the stream is observed routinely and discharge is estimated by stage-discharge relationship.

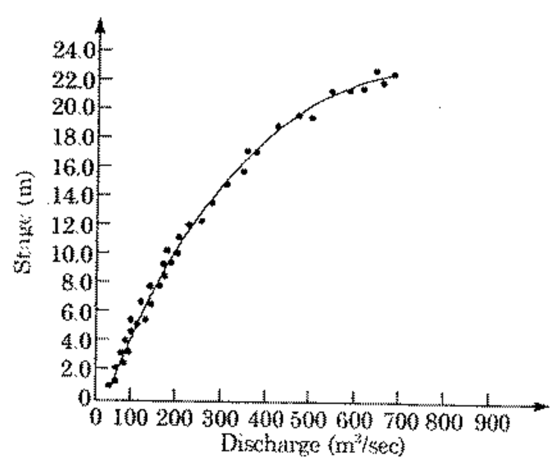


Fig 6.1. A stage discharge relationship curve.

Measurement of Stage

It is defined as the water surface elevation measured above a datum (can be MSL or any arbitrary datum).

- Manual stage measurements are done using
  (i) Staff gauge
  (ii) Wire gauge
- Automatic stage measurements are done using
  (i) Float gauge recorder
  (ii) Bubble gauge

Staff Gauge

The simplest of stage measurements are made by noting the elevation of the water surface in contact with a fixed graduated staff. The staff may be vertical or inclined. Sometimes, it may not be possible to read the entire range of water-surface elevations of a stream by a single gauge and in such cases the gauge is built in sections at different locations. Such gauges are called *sectional gauges*.

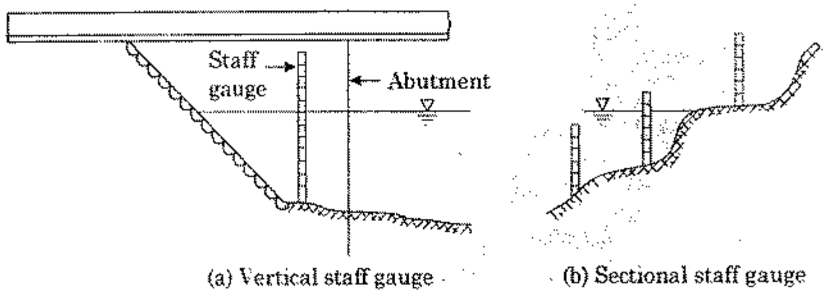


Fig. 6.2. Staff gauge. Subramaniam.

Wire Gauge

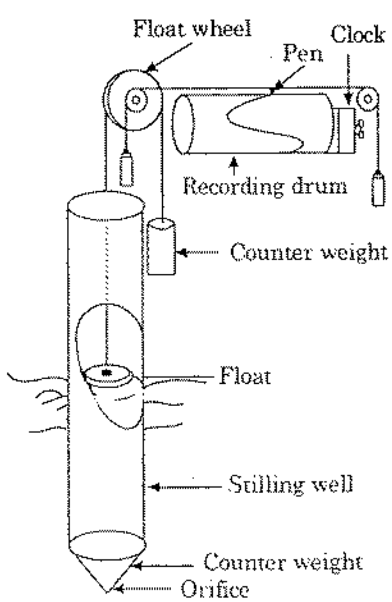
From a known elevation above the water surface such as bridge etc. wire is lowered and noting the length of wire, elevation of water surface can be calculated.

Automatic Stage Recorders

In manual gauge, reading is to be taken at frequent intervals. To overcome this, automatic recorders are used:

Float-Gauge Recorder

It is the most common type of automatic stage recorder in use. In this a float operating in a stilling well is balanced by means of a counterweight over the pulley of a recorder. Rising or lowering of the water surface elevation causes an angular displacement of the pulley which in turn causes linear displacement of a pen to record over a drum driven by clockwork.



Bubble Gauge

In this gauge compressed air or gas is fed through a tube and bubbled freely into stream through an orifice at a fixed elevation in the stream. The gas pressure in the tube is equal to the piezometric head on the bubble orifice. This is equal to the height of water above that level. Arragement is made to record this pressure head.

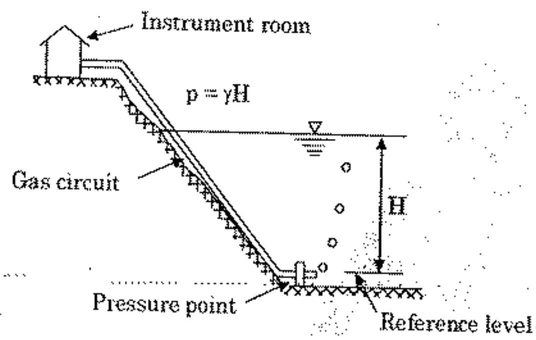
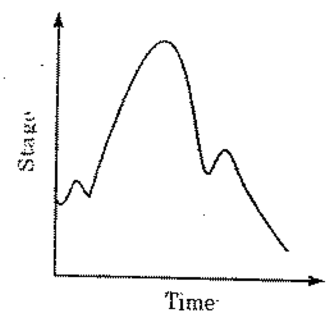


Fig. 6.3. Bubble gauge.

Stage Data Presentation

- Stage data is presented in the form of plot of stage against chronological time known as stage hydrograph.
- Uses:
  - Determination of stream discharge
  - Flood warning and flood protection works
  - Reliable long term data of peak floods can be analyzed statistically to estimate design peak river stages for use in the design of the hydraulic structures, such as bridges, weirs etc.



Measurement of Velocity

The 1st step in the measurement of discharge is the measurement of velocity. It is commonly measured by a mechanical device called current meter. Approximate stream velocity can be determined by floats.

- Current meter consists of a rotating element which rotates due to reaction of the stream current with an angular velocity proportional to the stream velocity.
- Two main types of current meter are:
  - Vertical axis meters
  - Horizontal axis meters.
- Current meter is so designed that its rotation speed varies linearly with the stream velocity at the location of the instrument.

  $v = a\,N_s + b$

  where

  v = stream velocity at the instrument location in m/s

  N_s = revolutions per second of the current meter

  a, b = constants of current meter

- Each instrument has a threshold velocity below which above equation is not valid.
- No of revolutions are counted for a known interval of time and from this N_s and hence V is known.

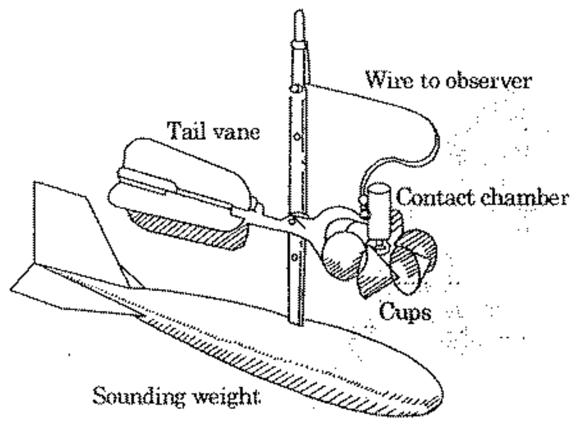


Fig. 6.4. Cup type current meter.

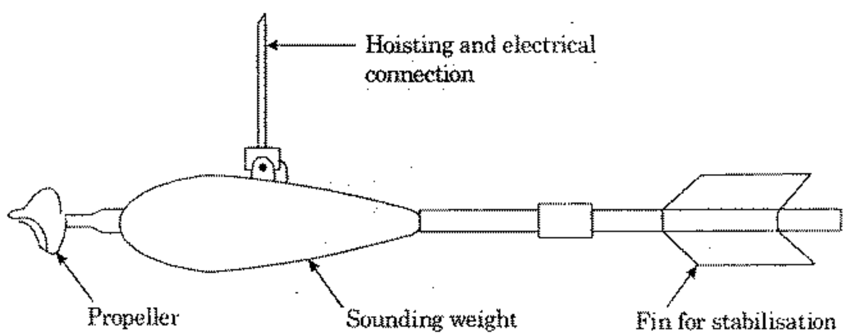


Fig. 6.5. Propeller type current meter.

Calibration of Current Meter

- The relation between stream velocity and revolutions per seconds of the current meter is called calibration equation.
- Calibration equation is unique to each instrument.
- It is determined by towing the instrument in a special tank.
- The towing tank is a long channel containing still water with arrangements for moving a carriage longitudinally over its surface at constant speed.
- The instrument to be calibrated is immersed to a specified depth and connected to a carriage.
- Average value of revoltuions per seconds (N_s) are determined for a predetermined constant speed (v) carriage. This is repeated over a complete range of velocities and a best fit linear relationship is formed.

Field Use

The velocity distribution in a stream across a vertical section is logarithmic in nature.

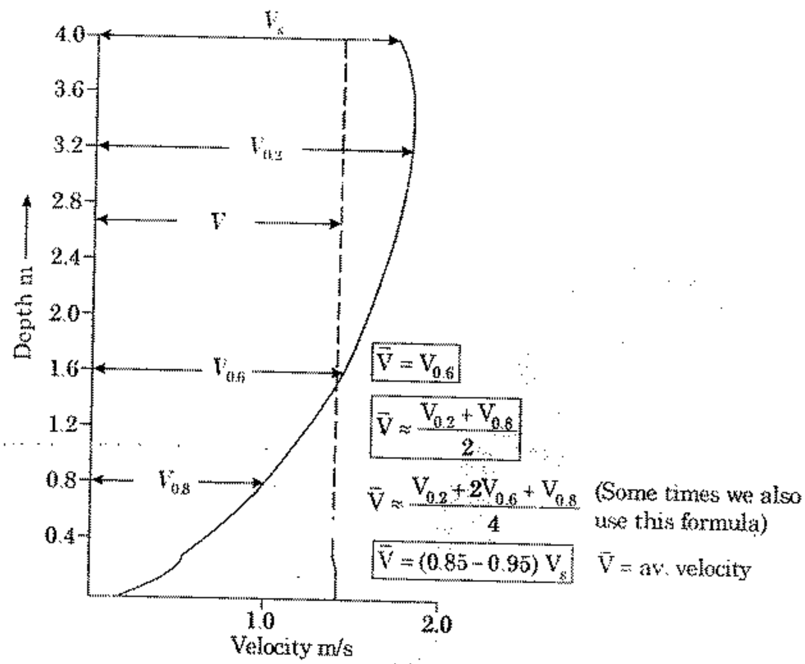


Fig. 6.6. Velocity distribution at a vertical.

To accurately determine the average velocity in a section an apox. method is used. In shallow streams of depth up to about 3.0 m, the velocity measured at 0.6 times the depth of flow below the water surface is taken as the average velocity $\bar{V}$ in the along the vertical line

$$\boxed{\bar{V} = V_{0.6}}$$

This is known as the single-point observation method.

In moderately deep streams the velocity is observed at two points; one at 0.2 times the depth of flow below the free surface ($V_{0.2}$) and other at 0.8 times the depth of flow below the free surface ($V_{0.8}$). The average velocity in the vertical $\bar{V}$ is taken as

$$\left[\bar{V} = \frac{V_{0.2}+V_{0.8}}{2}\right]$$

Sometimes the average velocity $\bar{V}$ is obtained by using a reduction factor K as

$$\boxed{\bar{V} = K\,V_s}$$

where, V_s = surface velocity.

The value of K is in the range of 0.85 to 0.95.

In small streams of shallow depth an observer stands in the water with the current meter held at requisite depth. This arragement is called ***wading***.

Velocity Measurement by Floats

When distance travelled (s) by a floating object in time (t) is noted, we can get surface velocity

$$\boxed{v_s = \frac{s}{t}}$$

From surface velocity average velocity is found out using correction factor

$$\boxed{\bar{v} = K.v_s}$$

This method is useful for:

(a) Small stream in floods

(b) Priliminary survey

Discharge Measurements

1. Direct determination of stream discharge is done using:

   (a) Area-velocity methods,

   (b) Dilution techniques,

   (c) Electromagnetic method, and

   (d) Ultrasonic method.

2. Indirect determination of flow is done using:

   (a) Hydraulic structures, such as weirs, flumes and gated structures and

   (b) Slope-area method.

Direct Method

(i) Area Velocity Method

After the velocities at no. of locations (A, B, C, D etc.) are noted using current meter, average velocity at various verticals are calculated like

$$\bar{V} = \frac{V_{0.2} + V_{0.8}}{2}$$

or $V_{0.6}$ as the case may be.

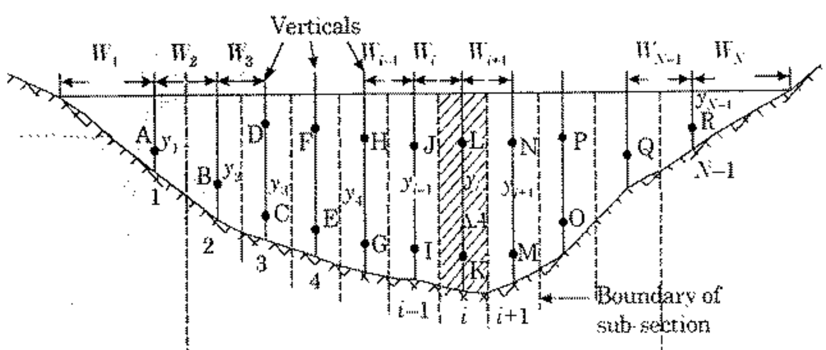


Fig. 6.7. Stream section for area-velocity method.

In the above figure discharge is calculated in three parts and added together to get total discharge.

$$Q_{total} = Q_{1st\ section} + Q_{last\ section} + Q_{remaining\ section}$$

1st section and last section area is taken as triangular area and remaining sections area is taken as rectangular. Thus

$$Q_{\text{1st section}} = \frac{1}{2}\times\left(W_1+\frac{W_2}{2}\right)\left[\left(\frac{y_1}{W_1}\right)\times\left(W_1+\frac{W_2}{2}\right)\right]\times\bar{V}_1$$

$$= \frac{\left(W_1+\frac{W_2}{2}\right)^2}{2W_1}\cdot y_1\cdot\bar{V}_1$$

$$\Rightarrow \text{ Average width} = \frac{\left(W_1+\frac{W_2}{2}\right)^2}{2W_1}$$

$$Q_{\text{last section}} = \frac{\left(W_N+\frac{W_{N-1}}{2}\right)^2}{2\,W_N}\times y_{N-1}.\bar{V}_{N-1}$$

$$Q_{\text{remaining section}} = \sum_{i=2}^{N-2}\left(\frac{W_i}{2}+\frac{W_{i+1}}{2}\right)y_i\times\bar{V}_i$$

No. of segments 'N' as above has to be chosen based on the following criteria.

1. The segment width should not be greater than 1/15 to 1/20 of the width of the river.
2. The discharge in each segment should be less than 10% of the total discharge.
3. The difference of velocities in adjacent segments should not be more than 20%.

Note: The depth a various verticals are measured using sounding rod or sounding weights. When the depth of stream is large, or when fast and accurate measurements are required, we use eco-depth recorder.

Other Approach of Area-Velocity Method

After the point velocities are obtained from current meter observations, they are indicated at the corresponding points on a cross-section of the gauging site, and the contours of equal velocity known as the *isovels* are then drawn as shown in figure. The areas between successive isovels are next planimetered and are multipled by the mean value of the isovels to give the incremental discharges. All these incremental discharges are then added to obtain the total flow. This method is more acurate than the previous method.

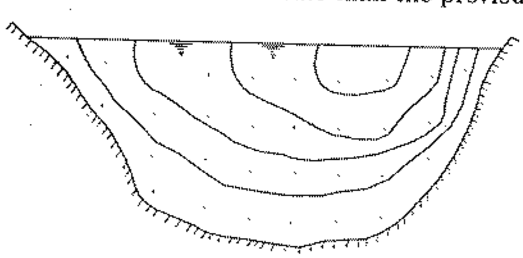


Fig. 6.8. Isovel plotted for discharge computation.

Example 6.1

The data pertaining to a stream-gauging operation at a gauging site are given below.
The rating equation of the current meter is v = 0.51 N_s + 0.03 m/s
Calculate the discharge in the stream

Distance from left water edge (m)	0	1.0	3.0	5.0	7.0	9.0	11.0	12.0
Depth (m)	0	1.1	2.0	2.5	2.0	1.7	1.0	0
Revolutions of a current meter kept at 0.6 depth	0	39	58	112	90	45	30	0
Duration of observation (s)	0	100	100	150	100	100	100	0

Solution: The calculations are performed in a tabular form.

For the first and last section,

Average width,
$$\bar{W} = \frac{\left(1+\frac{2}{2}\right)^2}{2\times 1} = 2.0 \text{ m}$$

For the rest of the segments,

$$\bar{W} = \left(\frac{2}{2}+\frac{2}{2}\right) = 2.0 \text{ m}$$

Since the velocity is measured at 0.6 depth, the measured velocity is the average velocity at that vertical ($\bar{v}$). The calculation of discharge by the mid-section method is shown in tabular form below:

Distance from left water edge (m)	Average width $\bar{W}$ (m)	Depth y (m)	Velocity $\bar{v}$ (m/s)	Segmental discharge ΔQ_i (m^3/s)
0	0	0	—	—
1	2.00	1.1	0.229	0.504
3	2.00	2.0	0.326	1.304
5	2.00	2.5	0.411	2.055
7	2.00	2.0	0.336	1.344
9	2.00	1.7	0.260	0.884
11	2.00	1.0	0.183	0.366
12	0	0		—
			$\Sigma\Delta Q_i =$	6.457

Total discharge $Q = 6.457$ m^3/s.

Example 6.2

Using the equation V(m/s) = 0.65 N + 0.03, obtain the velocity at 0.6 times the depth from the free surface. here N stands for revolutions/s. Data on current meter observations are given below in tabular form.

Distance from one bank (m)	Depth (m) (y)	Current meter observation at 6.0 y	
		No. of revolutions	Time in seconds
3.0	0.4	30	150
6.0	0.8	50	130
9.0	1.2	70	100
12.0	2.0	100	80
15.0	3.0	150	60
18.0	2.5	200	50
21.0	2.2	130	40
24.0	1.0	90	130

Also compute the discharge through the section:

Solution: The characteristic equation is given as

$$V(m/s) = 0.65\ N + 0.03$$

where N is the number of revolutions per second.

Assuming depth at a distance of 27 m from one bank to be zero.

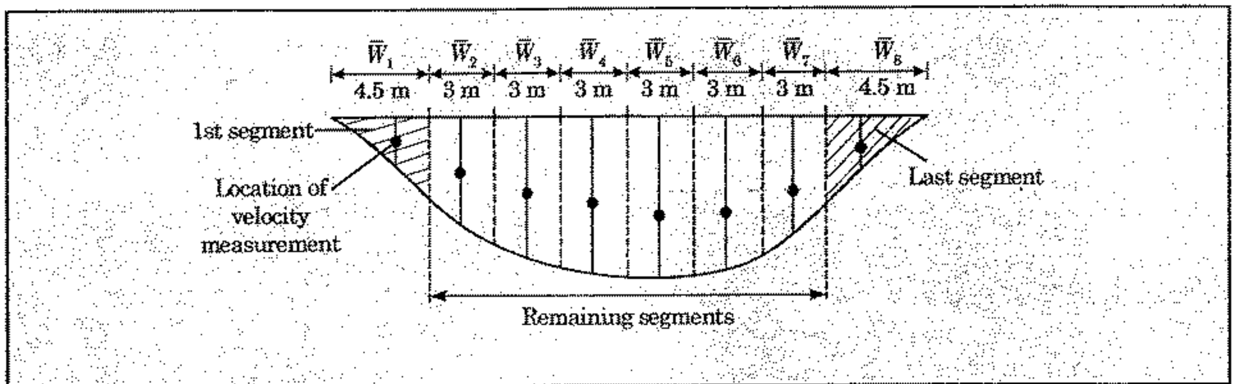
The total discharge is calculated by method of mid sections.

For the first and last section average width, $\bar{W} = \dfrac{\left(W_1 + \dfrac{W_2}{2}\right)^2}{2W_1} = \dfrac{\left(3+\dfrac{3}{2}\right)^2}{2\times 3} = 3.375$ m

For the rest of segments, $\bar{W} = \left(\dfrac{3}{2}+\dfrac{3}{2}\right) = 3$ m

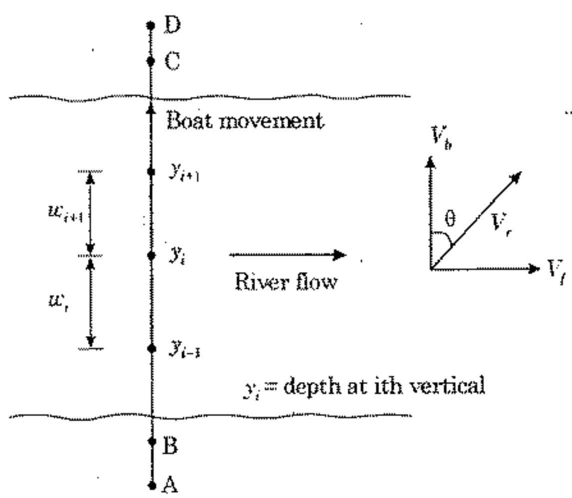
Distance from one bank (m)	Depth (m) (y)	Average width (m) $\overline{W}$	V	Segmental discharge $\Delta Q = y \times V \times \overline{W}$
3.0	0.4	3.375	0.16	0.216
6.0	0.8	3.0	0.28	0.672
9.0	1.2	3.0	0.485	1.746
12.0	2.0	3.0	0.843	5.058
15.0	3.0	3.0	1.655	14.895
18.0	2.5	3.0	2.63	19.725
21.0	2.2	3.0	2.14	14.124
24.0	1.0	3.375	0.48	1.62
27.0	0	–	—	

$\therefore$ Total discharge ΣQ, $\Sigma Q = 58.056\ m^3/sec.$



Moving Boat Method

The moving boat technique is similar to the conventional method of area-velocity approach. The main difference is in the method of data collection. In the conventional method the data at each point in the cross-section are collected while the observer is in a stationary position. In contrast, in the moving-boat method the data are collected while the observer is on a boat which is rapidly traversing the cross-section.



The boat is moved along a line perpendicular to the flow velocity and current meter is lowered upto a particular fixed depth below water surface. Thus while boat is moving velocity will be recorded at that depth. The point velocity is converted to average velocity.

The average velocity recorded by the current meter is V_r, which is the resultant of boat velocity (V_b) and flow velocity (V_f). Angle θ between V_r and V_b is noted using angle indicator arragement. Depth of water is also continuously noted using ecodepth recorder.

Thus the known information are V_r, Q_i, and cross-section of river.

Discharge is calculate as $\boxed{Q = \sum \Delta Q_i}$

where $\Delta Q_i = \left(\dfrac{y_i + y_{i+1}}{2}\right) W_{i+1} \times V_f$

$$= \left(\frac{y_i + y_{i+1}}{2}\right)\left[(V_r \cos\theta)(\Delta t)\right](V_r \sin\theta)$$

$$\boxed{\Delta Q_i} = \left(\frac{y_i + y_{i+1}}{2}\right) V_r^2 \cos\theta \sin\theta . \Delta t$$

Moving boat method is not recommended in stream with water depth less than 3 m.

Dilution Technique

In this method a solution of a stable chemical such as common salt or sodium dichromate or a radioactive chemical, known as the *tracer*, is injected into the stream at either a constant rate or all at once. When the tracer is introduced into the stream at a constant rate, it is called the *plateau* method and when it is introduced all at once it is called the '*gulp method*'.

The solution will be diluted by the discharge of the stream. The concentration of the tracer at a cross-section downstream (known as the sampling section) of the injection point is determined.

Let C_0 be the small initial concentration of the tracer in the streamflow. At section 1 a small quantity (volume V_1) of high concentration C_1 of this tracer is added all at once. The concentration profile at section 2, which is far away from 1 is schematically shown in the figure.

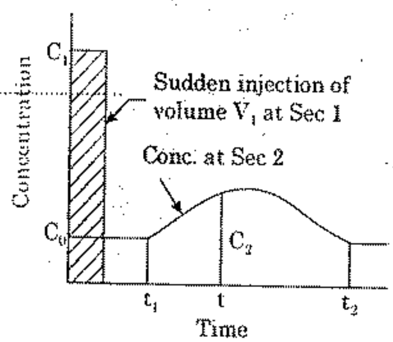


Fig. 6.9. Sudden-injection method.

The concentration will have a base value of C_0, increases from time t_1 to a peak value and gradually reaches the base value of C_0 at time t_2. The stream flow is assumed to be steady.

By Continuity Equation

Mass of tracer at section 1 = Mass of tracer in the original discharge of stream (Q) at section 2 + mass of tracer in the added additional volume (V_1) at section 2

$$\Rightarrow \quad V_1C_1 = \int_{t_1}^{t_2} Q\,(C_2 - C_0)\,dt + \int_{t_1}^{t_2} \frac{V_1(C_2 - C_0)}{(t_2 - t_1)}\,dt$$

The 2nd term is negligible and hence

$$\Rightarrow \quad \boxed{Q = \frac{V_1C_1}{\int_{t_1}^{t_2}(C_2 - C_0)\,dt}} = \text{Discharge of stream}$$

When the Tracer is Injected Continuously

Let concentration C_1 is added continuously at constant rate Q_t at section 1. Let the initial concentration in tracer in stream be C_0 and at section 2 after some time the concentration becomes constant at C_2 then

$$Q_tC_1 + Q\times C_0 = (Q + Q_t)\ C_2$$

$$\Rightarrow \quad \boxed{Q = \frac{Q_t(C_1 - C_2)}{(C_2 - C_0)}} = \text{discharge of stream}$$

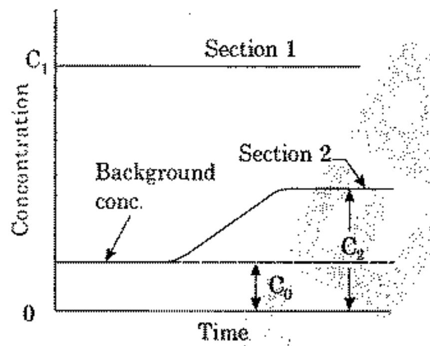
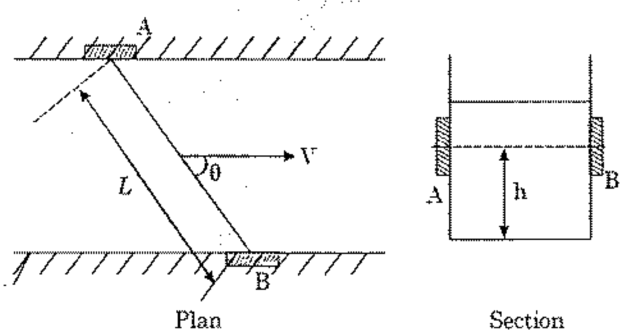


Fig. 6.10. Constant rate injection method.

Tracer method is most suitable for small turbulent streams in mountainous areas.

Ultrasonic Method

It is a type of area velocity method in which average velocity is measured by using ultrasonic signals.



Electronic transducer is placed as shown above, when signal is sent from 'A' it is recorded at B in time t_1

$$\Rightarrow \quad \frac{L}{u + V_P} = t_1,$$

u = velocity of sound in water, V_P = component of velocity along the sound path

Similarly if signal is sent from B it is received at A in time t_2

$$t_2 = \frac{L}{u - V_P}$$

$$\frac{1}{t_1} - \frac{1}{t_2} = \frac{2V_P}{L} = \frac{2v\cos\theta}{L}$$

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

v = velocity of flow of water at depth 'h' above stream bed. From this velocity average velocity of stream is calculated using caliberation.

$$\Rightarrow \boxed{\text{Discharge} = \text{Average velocity} \times \text{Area of x-section}}$$

Electro Magnetic Induction Method

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 this magnetic field, it produces an electromagnetic force (emf) which is related to the discharge in the river.

Indirect Method

In this methods we make use of the relationship between the flow discharge and the depths at specified locations. The field measurement is restricted to the measurements of these depths only and discharge is found out from the relation of depth with discharge.

Flow Measuring Structure

We know that there is fixed relationship between discharge and head 'H' existing at the location of flow measuring structures such as V-notch, broad crested weir etc.

like $\boxed{Q = KH^n}$

by meaning H, Q can be calculated.

However use of these is limited because of debris or sediment load of stream, backwater effects produced by their installation etc.

Slope Area Method

In this case discharge measurement is done using resistance equation like Manning's or Chazy's formula.

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

where h_L = head loss in the reach. The head loss h_L can be considered to be made up of two parts (i) frictional loss h_f and (ii) eddy loss h_e. Denoting $Z + y = h$ = water-surface elevation above the datum,

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

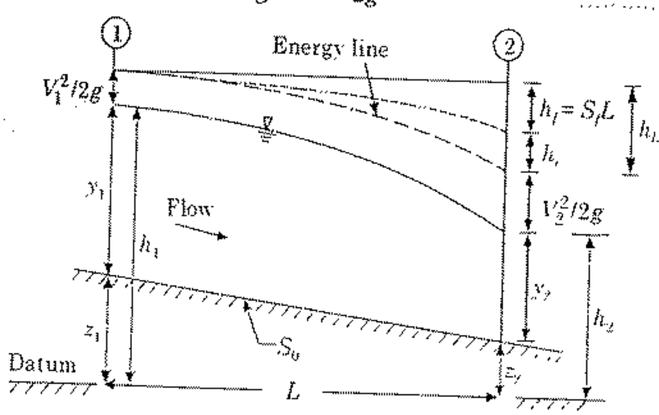


Fig. 6.11. Slope-area method.

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

If L = length of the reach, by Manning's formula for uniform flow,

$$\frac{h_f}{L} = S_f = \text{energy slope} = \frac{Q^2}{K^2}$$

where K = conveyance of the channel = $\frac{1}{n} AR^{2/3}$.

In nonuniform flow an average conveyance is used to estimate the average energy slope and

$$\frac{h_f}{L} = \bar{S}_f = \frac{Q^2}{K^2} \qquad \text{(ii)}$$

where $K = \sqrt{K_1 K_2}$; $K_1 = \frac{1}{n_1} A_1 R_1^{2/3}$ and $K_2 = \frac{1}{n_2} A_2 R_2^{2/3}$

n = Manning's roughness coefficient

The eddy loss h_e is estimated as

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

where K_e = eddy-loss coefficient having values as below.

Cross-section characteristic of the reach	Value of K	
	Expansion	Contraction
Uniform	0	0
Gradual transition	0.3	0.1
Abrupt transition	0.8	0.6

Equation (i), (ii) and (iii) together with the continuity equation $Q = A_1 V_1 = A_2 V_2$ enable the discharge Q to be estimated for known values of h, channel cross-sectional properties and n.

The discharge is calculated by a trial and error procedure using the following sequence of calculations:

1. Assume $V_1 = V_2$. This leads to $V_1^2/2g = V_2^2/2g$ and by Eq. (i) $h_f = h_1 - h_2 = F$ = fall in the water surface between sections 1 and 2.
2. From (ii) calculate discharge Q.
3. Compute $V_1 = Q/A_1$ and $V_2 = Q/A_2$. Calculate velocity heads and eddy-loss h_e.
4. Now calculate a refined value of h_f by Eq. (i) and go to step (2). Repeat the calculations till two successive calculations give values of discharge (or h_f) differing by a negligible margin.

   Some times when fairly straight reach of channel is selected, the eddy loss may be negligible.

- If area of x-section and wetted perimeters of three sections i.e. initial. Final and intermediate are given.

  Then conveyance of the channel $K = \frac{1}{n} AR^{2/3}$ can be calculated using

$$A = \frac{A_1 + 2A_3 + A_2}{4}$$

$$P = \frac{P_1 + 2P_3 + P_2}{4}$$

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

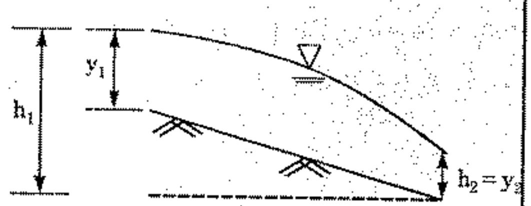
A_1 = Area of 1st section
A_2 = Area of 2nd section
A_3 = Area of intermediate section
P_1 = Wetted perimeter of 1st section
P_2 = Wetted perimeter of 2nd section
P_3 = Wetted perimeter of intermediate section

- This method can give only an approximate estimate of the discharge due to the following limitations: (i) Difficulty in proper selection of the rugosity coefficient n. (ii) The cross-sectional areas used in the discharge equation may be differing from those which existed during the time the flood passed the reach. (iii) The slope used in the discharge estimation cannot be precise as the water surface during the flood may be markedly warped.
- This method of discharge measurement is used mainly for the estimation of flood discharge when actual measurements for the area-velocity method are not possible for various reasons or when the estimation of discharge is to be made after the flood has passed the site.

Example 6.3

During a flood flow the depth of water in a 10 m wide rectangular channel was found to be 3.0 m and 2.9 m at two sections 200 m apart. The drop in the water-surface elevation was found to be 0.12 m. Assuming Manning's coefficient to be 0.025, estimate the flood discharge through the channel. Assume ($h_e = 0$).

Solution: Using suffixes 1 and 2 to denote the upstream and downstream section respectively, the cross-sectional properties are calculated as follows:



$h_1 - h_2 = 0.12$ m

Section 1	Section 2
$y_1 = 3.0$ m	$y_2 = 2.90$ m
$A_1 = 30$ m^2	$A_2 = 29$ m^2
$P_1 = 16$ m	$P_2 = 15.8$ m
$R_1 = 1.875$ m	$R_2 = 1.835$ m

$$K_1 = \frac{1}{0.025} \times 30 \times (1.875)^{2/3} = 1824.7 \qquad K_2 = \frac{1}{0.025} \times 29 \times (1.835)^{2/3} = 1738.9$$

Average K for the reach $= \sqrt{K_1 K_2} = 1781.3$

To start with h_f = fall = 0.12 m is assumed. (This follows from $V_1 = V_2$).

Eddy loss $h_e = 0$.

The calculations are shown in Table as shown below

$$\bar{S}_f = h_f/L = h_f/200$$

$$Q = K\sqrt{\bar{S}_f} = 1781.3\sqrt{\bar{S}_f}$$

$$\frac{V_1^2}{2g} = \left(\frac{Q}{30}\right)^2 \Big/ 2g$$

$$\frac{V_2^2}{2g} = \left(\frac{Q}{29}\right)^2 \Big/ 2g$$

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

$$h_f = fall + \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) = 0.12 + \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) \quad \text{(A)}$$

Trial	h_f (trial)	S_f (units of 10^{-4})	Q (m^3/s)	$V_1^2/2g$ (m)	$V_2^2/2g$ (m)	h_f by eq. (A)
1	0.1200	6.000	43.63	0.1078	0.1154	0.1124
2	0.1124	5.615	42.21	0.1009	0.1080	0.1129
3	0.1129	5.645	42.32	0.1014	0.1085	0.1129

The final value of Q may be taken as 42.32 m^3/s.

Example 6.4

In order to compute the flood discharge in a stream by the slope method, the following data have been obtained:

	Upstream section	Middle section	Downstream section
Area (m^2) →	108.6	103.1	99.80
Wetted perimeter (m) →	65.3	60.7	59.40
Gauge reading (m) →	316.8	–	316.55

Determine the flood discharge assuming Manning's n = 0.029 and length between downstream section and upstream section as 250 m.

Sol: Average area of reach $= \dfrac{A_1 + 2A_3 + A_2}{4}$

$$= \frac{108.6 + 2\times103.1 + 99.8}{4} = 103.65 \text{ m}^2$$

Average wetted perimeter of reach $= \dfrac{P_1 + 2P_3 + P_2}{4}$

$$= \frac{65.3 + 2\times60.7 + 59.4}{4} = 61.53 \text{ m}$$

Hydraulic radius $R = \dfrac{A}{P} = \dfrac{103.65}{61.53} = 1.685$ m

$$R^{2/3} = 1.416 \text{ m}$$

$$Q = \frac{1}{n}AR^{2/3}S^{1/2} = \frac{1}{0.029}\times 103.65\times 1.416\sqrt{S}$$

$$\boxed{Q = 5060.98\sqrt{S}\ m^3/s}$$

Assume $V_1 = V_2 \Rightarrow h_1 + \frac{V^2}{2g} = h_2 + \frac{V^2}{2g} + h_3$

$$\Rightarrow \boxed{h_L = h_1 - h_2}$$

The energy line slope $= \frac{h_L}{L} = S = \frac{316.8 - 316.53}{250}$

$$\Rightarrow \boxed{S = \frac{1}{1000}}$$

$$\Rightarrow Q = 5060.98\sqrt{\frac{1}{1000}} = 160.04\ m^3/s$$

Eddy losses are neglected.

Hence Next trial

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

$$\Rightarrow h_L = (h_1 - h_2) + \frac{V_1^2 - V_2^2}{2g} = (h_1 - h_2) + \frac{Q^2}{2g}\left(\frac{1}{A_1^2} - \frac{1}{A_2^2}\right)$$

$$S = \frac{(h_1 - h_2) + \frac{Q^2}{2g}\left(\frac{1}{A_1^2} - \frac{1}{A_2^2}\right)}{l}$$

$$= \frac{0.25 + \frac{(160.04)^2}{2\times 9.81}\left[\left(\frac{1}{108.6}\right)^2 - \frac{1}{(99.8)^2}\right]}{250}$$

$$= 9.184\times 10^{-4}$$

$$\Rightarrow Q = 5060.98\sqrt{S} = 153.38\ m^3/s$$

Next trial

$$\frac{V_1^2}{2g} = \frac{\left(\frac{153.38}{108.6}\right)^2}{2\times 9.81} = 0.10167$$

$$\frac{V_2^2}{2g} = \frac{\left(\frac{153.38}{99.8}\right)^2}{2\times 9.81} = 0.1204$$

$$\Rightarrow S = \frac{0.25 + (0.10167 - 0.1204)}{250} = 9.2508 \times 10^{-4}$$

$$\Rightarrow Q = 5060.98\sqrt{S} = 153.93 \text{ m}^3/\text{s}$$

Thus value is closer to the value of Q in previous trial and hence we should stop here.

The final value of Q can be taken as 153.93 m²/s.

Example 6.5

During the passage of a flood, the following data were estimated at two sections, 500 m apart:

Section	Water surface elevation (m)	Area of flow section (m^2)	Hydraulic mean depth (m)
Up stream, P	85.233	91.746	2.835
Down stream, R	85.176	84.354	2.917

The eddy loss coefficient for gradual contraction is to be taken as 0.1 and for gradual expansion as 0.35. Estimate the flood discharge passing through the reach, Manning's n = 0.022.

Sol:

$A_P = 91.746 \text{ m}^2$, $A_R = 84.354 \text{ m}^2$

$R_P = 2.835$ m, $R_R = 2.917$ m, $L = 500$ m

$h_P = 85.233$ m, $h_R = 85.176$ m, $n = 0.022$

$K = 0.1$ (since, the stream is contracting)

Conveyance is given by,

$$K_P = \frac{1}{n} A_P R_P^{2/3} = \frac{1}{0.022} \times 91.746 \times (2.835)^{2/3} = 8353.46$$

Similarly, $K_R = \dfrac{1}{0.022} \times 4.354 \times (2.917)^{2/3} = 7827.82$

$\therefore$ Average conveyance $(K_{avg}) = (K_P \times K_R)^{1/2} = (8353.46 \times 7827.82)^{1/2} = 8086.37$

Friction loss, $h_f = (h_P - h_R) + \left(\dfrac{V_P^2}{2g} - \dfrac{V_R^2}{2g}\right) - K\left|\dfrac{V_P^2}{2g} - \dfrac{V_R^2}{2g}\right|$ where K is 0.1

(i) 1st trial

Assuming $V_P = V_R$, we get

$h_f = h_P - h_R = 85.233 - 85.176 = 0.057$ m

But $Q = K_{avg}\sqrt{\dfrac{h_f}{L}} = 8086.37 \times \sqrt{\dfrac{0.057}{500}} = 86.34 \text{ m}^3/\text{sec}$

$$\therefore V_P = \frac{Q}{A_P} = \frac{86.34}{91.746} = 0.94 \text{ m/s}$$

$$V_R = \frac{Q}{A_R} = \frac{86.34}{84.354} = 1.02 \text{ m/s}$$

(ii) 2nd trial

$$V_P = 0.94 \text{ m/s},\ V_R = 1.02 \text{ m/s}$$

$$\therefore\ h_f = 0.057 + \left(\frac{0.94^2}{2\times 9.81} - \frac{1.02^2}{2\times 9.81}\right) - 0.1\left|\frac{0.94^2}{2\times 9.81} - \frac{1.02^2}{2\times 9.81}\right|$$

$$= 0.057 - 0.08 - 0.008$$

$$= 0.0482 \text{ m}$$

$$\therefore\ Q = K_{avg}\sqrt{\frac{h_f}{L}} = 8086.37 \times \sqrt{\frac{0.0482}{500}} = 79.39 \text{ m}^3/\text{sec}$$

$$\therefore\ V_P = \frac{Q}{A_P} = \frac{79.39}{91.746} = 0.86 \text{ m/s}$$

$$V_R = \frac{Q}{A_R} = \frac{79.39}{84.354} = 0.94 \text{ m/s}$$

(iii) 3rd trial

$$V_P = 0.86 \text{ m/s},\ V_R = 0.94 \text{ m/s}$$

$$\therefore\ h_f = 0.057 + \ 0.057 + \left(\frac{0.86^2}{2\times 9.81} - \frac{0.94^2}{2\times 9.81}\right) - 0.1\left|\frac{0.86^2}{2\times 9.81} - \frac{0.94^2}{2\times 9.81}\right|$$

$$= 0.057 - 0.0073 - 0.00073$$

$$= 0.04897 \text{ m}$$

$$\therefore\ Q = K_{avg}\sqrt{\frac{h_f}{L}} = 8086.37 \times \sqrt{\frac{0.04897}{500}} = 80.03 \text{ m}^3/\text{sec}$$

$$\therefore\ V_P = \frac{Q}{A_P} = \frac{80.03}{91.746} = 0.87 \text{ m/s}$$

$$\therefore\ V_R = \frac{Q}{A_R} = \frac{80.03}{84.354} = 0.95 \text{ m/s}$$

(iv) 4th trial

$$V_P = 0.87 \text{ m/s},\ V_R = 0.95 \text{ m/s}$$

$$\therefore\ h_f = 0.057 + \left(\frac{0.87^2}{2\times 9.81} - \frac{0.95^2}{2\times 9.81}\right) - 0.1\left|\frac{0.87^2}{2\times 9.81} - \frac{0.95^2}{2\times 9.81}\right|$$

$$= 0.057 - 0.0074 - 0.00074$$

$$= 0.04886 \text{ m}$$

$$\therefore\ Q = K_{avg}\sqrt{\frac{h_f}{L}} = 8086.37 \times \sqrt{\frac{0.04886}{500}} = 79.94 \text{ m}^3/\text{sec}$$

$$\therefore\ V_P = \frac{Q}{A_P} = \frac{79.94}{91.746} = 0.87 \text{ m/s}$$

$$V_R = \frac{Q}{A_R} = \frac{79.94}{84.354} = 0.95 \text{ m/s}$$

The final flood discharge can be adopted as, Q = 79.94 m³/sec.

Stage Discharge Relationship

After a sufficient number of discharge measurements have been made at a gauging station alongwith simultaneous stage observations, the results are plotted on an ordinary graph. Such a plot between the discharge (Q) and stage (G) is known as the *stage-discharge relation* or the *rating curve* of the gauging station. Once a stable stage-discharge relation is established, it is only a matter of recording the stage continuously which can be readily converted into the discharge through the above relation.

If the (G–Q) relationship for a gauging section is constant and does not change with time the control is said to be *permanent*. If it changes with time, it is called *shifting control*.

The term control has been used because at control section constant relationship exist between discharge and stage.

Permanent Control

For permanent control the relationship between the stage and the discharge is a single-valued relation which is expressed as:

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

in which Q = stream discharge, G = gauge height (stage), a = a constant which represent the gauge reading corresponding to zero discharge, C_r and β are rating curve constants.

For a given series of data, β and C_r are found out using regression analysis.

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

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

The coefficient of correlation r is given by

$$r = \frac{N(\Sigma XY) - (\Sigma X)(\Sigma Y)}{\left(\sqrt{N(\Sigma X^2) - (\Sigma X)^2}\right)\left(\sqrt{N(\Sigma Y^2) - (\Sigma Y)^2}\right)}$$

For a perfect correlation $r = 1.0$. If r is between 0.6 and 1.0, it is generally taken as a good correlation.

The constant a representing the stage (gauge height) for zero discharge in the stream is a hypothetical parameter and can not be measured in the field. It is calculated by some specialised method which is beyond the scope of this chapter.

Example 6.6

Following are the data of gauge and discharge collected at a particular section of the river by stream gauging operation. (a) Develop a gauge discharge relationship for this stream at this section for use in estimating the discharge for a known gauge reading. What is the coefficient of correlation of the derived relationship? Use a value of a = 7.50 m for the gauge reading corresponding to zero discharge. (b) estimate the discharge corresponding to a gauge readings of 10.5 m at this gauging section.

Gauge reading (m)	Discharge (m^3/s)	Gauge reading (m)	Discharge (m^3/s)
7.65	15	8.48	180
7.70	30	8.98	280
7.77	57	9.30	550
7.80	39	9.50	970
7.90	60	10.50	1900
7.91	100	11.10	1600
8.08	150	11.70	1200

Solution: (a) The gauge-discharge equation is

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

By taking logarithms

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

$$\text{or} \quad Y = \beta X + b$$

where $Y = \log Q$ and $X = \log (G - a)$.

Values of X, Y and XY are calculated for all the data as shown in the following table.

Gauge G(m)	$(G-a)$	Q (m^3/s)	$X = \log(G-a)$	$Y = \log Q$	(XY)
7.65	0.15	15	−0.824	1.176	−0.969
7.70	0.20	30	−0.699	1.477	−1.032
7.77	0.27	57	−0.569	1.756	−0.999
7.80	0.30	39	−0.523	1.591	−0.832
7.90	0.40	60	−0.398	1.778	−0.708
7.91	0.41	100	−0.387	2.000	−0.774
8.08	0.58	150	−0.237	2.176	−0.515
8.48	0.98	180	−0.009	2.255	−0.020
8.98	1.48	280	+0.170	2.447	0.416
9.30	1.80	550	0.255	2.740	0.699
9.50	2.00	970	0.301	2.987	0.899
10.55	3.05	1900	0.484	3.279	1.587
11.10	3.60	1600	0.556	3.204	1.781
11.70	4.20	1200	0.623	3.079	1.918

From the above table:

$$\Rightarrow \quad \Sigma X = -1.2545 \qquad \Sigma Y = 31.9460 \quad \Sigma XY = 1.456$$

$$\Sigma X^2 = 3.2457 \qquad \Sigma Y^2 = 79.0800$$

$$(\Sigma X)^2 = 1.5738 \quad (\Sigma Y)^2 = 1020.5$$

$$N = 14$$

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

$$= \frac{(14\times1.456)-(-1.2545)(31.9460)}{(14\times3.2457)-1.5738} = \frac{60.4644}{43.866} = 1.378$$

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

$$= \frac{(31.9460)-1.378(-1.2545)}{14} = 2.4054 = \log C_r$$

Hence $C_r = 254.3$

The required gauge – discharge relationship is therefore

$$Q = 254.3\,(G-a)^{1.378}$$

Coefficient of correlation

$$r = \frac{N(\Sigma XY)-(\Sigma X)(\Sigma Y)}{\sqrt{[N(\Sigma X^2)-(\Sigma X)^2][N(\Sigma Y^2)-(\Sigma Y)^2]}}$$

$$= \frac{60.4644}{\sqrt{(43.866)(14\times79.08-1020.5)}} = 0.981$$

As the value of r is very near 1.0 the correction is good.

(b) when $G = 10.05$, as $a = 7.5$ m

$$Q = 254.33\,(10.05-7.50)^{1.3378} = 924 \text{ m}^3/\text{s}$$

Shifting Control

The control that exists at a gauging section giving rise to a unique stage-discharge relationship can change due to: (i) changing characteristics caused by weed growth, dredging or channel encroachment, (ii) aggredation or degradation phenomenon in an alluvial channel, (iii) variable backwater effects affecting the gauging section and (iv) unsteady flow effects of a rapidly changing stage. There are no permanent corrective measure to tackle the shifting controls due to causes (i) and (ii) listed above. The only recourse in such cases is to have frequent current meter gauging and to update the rating curves.

Shifting Control Due to Backwater

If the shifting control is due to variable backwater curves, the same stage will indicate different discharges depending upon the backwater effect. To remedy this situation another gauge, called the *secondary gauge* or *auxiliary gauge* is installed some distance downstream of the gauging section and readings of both gauges are taken. The difference between the main gauge and the secondary gauge gives the *fall* (F) of the water surface in the reach.

The discharge is calculated as:

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

in which Q_0 = normalized discharge at the given stage when the fall is equal to F_0 and m = an exponent with a value close to 0.5.

Example 6.7

An auxiliary gauge was used downstream of a main gauge in a river to provide corrections to the gauge-discharge relationship due to backwater effects. The following data were noted at a certain main gauge readings:

Main gauge (m above datum)	Auxiliary gauge (m above datum)	Discharge (m^3/s)
86.00	85.50	275
86.00	84.80	600

If the main gauge reading is still 86.00 m and the auxiliary gauge reads 85.30 m, estimate the discharge in the river.

Solution: Fall (F) = main gauge reading – auxiliary gauge reading.

When $F_1 = (86.00 - 85.50) = 0.50$ m $\quad Q_1 = 275\ m^3/s$

$F_2 = (86.00 - 84.80) = 1.20$ m $\quad Q_2 = 600\ m^3/s$

$$\boxed{(Q_1/Q_2) = (F_1/F_2)^m}$$

$$(275/600) = (0.50/1.20)^m$$

Hence $m = 0.891$

When the auxiliary gauge reads 85.30 m, at a main gauge reading of 86.00 m.

Fall $F = (86.00 - 85.30) = 0.70$ m and

$$Q = Q_2\,(F/F_2)^m = 600\,(0.70/1.20)^{0.891} = 371\ m^3/s$$

Unsteady Flow Effect

When a flood wave passes a gauging station, in the advancing portion of the wave the approach velocities are larger than in the steady flow at corresponding stage. Thus for the same stage, more discharge than in a steady uniform flow occurs. In the retreating phase of the flood, reduced wave approach velocities gives lower discharges than in a equivalent steady flow case. Thus the stage-discharge relationship for an unsteady flow will not be a single-valued relationship as in steady flow but it will be a looped curve.

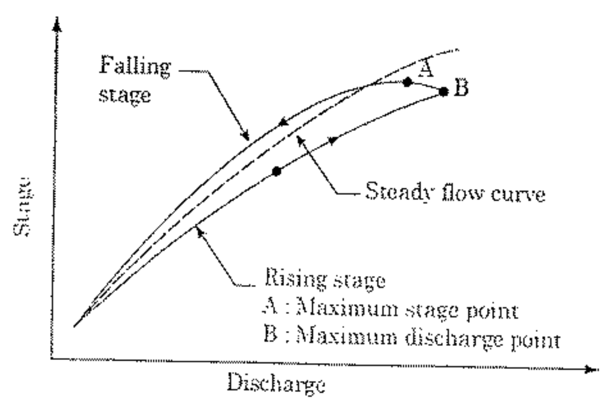


Fig. 6.12. Loop rating curve.

If Q_n is the normal discharge at a given stage under steady uniform flow and Q_M is the measured (actual) unsteady flow the two are related as

$$\frac{Q_M}{Q_n} = \sqrt{1+\frac{1}{V_w S_0}\frac{dh}{dt}}$$

where S_0 = channel slope = water surface slope at uniform flow, dh/dt = rate of change of stage and V_w = velocity of the flood wave.

Density of Hydrometry Stations (i.e. the station of discharge measurement)

S.No.	Region	Minimum density (km^2/station)	Tolerable density under difficult conditions (km^2/station)
1	Flat region of temperate, mediterranean and tropical zones	1,000–2,500	3,000–10,000
2	Mountainous regions of temperate mediterranean and tropical zones	300–1,000	1,000–5,000
3	Arid and polar zones	5,000–20,000	

As per WMO norms, India needs 1700 hydrometry stations.

□□□□

OBJECTIVE QUESTIONS

1. As a flood wave passes a given section of river, the time of occurrence of the maximum stage and that of the maximum discharge will be such that
   (a) the maximum discharge passes down before the maximum stage is attained
   (b) the maximum stage is attained before the maximum discharge passes down
   (c) the two events occur simultaneously
   (d) no specific sequence would be universally assignable

2. The slope area method is extensively used in
   (a) development of rating curve
   (b) estimation of flood discharge based on high-water marks
   (c) cases where shifting control exists
   (d) cases where back-water effect is present

3. The discharge per unit drawndown at the well is known as
   (a) Specific yield (b) Specific storage
   (c) Specific retention (d) Specific capacity

4. A catchment has an area of 150 hectares and a run-off/rainfall ratio of 0.40. If due to 10 cm rainfall over the catchment, a stream flow at the catchment outlet lasts for 10 hours, what is the average stream flow in the period?
   (a) 600,000 m^3/hr (b) 100 m^3/minute
   (c) 3.5 m^3/s (d) 1.33 m^3/s

5. Match **List-I** (Technique/Principle) with **List-II** (Purpose) and select the correct answer using the code given below the lists:

List-I

A. ϕ-Index
B. Slope - area method
C. Flow duration curve
D. Dilution technique

List-II

1. Dependable flow
2. Reservoir regulation
3. Steady stream discharge determination
4. Run-off volume
5. Unsteady stream discharge determination

Codes:

	A	B	C	D
(a)	3	5	1	4
(b)	4	1	2	3
(c)	3	1	2	4
(d)	4	5	1	3

6. In a river carrying a discharge of 142 m^3/s, the stage at a station A was 3.6 m and the water surface slope was 1 in 6000. If during a flood, the stage at A was 3.6 m and the water surface slope was 1 in 3000, what was the flood discharge (approximately)?
   (a) 284 m^3/s (b) 200 m^3/s
   (c) 164 m^3/s (d) 96 m^3/s

7. How is the average velocity along the vertical in a wide stream obtained?
   (a) By averaging the velocities at 0.2 and 0.8 depth from surface.
   (b) By measuring velocity at 0.6 depth below the surface.
   (c) By measuring velocity at half the depth.
   (d) By measuring velocity at 0.1 times the depth below the surface.

8. Consider the following with respect to measurement of stream flow during flood:
   1. Timing of the travel of floats released in the stream
   2. Use of weir formula for spillways provided on a dam
   3. Calculation of flow through a contracted opening at a bridge
   4. Using a current meter

   Which of the above is/are reliable and accurate?
   (a) 1 only (b) 4 only
   (c) 3 and 4 (d) 2 and 3

9. Calibration of a current meter for use, in channel flow measurement is done in a
   (a) wind tunnel (b) water tunnel
   (c) towing tank (d) flume

ANSWERS

1. (a)	3. (d)	5. (d)	7. (c)
2. (b)	4. (b)	6. (b)	8. (b)
			9. (c)

CHAPTER

7

Surface Water Hydrology (Hydrographs)

Hydrographs

In the previous chapter we discussed that annual, monthly and seasonal hydrographs are long term hydrographs which are used for studies like surface water potential of a stream, reservoir studies and drought studies. Whereas, flood hydrographs are used to study the flooding characteristics of a stream due to a rainfall. Hence flood hydrographs study is a short term study.

In this chapter our basic concern is the study of flood hydrographs. Flood hydrograph is important in flood control and flood forecasting and in establishing design flow for hydraulic structures which must pass the flood water.

Features of a Hydrograph

Hydrograph is the response of a given catchment to a rainfall input. it embodies in itself the combined effect of catchment and rainfall. The discharge noted in hydrograph is the combined effect of surface runoff, interflow and base flow.

If two storms occurs in a catchment such that the 2nd one doesnot start before the direct runoff due to 1st one has ceased, we get a single peaked hydrograph.

If however, the second storm starts before the direct runoff due to 1st storm has ceased, (complex storm) then multipeaked hydrograph are obtained.

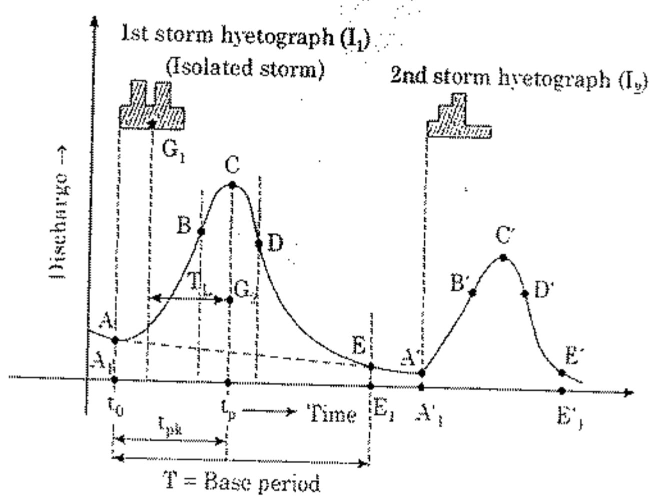


Fig. I. Single peaked hydrograph.

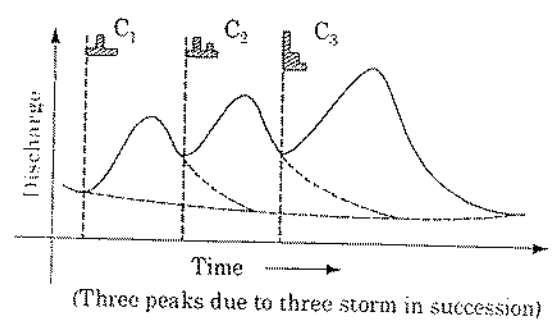


Fig. II. Multi Peak hydrograph.

In the above Figure (I):

- Hydrograph plotted as hydrograph $A_1ABCDEE_1$ is called hydrograph due to isolated storm I_1.
- Hydrograph due to isolated storm is a single peaked hydrograph.
- AB is rising limb or concentration curve.
- BCD is crest segment.
- DE is falling limb or recession limb.
- C is the point of crest or peak.
- t_p is the time of peak.
- B and D are inflection points.
- E is the end of direct runoff.
- EA' is the hydrograph in the period of ground water recession.
- A' is the beginning of direct runoff due to 2nd storm.
- $A_1'A'B'C'D'E'E_1'$ is called hydrograph due to isolated storm I_2.
- T = base period of 1st isolated storm hydrograph.
- A_1AEE_1 is the base flow contribution to total discharge.
- $ABCDEA$ is the direct runoff contribution to total discharge.
- AA_1 is the base flow discharge prior to the start of 1st isolated storm.
- G_1 is the centre of mass of rainfall.
- G_2 is the centre of mass of hydrograph.
- T_L = Lag time
- t_{PK} = is the time of peak from the starting point 'A'.

In the Figure (II) storm, C_1, C_2 and C_3 are called complex storms. A hydrograph due to complex storm (multi peaked hydrograph) can be resolved into corresponding single peaked hydrographs.

However, single peaked hydrographs resulting from isolated storms are generally preferred for hydrological analysis.

Factors Affecting Flood Hydrograph

Shape of hydrograph depends broadly on catchment and rainfall characteristics. The major factors affecting the shape are:

(i) Shape of the Catchment

A catchment that is shaped with the narrow end towards the upstream and the broader end nearer the catchment outlet (Figure 3(a)) shall have a hydrograph that is fast rising and has a rather concentrated high peak.

A catchment with the same area as in (Figure a) but shaped with its narrow end towards the outlet has a hydrograph that is slow rising and with somewhat lower peak (Figure 3(b)) for the same amount of rainfall. This is because for uniform rainfall distribution more rain fall is away from the outlet.

Though the volume of water that passes through the outlets of both the catchments is same (as areas and effective rainfall have been assumed same for both), the peak in case of the latter is ***attenuated.***

For other type of catchment the hydrograph may have the shape shown in Figure 3(c).

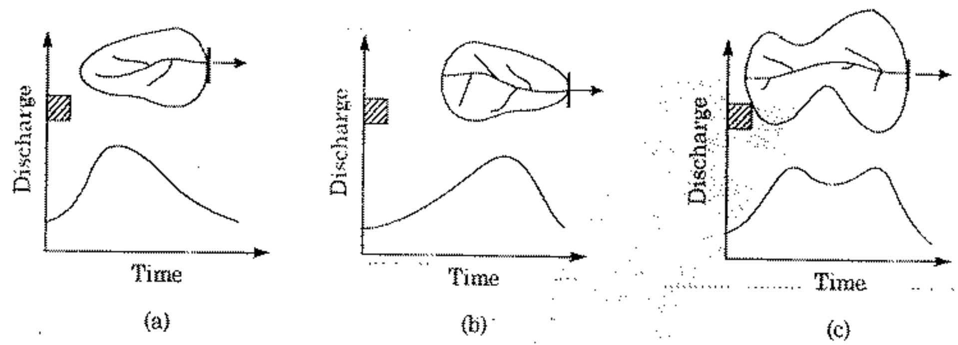


Fig. Effect of catchment shape on the hydrograph.

(ii) Size

Small basins behave different from the large ones in terms of the relative importance of various phases of the runoff phenomenon. In small catchments the overland flow phase is the more important than the channel flow, hence the land use and intensity of rainfall have important role on the peak flood. On large basins channel flow phase is more predominant. Hence drainage density has significant role in large catchment.

The time base of the hydrographs from larger basins will be larger than those of corresponding hydrographs from smaller basins.

(iii) Slope

Slope of the main stream or general land slope affects the shape of the hydrograph. Larger slopes generate more velocity than smaller slopes and hence can dispose off runoff faster. Thus the peak will come early and time base will be shorter.

General land slope is more important in smaller catchment where overland flow is predominant.Main stream slope is more important in large catchment because the channel flow is more important in this.

(iv) Drainage Density

Density of drainage has pronounced effect on peak of the hydrograph. If drainage density is higher, peak is more and if drainage density is low, peak is lower because in basins with smaller drainage densities, the overland flow is predominant and the resulting hydrograph is squat with a slowly rising limb.

(vi) Effect of Rainfall

(a) **Rainfall intensity:** For a given duration, the peak and volume of the surface runoff are essentially proportional to the intensity of rainfall.

(b) **Rainfall duration:** If the rainfall intensity is constant, then the rainfall duration determines the peak flow and time period of the surface runoff.

If uniform rainfall continues, the discharge will go on increasing upto the rainfall duration equal to time of concentration. If rainfall continues beyond the time of concentration the discharge will not increase only further. The neglecting base flow peaks discharge at the time of concentration is ($i \times A$), where i = intensity of uniform rainfall and A = Area of catchment.

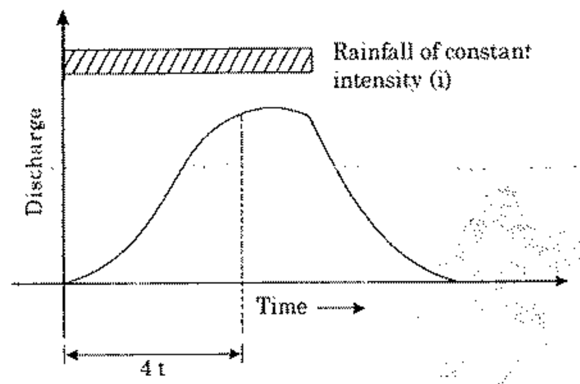
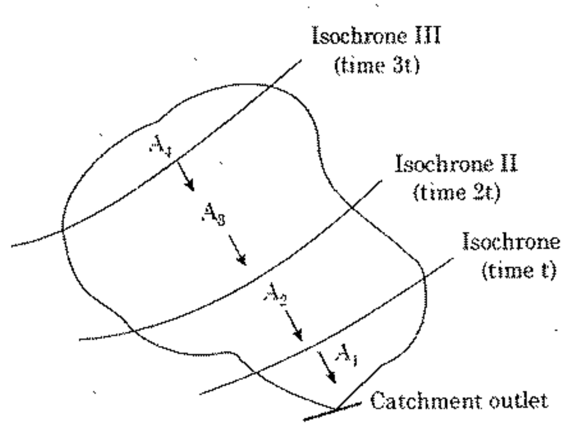


Fig. (Hydrograph due to constant rainfall intensity greater than the time in which entire catchment starts contributing.)

Note: *Isochrones are imaginary lines across the catchment from where water particles travelling downward take the same time to reach the catchment outlet.*

(c) Effect of distribution of rainfall in the catchment on hydrograph.



If only area A_1 receives rainfall but other areas do not then since the area is nearest to outlet, the resulting hydrograph immediately rises. Thus early peak will come. If the rainfall continues for duration greater than 't' in area A_1, discharge will reach a max constant value. [$i \times A_1$, if base flow is neglected].

If rainfall occurs with constant intensity in area A_4 only then there will be no direct runoff component in hydrograph upto time $3t$. Pictorially these things are shown as.

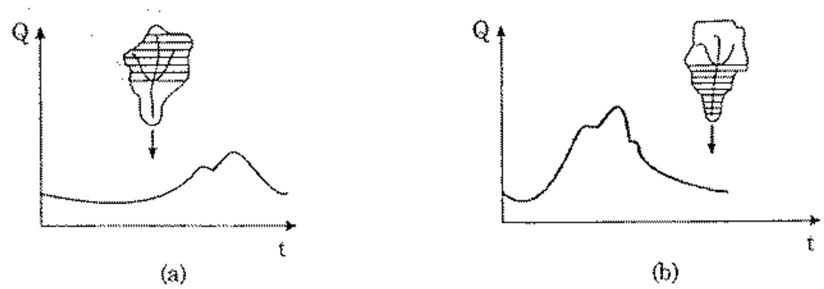


Fig. Effects of storm and basin characteristics on hydrograph shape.

(d) **Direction of storm movement:** If the storm moves from upstream of the catchment to the downstream end, there will be a quicker concentration of flow at the basin outlet. This results in a peaked hydrograph. However, if the storm movement is up the catchment, the resulting hydrograph will have a lower peak and longer time base.

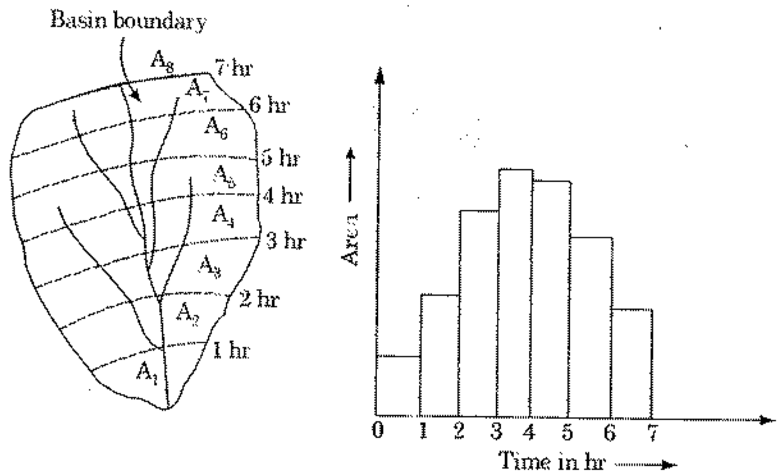
Components of a Hydrograph

The essential components of a hydrograph are: (i) the rising limb, (ii) the crest segment, and (iii) the recession limb.

Rising Limb

The shape of the rising limb depends mainly on the duration and the intensity distribution of rainfall, and to some extent on the antecedent condition and the shape of the *time area diagram* of the basin.

Note: The area between successive isochrones is measured and a *time area concentration diagram* (also called simply the *time area diagram*) is prepared.



(a) Isochrones (b) Time area diagram

Crest Segment

The peak discharge, included in the crest segment, represents the highest concentration of runoff from the basin. It occurs usually at a certain time after the rainfall has ended and this time depends on the aerial distribution of rainfall. The point of inflection at the end of crest segment commonly assumed to mark the time at which surface inflow to the channel system or the overland flow ceases.

Recession Limb

The recession limb extends from the point of inflection at the end of the crest segment to the commencement of the natural groundwater flow. It represents the withdrawal of water from the storage built up in the basin during the earlier phases of the hydrograph. The point of inflection represents the condition of maximum storage. Since the depletion of storage takes place after the cessation of rainfall, the shape of this part of the hydrograph is independent of storm characteristics and depends entirely on the catchment characteristics.

Equation of recession curve is generally given by

$$Q_t = Q_0 K_r^t$$

or

$$Q_t = Q_0 e^{-at} \text{ (where } a = -\ln K_r\text{)}$$

where Q_0 = initial discharge

Q_t = discharge at time t

K_r = recession constant having value less than unity.

Since Q_t represents the rate of depletion. Hence

$$\frac{-dS_t}{dt} = Q_t$$

where S_t = Storage in the catchment at time t

$$\Rightarrow \quad S_t = -\int Q_t\,dt$$

$$S_t = -\int Q_0 e^{-at} dt$$

$$S_t = \frac{-Q_0 e^{-at}}{-a} + C$$

$$\text{at} \quad t = \infty,\ S_t = 0$$

$$\Rightarrow \quad C = 0$$

$$\Rightarrow \quad S_t = \frac{Q_0}{a} e^{-at}$$

$$\boxed{S_t = \frac{Q_t}{-\ln K_r}}$$

Thus discharge at any time is proportional to storage remaining at that time.

Hydrograph Separation

Sometimes in hydrological analysis it is necessary to obtain direct runoff hydrograph (DRH) from total storm hydrograph (TSH).

When from total storm hydrograph base flow is separated we get surface runoff hydrograph or direct runoff hydrograph (DRH).

The various methods are as described below:

(i) Simply by drawing a line AC tangential to both the limbs at their lower portion. This method is very simple but is approximate and can be used only for preliminary estimates.

(ii) Extending the recession curve existing prior to the occurrence of the storm up to the point D directly under the peak of the hydrograph and then drawing a straight line DE, where E is a point on the hydrograph N days after the peak, and N (in days) is given by

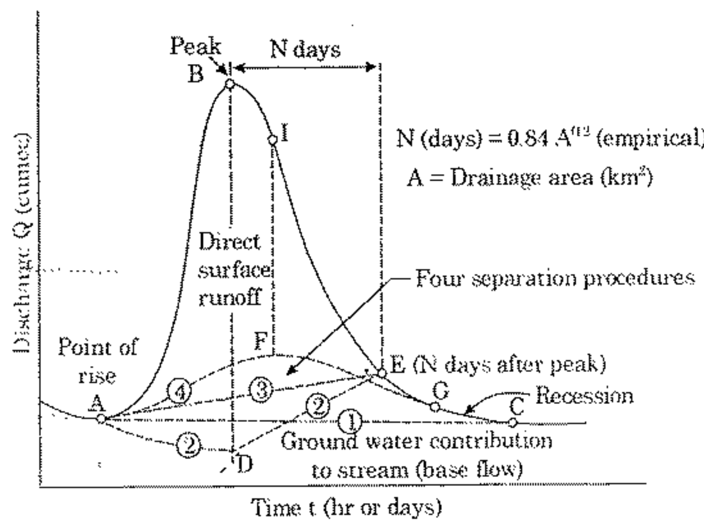


Fig. Hydrograph separation.

$$N = 0.83\ A^{0.2}, \text{ where } A = \text{area of the drainage basin, km}^2$$

(iii) Simply by drawing a straight line AE from the point of rise to the point E, on the hydrograph, N days after the peak.

(iv) Construct a line AFG by projecting backwards the ground water recession curve after the storm, to a point F directly under the inflection point of the falling limb and sketch an arbitrary rising line from the point of rise of the hydrograph to connect with the projected base flow recession.

In all the above four separation procedures, the area below the line constructed represents the base flow i.e., the ground water contribution to stream flow. Any further refinement in the base flow separation procedure may not be needed, since the base flow forms a very insignificant part of high floods. In fact, very often, a constant value of base flow is assumed.

Example 7.1

The flood data and base flow in a storm are estimated for a storm in a catchment area of 600 km^2. Estimate the rainfall excess.

Time in days	0	1	2	3	4	5	6	7	8	9
Discharge (m^3/sec)	20	63	151	133	90	63	44	29	20	20
Base flow (m^3/sec)	20	22	25	28	28	26	23	21	20	20

Solution: Ordinates of DRH after the separation of the base flow are: 0, 41, 126, 105, 62, 37, 21, 8, 0, 0. Then

$$\Sigma\Delta Q_t = (41 + 126 + 105 + 62 + 37 + 21 + 8) \times 1$$
$$= 400 \text{ m}^3$$
$$\Sigma\Delta Q_t = (400 \times 24 \times 60 \times 60) \text{ m}^3$$
$$\therefore \quad \text{Runoff depth} = \text{Rainfall excess} = \left(\frac{400\times24\times60\times60}{600\times10^6}\right)$$
$$= 0.0576 \text{ m} = 5.56 \text{ cm}$$

Effective Rainfall Hyetograph

When initial losses and infiltration losses are subtracted from the rainfall hyetrograph, we get effective rainfall hyetograph (ERH). It is also known as hyetograph of rainfall excess or supra rainfall.

Direct runoff hydrograph is the result of effective rainfall hyetograph:

(Area under ERH) × (Catchment area) = Volume of runoff = Area under direct runoff hydrograph

Example 7.2

A storm over a catchment of area 5.0 km^2 had a duration of 14 hours. The mass curve of rainfall of the storm is as follows:

Time from start of storm (h)	0	2	4	6	8	10	12	14
Accumulated rainfall (cm)	0	0.6	2.8	5.2	6.6	7.5	9.2	9.6

If the ϕ index for the catchment is 0.4 cm/h, determine the effective rainfall hyetograph and the volume of direct runoff from the catchment due to the storm.

Solution: First the depth of rainfall in a time interval Δt = 2 hours, in total duration of the storm is calculated, as show in table below.

In a given time interval Δt, effective rainfall (ER) is given by

$$ER = \text{(actual depth of rainfall} - \phi\Delta t\text{), negative value taken as zero.}$$

The calculations are shown in the Table. For plotting the hyetograph, the intensity of effective rainfall is calculated in col. 7.

The effective rainfall hyetograph is obtained by plotting ER intensity (col.7) against time from start of storm (col. 1) is shown in Fig. below.

$$\text{Total effective rainfall} = \text{Direct runoff due to storm}$$
$$= \text{area of ER hydrograph}$$
$$= (0.7 + 0.8 + 0.35 + 0.45) \times 2 = 4.6 \text{ cm}$$
$$\text{Volume of direct runoff} = \frac{4.6}{100} \times 5 \times (1000)^2$$
$$= 23000 \text{ m}^3$$

Time from start of storm (h)	Time interval Δt (h)	Accumulated rainfall in Δt (cm)	Depth of rainfall in Δt (cm)	φΔt (cm)	ER (cm)	Intensity of ER (cm/h)
1	2	3	4	5	6	7
0	-	0	-	-	-	-
2	2	0.6	0.6	0.8	0	0
4	2	2.8	2.2	0.8	1.4	0.7
6	2	5.2	2.4	0.8	1.6	0.8
8	2	6.7	1.5	0.8	0.7	0.35
10	2	7.5	0.8	0.8	0	0
12	2	9.2	1.7	0.8	0.9	0.45
14	2	9.6	0.4	0.8	0	0

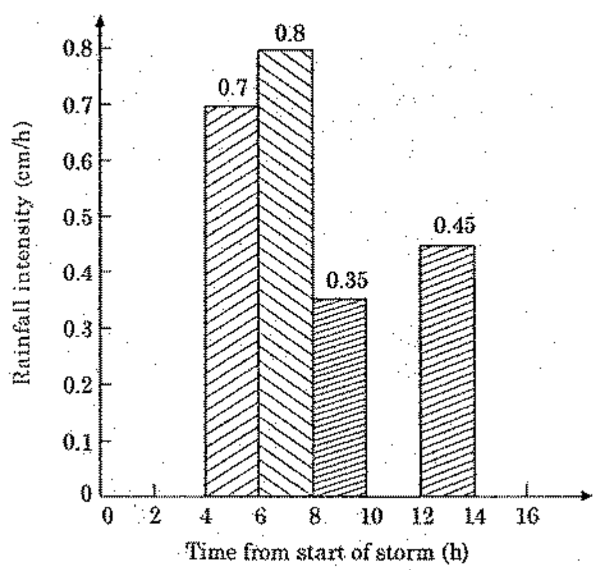


Fig. ERH of Storm.

Unit Hydrograph

- To predict the flood hydrograph from a known storm in a catchment one of the method is the use of unit hydrograph concept.
- The unit hydrograph of a drainage basin is defined as a hydrograph of direct runoff resulting from 1 cm of effective rainfall applied uniformly over the basin area at a uniform rate during a specified period of time (D-hr).
- Thus one can have a 6-hr unit hydrograph, 12-hr unit hydrograph etc.
- A 6-hr unit hydrograph will have an effective rainfall intensity of $\frac{1}{6}$ cm/hr.
- The effective rainfall intensity means the rainfall which will produce only runoff.
- In the D-hr unit hydrograph, D should not be more than any of the following (a) time of concentration, (b) lag time or (c) period of rise
- Volume of water contained inside the unit hydrograph (i.e. area of unit hydrograph) is equal to (1 cm × catchment area)

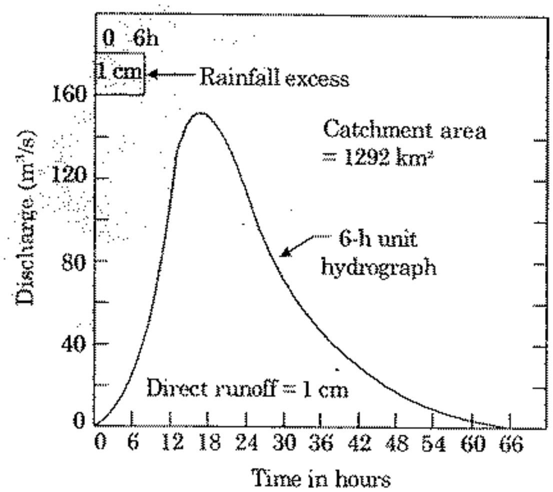


Fig. 6.9 Typical 6-h unit hydrograph.

Assumption made in the theory of unit hydrograph (As proposed by Sharman) are as follows:

Unit Hydrograph Assumptions

The following assumptions are made while using the unit hydrograph principle:

1. Effective rainfall should be uniformly distributed over the basin, that is, if there are 'N' rain gauges spread uniformly over the basin, then all the gauges should record almost same amount of rainfall during the specified time.

2. Effective rainfall is constant over the catchment during the unit time. i.e intensity is constant.

3. The direct runoff hydrograph for a given effective rainfall for a catchment is always the same irrespective of when it occurs.(Time invariance) Hence, any previous rainfall event is not considered to effect the new rainfall.

   Note: This antecedent precipitation is otherwise important because of its effect on soil-infiltration rate, depressional and detention storage, and hence, on the resultant hydrograph.

4. The ordinates of the unit hydrograph are directly proportional to the effective rainfall hyetograph ordinate. Hence, if a 6-h unit hydrograph due to 1 cm rainfall is given, then a 6-h hydrograph due to 2 cm rainfall would just mean doubling the unit hydrograph ordinates. Hence, the base of the resulting hydrograph (from the start or rise up to the time when discharge becomes zero) also remains the same. (Linear response)

   Note: This assumption of linear response in unit hydrograph enables the use of principle of superposition. Thus if two rainfalls of D-hr duration and magnitude M_1 and M_2 occur successively then combined direct runoff hydrograph due to these can be determined by multiplying the ordinate of a D-hr unit hydrograph 1st by M_1 and then by M_2 to obtain two DRHs and adding the ordinates of the two after lagging the ordinates of 2nd DRH by D-hr.

Assume that a 6-hour unit hydrograph (UH) of a catchment has been derived, whose ordinates are given in the following table and a corresponding graphical representation is shown in Figure A.

Time (hours):	0	6	12	18	24	30	36	42	48	54	60	66	72	78	84
Discharge (m^3/s):	0	5	15	50	120	201	173	130	97	66	40	21	9	3.5	2

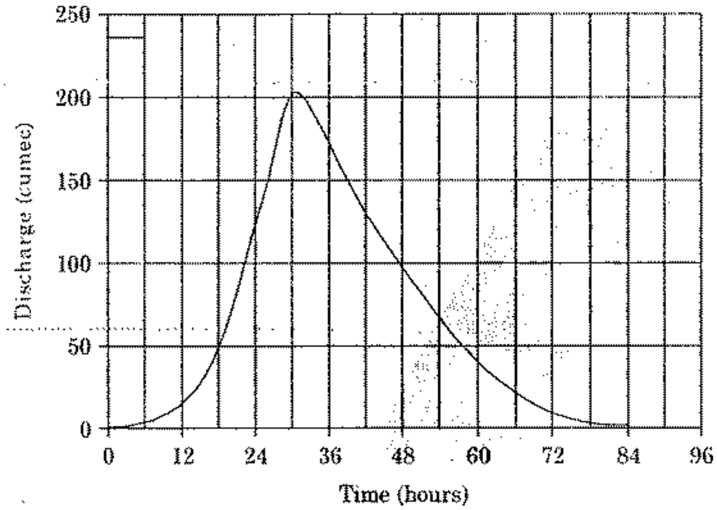


Fig. A 6-hour unit hydrograph.

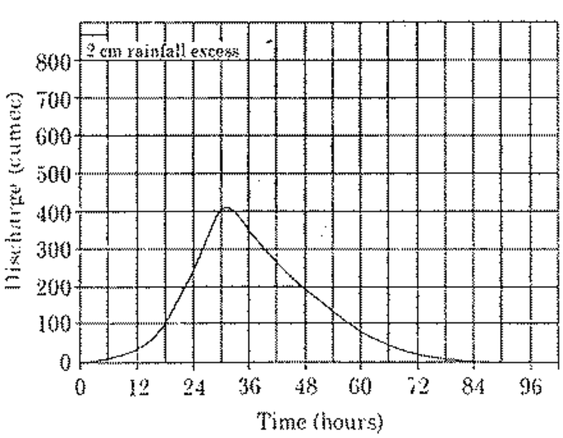
Assume further that the effective rainfall hyetograph (ERH) for a given storm on the region has been given as in the following table:

Time (hours):	0	6	12	18
Effective rainfall (cm):	0	2	4	3

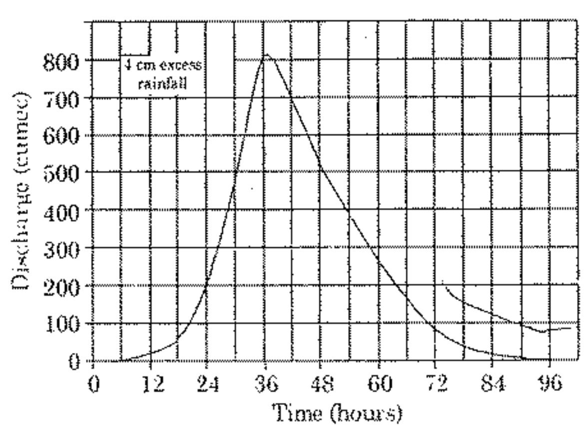
This means that in the first 6 hours, 2cm excess rainfall has been recorded. 4cm in the next 6 hours, and 3cm in the next.

The direct runoff hydrograph can then be calculated by the three separate hyetographs for the three excess rainfalls by multiplying the ordinates of the hydrograph by the corresponding rainfall amounts. Since the rainfalls of 2cm, 4cm and 3cm occur in successive 6-hour intervals, the derived DRH corresponding to each rainfall is delayed by 6 hours successively.

These have been shown in the figures indicated.



DRH for 2 cm excess rainfall in 0-6 hours



DRH for 4 cm excess rainfall in 6-12 hours

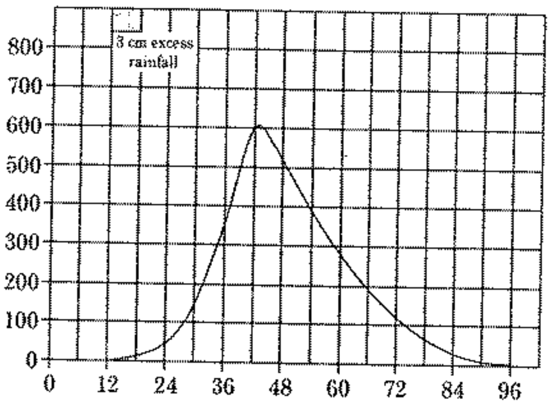


Fig. DRH corresponding to 3 cm excess rainfall during 12-18 hours.

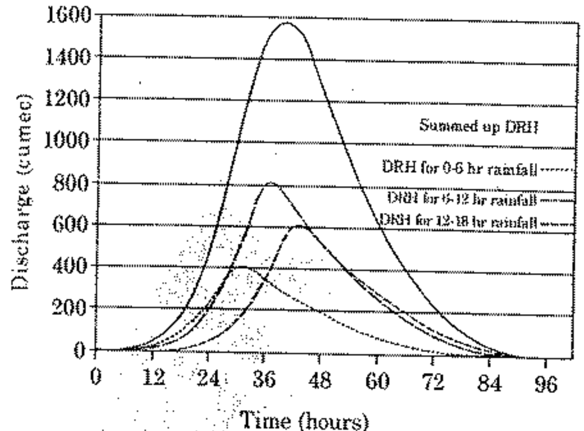


Fig. Final direct runoff hydrograph derived from summation of individual DRHs.

A sample calculation for the example solved graphically is given in the following table. Note the 6 hour shift of the DRHs in the second and subsequent hours.

Time (hours)	Unit hydrograph ordinates (m^3/s)	Direct runoff due to rainfall 2 cm excess in first 6 hours (m^3/s) (I)	Direct runoff due to 4 cm excess rainfall in second 6 hours (m^3/s) (II)	Direct runoff due to 3 cm excess rainfall in third 6 hours (m^3/s) (III)	Direct runoff hydrograph (m^3/s) (I)+(II)+(III)
0	0	0	0	0	0
6	5	10	0	0	10
12	15	30	20	0	50
18	50	100	60	15	175
24	120	240	200	45	485
30	201	402	480	150	1032
36	173	346	804	360	1510
42	130	260	692	603	1555
48	97	194	520	519	1233
54	66	132	388	390	910
60	40	80	264	291	635
66	21	42	160	198	400
72	9	18	84	120	222
78	3.5	7	36	63	106
84	2	4	14	27	45
90		0	8	10.5	18.5
96		0	0	6	6

The last column in the above table gives the ordinates of the DRH produced by the ERH. If the base flow is known or estimated, then this should be added to the DRH to obtain the 6-hourly ordinates of the flood hydrograph.

Example 7.3

Two storms each of 6-h duration and having rainfall excess values of 3.0 and 2.0 cm respectively occur successively. The 2-cm ER rain follows the 3-cm rain. The 6-h unit hydrograph for the catchment is the same. Calculate the resulting DRH.

Solution: First, the DRHs due to 3.0 and 2.0 cm ER are calculated by multiplying the ordinates of the unit hydrograph by 3 and 2 respectively. Noting that the 2-cm DRH occurs after the 3-cm DRH, the ordinates of the 2-cm DRH are lagged by 6 hrs as shown in column 4 of Table. Columns 3 and 4 give the proper sequence of the two DRHs. Using the method of superposition, the ordinates of the resulting DRH are obtained by combining the ordinates of the 3- and 2-cm DRHs at any instant by this process, the ordinates of the 5 cm DRH are obtained in column 5. Figure shown below shows the component 3- and 2-cm DRHs as well as the composite 5-cm DRH obtained by the method of superposition.

Time (h)	Ordinate of 6-h UH (m^3/s)	Ordinate of 3-cm DRH (col. 2) × 3	Ordinate of 2-cm DRH (col. 2) lagged by (6h) × 2	Ordinate of 5-cm DRH (col. 3 + 4) (m^3/s)	Remarks
1	2	3	4	5	6
0	0	0	0	0	
3	25	75	0	75	
6	50	150	0	150	
9	85	255	50	305	
12	125	375	100	475	
15	160	480	170	650	
18	185	555	250	805	
(21)	(172.5)	(517.5)	(320)	(837.5)	Interpolated value
24	160	480	370	850	
30	110	330	320	650	
36	60	180	220	400	
42	36	108	120	228	
48	25	75	72	147	
54	16	48	50	98	
60	8	24	32	56	
(66)	(2.7)	(8.1)	16	(24.1)	Interpolated value
69	0	0	(10.6)	(10.6)	Interpolated value
75	0	0	0	0	

Note the interpolation involved. Interpolation is required due to unequal time interval.

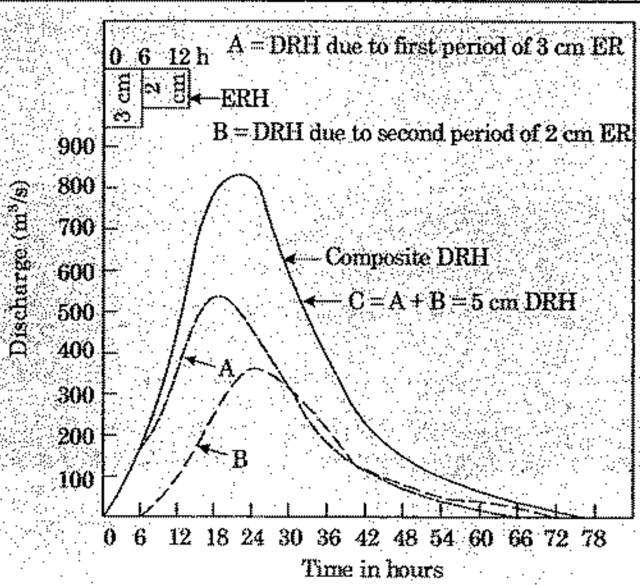


Fig. Principal of super position.

Example 7.4

The ordinates of a 6-hour unit hydrograph of a catchment is given below.

Time (h):	0	3	6	9	12	15	18	24	30	36	42	48	54	60	69
Ordinate of 6-h UH:	0	25	50	85	125	160	185	160	110	60	36	25	16	8	0

Derive the flood hydrograph in the catchment due to the storm given below:

Time from start of storm (h):	0	6	12	18
Accumulated rainfall (cm):	0	3.5	11.0	16.5

The storm loss rate (ϕ-index) for the catchment is estimated as 0.25 cm/h. The base flow can be assumed to be 15 m^3/s at the beginning and increasing by 2.0 m^3/s for every 12 hours till the end of the direct-runoff hydrograph.

Solution: The effective rainfall hyetograph is calculated as in the following table. The direct runoff hydrograph is next calculated by the method of superposition as discussed above. The ordinates of the unit hydrograph are multiplied by the ER values successively. The second and third set of ordinates are advanced by 6 and 12 h respectively and the ordinates at a given time interval added. The base flow is then added to obtain the flood hydrograph shown in the Table.

Interval	1st 6 hours	2nd 6 hours	3rd 6 hours
Rainfall depth (cm):	3.5	(11.0 – 3.5) = 7.5	(16.5 – 11.0) = 5.5
Loss @ 0.25 cm/h for 6 h:	1.5	1.5	1.5
Effective rainfall (cm):	2.0	6.0	4.0

Month	*Ordinates of U.H.*	*DRH due to 2 cm ER col. 2 × 2.0*	*DRH due to 6 cm ER × 6.0 (advanced by 6 h)*	*DRH due to 4 cm ER col.2 × 4.0 (advanced by 12 h)*	*Ordinates of final DRH (col. 3 + 4 + 5)*	*Base flow (m^3/s)*	*Ordinates of food hydrograph (m^3/s) (col. 6 + 7)*
1	2	3	4	5	6	7	8
0	0	0	0	0	0	15	15
3	25	50	0	0	50	15	65
6	50	100	0	0	100	15	115
9	85	170	150	0	320	15	335
12	125	250	300	0	550	17	567
15	160	320	510	100	930	17	947
18	185	370	750	200	1320	17	1337
(21)	(172.5)	(345)	960	340	1645	(17)	1662
24	160	320	1110	500	1930	19	1949
(27)	(135)	(270)	(1035)	640	1945	19	1964
30	110	220	960	740	1920	19	1939
36	60	120	660	640	1420	21	1441
42	36	72	360	440	872	21	893
48	25	50	216	240	506	23	529
54	16	32	150	144	326	23	349
60	8	16	96	100	212	25	237
66	(2.7)	(5.4)	48	64	117	25	142
69	0	0	—	—	—	—	—
72		0	16	32	48	27	75
75		0	0	—	—	—	—
78		0	0	(10.8)	(11)	27	49
81				0	0	27	27

Note: Due to the unequal time intervals of unit hydrograph ordinates, a few entries, indicated in paraemters have to be interpolated to complete the table.

Derivation of UH from a Simple Flood Hydrograph of Isolated Storm

Different steps required to derive UH are:

Step 1: From the given flood hydrograph, separate the base flow by any one of the methods. Most commonly used method to draw a straight line for simplicity.

Step 2: Determine the volume of DRH by the formula:

$$\text{Volume of DSR} = \Sigma Q\Delta t = \text{area under DRH}$$

Step 3: Divide this volume by known area of catchment to get rainfall or rainfall excess in (cm).

Step 4: Divide the ordinates of DRH by the depth of rainfall excess to obtain ordinates of UH.

Step 5: Plot the ordinates of UH against time to get the UH of the catchment.

Example 7.4 [illegible]

The following are the ordinates of the flood hydrograph from a catchment area of 780 km^2 due to 6 hr storm. Derive the 6 hr unit hydrograph of the basin.

Time (hrs):	6	12	18	24	6	12	18	24	6	12	18	24	6
Discharge (m^3/sec):	40	64	215	360	405	350	270	205	145	100	70	50	40

Sol: Assume a base flow of 40 m^3/sec. Then

ΣQ for direct surface runoff

$$= (64-40)+(215-40)+(360-40)+(405-40)+(350-40)+(270-40)+(205-40)+(145-40)+(100-40)+(70-40)+(50-40)$$

$$= (24+175+320+365+310+230+165+105+60+30+10)$$

$$= 1794 \text{ m}^3/\text{sec}$$

$$\therefore \text{ DRH in depth} = \left(\frac{1794\times6\times60\times60}{780\times10^6}\times100\right) \text{ cm} = 4.968 \text{ cm}$$

Therefore, the ordinates of UH are obtained by dividing the ordinates of DRH hydrograph by rain excess 4.968 cm to get ordinates of UH.

Time (hrs):	6	12	18	24	6	12	18	24	6	12	18	24	6
Ordinates of UH:	0	4.83	35.22	64.42	74.47	62.4	46.29	33.21	21.13	12.077	6.04	2.01	0

Example 7.5

(a) The peak of flood hydrograph due to a 3-h duration isolated storm in a catchment is 270 m^3/s. The total depth of rainfall is 5.9 cm. Assuming an average infiltration loss of 0.3 cm/h and a constant base flow of 20 m^3/s estimate the peak of the 3-h unit hydrograph (UH) of this catchment.

(b) If the area of the catchment is 567 km^2 determine the base width of the 3-h unit hydrograph by assuming it to be triangular in shape.

Sol: (a) Duration of rainfall excess = 3 h

Total depth of rainfall = 5.9 cm

Loss @ 0.3 cm/h for 3 h = 0.9 cm

Rainfall excess = 5.9 − 0.9 = 5.0 cm

Peak flow:

Peak of flood hydrograph = 270 m^3/s

Base flow = 20 m^3/s

Peak of DRH = 250 m^3/s

$$\text{Peak of 3-h unit hydrograph} = \frac{\text{peak of DRH}}{\text{rainfall excess}}$$

$$= \frac{250}{5.0} = 50 \text{ m}^3/\text{s}$$

(b) Let B = base width of the 3-h UH in hours.

Volume represented by the area of UH = volume of 1 cm depth over the catchment

Area of UH = (Area of catchment × 1 cm)

$$\frac{1}{2} \times B \times 60 \times 60 \times 50 = 567 \times 10^6 \times \frac{1}{100}$$

$$B = \frac{567 \times 10^4}{9 \times 10^4} = 63 \text{ hours.}$$

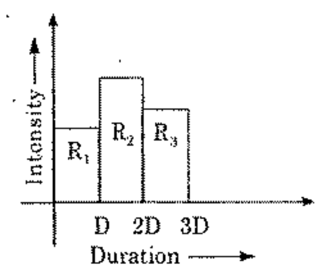
Unit Hydrograph from Complex Storm

Derivation of unit hydrograph of D-hr duration from a complex storm hydrograph is just the reverse of determining DRH from various rainfall of D-hrs occurring successively.

Steps:

1. From complex storm hydrograph base flow is separated.
2. Effective rainfall hyetograph is obtained. (using ϕ-index approach).
3. If R_1, R_2 and R_3 are the rainfall excesses of D-hr duration each

   ~~R_1, R_2, R_3 are depths of rainfall in D-hr each~~ and u_1, u_2, u_3, u_4, u_5 .... are the ordinates of unit hydrograph, then by obtaining DRH for R_1, DRH for R_2 and DRH for R_3 and lagging each DRH by D-hr combined DRH is obtained.



4. Combined DRH so obtained is equated to the known values of DRH and hence, u_1, u_2, u_3, u_4, ... etc. is obtained.

Example 7.6

The stream flows due to three successive storms of 2.9, 4.9 and 3.9 cm of 6 hours duration each on a basin are given below. The area of the basin is 118.8 km². Assuming a constant base flow of 20 cumec, derive a 6-hour unit hydrograph for the basin. An average storm loss of 0.15 cm/hr can be assumed.

Time (hr):	0	3	6	9	12	15	18	21	24	27	30	33
Flow (cumec):	20	50	92	140	199	202	204	144	84.5	45.5	29	20

Solution: Let the 6-hour unit hydrograph ordinates be u_0, u_1, u_2, u_3, u_4, ..., u_7 at 0, 3, 6, 12, ..., 21 hours, respectively. The direct runoff ordinates due to the three successive storms (of 6 hours duration each) are obtained by deducting the base flow of 20 cumec from the stream flows at the corresponding time intervals as shown in Table. The net storm/rains are obtained by deducting the average storm loss as

0-6 hr: $x = 2.9 - 0.15 \times 6 = 2$ cm

6-12 hr: $y = 4.9 - 0.15 \times 6 = 4$ cm

12-18 hr: $z = 3.9 - 0.15 \times 6 = 3$ cm

Time (hr)	UGO*	DRO due to** 1st storm UGO × x	2nd storm UGO × y	3rd storm UGO × z	Equation Total DRO = TRO—BFO	Solution 6-hr- UGO
0	$u_0 = 0$	0	—	—	$0 = 20 - 20$	$u_0 = 0$
3	u_1	$2u_1$	—	—	$2u_1 = 50 - 20$	$u_1 = 15$
6	u_2	$2u_2$	0	—	$2u_2 = 92 - 20$	$u_2 = 36$
9	u_3	$2u_3$	$4u_1$	—	$2u_3 + 4u_1 = 140 - 20$	$u_3 = 30$
12	u_4	$2u_4$	$4u_2$	0	$2u_4 + 4u_2 + 0 = 199 - 20$	$u_4 = 17.5$
15	u_5	$2u_5$	$4u_3$	$3u_1$	$2u_5 + 4u_3 + 3u_1 = 202 - 20$	$u_5 = 158.5$
18	u_6	$2u_6$	$4u_4$	$3u_2$	$2u_6 + 4u_4 + 2u_2 = 204 - 20$	$u_6 = 3$
21	u_7	$2u_7$	$4u_5$	$3u_3$	$2u_7 + 4u_5 + 3u_3 = 144 - 20$ $\therefore u_5 = 8.5$	$u_7 = 0$
24			$4u_6$	$3u_4$	$4u_6 + 3u_4 = 84.5 - 20$ $\therefore u_6 = 3$	$\Sigma u = 15$
27			$4u_7$	$3u_5$	$4u_7 + 3u_5 = 45.5 - 20$ $\therefore u_7 = 0$	check for
30				$3u_6$	$3u_6 = 29 - 20$	UGO derived
33				$3u_7$	$3u_7 = 20 - 20$	above

*Except the first column, all other columns are in cumec.

**x = 2 cm, y = 4 cm, z = 3 cm.

Checks:

$$\frac{\Sigma u\, t}{A} = 1 \text{ cm, in consistent units}$$

Σu = sum of the Unit hydrograph ordinates = 110 cumec

$$\frac{110\,(3 \times 60 \times 60)}{118.8 \times 10^6} = 0.01 \text{ m, or } \mathbf{1\ cm}$$

Hence, the *UGO's* derived are correct and is plotted in Figure as shown.

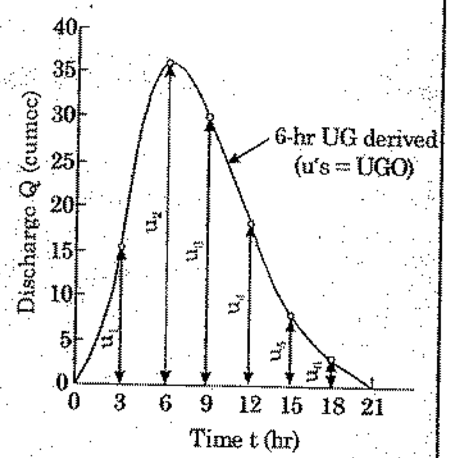


Fig. 6-hr unit hydrograph (derived).

Unit Hydrograph of Different Duration

A unit hydrograph where duration is D-hr can only be applied to storm of duration D-hr to obtain direct runoff. If however the rainfall duration is different, the unit hydrograph of duration equal to the duration of rainfall is to be used. If such unit hydrograph is not available and only D-hr unit hydrograph is available then the D-hr unit hydrograph has to be converted into the unit hydrograph of duration equal to that of the rainfall. In this process two conditions arise:

Case (i) Changing a Short Duration Unit Hydrograph to Longer Duration.

- If the desired long duration of the unit graph is an integral multiple of the short, (say a 3-hour unit graph is given and a 6-hour unit graph is required) assume two consecutive unit storms of 3 hr duration, producing a net rain of 1 cm each.
- Draw the two unit hydrographs, the second unit graph being lagged by 3 hours.

- Draw now the combined hydrograph by superposition. This combined hydrograph will now produce 2 cm in 6 hours.
- To obtain the 6-hour unit graph divide the ordinates of the combined hydrograph by 2.
- It can be observed that this 6-hour unit graph derived has a longer time base by 3 hours than the 3-hour unit graph, because of a lower intensity storm for a longer time.

Example 7.7

From the 4 h unit hydrograph, derive the 8 h unit hydrograph.

Sol: The necessary computations required to obtain the ordinates of 8 h unit hydrograph are given in Table below.

Time (h)	Ordinates of 4 h U.H. in m^3/s		Combined hydrograph (m^3/s)	Ordinates of 8 h U.H. (m^3/s)
	without lag	lagged by 4 h		
(1)	(2)	(3)	(4) = (2) + (3)	(5) = (4)/2
0	0		0	0
2	12.52		12.52	6.26
4	21.32	0	21.32	10.66
$\boxed{6}$	$\boxed{23.54}$	12.52	36.06	18.03
$\boxed{8}$	17.84	21.32	39.16	$\boxed{19.58}$
10	14.79	23.54	38.33	19.17
12	12.18	17.84	30.02	15.01
14	10.04	14.79	24.83	12.42
16	8.26	12.18	20.44	10.22
18	6.51	10.04	16.59	8.30
20	4.98	8.26	13.24	6.62
22	3.95	6.51	10.46	5.23
24	3.05	4.98	8.03	4.02
26	2.26	3.95	6.21	3.11
28	1.60	3.05	4.65	2.33
30	1.07	2.26	3.33	1.67
32	0.53	1.60	2.13	1.07
$\boxed{34}$	0	1.07	1.07	0.54
36		0.53	0.53	0.27
$\boxed{38}$		0	0	0

The features of the two unit hydrographs are compared below.

	4 *h U.H.*	8 *h U.H.*
Peak ordinate	23.54 m^3/s	19.58 m^3/s
Time to peak	6 *h*	8 *h*
Base period	34 *h*	38 *h*

Case (ii): Unit hydrograph of duration mD from unit hydrograph of duration D. Where m is not an integer.

- The simple procedure of converting the unit hydrograph of short duration into unit hydrograph of longer duration, explained in the previous section, cannot be adopted if the duration of the required unit hydrograph is either less than, or not an integral multiple of the duration of the given unit hydrograph.

- In such situations the U.H. of any duration can be obtained from the U.H. of given duration using the S-curve technique.
- A S-curve hydrograph may be defined as the hydrograph of direct runoff resulting from a continuous effective rainfall of uniform intensity $\frac{1}{D}$ cm/h.
- As shown below, the S-curve is constructed by adding together a series of D h unit hydrographs, each lagged by D h with respect to the previous one.
- The S-curve hydrograph attains a constant ordinate, called the *equilibrium discharge* denoted by Q_e, approximately at the end of the base period T_B of the unit hydrograph.
- Thus the number of unit hydrographs needed to produce the S-curve is $\frac{T_B}{D}$.

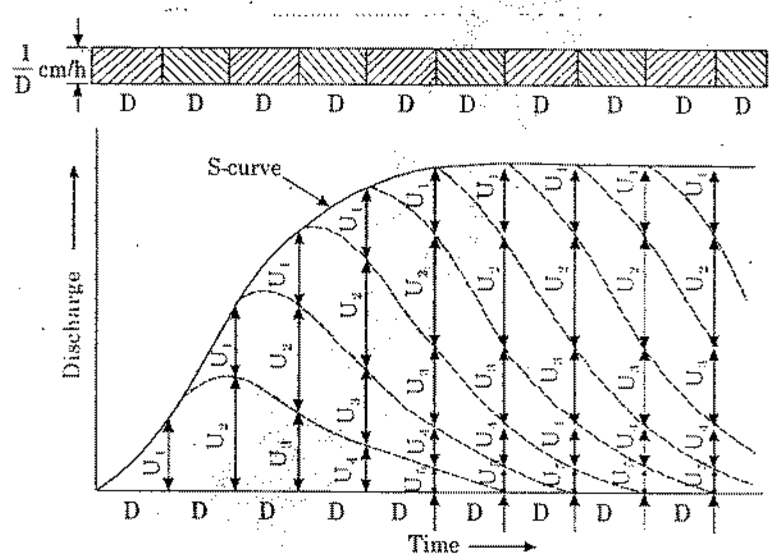


Fig. S-curve hydrograph.

- Since the rainfall rate is equal to the runoff rate at the equilibrium state, it follows that

$$Q_e = A \cdot \frac{1}{D}\ \text{km}^2\text{-cm/h} = 2.778\,\frac{A}{D}\ \text{m}^3/\text{s}$$

where A is the area of the basin in km^2 and D is the duration of unit hydrograph in hours which is used in the construction of the S-curve.

- Consider two D-h S-curves A and B displaced by T h. If the ordinates of B are subtracted from that of A, the resulting curve is a DRH produced by a rainfall excess of duration T h and magnitude $\left(\frac{1}{D}\times T\right)$ cm. Hence if the ordinate difference of A and B, i.e. $(S_A - S_B)$ are divided by T/D, the resulting ordinates denote a hydrograph due to an ER of 1 cm and of duration T. i.e. a T-h unit hydrograph.

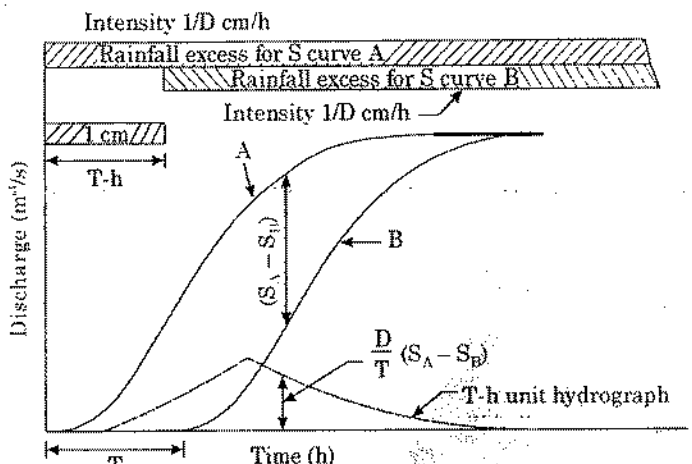


Fig. Derivation of T-h unit hydrograph by *S*-curve lagging method.

Example 7.8

The ordinates of a 4-hour unit hydrograph for a particular basin are given below. Derive the ordinates of (i) the *S*-curve hydrograph, and (ii) the 2-hour unit hydrograph, and plot them, area of the basin is 630 km^2.

Time (h)	Discharge (cumec)	Time (hr)	Discharge (cumec)
0	0	14	70
2	25	16	30
4	100	18	20
6	160	20	6
8	190	22	1.5
10	170	24	0
12	110		

Sol:

Time (hr)	4-hr UGO (cumec)	S-curve additions (cumec) (unit storms after every 4 hr = t_r)					S-curve ordinates (cumec) (2) + (3)	lagged *S*-curve (cumec)	*S*-curve difference (cumec) (4) − (5)	2-hr UGO (6) × 4/2 (cumec)
1	2	3					4	5	6	7
0	0	—	—	—	—	—	0	—	0	0
2	25	—	—	—	—	—	25	0	25	50
4	100	0	—	—	—	—	100	25	75	150
6	160	25	—	—	—	—	185	100	85	170
8	190	100	0	—	—	—	290	185	105	210
10	170	160	25	—	—	—	335	290	65	130
12	110	190	100	0	—	—	400	355	45	90
14	70	170	160	25	—	—	425	400	25	50
16	30	110	190	100	0	—	430	425	5	10*
18	20	70	170	160	25	—	445	430	15	30
20	6	30	110	190	100	0	436	445	−9	−18*
22	1.5	20	70	170	160	25	446.5	436	10.5	21
24	0	6	30	110	190	100	436	446.6	−10.5	−21*

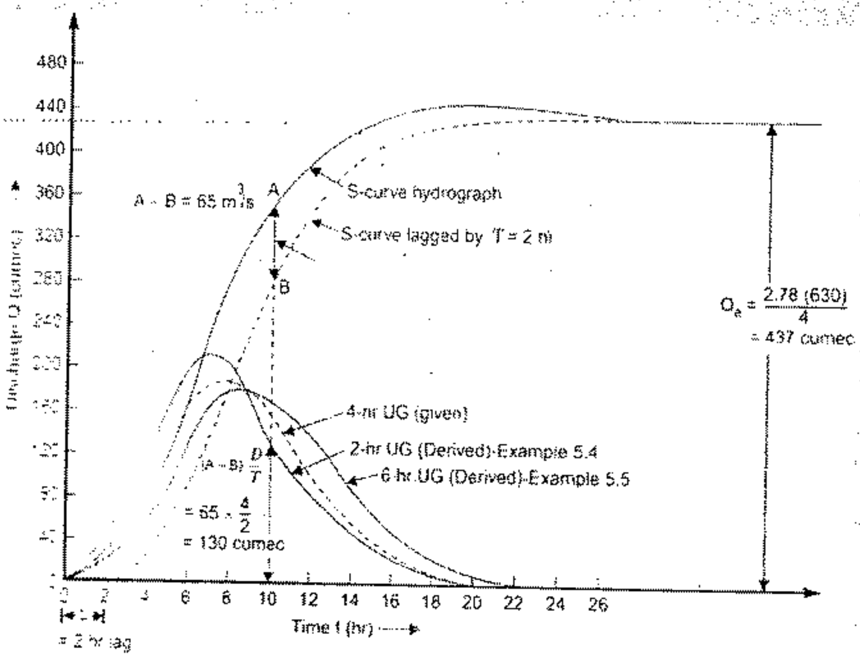
***Slight adjustment is required to the tail of the 2-hour unit hydrograph.**

Col. (5): lagged S-curve is the same as col. (4) but lagged by $T_r = 2$ hr

Col. (7): col. (6) $\times \dfrac{D}{T}$, $D = 4$ hr, $T = 2$ hr

Col. (3): No. of unit storms in succession = $T/D = 24/4 = 6$, to produce a constant outflow.

$Q_e = \dfrac{2.78A}{T} = \dfrac{2.78 \times 630}{4} = 437$ cumec, which agrees very well with the tabulated S-curve terminal value of 436.



Example 7.9

The ordinates of a 4-hour unit hydrograph for a particular basin are given below. Determine the ordinates of the S-curve hydrograph and therefrom the ordinates of the 6-hour unit hydrograph.

Time (hr)	4-hr UGO (cumec)	Time (hr)	4-hr UGO (cumec)
0	0	12	110
2	25	14	70
4	100	16	30
6	160	18	20
8	190	20	6
10	170	22	1.5
		24	0

Sol:

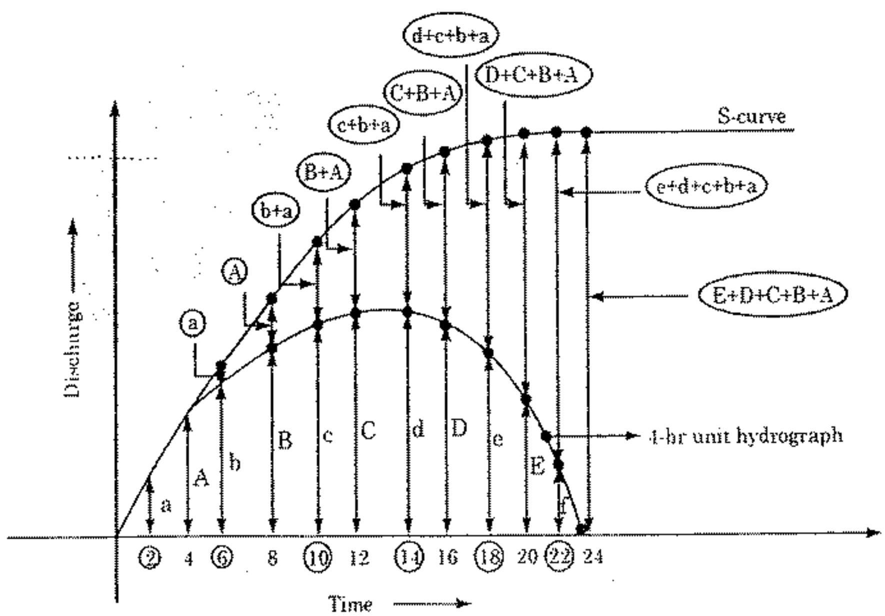
Time (hr)	4-hour UGO (cumec)	S-curve ordinates (cumec)	Lagged[2] C-curve (cumec)	S-curve difference (cumec) (4) – (5)	6-hr UGO (cumec) (6) × 4/6
1	2	3	4	5	6
0	0 →	0	—	0	0
2	25 →	25	—	25	16.7
4	100 + →	100	—	100	66.7
6	160 + →	185	0	185	123.3
8	190	290	25	265	176.7
10	170	355	100	255	170.0
12	110	400	185	215	143.3
14	70	425	290	135	90.0
16	30	430	355	75	50.0
18	20	445	400	45	30.0
20	6	436	425	11	7.3
22	1.5	446.6	430	16.5	11.0
24	0	436	445	–9	–6.0

1—Start the operation shown with 0 cumec after $t_r = 4$ hr.
2—Lag the *S*-curve ordinates by $t'_r = 6$ hr.
Plot col. (3) vs. col. (1) to get *S*-curve hydrograph and col. (6) vs. col. (1) to get 6-hr unit hydrograph.
Note that S-curve ordinate has been written in single column as against the procedure discussed in previous problems.

Use and Limitations of Unit Hydrograph

Uses:

(i) The development of flood hydrographs for extreme rainfall magnitudes for use in the design of hydraulic structure,



(ii) Extension of flood-flow records based on rainfall records and

(iii) Development of flood forecasting and warning systems based on rainfall.

Cumulative ordinates of unit hydrograph which are at a difference of 4-hr are taken and written against the corresponding time. Thus cumulative ordinates of 0, 4, 8, 12, 16, 20, 24hr are taken and written against these times. Again, cumulative ordinates of 2, 6, 10, 14, 18 and 22 are taken and written against these times. The following figure will illustrates how to find out the ordinates of S-curve.

Limitations

1. Unit hydrograph cannot be used for catchment area greater than 5000 km^2 because similar rainfall distribution from storm to storm over a large area is rare.
2. Unit hydrograph can not be used for catchment < 200 km^2.
3. Precipitation must be from rainfall only. Snow-melt runoff cannot be satisfactorily represented by unit hydrograph.
4. The catchment should not have unusually large storage in terms of tanks, ponds, large flood-bank storage, etc. which affect the linear relationship between storage and discharge.
5. If the precipitation is decidedly nonuniform, unit hydrographs cannot be expected to give good results.

Synthetic Unit Hydrograph

Unit hydrographs can be derived if rainfall and runoff records are available for the basin under consideration. But there are many basins, which are not gauged and for which unit-graphs may be required. Hence some method of deriving unit hydrographs for ungauged basins is necessary.

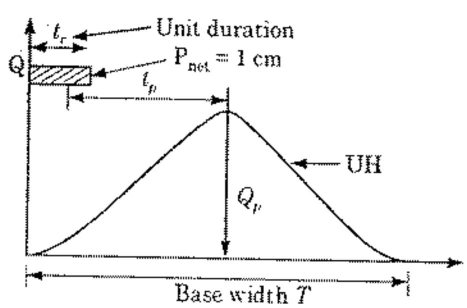
This is usually done by relating the selected basin characteristics to the unit hydrograph shape. Once such relations are established between the basin parameters and unit hydrograph parameters for the basins having sufficient data, the same relations are applied to get the unit hydrograph of ungauged basins in the same hydrometeorologically homogeneous area from the known basin parameters. The unit hydrograph thus obtained is known as *Synthetic unit hydrograph.*

Synder selected three parameters for development of SUH. They are:

(i) Basin time width T

(ii) Peak discharge Q_p

(iii) Lag time i.e. basin lag time t_p.



He proposed the following three equations for these three parameters

$$\boxed{\text{Lag time, } t_p = C_t (LL_{ca})^{0.3}} \quad \text{(i)}$$

$$\boxed{\text{Basin time width } T = (72 + 3t_p)} \qquad \text{(ii)}$$

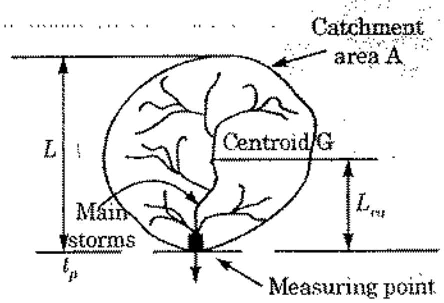
$$\boxed{\text{Peak discharge } Q_p = \frac{2.78\, C_p A}{t_p}} \qquad \text{(iii)}$$

where t_p is in hr.

C is a coefficient reflecting slope, land use, and associated storage characteristics of basin. Its value varies between 1.35 to 1.65, average being 1.5

L = basin length measured along the water course from the the basin divide to the gauging station in km.

L_{ca} = Distance of centroid of catchment from the gauging point (in km)



T is in hr.

Q_p is in m^3/s

A = Catchment area in km^2.

C_p = a regional constant having value between 0.56 to 0.69.

Synder used the standard duration t_r (or D hr) in hr for unit hydrograph.

where
$$\boxed{t_r = D_{hr} = \frac{t_p}{5.5}} \qquad \text{(iv)}$$

If a synthetic unit hydrograph of other duration then D'_{h} is required, then lag time, $t_{pr'}$ is given by

$$\boxed{t_{pr} = t_p + \frac{D' - t_r}{4}} \qquad \text{(v)}$$

To plot the smooth synthetic unit hydrograph, US army corps of engineering gave the width of SUH as

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

$$W_{75} = \frac{3.35}{\left(\frac{Q_p}{A}\right)^{1.08}}$$

Where W_{50} and W_{75} are the width of synthetic unit hydrograph in hr at 50% and 75% of Q_p respectively. where Q_p is in m^3/s and A is area of catchment in km^2.

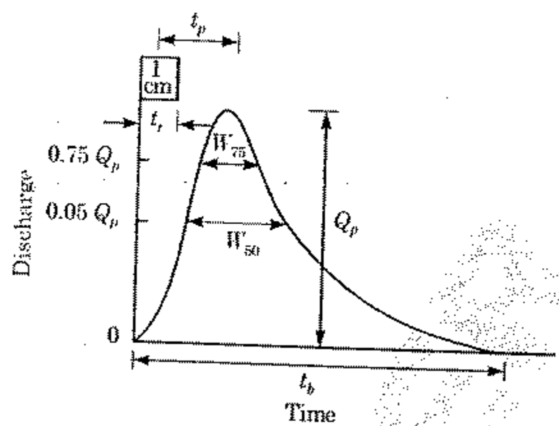


Fig. Elements of a synthetic unit hydrograph.

Example 7.10

Derive 3-hr synthetic unit hydrograph of basin with following data with a catchment area of 2500 km²:

Length of main stream = 120 km

Distance from central outlet = 80 km

C_t and C_p of the catchment are assumed to be 1.5 and 0.6 respectively. Use Synder's method.

Sol:

$$t_p = C_t(LL_{ca})^{0.3}$$

Here $C_t = 1.5,\ L = 120 \text{ km},\ L_{ca} = 80 \text{ km}$

$\therefore$ $t_p = 23.484 \text{ hrs.}$

$$\text{Standard duration } t_r = D = \left(\frac{t_p}{5.5}\right) = \left(\frac{23.484}{5.5}\right) = 4.27 \text{ hrs}$$

$$\text{Required duration } D' = 3 \text{ hrs}$$

$$\text{Lag time } t_{pr} = t_p + \frac{D' - t_r}{4} = t_p + 0.25\,(D' - t_r)$$

or
$$t_{pr} = 23.484 + 0.25\,(3 - 4.27) = 23.16 \text{ hrs}$$

Now
$$Q_p = 2.78\left(\frac{0.6 \times 2500}{23.16}\right) = 180 \text{ m}^3/\text{sec}$$

and
$$T = 72 + 3 \times 23.16 = 141.48 \text{ hours}$$

With these values Q_p, T and t_{pr} a smooth curve may be drawn.

To plot the smooth synthetic unit hydrograph (SUH), US Army Corps Engineers gave the width of SUH as:

$$W_{50} = \frac{5.87}{\left(\frac{Q_P}{A}\right)^{1.08}}$$

$$W_{75} = \frac{3.35}{\left(\frac{Q_P}{A}\right)^{1.08}}$$

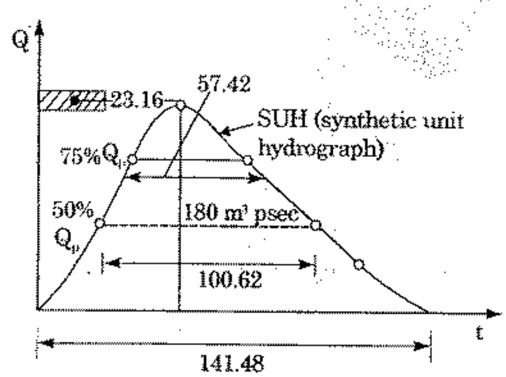
where W_{50} and W_{75} are widths in hrs of SUH at 50% and 75% of Q_p.

$$\therefore \quad W_{50} = \frac{5.87}{\left(\frac{180}{2500}\right)^{1.08}} = 100.62 \text{ hrs}$$

and

$$W_{75} = \frac{3.35}{\left(\frac{180}{2500}\right)^{1.08}} = 57.42 \text{ hrs}$$

Hence the synthetic unit hydrograph (SUH) may be drawn as shown in Figure below.



Example 7.11

Two catchments x and y are considered meteorologically homogeneous. The characteristics of the two are:

x	y
$L_x = 50$ km	$L_y = 80$ km
$L_{ca\,x} = 30$ km	$L_{ca\,y} = 60$ km
$A_x = 500$ km²	$A_y = 1000$ km²

For catchment x, UH developed is of 2 hr duration and peak discharge $Q_{px} = 80$ m³ sec. The time of peak from beginning of rainfall in x is 13 hrs. Using synder's method, develop SUH for catchment y.

Sol: For catchment x, t_{px} is obtained as:

$$13 = t_{px} + \frac{t_r}{2} = t_{px} + \frac{2}{2}$$

$$\therefore \quad t_{px} = 12 \text{ hrs}$$

Now,

$$t_{px} = C_t(L_x L_{ca})^{0.3}$$

$$12 = C_t(50 \times 30)^{0.3}$$

$$\therefore \quad C_t = 1.3376$$

Again

$$Q_{px} = 2.78\left(\frac{C_p A_x}{t_{px}}\right)$$

$$\Rightarrow \quad 80 = \frac{2.78\,(C_p \times 500)}{12}$$

$$\therefore \quad C_p = 0.69$$

Since x and y are homogeneous, the values of C_t and C_p for x may be used to develop SUH for catchment y.

$$\Rightarrow \quad t_{py} = C_t(L_y L_{cay})^{0.3} = 1.3376\,(80 \times 60)^{0.3}$$

$$\text{or} \quad t_{py} = 17.00 \text{ hrs}$$

$$\therefore \quad \text{Standard duration } t_{ry} = D_y = \frac{t_{py}}{5.5} = \frac{17.00}{5.5} = 3.09 \text{ hrs}$$

$$\cong 3 \text{ hrs}$$

Thus we have to SUH of 3 hr for catchment y

Now,

$$\therefore \quad Q_{py} = 2.78\left(\frac{C_p \times A_y}{t_{py}}\right) = \left(\frac{0.69 \times 1000}{17}\right) = 112.83 \text{ m}^3\text{/s}$$

$$T_y = 72 + 3 \times 17 = 123 \text{ hr}$$

$$\Rightarrow \quad Q_p = 112.83 \text{ m}^3\text{/s}, \; T = 123 \text{ hrs}, \; t_p = 17 \text{ hrs}$$

Now,
$$W_{50} = \frac{5.87}{\left(\frac{Q_P}{A}\right)^{1.08}}$$

$$= \frac{5.87}{\left(\frac{Q_P}{A}\right)^{1.08}} = 61.95 \text{ hrs}$$

and
$$W_{75} = \frac{3.35}{\left(\frac{112.83}{1000}\right)^{1.08}} = 35.35 \text{ hr}$$

Triangular Unit Hydrograph

The area triangular unit hydrograph gives volume Q_v, i.e.,

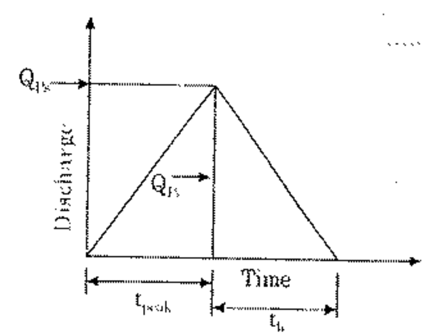


Fig. A triangular UH.

$$Q_v = \frac{1}{2}\,(t_{peak} + t_b)\,Q_{ps}$$

$$\text{or} \quad Q_{ps} = \frac{2Q_v}{t_{peak} + t_b}$$

According to US soil conservation service,

$$t_{peak} = 0.7\, t_c$$

or
$$t_{peak} = \frac{\text{Rainfall effective time}}{2} + 0.6\, t_c$$

Here t_c is the time of concentration

$$\text{Time lag} = 0.6\, t_c$$

$$Q_{ps} = \frac{2Q_v}{2.67\, t_{peak}} = \frac{0.75\, Q_v}{t_{peak}}$$

Instantaneous Unit Hydrograph (IUH)

A unit hydrograph of infinitesimal duration is called instantaneous unit hydrograph.

- Main advantage of IUH is that it is independent of the duration of effective rainfall hyetograph and has one parameter less than D-hr unit hydrograph.
- IUH is independent of rainfall characteristics. It is indicative of catchment storage characteristics.
- Instantaneous unit hydrograph is obtained from S-curve.
- Ordinate of instantaneous hydrograph is given by $\boxed{u(t) = \frac{1}{i}\frac{dS}{dt}}$, i = intensity of rainfall, S = ordinate of S-curve.

Thus ordinate of instantaneous unit hydrograph is the slope of S-curve of intensity 1 cm/hr (i.e. S-curve derived from a unit hydrograph of 1 hr duration).

Clark Model

- Clark showed that IUH may be obtained by routing the rainfall excess.
- He used the concept of time area diagram.

□□□□

OBJECTIVE QUESTIONS

1. Which one of the following constitute the basic assumption of Unit Hydrograph theory?
   (a) Non-linear response and time invariance
   (b) Non-linear time variance and linear response
   (c) Linear response and linear time variance
   (d) Time invariance and linear response

2. The following four hydrological features have to be estimated or taken as inputs before one can compute the flood hydrograph at any catchment outlet:
   1. Unit hydrograph
   2. Rainfall hydrograph
   3. Infiltration index
   4. Base flow

   The correct order in which they have to be employed in the computations is
   (a) 1, 2, 3, 4 (b) 2, 1, 4, 3
   (c) 2, 3, 1, 4 (d) 4, 1, 3, 2

3. The following steps are involved in arriving at a unit hydrograph:
   1. Estimating the surface runoff in depth.
   2. Estimating the surface runoff in volume.
   3. Separation of base flow.
   4. Dividing surface runoff ordinates by depth of runoff.

   The correct sequence of these steps is
   (a) 3, 2, 1, 4 (b) 2, 3, 4, 1
   (c) 3, 1, 2, 4 (d) 4, 3, 2, 1

4. If a 4-hour unit hydrograph of a certain basin has a peak ordinate of 80 m^3/s, the peak ordinate of a 2-hour unit hydrograph for the same basin will be
   (a) equal to 80 m^3/s
   (b) greater than 80 m^3/s
   (c) less than 80 m^3/s
   (d) between 40 m^3/s to 80 m^3/s

5. Match **List-I** (Name of scientists) with **List-Ii** (Contribution to field of hydrology) and select the correct answer using the codes given below the lists:

   List-I
   A. Dalton
   B. Snyder
   C. Blaney-Criddle
   D. Sherman

   List-II
   1. Unit Hydrograph
   2. Evaporation
   3. Empirical flood formula
   4. Synthetic Unit Hydrograph
   5. Consumptive use equation

   Codes:

	A	B	C	D
(a)	2	3	5	1
(b)	1	4	3	2
(c)	2	4	5	1
(d)	1	3	4	5

6. If the base period of a 6-hour hydrograph of a basin is 84 hours, then a 12 hours unit hydrograph derived from this 6 hour unit hydrograph will have a base period of
   (a) 72 hours (b) 78 hours
   (c) 84 hours (d) 90 hours

7. Which of the following principles related to a Unit Hydrograph?
   1. The hydrographs of direct runoff due to effective rainfall of equal duration have the same time base.
   2. Effective rainfall is not uniformly distributed within its duration.
   3. Effective rainfall is uniformly distributed throughout the whole area of drainage basin.

4. Hydrograph of direct run off from a basin due to a given period of effective rainfall reflects the combination of all the physical characteristics of the basin.

Select the correct answer using the codes given below:

(a) 1, 2 and 3 (b) 1, 2 and 4

(c) 2, 3 and 4 (d) 1, 3 and 4

8. The Unit Hydrograph theory is based on the assumption of

(a) nonlinear response and time invariance

(b) linear response and nonlinear time variance

(c) time variance and linear response

(d) nonlinear response and nonlinear time variance

9. A two hour storm hydrograph has 5 units of direct runoff. The two-hour unit hydrograph for this storm can be obtained by dividing the ordinates of the storm hydrograph by

(a) 2 (b) 2/5

(c) 5 (d) 5/2

10. **Assertion (A)**: Unit hydrograph theory is not applicable to catchment areas larger than 5000 sq. km.

Reason (R): Rainfall is not uniformly distributed on large catchment areas.

11. In Synder's method of synthetic unit hydrograph development, basin lag is taken as

(a) the time interval between centroid of the rainfall excess and surface runoff

(b) the time interval from mid point of the unit rainfall excess to the peak of the unit hydrograph

(c) independent of rainfall duration

(d) independent of catchment characteristics

12. Match **List-I** with **List-II** and select the correct answer using the codes given below the lists:

List-I

A. Rising limb of a hydrograph

B. Falling limb of a hydrograph

C. Peak rate of flow

D. Drainage density

List-II

1. Depends on intensity of rainfall

2. Function of total channel length

3. Function of catchment slope

4. Function of storage characteristics

Codes:

	A	B	C	D
(a)	3	4	1	2
(b)	1	4	3	2
(c)	3	2	1	4
(d)	1	2	3	4

13. Match **List-I** with **List-II** and select the correct answer using the codes given below the lists:

List-I

A. Unit Hydrograph
B. Synthetic unit Hydrograph
C. Darcy's law
D. Rational method

List-II

1. Design flood
2. Permeability
3. Ungauged basin
4. 1 cm runoff

Codes:

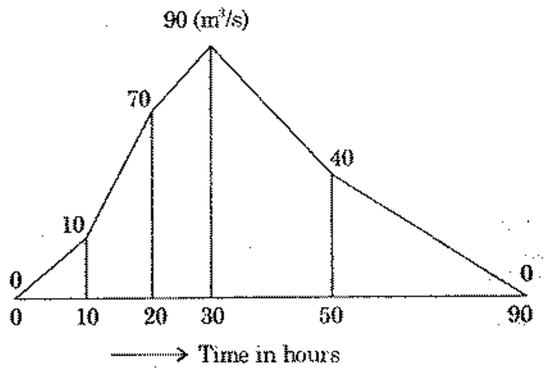
	A	B	C	D
(a)	2	3	4	1
(b)	2	1	4	3
(c)	4	3	2	1
(d)	4	1	2	3

14. Clark's method aims at which one of the following?
(a) Developing on IUH due to a instantaneous rainfall excess over catchment
(b) Developing stage-discharge relationship
(c) Measurement of infiltration
(d) Flood routing through channels

15. Viewing watershed as a system, which one of the following assumptions is made in the Unit Hydrograph theory?
(a) Non-linearity
(b) Both linearity and time variance
(c) Both time invariance and nonlinearity
(d) Both linarity and time invariance

16. For a given storm, other factors remaining same,
(a) basins with large drainage densities given smaller flood peaks
(b) low drinage density basins give shorter time bases of hydrographs
(c) the flood peak is independent of the drinage density
(d) basins having low drinage density give smaller peaks in flood hydrographs

17. A DRH due to a storm over a basin has a time base of 90 hours with straight line portions of the hydrograph with flow rates of 0, 10, 70, 90, 40 and 0 m^3/s at elapsed durations of 0, 10, 20, 30, 50 and 90 hours as indicated on the bove diagram, respectively. The catchment area is 300 km^2. What is the rainfall excess in the storm?



(a) 2.83 cm (b) 3.46 cm
(c) 3.87 cm (d) 4.02 cm

18. A 252 km^2 catchment area has a 6 hr UH which is a triangle with time base of 35 hours. What is the peak discharge of the DRH due to 5 cm effective rainfall in 6 hr from that catchment?
(a) 45 cumecs (b) 115 cumecs
(c) 200 cumecs (d) 256 cumecs

19. A 4 hour unit hydrograph of a basin can be approximated as a triangle with a base period of 48 hours and peak ordinate of 300 m^3/s. What is the area of the catchment basin?
(a) 7776 km^2 (b) 5184 km^2
(c) 2592 km^2 (d) 1294 km^2

20. Consider the following statements:
Assertion (A): The unit hydrograph cannot be applied for areas less then 5000 sq. km.
Reason (R): The run-off hydrograph relfects the physiographic factors of a catchment.
Of these statements:
(a) both A and R are true and R is the correct explanation of A
(b) both A and R are true but R is not a correct explanation of A
(c) A is true but R is false
(d) A is false but R is true

21. Match **List-I** (Parameter) with **List-II** (Relatable Term) and select the correct answer using the codes given below the lists:
List-I
A. Rainfall Intensity
B. Rainfall Excess
C. Rainfall Averaging
D. Mass Curve
List-II
1. Isohyets
2. Cumulatie rainfall
3. Hyetograph
4. Direct runoff hydrograph

Codes:

	A	B	C	D
(a)	1	3	2	4
(b)	3	4	1	2
(c)	1	3	4	2
(d)	2	4	2	1

22. In constructing a 4 hour synthetic unit hygrograph for a basin, the lag time is estimated to be 40 hours. When will the peak discharge in the Synthetic Unit Hydrograph occur from the start of the storm?

(a) 36 hours (b) 40 hours
(c) 42 hours (d) 44 hours

23. Consider the following statements:

1. Only the surface flow consitutes the flood hydrograph due to an isolated storm.
2. For a given storm, the flood peak is dependent on the drainage density.
3. Fan shaped catchments give narrow hydrograph with low peak.

Which of these statements is/are correct?

(a) 1, 2 and 3 (b) 1 and 3
(c) 2 only (d) 3 only

24. A unit hydrograph for a watershed is triangular in shape with base period of 20 hours. The area of the watershed is 500 ha. What is the peak discharge in m^3/hrour?

(a) 7000 (b) 6000
(c) 5000 (d) 4000

25. A triangular direct runoff hydrograph due to a storm has a time base of 60 hr and a peak flow of 30 m^3/s occurring at 20 hr from the start. If the catchment area is 300 km^2, what is the rainfall excess in the storm?

(a) 50 mm (b) 20 mm
(c) 10.8 mm (d) 8.3 mm

26. A 3 hr unit hydrograph U_1 of a catchment of area 235 km^2 is in the form of a triangle with peak discharge 30 m^3/s. Another 3 hr unit hydrograph U_2 is also triangular is shape and has the same base width as U_1, but has a peak flow of 90 m^3/s. What is the catchment area of U_2?

(a) 117.5 km^2 (b) 235 km^2
(c) 470 km^2 (d) 705 km^2

27. A direct-runoff hydrograph due to isolated storm was triangular in shape with a base of 80 h and peak of 200 m^3/s. If the catchment area is 1440 km^2, the effective rainfall of the storm is

(a) 20 cm (b) 10 cm
(c) 5 cm (d) 2 cm

28. The shape of the recession limb of a hydrograph depends on

(a) basin as well as storm characteristics
(b) storm characteristics only
(c) basin characteristics only
(d) base flow only

29. An S-curve hydrograph has been obtained for catchments of 270 km^2 from a 3 hour unit hydrograph. The equilibrium discharge for the S-curve is
    (a) 750 m^3/s (b) 277.8 m^3/s
    (c) 250 m^3/s (d) 187 m^3/s

30. Match **List-I** with **List-II** and select the correct answers using the codes given below the lists:

List-I	**List-II**
A. Rainfall intesnity	**1.** Isohyets
B. Rainfall excess	**2.** Cumulative rainfall
C. Rainfall averaging	**3.** Hyetograph
D. Mass curve hydrograph	**4.** Direct runoff

Codes:

	A	B	C	D
(a)	1	3	2	4
(b)	3	4	1	2
(c)	1	2	4	3
(d)	3	4	2	1

Common Data for Questions 31 and 32:

An average rainfall of 16 cm occurs over a catchment during a period of 12 hours with a uniform intensity. The unit hydrograph (unit depth = 1 cm, duration = 6 hours) of the catchment rises linearly from 0 to 30 cumecs in six hours and then falls linearly from 30 to 0 cumecs in the next 12 hours. ϕ index of the catchment is known to be 0.5 cm/hr. Base flow in the river is known to be 5 cumecs.

31. Peak discharge of the resulting direct runoff hydrograph shall be
    (a) 150 cumecs (b) 225 cumecs
    (c) 230 cumecs (d) 360 cumecs

32. Area of the catchment in hectares is
    (a) 97.20 (b) 270
    (c) 9720 (d) 27000

33. The average rainfall for a 3 hour duration storm is 2.7 cm and the loss rate is 0.3 cm/hr. The flood hydrograph has a base flow of 20 m^3/s and produces a peak flow of 210 m^3/s. The peak of a 3-h unit hydrograph is
    (a) 125.50 m^3/s (b) 105.50 m^3/s
    (c) 77.77 m^3/s (d) 70.37 m^3/s

34. When the outflow from a storage reservoir is uncontrolled as in a freely operating spillway, the peak of outflow hydrograph occurs at
    (a) the point of intersection of the inflow and outflow hydrographs
    (b) a point after the intersection of the inflow and outflow hydrographs
    (c) the tail of inflow hydrographs
    (d) a point, before the intersection of the inflow and outflow hydrographs

Statement for Linked Answer Questions 35 and 36:

A four hour unit hydrograph of a catchment is triangular in shape with base of 80 hours. The area of the catchment is 720 km^2. The base flow and ϕ-index are 30 m^3/s and 1 mm/hr, respectively. A storm of 4 cm occurs uniformly in 4 hours over the catchment.

35. The peak discharge of four hour unit hydrograph is

(a) 40 m^3/s (b) 50 m^3/s

(c) 60 m^3/s (d) 70 m^3/s

36. The peak flood discharge due to the storm is

(a) 210 m^3/s (b) 230 m^3/s

(c) 260 m^3/s (d) 720 m^3/s

Common Data for Questions 37 and 38:

Ordinates of a 1-hour unit hydrograph at 1 hour intervals, starting from time t = 0 and 0, 2, 6, 4, 2, 1 and m^3/s.

37. Catchment area respresented by this unit hydrograph is

(a) 1.0 km^2 (b) 2.0 km^2

(c) 3.2 km^2 (d) 5.4 km^2

38. Ordinate of a 3-hour unit hydrograph for the catchment at t = 3 hours is

(a) 2.0 m^3/s (b) 3.0 m^3/s

(c) 4.0 m^3/s (d) 5.0 m^{3}s

39. A flood wave with a known inflow hydrograph is routed through a large reservoir. The outflow hydrograph will have

(a) attenuated peak with reduced time-base

(b) attenuated peak with increased time-base

(c) increased peak with increased time-base

(d) increased peak with reduced time-base

Common Data for Questions 40 and 41:

One hour triangular unit hydrograph of a watershed has the peak discharge of 60 m^3/sec.cm at 10 hours and time base of 30 hours. The ϕ index is 0.4 cm per hour and base flow is 15 m^3/sec.

40. The catchment area of the watershed is

(a) 3.24 km^2 (b) 32.4 km^2

(c) 324 km^2 (d) 3240 km^2

41. If there is rainfall of 5.4 cm in 1 hour, the ordinate of the flood hydrograph at 15th hour is

(a) 225 m^3/sec (b) 240 m^3/sec

(c) 249 m^3/sec (d) 258 m^3/sec

ANSWERS

1. (d)	12. (a)	23. (c)	34. (b)
2. (c)	13. (c)	24. (c)	35. (b)
3. (a)	14. (a)	25. (c)	36. (a)
4. (b)	15. (d)	26. (d)	37. (d)
5. (c)	16. (d)	27. (d)	38. (c)
6. (d)	17. (d)	28. (c)	39. (b)
7. (d)	18. (c)	29. (c)	40. (c)
8. (c)	19. (c)	30. (b)	41. (b)
9. (c)	20. (d)	31. (d)	
10. (a)	21. (b)	32. (c)	
11. (b)	22. (c)	33. (b)	

CHAPTER

8

Floods

- Any flow which is relatively high and which overtops the natural or artifical banks in any reach of a river may be called a flood.
- Design of culverts, road and rail bridges, drainage works and irrigation diversion works, needs a reliable estimate of the flood at the site concerned.
- The maximum flood that any structure can safely pass is called the 'design flood'.
- As the magnitude of the design flood increase, the capital cost of the structure also increases but the probability of annual damages will decrease.
- Magnitude of design flood is decided based on acceptable risk of exceedence.
- In the design flood estimates, reference is usually made to the following:

  (a) ***Standard project flood (SPF):*** This is the estimate of the flood likely to occur from the most severe combination of the meterological and hydrological conditions, which are reasonably characteristic of the drainage basin being considered, but excluding extremely rare combination.

  (b) ***Maximum probable flood (MPF):*** This differs from the SPF in that it includes the extremely rare and catastrophic floods and is usually confined to spillway design of very high dams. The SPF is usually around 80% of the MPF for the basin.

  (c) ***Probable maximum precipitation (PMP):*** The probable maximum precipitation (PMP) is defined as the greatest or extreme rainfall of a given duration that is physically possible over a station or basin.

  Depth area duration curve for PMP is usually derived by taking the results of maximum depth area duration analysis and adjusting them for maximum moisture change rate of moisture inflow and other hydrometerological conditions that would maximise the rainfall.

  PMP, when applied on the design unit hydrograph for the basin, will produce the MPF.

  (d) ***Design flood:*** It is the flood adopted for the design of hydraulic structures like spillways, bridge openings, flood banks, etc. It may be the MPF or SPF or a flood of any desired recurrence interval depending upon the degree of flood protection to be offered and cost economics of construction of structures. The design flood is usually selected after making a cost-benefit analysis, the ratio of benefit to cost may be desired to be the maximum.

- The methods used in the estimation of the design flood can be grouped as under.

  (i) Physical indication of past flood

(ii) Envelope curves

(iii) Empirical flood formulae

(iv) Rational method

(v) Unit hydrograph application

(vi) Frequency analysis (or Statistical methods).

(i) Physical Indications of Past Floods

By noting the flood marks (and by local enquiry), depths, affluxes (heading up of water near bridge openings, or similar obstructions to flow), the maximum flood discharge may be estimated.

(ii) Envelope Curve

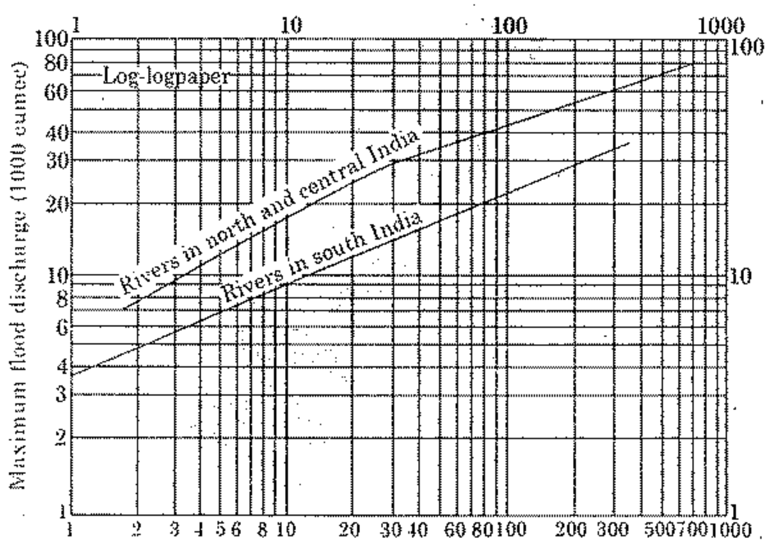


Fig. 8.1 (a) Enveloping curves of Kanwar Sain and Karpov.

Areas having similar topographical features and climatic conditions are grouped together. All available data regarding discharges are compiled along with their respective catchment areas. Peak flood discharges are then plotted against the drainage areas and a curve is drawn to cover or envelope the highest plotted points. Thus for any drainage area, peak flood can be read from the envelope curves. This method gives a rough estimate of peak flood. Using this we cannot assign any return period to the peak value.

Envelope curves are generally used for comparison only and the design floods got by other methods, should be higher than those obtained from envelope curves.

Following equation is sometimes used for enveloping curve of maximum floods throughout the world

$$Q = \frac{3010\,A}{(277 + A)^{0.78}}$$

where Q is in m^3/s. A is in km^2.

(iii) Emperical Formula

The simplest of the empirical relationships are those which relate the flood peak to the drainage area.

The various emperical formulas are:

Dickens Formula

$$Q_p = C_D A^{3/4}$$

where Q_p = maximum flood discharge (m^3/s)

A = catchment area (km^2)

C_D = Dickens constant with value between 6 to 30.

Dickens formula is used in the central and northern parts of the country (India.)

Ryves Formula

$$Q_p = C_R A^{2/3}$$

where Q_p = maximum flood discharge (m^3/s)

A = catchment area (km^2)

C_R = Ryves coefficient

It is in use in Tamil Nadu and parts of Karnataka and Andhra Pradesh.

C_R = 6.8 for areas within 80 km from the east coast

= 8.5 for areas which are 80–160 km from the east coast

= 10.2 for limited areas near hills

Inglis Formula

This formula is based on flood data of catchments in Western Ghats in Maharashtra. Q_p in m^3/s is expressed as

$$Q_p = \frac{124\,A}{\sqrt{A+10.4}}$$

where A is the catchment area in km^2, Q_p is peak flow in m^3/s.

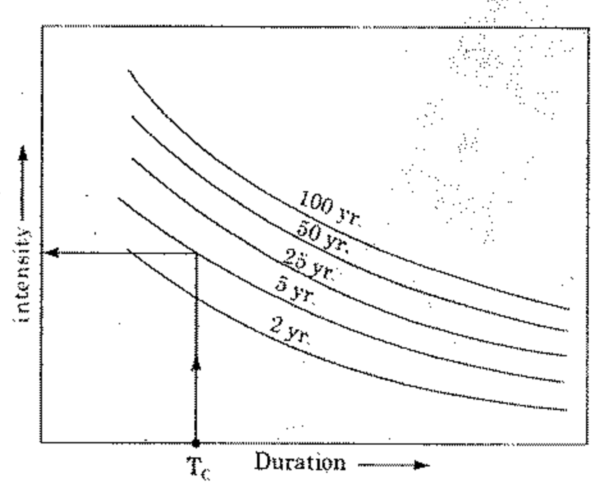
Rational Method

- In this method it is assumed that the maximum flood flow is produced by a certain rainfall intensity which lasts for a time equal to or greater than the period of *concentration time*.
- The concentration time is the time required for the surface runoff from the remotest part of the catchment area to reach the basin outlet.
- When a storm continues beyond concentration time every part of the catchment would be contributing to the runoff at outlet and therefore it represents condition of peak runoff.
- The runoff rate corresponding to this condition is given by

$$Q = CAI$$

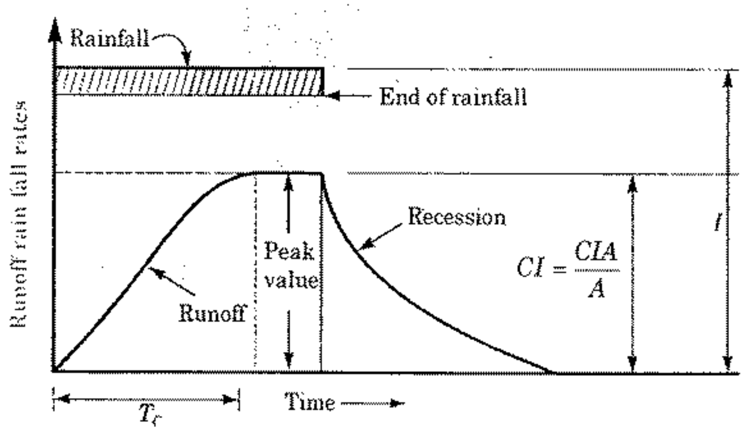
where A is the area of the catchment, I is the intensity of rainfall and C is a runoff coefficient to account for the abstractions from the rainfall.

The intensity of rainfall used should be corresponding to a duration equal to concentration time and desired return period as obtained from *IDF* curve. [i.e. $I = i_{tc,p}$ i.e. rainfall intensity corresponding to a duration of time of concentration (t_c) and probability of exceedance $P\left(T = \frac{1}{P}\right)$].



T_C = Time of concentration

Fig. 8.2. Intensity-frequency-duration curves.



(Volume of the two hatched portions are equal)

Fig. 8.3. Runoff hydrograph due to uniform rainfall.

Rainfall Intensity ($i_{tc,p}$)

The rainfall intensity corresponding to a duration t_c (time of concentration) and the desired probability of exceedence P (i.e. return period $T = 1/P$) can also be found from the rainfall frequency duration relationship using the formula

$$i_{tc,p} = \frac{KT^x}{(t_c + a)^m}$$

in which K, a, x and m are constant.

Time of Concentration (t_c)

Time of concentration can be calculated using the following formula:

US Practice

For small drainage basins, the time of concentration is approximately equal to the lag time of the peak flow. Thus

$$t_c = t_p = C_{tL}\left(\frac{LL_{ca}}{\sqrt{S}}\right)^n$$

where t_c = time of concentration in hours.

L = basin length measured along the water course from the basin divide to the gauging station in km.

L_{ca} = distance along the main water course from the gauging station to centroid of the water shed in km.

S = basin slope

C_{tL} and n are basin constants.

Kirpich Equation (1940)

t_c = **0.01947** $L^{0.77}\, S^{-0.385}$

t_c = time of concentration (minutes)

L = maximum length of travel of water in meter

S = slope of the catchment = $\Delta H/L$ in which

ΔH = difference in elevation between the most remote point on the catchment and the outlet.

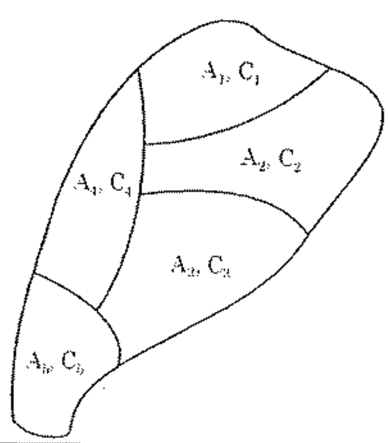
Runoff Coefficient

$$C = \frac{\text{runoff}}{\text{rainfall}} = \text{Runoff coefficient} = \text{impermeability factor.}$$

if different portions of catchment have different runoff coefficient then C_{eq} is calculated as

$$\boxed{C_{eq} = \frac{C_1A_1 + C_2A_2 + C_3A_3 + \ldots}{A_1 + A_2 + A_3 + \ldots}}$$

where C_i = Runoff coefficient pertaining to area A_i etc.



The IT rational formula is found to be suitable for peak-flow prediction in small catchments up to 50 km^2 in area. If finds considerable application in urban drainage designs and in the design of small culverts and bridges.

Example 8.1

An urban catchment has an area of 0.85 km^2. The slope of the catchment is 0.006 and the maximum length of travel of water is 950 m. The maximum depth of rainfall with a 25-year return period is as below:

Duration (min)	5	10	20	30	40	60
Depth of rainfall (mm)	17	26	40	50	57	62

If a culvert for drainage at the outlet of this area is to be designed for a return period of 25 years, estimate the required peak-flow rate, by assuming the runoff coefficient as 0.3.

Sol: The time of concentration is obtained by the Kirpich formula as

$$t_c = 0.01947 \times (950)^{0.77} \times (0.006)^{-0.385}$$

$$= 27.4 \text{ minutes}$$

By interpolation,

Maximum depth of rainfall for 27.4-min duration

$$= \frac{(50-40)}{10} \times 7.4 + 40 = 47.4 \text{ mm}$$

$$\text{Average intensity} = i_{tc,p} = \frac{47.4}{27.4} \times 60 = 103.8 \text{ mm/h}$$

$$Q_p = \frac{0.30 \times 103.8 \times 0.85}{3.6} = 7.35 \text{ m}^3\text{/s}$$

Example 8.2

If in the urban area, the land use of the area and the corresponding runoff coefficients are as given below, calculate the equivalent runoff coefficient.

Land use	Area (ha)	Runoff coefficient
Roads	8	0.70
Lawn	17	0.10
Residential area	50	0.30
Industrial area	10	0.80

Sol: Equivalent runoff coefficient $C_e = \dfrac{\sum_1^N C_i A_i}{A}$

$$C_e = \frac{[(0.7\times 8)+(0.1\times 17)+(0.3\times 50)+(0.8\times 10)]}{[8+17+50+10]}$$

$$= \frac{30.3}{85} = 0.36$$

Example 8.3

A small watershed consists of 1.5 km^2 of cultivated area ($C = 0.20$), 2.5 km^2 under forest ($C = 0.10$) and 1.0 km^2 under grass cover ($C = 0.35$). There is a fall of 22 m in a watercourse length of 1.8 km. The intensity-frequency-duration relation for the area may be taken as

$$I = \frac{80\,T_r^{0.2}}{(t+13)^{0.46}}$$

where I is in cm/h, T_r is in years and t is in minutes. Estimate the peak rate of runoff for a 25 year frequency.

Sol: Length of water course = 1.8 km = 1800 m

Fall in the elevation = 22 m

$$S = \frac{22}{1800}$$

$$t_c = 0.0195\, L^{0.77}\, S^{-0.385}$$

$$= 0.0195\,(1800)^{0.77}\left(\frac{22}{1800}\right)^{-0.385}$$

$$= 34 \text{ minutes}$$

$$I = \frac{80\,(25)^{0.2}}{(34+13)^{0.46}} = 25.9 \text{ cm/h}$$

$$A_1 = 1.5 \text{ km}^2 \qquad C_1 = 0.20$$

$$A_2 = 2.5 \text{ km}^2 \qquad C_2 = 0.10$$

$$A_3 = 1.0 \text{ km}^2 \qquad C_3 = 0.35$$

$$C_{eq} = \frac{(0.2\times1.5)+(0.1\times2.5)+(0.35\times1)}{1.5+2.5+1}$$

$$= 0.18$$

$$\Rightarrow \quad Q_p = 0.18 \times 0.259 \times \frac{1}{3600}\times5.0\times10^6 = 64.75 \text{ m}^3/\text{s}$$

Unit Hydrograph Method

The unit hydrograph technique can be used to predict the peak-flood hydrograph if the rainfall producing the flood, infiltration characteristics of the catchment and the appropriate unit hydrograph are available. For design flood extreme rainfall situations are used (design storm). The unit hydrograph of the catchment is then operated upon by the design storm to generate the desired flood hydrograph.

Frequency Analysis

Frequency analysis makes use of the observed data in the past to predict the future flood events along with their probabilities and return periods.

For frequency analysis and adequate and accurate data of previous floods are required. Generally records of less than 20 years should not be used in frequency analysis.

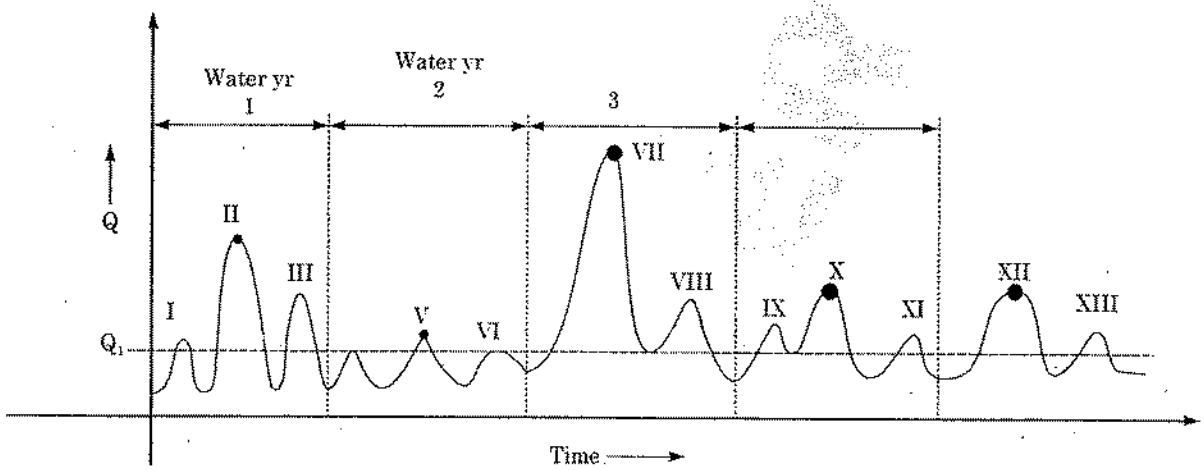
Dams, diversion, urbanisation and other land use changes introduce the inconsistency in data hence the data must be modified to bring it at par with the current condition of the catchment.

What data should be used for analysis?

If data for 50 yrs are available, then in that 50 yrs, may be, the flood may have come 200 times. But for analysis purpose we do not take all the 200 data into account. Data selected for analysis are:

(a) Data of annual series

(b) Data of partial series.



If peak discharge of II, V, VII, X, XII etc. (i.e. max flood of a year) are arranged in decreasing order of magnitude we get annual series. If however, all floods above a base value Q_1 is considered, we get *partial data series*.

Note: Note that the next higher peak of 1st year year (i.e. III) may be more than the peak of 2nd yr (i.e. V). However in annual series we take only the peak of each year into account.

For information about floods of fairly frequent occurrences, as is required during the construction period of a large dam (say 4-5 yrs), partial series are the best. While, for the spillway design flood, the annual series are preferred.

Hence normally for floods of large reccurance, interval (i.e. greater than 20 yrs, or in other words flood of exceedance probability ≤ 0.05) annual data series is used and for small recurrance interval (4-5 yrs) partial data series is used.

In our course we will concentrate only on annual data series.

Estimation of Design Flood for a Particular Return Period

Annual series are arranged in decending order of magnitude and rank no. is assinged to each with rank m = 1 for highest observed flood and $m = n$ (yr. of record) for lowest observed flood.

The return period is calculated for each event using Weibull's formula

$$\boxed{T_r = \frac{n+1}{m}}$$

T_r = return period in yr.

m = order no.

n = no. of yr of records

Return period represents the average no. of years within which a given event will be equalled or exceeded.

Probability of exceedance = $\frac{1}{T_r} = P$

If a graph is plotted between flood magnitude and its return period in simple plane co-ordinates, the plot is called *probability* or an *emperical distribution.*

This graph may be extrapolated or interpolated to get the design flood of any specific return period. We may also plot flood megnitude (y-axis, simple scale) and return period (x-axis log-scale) and use it for extropolation. [This curve may yield straight line plot].

However, the extrapolation for large return period may yield erroneous results. Hence for longer return period, theoretical probability distributions have to be used. The most commonly used distributions are:

(a) Gumble's distribution

(b) Log Pearson Type III distribution.

Gumble's Method

As per Gumble's method

$$X_T = \bar{X} + K.\sigma_{n-1}$$

where X_T = Value of variate (i.e. flood) with a return period of T

$\bar{X}$ = Mean value of variate $\frac{\sum X}{n} = \bar{X}$ (from annual series)

n = no. of yrs of record

σ_{n-1} = Standard deviation of the sample of size n

$$\sigma_{n-1} = \sqrt{\frac{\sum(X - \bar{X})^2}{n-1}}$$

K = Frequency factor

$$K = \frac{y_T - \bar{y}_n}{S_n}$$

y_T = reduced variate

$$\frac{1}{t} = (-)\left[\ln \ln \frac{T}{T-1}\right]$$

$\bar{y}_n$ = Mean of reduced variate

S_n = Standard deviation of reduced variate.

$\bar{y}_n$ and S_n are function of n (no. of yr. of record). Table is available for $\bar{y}_n$ and S_n against n. Hence S_n and $\bar{y}_n$ will be obtained from table.

However if, n is large (generally > 200)

$$y_n \rightarrow 0.577$$

$$S_n \rightarrow 1.2825$$

[Normally for n > 50 also some time we use $y_n = 0.577$, $S_n = 1.2825$ without much error]

Hence the step involved are:

1. Assemble the discharge data and note the sample size n. Here the annual flood value is the variate X. Find $\bar{X}$ and σ_{n-1} for the given data.
2. Using Tables determine $\bar{y}_n$ and S_n for given n.
3. Find y_T for a given T.
4. Find K.
5. Determine the required x_T.

Example 8.4

Flood-frequency computations for the river Chambal at Gandhisagar dam, by using Gumbel's method, yielded the following results:

Return period T (years)	Peak flood (m^3/s)
50	40,809
100	46,300

Estimate the flood magnitude in this river with a return period of 500 years.

Sol: Gumble's equation

$$x_{100} = \bar{x} + K_{100}\sigma_{n-1}$$

$$x_{50} = \bar{x} + K_{50}\sigma_{n-1}$$

$$(K_{100} - K_{50})\,\sigma_{n-1} = x_{100} - x_{50} = 46300 - 40809 = 5491$$

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

where S_n and $\bar{y}_n$ are constant for the given data series.

$$\therefore \quad (y_{100} - y_{50})\,\frac{\sigma_{n-1}}{S_n} = 5491$$

Also,

$$y_{100} = -[\ln \cdot \ln (100/99)] = 4.60015$$

$$y_{50} = -[\ln \cdot \ln (50/49)] = 3.90194$$

$$\Rightarrow \quad \frac{\sigma_{n-1}}{S_n} = \frac{5491}{(4.60015 - 3.90194)} = 7864$$

For $T = 500$ years, by eq.

$$\text{Now,} \quad y_{500} = -[\ln \cdot \ln (500/499)$$

$$\text{Now,} \ (y_{500} - y_{100})\,\frac{\sigma_{n-1}}{S_n} = x_{500} - x_{100}$$

$$(6.21361 - 4.60015) \times 7864 = x_{500} - 46300$$

$$x_{500} = 58988, \quad \text{say } 59{,}000 \text{ m}^3/\text{s}$$

Example 8.5

From the analysis of available data on annual flood peaks of a small stream for a period of 35 years, the 50 year and 100 year flood have been estimated to be 660 m³/s and 740 m³/s. Using Gumbel's method. Estimate the 200 year flood for the stream. (Take $y_n = 0.54034$ and $S_n = 1.12843$ for $n = 35$ yrs].

Sol:
$$\bar{y}_n = 0.54034 \text{ and } S_n = 1.12847$$

Using Gumbel's method

$$y_{50} = -\ln \times \ln\left(\frac{50}{49}\right) = 3.90194$$

$$K_{50} = \frac{3.90194 - 0.54034}{1.12847} = 2.9789$$

Similarly

$$y_{100} = -\ln \cdot \ln\left(\frac{100}{99}\right) = 4.60015$$

$$K_{100} = \frac{4.60015 - 0.54034}{1.12847} = 3.59762$$

$$x_{50} = \bar{x} + K_{50}\sigma_{n-1}$$

or
$$660 = \bar{x} + 2.9789\,\sigma_{n-1} \quad \text{(a)}$$

$$x_{100} = \bar{x} + K_{100}\sigma_{n-1}$$

or
$$\Rightarrow \quad 740 = \bar{x} + 3.59762\,\sigma_{n-1} \quad \text{(b)}$$

Solving the two simultaneous equations (a) and (b) for $\bar{x}$ and σ_{n-1}, we obtain $\bar{x} = 274.83$ m³/s and $\sigma_{n-1} = 129.3$ m³/s.

Now
$$y_{200} = -\ln \cdot \ln\left(\frac{200}{199}\right) = 5.29581$$

$$K_{200} = \frac{5.29581 - 0.54034}{1.12847} = 4.2141$$

$$x_{200} = \bar{x} + K_{200}\sigma_{n-1}$$

$$= 274.83 + 4.2141 \times 129.3 = 819.71 \text{ m}^3/\text{s}.$$

∴ The 200 year flood for the stream would be 820 m³/s.

Example 8.6

The maximum annual floods for river Kamala at Darbhanga were statically analysed for 93 yrs (1876-1968). The mean annual flood and standard deviation are 14210 and 9700 m³/s respectively. Determine:

(i) The recurrance interval of the heightest flood 42500 m^3/s in (1968) by Weibull's method and what is its percentage chance of occuring in

(a) any year

(b) in 10 yrs

(ii) What is the recurrance interval of the design flood adopted by CWPC (49500 m^3/s) and the heightest flood (42500) by Gumble's method.

Sol: For the heightest flood, its rank is $m = 1$ by Weibull's formula

$$T = \frac{n+1}{m} = \frac{93+1}{1} = 94 \text{ yr}$$

Its percentage chance of occuring $P = \frac{1}{T} = \frac{1}{94} = 0.01065 = 1.065\%$

i.e. its chance of occurrance in any years = 1%

Probability of its occurance in 10 yr is the probability of its occurance at least once in 10 yrs

$$= 1 - (1-p)^{10} = 1 - \left(1 - \frac{1}{94}\right)^{10} = 0.1014 = 10.14\%$$

(ii) By Gumble Method

$$X_T = \bar{X} + K\sqrt{\sigma_{n-1}}$$

$$\Rightarrow \quad 49500 = 14210 + K\,(9700)$$

$$\Rightarrow \quad K = 3.6381$$

$$K = \frac{y_T - y_n}{S_n}$$

but for large years of record (say $n > 50$ yrs)

$$y_n = 0.577$$

$$S_n = 1.2825$$

$$\Rightarrow \quad \frac{y_T - 0.577}{1.2825} = 3.6381$$

$$\Rightarrow \quad y_T = 5.24292$$

$$y_T = -\ln \cdot \ln \frac{T}{T-1}$$

$$\Rightarrow \quad \frac{T}{T-1} = 1.005$$

$$\Rightarrow \quad T = 189 \text{ yrs}$$

Return period for flood of magnitude 49500 is 189 yrs,

Return Period for Design Flood (42500 m³/s)

$$42500 = 14210 + 9700\,K$$

$$\Rightarrow \quad K = 2.916495$$

But $K = \dfrac{y_T - y_n}{S_n}$, for large years of record $y_n = 0.577$; $S_n = 1.2825$

$$\Rightarrow \quad 2.916495 = \frac{y_T - 0.577}{1.2825}$$

$$\Rightarrow \quad y_T = 4.3174$$

$$-\ln\ln\frac{T}{T-1} = 4.3174$$

$$\Rightarrow \quad \frac{T}{T-1} = 1.013424$$

$$\Rightarrow \quad T = 75.495 \text{ yrs}$$

Example 8.7

Statistical analysis of the annual floods of the river Tapti (1876-1968) using Gumbel's method yielded the 100-hr and 10-hr floods as 42800 and 22700 cumec, respectively. Determine:

(a) the matnitude of a 20-yr flood

(b) the probability of a flood of magnitude 35000 cumec (i) occurring in the next 10 years,

(ii) in the next year itself.

Sol: (a) $X_{100} = 42800$

$$X_{10} = 22700$$

$$X_{20} = ?$$

$$X_{100} = \bar{X} + K_{100}\sigma_{n-1} \quad \text{(i)}$$

$$X_{10} = \bar{X} + K_{10}\sigma_{n-1} \quad \text{(ii)}$$

$$X_{20} = \bar{X} + K_{20}\sigma_{n-1} \quad \text{(iii)}$$

$$\frac{X_{20} - X_{10}}{X_{100} - X_{10}} = \frac{(K_{20} - K_{10})}{(K_{100} - K_{10})} = \frac{\dfrac{y_{20} - y_n}{S_n} - \dfrac{y_{10} - y_n}{S_n}}{\dfrac{y_{100} - y_n}{S_n} - \dfrac{y_{10} - y_n}{S_n}}$$

$$\frac{X_{20} - X_{10}}{X_{100} - X_{10}} = \frac{y_{20} - y_{10}}{y_{100} - y_{10}}$$

$$y_{20} = -\ln\ln\frac{20}{19} = 2.9702$$

$$y_{10} = -\ln\ln\frac{10}{9} = 2.25037$$

$$y_{100} = -\ln\ln\frac{100}{99} = 4.60015$$

$$\Rightarrow \quad \frac{X_{20}-22700}{42500-22700} = \frac{2.9702-2.25037}{4.60015-2.25037}$$

$$\boxed{X_{20} = 28857.42 \text{ m}^3/\text{s}}$$

(b) Return period of $Q = 35000$ m³/s is T_s

$$X_T = 35000 = \bar{X} + K_T\sigma_{n-1}$$

$$\frac{X_T - X_{10}}{X_{100} - X_{10}} = \frac{y_T - y_{10}}{y_{100} - y_{10}}$$

$$\Rightarrow \quad \frac{35000-22700}{42800-22700} = \frac{y_T - 2.25037}{4.60015-2.25037}$$

$$\Rightarrow \quad y_T = 3.688295$$

$$-\ln\ln\frac{T}{T-1} = 3.688295$$

$$\Rightarrow \quad T = 40.479 \text{ yrs}$$

$$\Rightarrow \quad P = \frac{1}{T} = 0.0247$$

Probability of occurrance of this in next 10 yrs means probability of occurrance of at least once in next 10 yrs.

$$\Rightarrow \quad \text{Probability} = 1 - (1 - P)^{10} = 0.2213 = 22.13\%$$

Probability of occurrance in next yr itself = probability of occurrance of at least once in next one yr

$$= 1 - (1 - P)^1 = 1 - (1 - 0.0247) = 0.0247 = 2.47\%$$

SOME IMPORTANT POINTS

- For average of annual series, Gumbles method gives $T = 2.33$ yrs.

  Thus the value of flood with $T = 2.33$ yrs is called mean annual flood.
- X_T vs T plot on Gumble probability paper is a straight line. This property can be used for calculation of X_T for any T using Gumble probability peper plot.

Confidence Limit

Due to limitted data, the value of variate X_T determined using Gumble's method can have error. Hence it is desirable that confidence limits of the estimate be determined.

Confidence interval indicates the limits about the calculated value between which the true value can be said to lie with a specific probability based on sampling errors only.

For confidence probability 'α', the confidence interval of variate X_T is bounded by value X_1 and X_2 given by

$$\boxed{X_{1/2} = X_T \pm f(\alpha).S_e}$$

where $f(\alpha)$ = function of confidence probability 'α'. It can be found using the following table

α in percent	50	68	80	90	95	99
$f(\alpha)$	0.674	1.0	1.282	1.645	1.96	2.58

S_e = Probable error = $b\dfrac{\sigma_{n-1}}{\sqrt{n}}$

$$b = \sqrt{1+1.3K+1.1\,K^2}$$

$$K = \frac{y_T - y_n}{S_n}$$

$$y_T = -\ln\ln\frac{T}{T-1}$$

n = Sample size

T = Return period

σ_{n-1} = Standard deviation of sample

Example 8.8

Data covering a period of 92 years for the river Ganga at Raiwala yielded the mean and standard deviation of the annual flood series as 6437 and 2951 m³/s respectively. Using Gumbel's method estimate the flood discharge with a return period of 500 years. What is the 95% confidence limits for this estimate. [Take $\bar{y}_n$ = 0.5589 and S_n = 1.2020 for n = 92 yrs] also take the value of function of confidence probability for 95.1 and 80% confidence limit to be 1.96 and 1.282 respectively.

Sol: $\bar{y}_n$ = 0.5589 and S_n = 1.2020

$$\Rightarrow \quad y_{500} = -[\ln\cdot\ln(500/499)]$$

$$= 6.21361$$

$$K_{500} = \frac{6.21361 - 0.5589}{1.2020} = 4.7044$$

$$x_{500} = 6437 + 4.7044 \times 2951 = 20320 \text{ m}^3\text{/s}$$

also

$$b = \sqrt{1+1.3(4.7044)+1.1(4.7044)^2}$$

$$= 5.61$$

$$S_e = \text{probable error} = 5.61 \times \frac{2951}{\sqrt{92}} = 1726$$

(a) For 95% confidence probability $f(\alpha)$ = 1.96

$x_{1/2} = 20320 \pm (1.96 \times 1726)$

$x_1 = 23703$ m³/s , and $x_2 = 16937$ m³/s

Thus estimated discharge of 20320 m³/s has a 95% probability of lying between 23700 and 16940 m³/s.

(b) For 80% confidence probability, $f(\alpha) = 1.282$

$\Rightarrow x_{1/2} = 20320 \pm 1.282 \times 1726$

$= 20320 \pm 2212.7$

$= 22532.7$ and 18107.3

Note: As confidence probability increases, confidence interval also increases.

Risk Reliability and Safety Factor

The probability of occurrence of an even ($x \geq x_T$) at least once over a period of n successive years is called the risk R. Thus the risk is given by

$\bar{R} = 1 -$ (probability of non-occurrence of the even $x \geq x_T$ in n years)

$$\bar{R} = 1 - (1 - P)^n$$

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

where P = probability $P(x \geq x_T) = \frac{1}{T}$

T = return period

The reliability R_e, is defined as

$$R_e = 1 - \bar{R} = \left(1 - \frac{1}{T}\right)^n$$

Example 8.9

A bridge has an expected life of 25 years and is designed for a flood magnitude of return period 100 years. (a) What is the risk of this hydrologic design? (b) If a 10% risk is acceptable, what return period will have to be adopted?

Sol: (a) The risk $\bar{R} = 1 - \left(1 - \frac{1}{T}\right)^n$

Here $n = 25$ years and $T = 100$ years

$$\bar{R} = 1 - \left(1 - \frac{1}{100}\right)^{25} = 0.222$$

Hence the inbuilt risk in this design is 22.2%.

(b) If $\bar{R} = 10\% = 0.10$

$$0.10 = 1 - \left(1 - \frac{1}{T}\right)^{25}$$

$$\left(1 - \frac{1}{T}\right)^{25} = 0.90 \text{ and } T = 238 \text{ years}$$

Hence to get 10% acceptable risk, the bridge will have to be designed for a flood of return period $T = 238$ years.

Example 8.10

Annual flood data of the river Narmada at Garudeshwar covering the period 1948 to 1979 yielded for the annual flood discharges a mean of 29,600 m^3/s and a standard deviation of 14,860 m^3/s. 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 m^3/s what are the safety factor and safety margin relating to maximum flood discharge? [Take for n = 32 yrs, $\bar{y}_n = 0.5380$, $S_n = 1.1193$].

Sol: Risk $\bar{R} = 0.10$

Life period of the structure $n = 50$ years

Hence $\bar{R} = 1.10 = 1 - \left(1 - \frac{1}{T}\right)^{50}$

$T = 475$ years

Gumbel's method is now used to estimate the flood magnitude for this return period of $T = 475$ years.

$$y_T = -\left[\ln . \ln \frac{T}{T-1}\right] - \left[\ln . \ln\left(\frac{475}{474}\right)\right] = 6.16226$$

$$K = \frac{y_T - \bar{y}_n}{S_n} = \frac{(6.16226 - 0.5380)}{1.1193} = 5.0248$$

$x_T = \bar{x}_T + K\sigma_{n-1}$

$= 29600 + (5.0248 \times 14860) = 104268$ = hydrological design flood magnitude.

Actual flood magnitude adopted in the project is = 125,000 m^3/s

Safty factor = $(SF)_{\text{flood}}$ = 125,000/104,268 = 1.199

Safety margin for flood magnitude = 125,000 − 104,268 = 20,732 m^3/s

CHAPTER

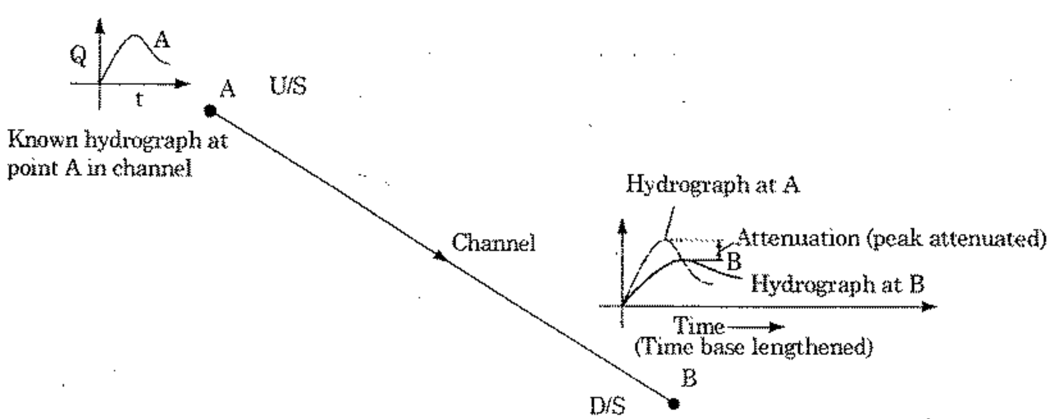
9

Flood Routing

Introduction

Flood routing is a procedure whereby the shape of a flood hydrogrpah at a particular location on the stream is determined from the known or assumed flood hydrograph at some other location upstream.

As the discharge in a stream due to flood increases, the stage also increases and hence the volume of water in temporary storage in the channel increases. As the flood recedes, this temporary storage depletes. As a result, a flood hydrograph moving down a channel has its time base lengthened and peak gets reduced. The flood wave then is said to be attenuated. Similar is the effect of reservoir storage.



In flood routing our aim is to determine hydrograph *B* from known hydrograph A at the upstream location.

Flood routing is used in:

(a) Establishing the flood peak at a downstream location (i.e. prediction of flood).

(b) Establishing the effect of construction of reservoir on flood (i.e. to establish how much of reduction in peak results as a result of construction of reservoir).

(c) Determining the required levee height for flood protection.

(d) Predicting the behaviour of river after a change has been done in the channel conditions.

(e) Determining the adequacy of spillway.

The basic techniques used in flood routing are:

(i) Lumped routing (hydrologic routing).

(ii) Distributed routing (hydraulic routing)

- In Lumped routing we find out:

  $Q = f(t)$ at a given x-location

  i.e. discharge as a function of time at any given x

- In distributed routing we find out

  $Q = f(x, t)$

  i.e. discharge as a function of time and space.

- In Lumped routing we use continuity equation.
- In distributed routing we use continuiety equation and momentum equation.

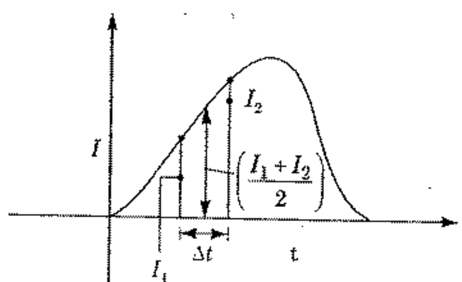
Basic Equations Used in Flood Routing

1. **Continuity equation:**

$$I - Q = \frac{ds}{dt} \quad \text{i.e. inflow-outflow = storage}$$

for small time increment Δt

$$\left(\frac{I_1 + I_2}{2}\right)\Delta t - \left[\frac{Q_1 + Q_2}{2}\right]\Delta t = \left[S_2 - S_1\right] \qquad \text{(A)}$$



Note that Δt should be small so that inflow and outflow hydrograph can be assumed as straight line in time interval Δt.

2. In differential form the continuity equation for unsteady flow in a reach with no lateral inflow given by

$$\boxed{\frac{\partial Q}{\partial x} + T\frac{\partial y}{\partial t} = 0} \qquad \text{(B)}$$

where T = Top surface width, y = depth of flow.

3. The momentum equation is given by

$$\boxed{\frac{\partial y}{\partial x} + \frac{V}{g}\frac{\partial V}{\partial x} + \frac{1}{g}\frac{\partial V}{\partial t} = S_0 - S_f} \qquad \text{(C)}$$

V = velocity of flow, S_0 = bed slope, S_f = energy line slope

Euqation (B) and (C) are Saint Venants Equation for gradually valued unsteady flow.

In our course we will concentrate only on Lumped routing (hydrologic routing). There are two type of routing used:

(i) Storage routing or reservoir routing.

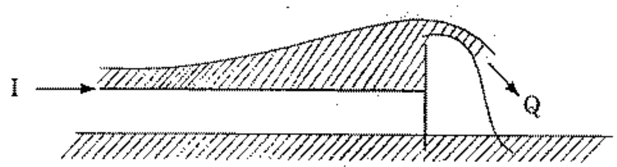
(ii) Channel routing.

Storage Routing

- In storage routing, we assume level pool routing concept i.e. the water level will be assumed horizontal. [Although the actual water surface will not be level due to varied flow phenomenon].
- In this case storage will be function of elevation only

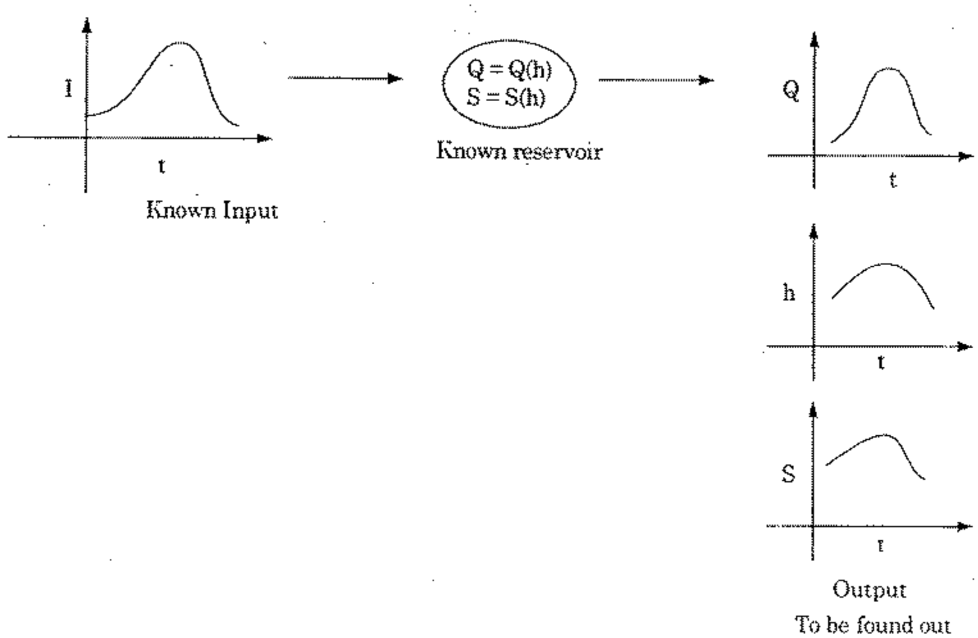
$$S = S(h)$$

Note: If we assumed the varied flow concept with water level not level, than storage will be a function of inflow and outflow.



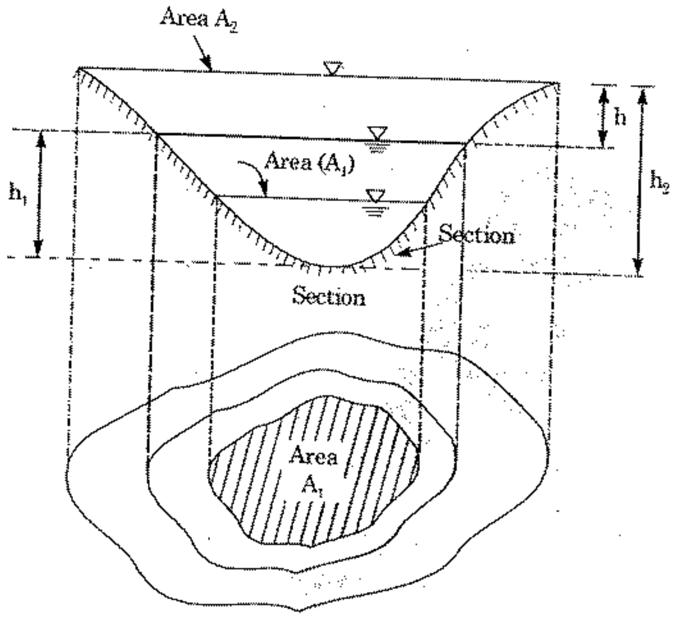
The shaded portion (storage) will be a function of inflow and outflow.

- Thus, in level pool routing the inflow hydrograph is known and at the same time the outflow-elevation and storage elevation curve can be established. We will have to find out <u>outflow discharge vs time</u> or <u>elevation vs time</u> or <u>storage vs time curves</u>.



- The outflow elevation and storage elevation relationship will be establish as follows:

Storage Elevation Curve



Plan

- The area at the reservoir site is surveyed and contour maps are prepared. From this area enclosed by various contours are planimetered. Once the areas enclosed by contours are known, the incremental volumes of water stored between any two successive contours can be determined using one of the following equations

$$\Delta V = \frac{h}{2}(A_1 + A_2) \quad \text{[Trapezoidal formula]}$$

$$\Delta V = \frac{h}{3}(A_1 + A_2 + \sqrt{A_1 A_2}) \quad \text{[Cone formula]}$$

$$\Delta V = \frac{h}{6}[A_1 + 4A_m + A_2] \quad \text{[Prismoidal formula]}$$

*Prismoidal rule is more accurate.

A_1 and A_2 are the areas corresponding to the successive contours and 'h' is the difference between elevation (contour internal). A_m is the area enclosed by a contour line midway between the two adjacent contours.

- If these incremental volumes are accumulated upto any contour value, then the sum represents the storage volume of the reservoir upto the elevation of that contour. Thus elevation-storage relationship is established

Outflow-Storage Relationship

$$Q_{spillway} = \frac{2}{3} C_{d_1} \sqrt{2g}\, L \left(H - H_s\right)^{3/2}$$

$$Q_{sluice} = C_{d_2} A \sqrt{2g(H - H_{sl})}$$

$$Q_{total} = Q_{spillway} + Q_{sluice}$$

Thus $Q = Q(H)$ is obtained.

In the above formulation

C_{d_1} = Coefficient of discharge for spillway

C_{d_2} = Coefficient of discharge for sluices

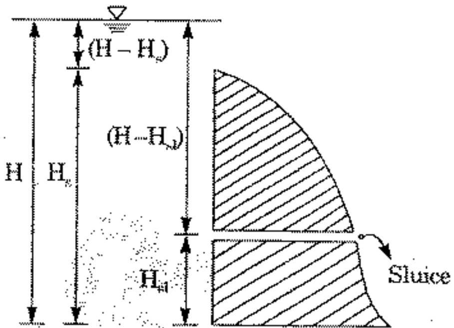
H_s = Sill level of spillway

H_{sl} = Centre of sluice height

A = Area of sluices

L = Length of spillway crest

H = Water surface elevation



Example 9.1

The topographical survey of a proposed reservoir site yielded the following data:

Contour elevation (m)	470	472	474	478	480	482	484	486
Contour area (hectare)	219	227	257	278	303	330	362	396

There are two circular sluices with diameter of 2.5 m and with their centres at an elevation 470 m. A spillway with an effective crest length of 20 m is also provided with its sill at 480 m. The C_d for sluices may be taken as 0.8 and for spillway $\frac{2}{3}C_d\sqrt{2g} = C = 2.25$. Prepare the storage-discharge curve for the reservoir. (Use trapezoidal rule).

Sol: *Storage-elevation relation:*

Elevation	Area	$\frac{A_1+A_2}{2}$	Incremental storage = $h\frac{(A_1+A_2)}{2}$ h = 2m = contour interval	Cumulative storage in hac – m	Storage (m^3/s)-(day)
470	219	—	—	0	0
472	227	223	446	446	51.62
474	240	233.5	467	913	105.67
476	257	248.5	497	1410	163.19
478	278	267.5	535	1945	225.12
480	303	290.5	581	2526	292.36
482	330	316.5	633	3159	365.63
484	362	346.0	692	3851	445.72
48	396	379.0	758	4609	533.45

$$\text{Discharge from sluice} = 0.8\times2\times\frac{\pi}{4}(2.5)^2\times\sqrt{2\times9.81\times(H-470)}$$

$$Q_{sluice} = 34.79\sqrt{H-470}$$

Discharge from spillway

$$Q_{spillway} = 2.25\times20\,(H-480)^{3/2}$$
$$Q_{spillway} = 45\,(H-480)^{3/2}$$

Computation of Discharge

Eleavation H (m)	$Q_{spillway}$ (m^3/s)	Q_{sluice} (m^3/s)	$Q = Q_{spillway} + Q_{sluice}$ (m^3/s)
470	0	0	0
472	0	49.2	49.2
474	0	69.6	69.6
476	0	85.2	85.2
478	0	98.4	98.4
480	0	110.0	110.0
482	127.3	120.5	247.8
484	360.0	130.2	490.2
486	661.4	139.2	800.6

Thus storage discharge relationship is as under:

Elevation (m)	Storage (m^3/s-day)	Discharge (m^3/s)
470	0	0
472	51.62	49.2
474	105.67	69.6
476	163.19	85.2
478	225.12	98.4
480	292.36	110.0
482	365.63	274.8
484	445.72	490.2
486	533.45	800.6

Note: Note that there could be storage below elevation of 470 m but it would be dead storage. It cannot affect the flood hydrograph and hence is irrelevant for flood routing.

Various Methods of Reservoir Routing

(a) Modified Pul's method
(b) Goodrich method
} (Graphical method)

(c) Standard 4th order Runge-Kutta method (Numerical method)

Modified Pul's Method

$$\left(\frac{I_1 + I_2}{2}\right)\Delta t - \left(\frac{Q_1 + Q_2}{2}\right)\Delta t = S_2 - S_1$$

The only unknowns in this are S_2 and Q_2 [i.e. storage and discharge after time Δt, Q_1, S_1 being known at t = 0]. Hence we rearange the term such that all knowns are on one side and all unknown are on other side.

$$\boxed{\left(\frac{I_1+I_2}{2}\right)\Delta t+\left(S_1-\frac{Q_1\Delta t}{2}\right)=\left(S_2+\frac{Q_2\Delta t}{2}\right)}$$

As storage and discharge both are functions of elevation hence we find a relationship of $\left(S+\frac{Q\Delta t}{2}\right)$ and elevation.

Thus we have $Q = Q(H)$, $S = S(H)$ and $\left(S+\frac{Q\Delta t}{2}\right) = f(H)$ are known.

The various steps in the routing are:

(i) For the 1st time interval Δt

$\left(\frac{I_1+I_2}{2}\right)\Delta t$ and $\left(S_1-\frac{Q_1\Delta t}{2}\right)$ are known, hence $\left(S_2+\frac{Q_2\Delta t}{2}\right)$ is determined.

(ii) From the relationship of $\left(S+\frac{Q\Delta t}{2}\right)$ vrs (H), H is found out. At the same time, from H outlet discharge is found out using discharge-elevation relationship.

(iii) For the next time increment, $\left(S_1-\frac{Q_1\Delta t}{2}\right)$ at the beginning is found out from $\left[\left(S_2+\frac{Q_2\Delta t}{2}\right)-Q_2\Delta t\right]$ and the procedure described above is repeated till the entire inflow hydrograph is routed.

Example 9.2

If the storage-discharge-elevation relationship of a reservoir is as folows:

Elevation (m)	Storage (m^3/s-day)	Discharge (m^3/s)
470	0	0
742	51.62	49.2
474	105.67	69.6
476	163.19	85.2
478	225.12	98.4
480	292.36	110
482	365.63	247.8
484	445.72	490.2
486	533.45	800.6

and a flood passes through the reservoir like

Time (h)	0	6	12	18	24	30	36	42	48	54	60	66	72
Flow (m^3/s)	50	180	270	360	410	370	300	230	155	90	60	35	20

Find the flood hydrograph after routing, if the outflow from reservoir just before the flood was 200 m^3/s

Sol: Take: Note 6 hr = 0.25 days, Δt = 6 hr = 0.25 days

Table-I

Storage (m^3/s-days)	Discharge (m^3/s)	Elevation	$\left(S+\frac{Q\Delta t}{2}\right)$	$\left(S-\frac{Q\Delta t}{2}\right)$
0	0	470		
51.62	49.2	472	57.77	45.47
105.67	69.6	474	114.37	96.97
163.19	85.2	476	173.84	152.54
225.12	98.4	478	237.42	212.82
292.36	110	480	306.11	278.61
365.63	247.8	482	396.61	334.65
445.72	490.2	484	507.00	384.44
533.45	800.6	486	633.53	433.37

Note that $51.62+\frac{49.2}{2}\times 0.25 = 57.77$

$$105.67+\frac{69.6}{2}\times 0.25 = 114.37$$

$$51.53-\frac{49.2}{2}\times 0.25 = 45.47$$

$$105.67-\frac{69.6}{2}\times 0.25 = 96.97$$

Table-II

Time (hr)	Inflow (m^3/s)	$\left(\frac{I_1+I_2}{2}\right)\Delta t$ (m^3/s-day)	$S_1-\frac{Q_1\Delta t}{2}$ (m^3/s-day)	$S_2+\frac{Q_2}{2}\Delta t$ (m^3/s-day)	Elevation (m)	Outflow discharge (m^3/s)
0	50				481.3	200
6	180	28.75	315	343.75	480.83	167.3
12	270	56.25	301.925	358.175	481.15	189.235
18	360	78.75	310.866	389.62	481.84	236.78
24	410	96.25	330	426.25	482.54	313
30	370	97.5	348	445.50	482.88	355
36	300	83.75	356	439.75	482.78	372
42	230	66.25	354	420.25	482.43	300
48	155	48.13	345	393.13	481.92	242
54	90	30.63	332	362.63	481.25	196
60	60	18.75	314	332.75	480.58	150
66	35	11.88	295	306.88	480.00	110
72	20	6.88	280	286.88	479.78	106

Note: For initial discharge of 200 m^3/s, elevation is obtained by inter polation as follows:

$$\text{Elevation for } Q = 200 \text{ m}^3/\text{s} = 480+\frac{(200-110)\times(482-480)}{(247.8-110)}$$

$$= 481.3 \text{ m}$$

Note that

For elevation = 481.3, $\left(S_1 - Q_1 \frac{\Delta t}{2}\right)$ is calculated as

$$S_1 - Q_1 \frac{\Delta t}{2} = 278.61 + \frac{1.3}{2} \times (334.65 - 278.61) \text{ [By interpolation from table I]}$$

$$= 315 \text{ m}^3\text{/s-day}$$

Also from modified Pul's method

$$S_2 + \frac{Q_2 \Delta t}{2} = \frac{I_1 + I_2}{2} \Delta t + \left(S_1 - \frac{Q_1 \Delta t}{2}\right) \ldots$$

$$= 28.75 + 315 = 343.75$$

Corresponding discharge from Table 1

$$= 110 + \frac{247.8 - 110}{(396.61 - 306.11)} (343.75 - 306.11)$$

$$= 167.3 = Q_1 \text{ for next time increment}$$

and corresponding elevation is $= 480 + \frac{(343.75 - 306.11)}{(396.61 - 306.11)} \times 2$

$$= 480.83$$

For next time increment

$$S_1 - \frac{Q_1 \Delta t}{2} = (343.75) - Q_2 \Delta t$$

$$= 343.75 - 167.3 \times 0.25$$

$$= 301.925 \text{ m}^3\text{/s-day}$$

and hence $S_2 + \frac{Q_2 \Delta t}{2} = \left(\frac{I_1 + I_2}{2}\right) \Delta t + \left(S_1 - \frac{Q_1 \Delta t}{2}\right)$

$$= 56.25 + 301.925$$

$$= 358.175$$

Corresponding elevation from table 1 is

$$= 480 + \frac{2 \times (358.175 - 306.11)}{(396.61 - 306.11)}$$

$$= 481.15 \text{ m}$$

and discharge is

$$= 110 + \frac{(247.8 - 110)(481.15 - 480)}{(482 - 480)}$$

$$= 189.235 \text{ m}^3\text{/s} = Q_1 \text{ for next time increment}$$

For next time increment

$$S - \frac{Q \Delta t}{2} = 358.175 - 189.235 \times 0.25$$

$$= 310.866$$

and hence

$$S+\frac{Q\Delta t}{2} = \left(\frac{I_1+I_2}{2}\right)\Delta t+\left(S-\frac{Q\Delta t}{2}\right)$$

$$= 78.75 + 310.866$$

$$= 389.62$$

for which elevation from table 1 is

$$= 480+\frac{(389.62-306.11)}{(396.61-306.11)}$$

$$= 481.84$$

and discharge is

$$= 110+\frac{481.84-480}{(482-480)}\times(247.8-110)$$

$$= 236.78$$

On similar line table II will be completed.

Goodrich method: (Hydrological reservoir routing)

$$\left(\frac{I_1+I_2}{2}\right)\Delta t-\left(\frac{Q_1+Q_2}{2}\right)\Delta t = S_2-S_1$$

$$\boxed{(I_1+I_2)+\left(\frac{2S_1}{\Delta t}-Q_1\right)=\left(\frac{2S_2}{\Delta t}+Q_2\right)}$$

All terms in LHS is known, and that on RHS is unknown for a given time increment.

$\left(\frac{2S}{\Delta t}+Q\right)_2$ is determined from above equation and from storage – elevation discharge data $\left(\frac{2S}{\Delta t}+Q\right)_2$ is known as a function of elevation.

Hence discharge, elevation and storage at the end of time increment is known.

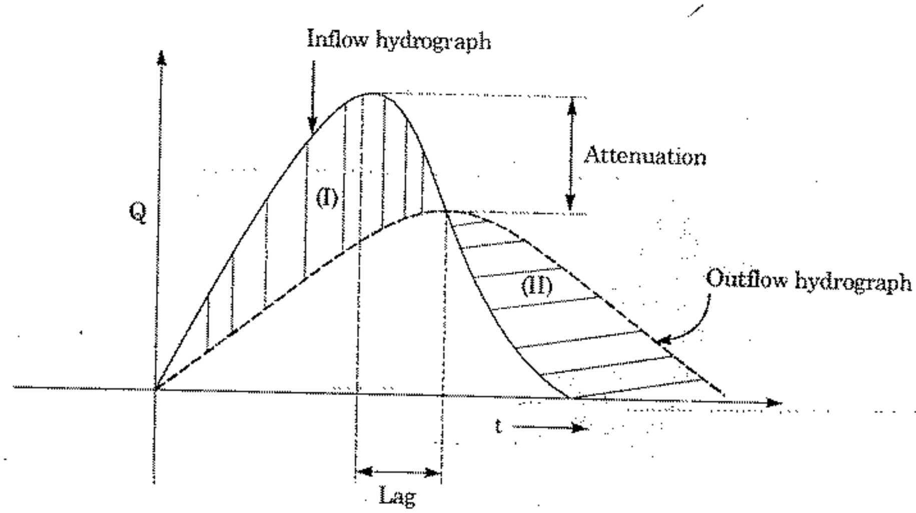
For next time increment

$$\left[\left(\frac{2S}{\Delta t}+Q\right)_2-2Q_2\right] \text{ of previous time increment}$$

$$= \left(\frac{(2S)}{\Delta t}-Q\right) \text{ for use as initial value of next time increment.}$$

Attenuation, Lag and Storage Characteristic

If we plot the inflow hydrograph and outflow hydrograph (routed hydrograph) the fig. obtained is as under



Note that the peak of outflow hydrograph is lowered and its time base is lengthened.

- This reduction in peak is called attenuation.
- The time difference between the two peaks is called lag.
- If the outflow from the reservoir is uncontrolled, then peak of outflow hydrograph will occur at the point of intersection of inflow and outflow hydrograph (i.e. when inflow discharge = outflow discharge).
- Storage is maximum at the point of intersection of inflow and outflow hydrograph.

Note: In level pool routing, when water surface is assumed horizontal, the storage will be a function of elevation (H) only

$$\Rightarrow \quad S = S(H)$$

$$\Rightarrow \quad \frac{ds}{dt} = A.\frac{dH}{dt},\ A = \text{Area of reservoir at elevation H} \cdots\cdots [\alpha]$$

The outflow is also a function of H

$$\Rightarrow \quad Q = Q(H)$$

$$\Rightarrow \quad \frac{dQ}{dt} = 0 \text{ at peak flow}$$

$$\Rightarrow \quad \frac{dH}{dt} = 0 \text{ and } \frac{ds}{dt} = 0 \text{ at peak flow} \quad [\text{from } (\alpha)]$$

$\Rightarrow$ At peak flow storage is max also

$$I - Q = \frac{ds}{dt}$$

$\Rightarrow$ At peak flow

$$I = Q$$

For regulated (controlled) reservoir, outflow discharge will not be maximum at point of intersection of inflow and outflow reservoir. i.e. peak will not occur when inflow discharge = outflow discharge.

Hydrological Channel Routing

- In a stream channel (river) a flood wave may be reduced in magnitude and lengthened in travel time i.e., attenuated, by storage in the reach between two sections.
- The storage in the reach can be divided into two parts-prism storage and wedge storage.
- The volume that would be stored in the reach if the flow were uniform throughout, i.e., when water surface line is parallel to bed line, at a level of downstream water level is called prism storage and the volume stored between this line and the actual water surface profile due to outflow being different from inflow into the reach is called 'wedge storage'.
- During rising stages the wedge storage volume is considerable while during falling stages, as inflow drops more rapidly than outflow, the wedge storage becomes negative.

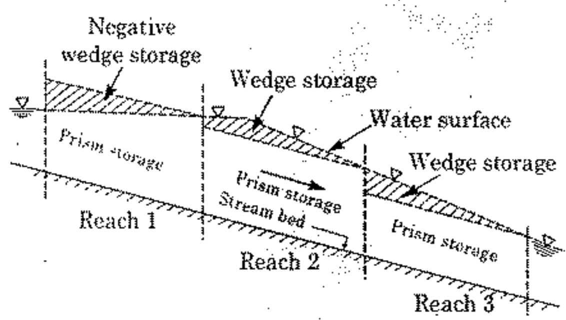


Fig. Flood is in rising stage in reach 2 and 3 while it is in falling stage in reach 1.

- In the case of stream-flow routing the solution of the storage equation is more complicated than in the case of reservoir routing, because the wedge storage is involved.
- While the storage in a reach depends on both the inflow and outflow, prism storage depends on the outflow alone (uniform flow condition) and the wedge storage depends on the difference of inflow and outflow $(I - O)$.
- A common method of stream/channel flow routing is the Muskingum method where the storage is expressed as a function of both inflow and outflow in the reach as

$$S = K[Q + x[I - Q] = K\,[x\,I + (1 - x)\,Q]$$

and this relationship is known as the *Muskingum equation*. In this the parameter x is known as *weighting factor* it takes a value between 0 and 0.5. The value of x depends on the shape of the wedge. When $x = 0$, obviously the storage is a function of discharge only

i.e. $$S = KQ$$

Such a storage is known as *linear storage* or *linear reservoir* (or reservoir type storage).

The coefficient K is known as *storage-time constant* and has the dimensions of time. It is approximately equal to the time of travel of a flood wave through the channel reach. The value of K depends on the length of the reach and other roughness characteristics.

Note: By chosing arbitrary value of x between 0–0.5 and by finding out accumulated storage as $\Sigma(I - Q)\Delta t$, a graph is plotted between accumulated storage and $[xI + (1 - x)Q]$ as ordinate. If nearly straight line is obtained, the corresponding value of x is the correct value and inverse slope of the above line is the K value.

For a given reach x and K are assumed cosntant.

Muskingum Method of Routing

For a given channel reach by selecting a routing interval Δt and using the Muskingum equation, the change in storage is

$$S_2 - S_1 = K[x(I_2 - I_1) + (1 - x)(Q_2 - Q_1)] \quad \text{(A)}$$

where suffixes 1 and 2 refer to the conditions before and after the time interval Δt. The continuity equation for the reach is

$$S_2 - S_1 = \left(\frac{I_2 + I_1}{2}\right)\Delta t - \left(\frac{Q_2 + Q_1}{2}\right)\Delta t \quad \text{(B)}$$

From eqs. (A) and (B), Q_2 is evaluated as

$$Q_2 = C_0 I_2 + C_1 I_1 + C_2 Q_1 \quad \text{(Muskingum routing equation)} \quad \text{(C)}$$

$$\text{where } C_0 = \frac{-Kx + 0.5\Delta t}{K - Kx + 0.5\Delta t}$$

$$C_1 = \frac{Kx + 0.5\Delta t}{K - Kx + 0.5\Delta t}$$

$$C_2 = \frac{K - Kx - 0.5\Delta t}{K - Kx + 0.5\Delta t}$$

Note that $C_0 + C_1 + C_2 = 1.0$, C_0, C_1 and C_2 are constant for a reach. In general, for the n th time step $Q_n = C_0 I_n + C_1 I_{n-1} + C_2 Q_{n-1}$

- For best results the routing interval Δt should be so chosen that $K > \Delta t > 2Kx$. Generally, negative values of coefficients are avoided by choosing appropriate values of Δt.

Following steps are used for channel routing using Muskingum method:

(i) Knowing K and x, select an appropaite value of Δt. $[K > \Delta t > 2Kx]$

(ii) Calculate C_0, C_1 and C_2.

(iii) Starting from the initial conditions I_1, Q_1 and known I_2 at the end of the first time step Δt, calculate Q_2 by eq. (C)

(iv) The outflow calculated in step (iii) becomes the known initial outflow for the next time step. Repeat the calculations for the entire inflow hydrograph.

Example 9.3

The inflow hydrograph readings for a stream reach are given below for which the Muskingum coefficients of $K = 36$ hr and $x = 0.15$ apply. Route the flood through the reach and determine the outflow hydrograph. Also determine the reduction in peak and the time of peak of outflow.

Outflow at the beginning of the flood may be taken as the same as inflow.

Time (hr)	0	12	24	36	48	60	72	84	96	108	120
Inflow (cumec)	42	45	88	272	342	288	240	198	162	133	110

Time (hr)	132	144	156	168	180	192	204	216	228	240
Inflow (cumec)	90	79	68	61	56	54	51	48	45	42

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

$x = 0.15$, $K = 36$ hr $= 1.5$ day; take the routing period (from the inflow hydrograph readings) as 12 hr $= \frac{1}{2}$ day. Compute C_0, C_1 and C_2 as follows:

$$C_0 = \frac{-Kx + 0.5t}{K - Kx + 0.5t} = \frac{-1.5 \times 0.15 + 0.5 \times \frac{1}{2}}{1.5 - 1.5 \times 0.15 + 0.5 \times \frac{1}{2}} = \frac{0.025}{1.525} = 0.02$$

$$C_1 = \frac{Kx + 0.5t}{K - Kx + 0.5t} = \frac{1.5 \times 0.15 + 0.5 \times \frac{1}{2}}{1.525} = 0.31$$

$$C_2 = \frac{K - Kx - 0.5t}{K - Kx + 0.5t} = \frac{1.5 - 1.5 \times 0.15 - 0.5 \times \frac{1}{2}}{1.525} = 0.67$$

Check: $C_0 + C_1 + C_2 = 0.02 + 0.31 + 0.67 = 1$.

$$Q_2 = 0.02\ I_2 + 0.31\ I_1 + 0.67\ Q_1$$

In the following table I_1, I_2 are known from the inflow hydrograph, and Q_1 is taken as I_1 at the beginning of the flood since the flow is almost steady.

Table: Stream flow routing–Muskingum method

Time (hr)	Inflow (I) (cumec)	$0.02\ I_2$ (cumec)	$0.31\ I_1$ (cumec)	$0.67\ Q_1$ (cumec)	Outflow (Q) (cumec)
0	42	—	—	—	42*
12	45	0.90	13.0	28.2	42.1
24	88	1.76	14.0	28.3	44.0
36	272	5.44	27.3	29.5	62.2
48	$\boxed{342}$	6.84	84.3	41.7	132.8
60	288	5.76	106.0	89.0	200.7
72 36 hr	240	4.80	89.2	139.0	233.0
84	198	3.96	74.4	156.0	$\boxed{234.0}$
96	162	3.24	61.4	157.0	221.6
108	133	2.66	50.2	148.2	201.0
120	110	2.20	41.2	134.5	178.9
132	90	1.80	34.1	119.8	155.7
144	79	1.58	27.9	104.0	133.5
156	68	1.36	24.4	89.5	115.3
163	61	1.22	21.1	77.4	99.7
180	56	1.12	18.9	66.8	86.8
192	54	1.08	17.4	58.2	76.7
204	51	1.02	16.7	51.4	69.1
216	48	1.00	15.8	46.3	63.1
228	45	0.90	14.8	42.3	58.0
240	42	0.84	13.9	38.9	53.6

*Q_1 is assumed equal to $I_1 = 42$ cumec.

$$\therefore \quad Q_2 = 0.02 \times 45 + 0.31 \times 42 + 0.67 \times 42 = 42.06 \text{ cumec} \approx 42.1$$

This value of Q_2 becomes Q_1 for the next routing period and the process is repeated till the flood is completely routed through the reach. The resulting outflow hydrograph is plotted as shown in figure. The reduction in peak is 108 cumec and the lag time is 36 hr, i.e., the peak outflow is after 84 ($= 3\frac{1}{2}$ days) after the commencement of the flood through the reach.

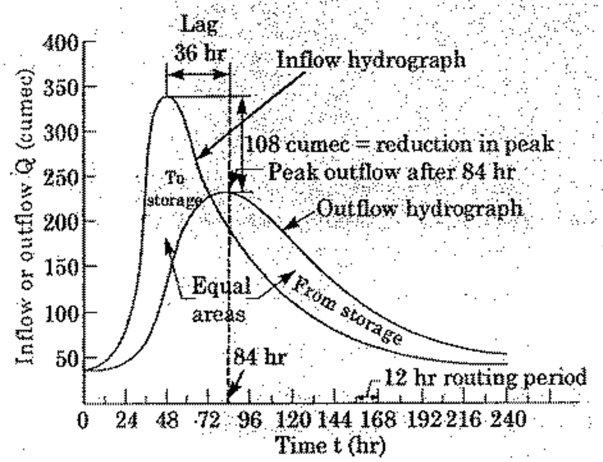


Fig. Streamflow routing by Muskingum method.

Flood Control

1. Structural methods:
   (i) Storage and detention reservoirs,
   (ii) Levees (flood embankments),
   (iii) Channel improvement,
   (iv) Flood ways (new channels), and
   (v) Soil conservation
2. Non-structural methods:
   (i) Flood plain zoning, and
   (ii) Flood warning, evacuation and relocation.

Flood Control by Reservoirs

The purpose of a flood control reservoir is to temporarily store a portion of the flood so that the flood peaks are flattened out. The reservoir may be ideally situated immediately upstream of the area to be potected and the water discharge in the channel downstream at its safe capacity. All the inflow into the reservoir in excess of the safe channel capacity is stored until the inflow drops below the channel capacity and the stored water is released to recover the storage capacity for the next flood.

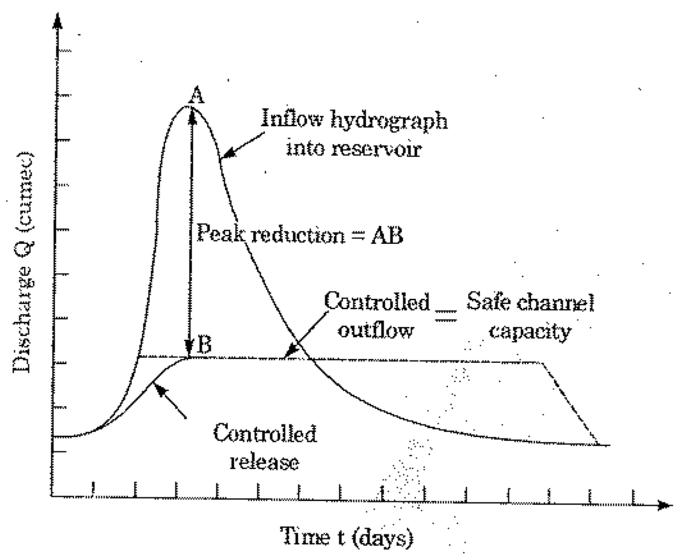


Fig. Flood control by reservoirs.

Detention Reservoirs

A detention reservoir consists of an obstruction to a river with an uncontrolled outlet. These are essentially small structures and operate to reduce the flood peak by providing temporary storage and by restriction of the outflow rate.

Levees

Levees, also known as *dikes or flood embankments* are earthen banks constructed parallel to the course of the river to confine it to a fixed course and limited cross-sectional width. The heights of levees will be higher than the design flood level with sufficient free board. The confinement of the river to a fixed path frees large tracts of land from inundation and consequent damage.

Masonry structures used to confine the river in a manner similar to levees are known as *flood walls*.

Floodways

Floodways are natural or manmade channels into which a part of the flood will be diverted during high stages.

Channel Improvement

The works under this category involve:

1. Widening or deepening of the channel to increase the cross-sectional area;
2. Reduction of the channel roughness, by clearing of vegetation from the channel perimeter;
3. Short circuiting of meander loops by cutoff channels, leading to increased slopes.

Soil Conservation

Soil-conservation measures leads to increased infiltration, greater evapotranspiration and reduced soil erosion. Small and medium floods are reduced by soil-conservation measures.

□□□□

OBJECTIVE QUESTIONS

1. Match **List-I** with **List-II** and select the correct answer using the codes given below the lists:

   List-I
   A. Conservation reservoirs
   B. Retarding basins
   C. Flood plains
   D. Flood walls

   List-II
   1. Uncontrolled outlets
   2. Flood-fighting
   3. Temporary storage of flood water
   4. Controlled outlets

   Codes:

	A	B	C	D
(a)	1	4	3	2
(b)	1	4	2	3
(c)	4	1	3	2
(d)	4	1	2	3

2. Probability of a 10 year flood to occur at least once in the next 4 years is
   (a) 25% (b) 35%
   (c) 50% (d) 65%

3. The Standard Project Flood is
   (a) derived from the probable maximum precipitation in the region
   (b) derived from the severemost meteoro-logical conditions anywhere in the country
   (c) the flood with return period of 1000 years
   (d) the same as the probable maximum flood

4. Consider the following statements:
   1. A 100 year flood sicharge is greater than a 50 year flood discharge.
   2. 90% dependable flow is greater than 50% dependable flow.
   3. Evaporation from salt-water surface is less than that from fresh-water surface.

   Which of these statements are correct?
   (a) 1 and 2 (b) 2 and 3
   (c) 1 and 3 (d) 1, 2 and 3

5. In a linear reservoir, the
   (a) volume varies linearly with elevation
   (b) outflow rate varies linearly with storage
   (c) storage varies linearly with time
   (d) storage varies linearly with inflow

6. A cuvert is designed for a peak flow Q_p on the basis of rational formula. If a storm of the same intensity as used in the design and twice the duration occurs, then the resulting peak discharge will be

(a) Q_p (b) $Q_p/2$

(c) $\sqrt{2}\,Q_p$ (d) $2Q_p$

7. The probability that a 100 year flood is equalled or exceeded, at least once in 100 years is

(a) 99% (b) 64%

(c) 36% (d) 1%

8. An effective storage of a flood control reservoir is

(a) the amount of water which can be supplied from it in a particular interval of time

(b) the storage betwen the minimum and maximum reservoir levels under ordinary operating conditions

(c) the useful storage plus the surcharge storage less the valley storage

(d) the storage volume of flood water above maximum reservoir level

9. **Assertion (A):** In routing a flood hydrograph through a reservoir, the peak of the outflow hydrograph will be smaller than the inflow hydrograph and it occurs after the peak of the inflow.

Reason (R): In linear reservoir routing, the storage is a function of both outflow and inflow discharges.

10. Consider the following statements:

1. Time-area histogram method aims at developing on IUH.
2. Isochrone in a line joining equal rainfall on a map.
3. Linear reservoir is a reservoir having straight boundaries.
4. Linear channel is a fictitious channel in which an inflow hydrograph passes through with only translation and no through with only translation and no attenuation.

Which of these statements are correct?

(a) 1, 2 and 3 (b) 1 and 4

(c) 2, 3 and 4 (d) 1, 2, 3 and 4

11. Match **List-I** (Floods) with **List-II** (Parameters) and select the correct answer using the codes given below the lists:

List-I

A. Standard Project Flood (SPF)

B. Maximum Probable Flood (MPF)

C. Design Flood

D. Maximum Flood

List-II

1. Includes catastrophic floods
2. Includes floods of severe conditions
3. Peak flow obtained from observed data
4. Flood of desired recurrence interval

Codes:

	A	B	C	D
(a)	2	1	4	3
(b)	1	2	3	4
(c)	2	1	3	4
(d)	1	2	4	3

12. A bridge has an expected life of 50 years and is designed for a flood magnitude of return period 100 years. What is the risk associated with this hydrologic design?

(a) $1-(0.99)^{50}$
(b) $(0.5)^{50}$
(c) $(0.99)^{50}$
(d) $(0.99)^{100}$

13. Which one of the following statements is correct in respect of the two important aspects offlood forecast – (1) reliability of the forecast, and (2) the time available in between the forecast and the occurrence of flood?
(a) Meteorological forecast is least reliable and time available is also the least.
(b) Hydrological forecast is most reliable but the time available is the least
(c) River forecast is least reliable and the time available is the maximum
(d) River forecast is most reliable but the time available is the least

14. Which one of the following flood routing methods involve the concepts of wedge and prism storages?
(a) Coefficient method
(b) Muskingum method
(c) Pul's method
(d) Lag method

15. The time of concentration at the outlet in an urban area catchment of 1.5 km^2 area with a run off coefficient of 0.42 is 28 minutes. The maximum depth of rainfall with a 50 year return period for this time of concentration is 48 mm. What is the peak flow rate at the outlet for this return period?
(a) 12 m^3/s
(b) 14 m^3/s
(c) 16 m^3/s
(d) 18 m^3/s

16. The basic equation of flood routing through a reservoir can be modified for discrete successive intervals Δt by which one of the following?

(a) $\left[\frac{I_1+I_2}{2}\right]\Delta t+\left[S_1+\frac{Q_1\Delta t}{2}\right]=\left[S_2+\frac{Q_2\Delta t}{2}\right]$

(b) $\left[\frac{I_1+I_2}{2}\right]\Delta t+\left[S_1-\frac{Q_1\Delta t}{2}\right]=\left[S_2+\frac{Q_2\Delta t}{2}\right]$

(c) $\left[\frac{I_1+I_2}{2}\right]\Delta t+\left[S_1+\frac{Q_1\Delta t}{2}\right]=\left[S_2-\frac{Q_2\Delta t}{2}\right]$

(d) $\left[\frac{I_1+I_2}{2}\right]\Delta t+\left[S_1-\frac{Q_1\Delta t}{2}\right]=\left[S_2-\frac{Q_2\Delta t}{2}\right]$

17. Kirpich equation is used to determine which one of the following?
(a) Run-off from a given rainfall
(b) Base time of a unit hydrograph
(c) Time of concentration in run-off hydrograph
(d) None of the above

18. A catchment area of 60 ha has a runoff coefficient of 0.40. If a storm of intensity 3 cm/h and duration longer than the time of concentration occurs in the catchment, then what is the peak discharge?
(a) 2.0 m^3/s
(b) 3.5 m^3/s
(c) 4.5 m^3/s
(d) 2.5 m^2/s

19. A catchment of area 200 ha has a runoff coefficient 0.5. A storm of duration larger than the time of concentration of the catchment and of intensity 3.6 cm/h causes a peak discharge of
(a) 5 m^3/s
(b) 10 m^3/s
(c) 100 m^3/s
(d) 360 m^3/s

20. A culvert is designed for a flood magnitude of return period 100 years and has expected life of 20 years. The risk in this hydrologic design is

(a) $1 - 0.99^{20}$ (b) $1 - 0.01^{20}$

(c) $1 - 0.09^{20}$ (d) $1 - 0.10^{20}$

ANSWERS

1. **(c)**	**7.** **(b)**	**13.** **(d)**	**19.** **(b)**
2. **(b)**	**8.** **(c)**	**14.** **(b)**	**20.** **(a)**
3. **(b)**	**9.** **(c)**	**15.** **(d)**	
4. **(c)**	**10.** **(b)**	**16.** **(b)**	
5. **(b)**	**11.** **(a)**	**17.** **(c)**	
6. **(a)**	**12.** **(a)**	**18.** **(a)**	

CHAPTER

10

Ground Water

Distribution of Sub-Surface Water

Sub-surface water (i.e., all forms of groundwater) can broadly be classified as:

(i) the portion in *unsaturated zone*, (Aeration zone)

(ii) the portion in *saturated zone* (ground water zone)

The unsaturated zone comprises of three sub surface zones – **soil water zone**, **intermediate zone**, **capillary zone**.

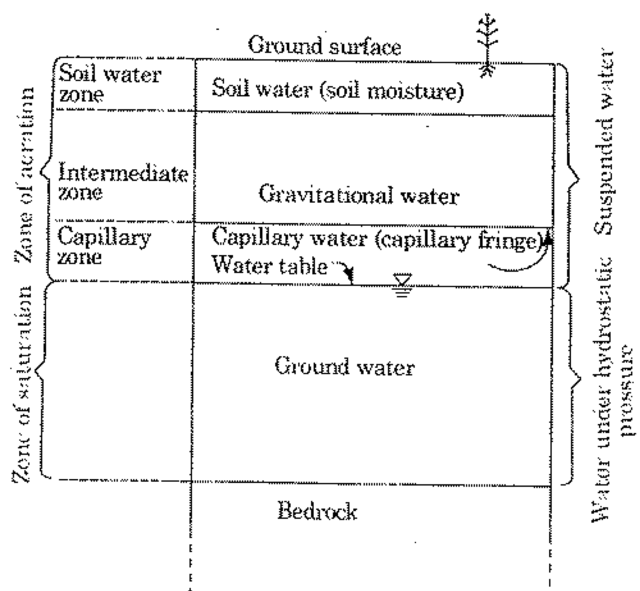


Fig. 10.1. Distribution of sub-surface water.

Soil Water Zone

Soil water zone encompasses the zone from the ground surface down to the roots from where water is drawn by vegetation; it is also called soil moisture belt and its thickness depends upon the type of vegetation that is being fed. This zone remains unsaturated except during periods of heavy infiltration. In this region, soil water is classified in three main classes. They are:

Hygroscopic water: Water is held tightly to the surface of soil particles by adsorption forces in the form of a thinnest film with soil water tension about 31 atmosphere and above.

Capillary water: Water held by surface tension in the capillary space in the form of a thickest film with tension about 1/3 atmosphere.

Gravitational water: Water that moves freely in response to gravity through macro-pores and drains out of soil.

Intermediate Zone

Intermediate zone extends from the bottom of the soil water zone down to the top of the capillary fringe.

- It greatly varies in thickness from no thickness to several hundred metres.
- All the infiltration water must pass through this region.

Capillary Zone

- Capillary zone is the zone of soil commencing from the water table to the top of the capillary-rise zone.
- It is the zone which is fully saturated at the equilibrium stage; however, the pressure in this zone is less than atmospheric because of the capillary potential within the capillary fringe. For this reason this zone is taken as a part of the unsaturated zone.
- Capillary rise depends on the size of pores (which is a function of soil particle size) and further on rise and fall in water table.
- The thickness of this zone is a function of the texture of soil; therefore, it varies from region to region as well as from place to place within a given area.

Saturated Zone

In the saturated zone, groundwater fills the pore spaces completely, and water is stored as in a reservoir, having a hydrostatic pressure variation throughout its depth with atmospheric pressure assumed to exist at the water table.

All earth materials, from soils to rocks have pore spaces. Although these pores are completely saturated with water below the water table, from the groundwater utilization point of view, only such material through which water moves easily and hence can be extracted with ease are significant. On this basis, the saturated formation are classified into four categories:

1. Aquifer,
2. Aquitard
3. Aquiclude, and
4. Aquifuge.

Aquifer

An aquifer is a saturated formation of earth material which not only stores water but yields it in sufficient quantity relatively easily due to its high permeability. Deposits of sand and gravel form good aquifers.

Aquitard

It is a formation through which only seepage is possible and thus the yield is insignificant compared to an aquifer. A sandy clay unit is an example of aquitard.

Aquiclude

Formations like clay which is highly porous but not permeable due to very small size of pores.

Aquifuge

It is a geological formation which is neither porous nor permeable. Massive compact rock without any fractures is an aquifuge.

Types of Aquifers

Unconfined Aquifers

- An unconfined aquifers is one which signifies the absence of any geological layer confining the zone of saturation (above the water table). The unconfined aquifer is in direct contact with atmospheric through the zone of aeration. The hydraulic pressure head at any point within the unconfined aquifer is equal to depth of the point from the water table.

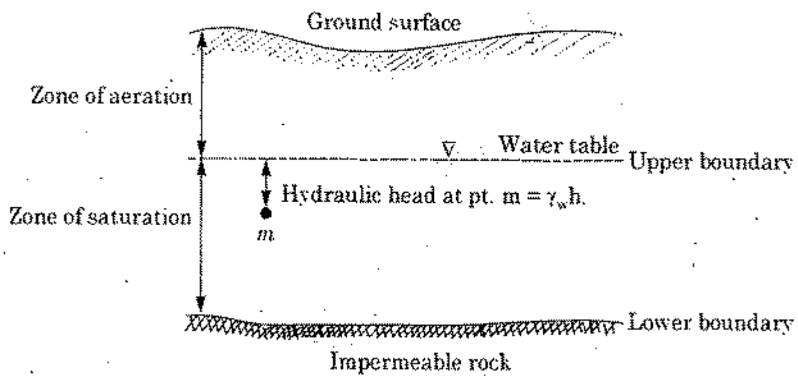


Fig. 10.2. Unconfined aquifer.

- In unconfined aquifer the water table goes down if water is withdrawn from the aquifer storage and the water table moves up if water is added into the aquifer storage.
- The water level in a large diameter dug wells tapping unconfined aquifer represents water table. This aquifer is also known as water table aquifer or phreatic aquifer.
- A special case of unconfined aquifer is known as *perched aquifer*. A perched aquifer is formed when the infiltrated rain water is intercepted within the zone of Aeration by an impermeable layer and a local zone of saturation is formed. The upper surface of such local zone of saturation is known as *perched water table*. The perched aquifer occurs at higher elevation than the regional water table.

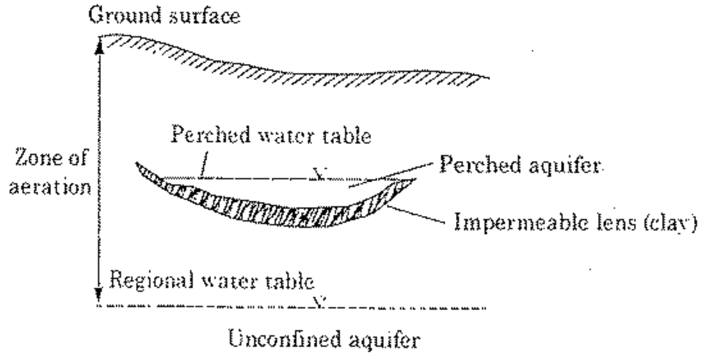


Fig. 10.3. Perched aquifer.

Confined Aquifers

A confined aquifer (also called artisan aquifer) is the one which is overlained by a impermeable layer or an Aquiclude. Unlike the unconfined aquifer, the water in the confined aquifer is not in direct contact with the atmosphere.

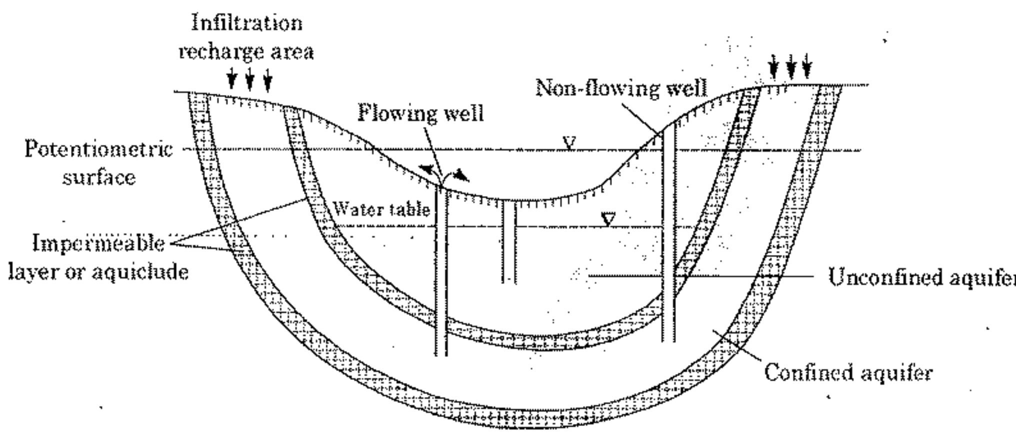


Fig. 10.4. Idealised synclinal case of confined aquifer.

The ground water within a confined aquifer occurs under pressure (known as confined pressure or artisan pressure) greater than atmospheric pressure. When such confined aquifer is pierced by a well, the water rises in the well due to release of pressure within the confined aquifer. The level upto which water will rise in the well is known as *potentiometric level*. This potentiometric level indicates the magnitude of pressure within the confined aquifer. If the potentiometric level is above the ground surface a *flowing well* results.

The area from which the infiltrated water enters the confined aquifer is known as *Recharge area.*

Aquifer Properties

An aquifer performs two functions:

1. storage of water, and
2. transmission of stored water.

The porosity and the hydraulic conductivity (permeability) explain the storage and transport of water.

Porosity

The amount of pore space per unit volume of the aquifer material is called *porosity*. It is expressed as

$$n = \frac{V_v}{V_0}$$

where n = porosity, V_v = volume of voids and V_0 = volume of the porous medium.

Specific Yield

- While porosity gives a measure of the water-storage capability of a formation, not all the water held in the pores is available for extraction by pumping or draining by gravity. The pores hold back some water by molecular attraction and surface tension.

- The actual volume of water that can be extracted by the force of gravity from a unit volume of aquifer material is known as the *specific yield*, S_y. The fraction of water held back in the aquifer is known as *specific retention* also called field capacity S_r. Thus porosity

$$n = S_y + S_r$$

Porosity and specific yield of selected formations

Formation	*Porosity* (%)	*Specific yield* (%)
Clay	45-55	1-10
Sand	35-40	10-30
Gravel	30-40	15-30

Note that although clay and sand have high porosity, the specific yield of clay is very small compared to that of sand.

Storage Coefficient (or Storativity) (S)

- In case of confined aquifer, volume of water given by unit plan area of aquifer when piezometric surface falls by unity is called storage coefficient.
- For unconfined aquifer storage coefficient is assumed to be equal to specific yield.

  Storage coefficient of an artesian aquifer is given by the relation

$$S = \gamma_w nb\left(\frac{1}{K_w} + \frac{1}{nE_s}\right)$$

where S = storage coefficient

γ_w = specific weight of water

n = porosity of soil

b = thickness of the confined aquifer

K_w = bulk modulus of elasticity of water

E_s = modulus of compressibility (elasticity) of the soil grains of the aquifer.

Since water is practically incompressible, expansibility of water as it comes out of the pores has a very little contribution to the value of the storage coefficient.

Since water is under pressures in confined aquifer, draining of water leads to decrease in pore pressure and hence increase in effective stress. This increase in effective stress leads to compression of soil Skelton. Thus specific storage is solely due to compression of aquifer and expansion of water.

- Storage coefficient per unit depth of confined aquifer is called specific storage (S_s)

$$S = S_s . b$$

b = depth of confined aquifer.

Thus sp. storage is defined as volume of water released from storage from a unit volume of aquifer due to unit decrease in pierometric head.

Note: When we talk of well we define a new quantity called specific capacity. Sp. capacity is the discharge from well per unit drawdown of well.

Darcy's Law

$$V = K\,i \quad \text{(Darcy's law)}$$

where V = Apparent velocity of seepage = Q/A in which Q = discharge and A = cross-sectional area of the porous medium. V is sometimes also known as discharge velocity. $i = -\frac{dh}{ds}$ = hydraulic gradient, in which h = piezometric head and s = distance measured in the general flow direction; the negative sign emphasizes that the piezometric head drops in the direction of flow. K = a coefficient, called *coefficient of permeability* (hydraulic conductivity) having the units of velocity.

The discharge Q can be expressed as

$$Q = K\,i\,A$$

Darcy's law is a particular case of the general viscous fluid flow. It has been shown valid for laminar flows only.

Darcy law is valid for Re < 1

$$\mathbf{Re} = \frac{V\,d_a}{\nu}$$

where **Re** = Renolds number

d_a = representative particle size, usually $d_a = d_{10}$ where d_{10} represents a size such that 10% of the aquifer material is of smaller size.

ν = kinematic viscosity of water.

Apparent velocity V used in Darcy's law is not the actual velocity of flow through the pores. Actual speed of travel of water in the porous media is expressed as

$$v_a = \frac{V}{n}$$

where n = porosity. The actual velocity v_a is the velocity that is obtained by tracking a tracer added to the groundwater.

Coefficient of Permeability

The coefficient of permeability, also designated as *hydraulic conductivity* relfects the combined effects of the porous medium and fluid properties. The coefficient of permeability K can be expressed as

$$K = C d_m^2 \frac{\gamma}{\mu}$$

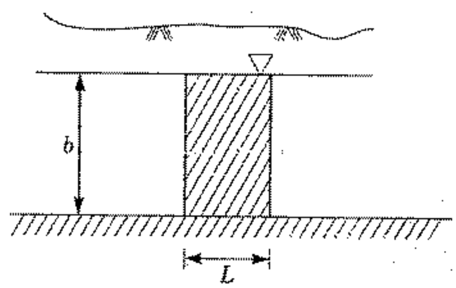
where d_m = mean particle size of the porous medium, $\gamma = \rho g$ = unit weight of fluid, ρ = density of the fluid, g = acceleration due to gravity, μ = dynamic viscosity of the fluid and C = a shape factor which depends on the porosity, packing, shape of grains and grain-size distribution of the porous medium.

The coefficient of permeability is often considered in two components, one reflecting the properties of the medium only and the other incorporating the fluid properties.

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

where $K_0 = C\,d_m^2$. The parameter K_0 is called *specific* or *intrinsic permeability* which is a function of the medium only. K_0 has dimensions of $[L^2]$. It is expressed in units darcys where, 1 Darcy = 9.87×10^{-13} m^2.

Coefficient of transmissibility (T)

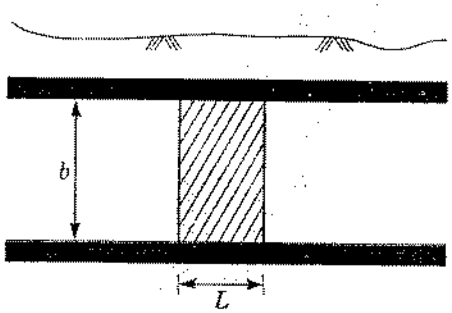


Unconfined aquifer

Fig. 10.5

$$\text{Area of flow} = bL$$
$$Q = KiA$$
$$= KibL$$
$$= (Kb)\,iL$$
$$Q = TiL$$

T is called coefficient of transmissibility.



Confined aquifer

Fig. 10.6

$$Q = TiL, \quad T = Kb$$

T is called coefficient of transsibility.

We know that coefficient of permeability is defined as the discharge through unit area under unit hydraulic gradient. However in reality ground water travels through entire thickness of aquifer (b).

Hence coefficient of transmissibility is defined to find out the discharge. [co-efficient of transmissibility is thus defined as, discharge per unit length of aquifer].

T has a unit of $(\text{length})^2/\text{time}$.

Storage coefficient (S) and Transmissibility coefficient (T) are known a formation constants of an aquifer and plays an important role in the unsteady flow through porous media.

Storage coefficient and transmissibility or transmissivity is determined in the field by carrying out pumping test on wells and measuring the discharge and lowering of water levels in the observation wells.

Stratification

Sometimes the aquifers may be stratified, with different permeabilities in each strata. Two kinds of flow situations are possible in such a case.

(i) When the flow is parallel to the stratification, equivalent permeability K_e of the entire aquifer of thickness $D = \sum_1^n D_i$ is

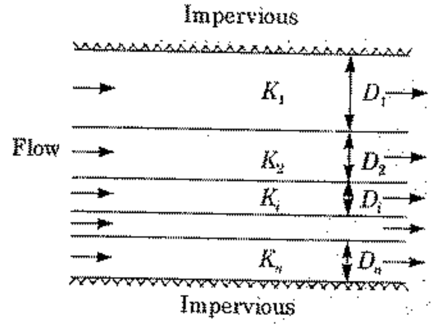


Fig. 10.7(a). Flow parallel to stratification.

$$\boxed{K_e = \frac{\sum_1^n K_i D_i}{\sum_1^n D_i}}$$

The transmissivity of the formation is

$$\boxed{T = K_e \Sigma D_i = \sum_1^n K_i D_i}$$

(ii) When the flow is normal to the stratification the equivalent permeability K_e of the aquifer of length $L = \sum_1^n L_i$ is

$$K_e = \frac{\sum_1^n L_i}{\sum_1^n (L_i/K_i)}$$

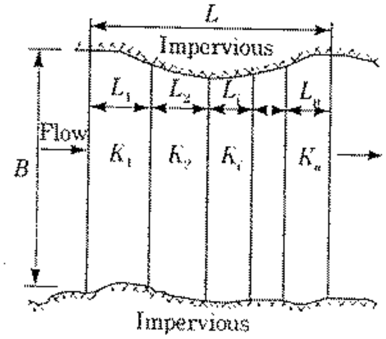


Fig. 10.7(b). Flow normal to stratification.

The transmissivity of the aquifer is $T = K_e.B.$

Example 10.1

When 3.68 million m^3 of water was pumped out from an unconfined aquifer of 6.2 km^2 areal extent, the water table was observed to go down by 2.6 m. What is the specific yield of the aquifer? During a monsoon season if the water table of the same aquifer goes up by 10.8 m what is the volume of recharge?

Sol: Water released from the aquifer $= (6.2 \times 10^6) \times 2.6 \times s_y$

$$\text{Water pumped out} = 3.68 \times 10^6 \text{ m}^3$$

$$\text{Equating these two quantities } s_y = \frac{3.68}{6.2 \times 2.6} = 2.2283$$

$$\therefore \text{ Specific yield of the aquifer} = 0.2283 \text{ or } 22.83\%.$$

$$\text{Volume of recharge} = 6.2 \times 10^6 \times 10.8 \times s_y$$

$$= 6.2 \times 10^6 \times 10.8 \times 0.2283$$

$$= 15.287 \text{ million m}^3.$$

Example 10.2

In a certain alluvial basin of 100 km^2, 90 mm^3 of ground water was pumped in a year and the ground water table dropped by about 5 m during the year. Assuming no replenishment, estimate the specific yield of the aquifer. If the specific retention is 12%, what is the porosity of the soil?

Sol: (i) Change in ground water storage

$$\text{Change in ground water storage} = 90 \times 10^6$$

$$90 \times 10^6 = (100 \times 10^6) \times 5 \times S_y$$

$$\therefore S_y = \mathbf{0.18}$$

(ii) Porosity $n = S_y + S_r = 0.18 + 0.12 = 0.30$ or **30%**

Example 10.3

An artesian aquifer, 30 m thick has a porosity of 25% and bulk modulus of compression 2000 kg/cm^2. Estimate the storage coefficient of the aquifer. What fraction of this is attributable to the expansibility of water? Take Bulk modulus of elasticity of water = 2.4×10^4 kg/cm^2.

Sol: $S = \gamma_w nb\left(\frac{1}{K_w} + \frac{1}{nK_s}\right) = 1000 \times 0.25 \times 30\left(\frac{1}{2.14 \times 10^8} + \frac{1}{0.25 \times 2 \times 10^7}\right)$

$$= 7500\ (0.467 \times 10^{-8} + 20 \times 10^{-8}) = 1.54 \times 10^{-3}$$

Storage coefficient due to the expansibility of water as a percentage of S above

$$= \frac{7500 \times 0.467 \times 10^{-8}}{7500 \times 20.467 \times 10^{-8}} \times 100 = 2.28\%, \text{ which is negligible.}$$

Equation of Motion

1. **One dimensional confined ground water flow between two water bodies (confined flow).**

Following assumptions are made in this analysis:

(a) Aquifer is homogeneous and isotropic.

(b) Flow is steady.

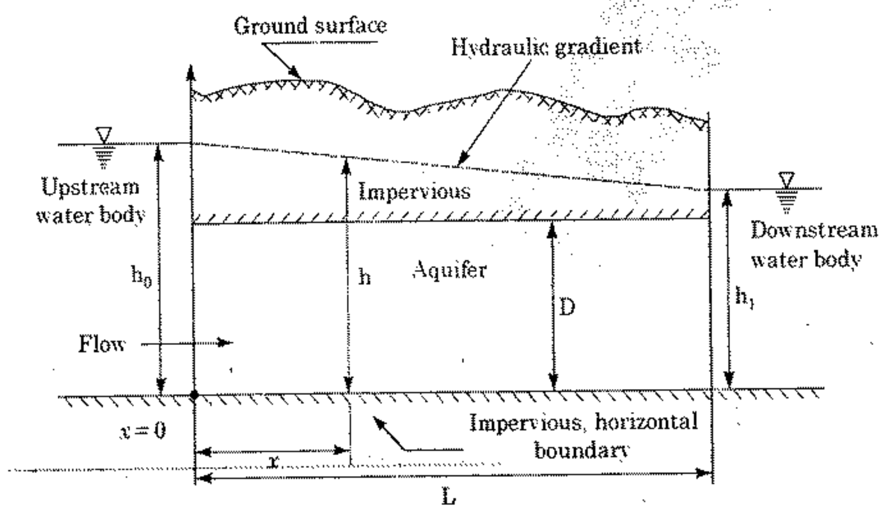


Fig. 10.8. Confined groundwater flow between two water bodies.

$$q = K\left(-\frac{dh}{dx}\right)D \times 1 = \text{Discharge per unit width}$$

$$\Rightarrow \int_0^L q\,dx = -kD \int_{h_0}^{h_1} dh$$

$$\Rightarrow qL = -K(h_1 - h_0)D$$

$$\Rightarrow \boxed{q = \frac{KD(h_0 - h_1)}{L}} \qquad (\alpha)$$

Shape of water surface profile

$$q = K\left(-\frac{dh}{dx}\right)D \times 1$$

$$\int q\,dx = -KD\int dh$$

$$\Rightarrow qx = -KDh + C$$

at $x = 0$, $h = h_0$

$$\Rightarrow C = KDh_0$$

$$\Rightarrow qx = -KD(h - h_0) \qquad (\beta)$$

at $x = L$, $h = h_1$ $\Rightarrow$ $qL = -KD(h_1 - h_0)$ (γ)

From (β) and (γ)

$$\Rightarrow \quad \frac{qx}{qL} = \frac{-KD\,(h-h_0)}{-KD(h_1-h_0)}$$

$$\Rightarrow \quad \frac{x}{L} = \frac{h-h_0}{h_1-h_0}$$

$$\Rightarrow \quad \boxed{h = h_0 - \frac{x}{L}(h_1-h_0)} \qquad (\delta)$$

In case of confined flow between two reservoir, hydraulic grade line varies linearly from h_0 to h_1.

Unconfined ground water flow between two water bodies (Unconfined flow)

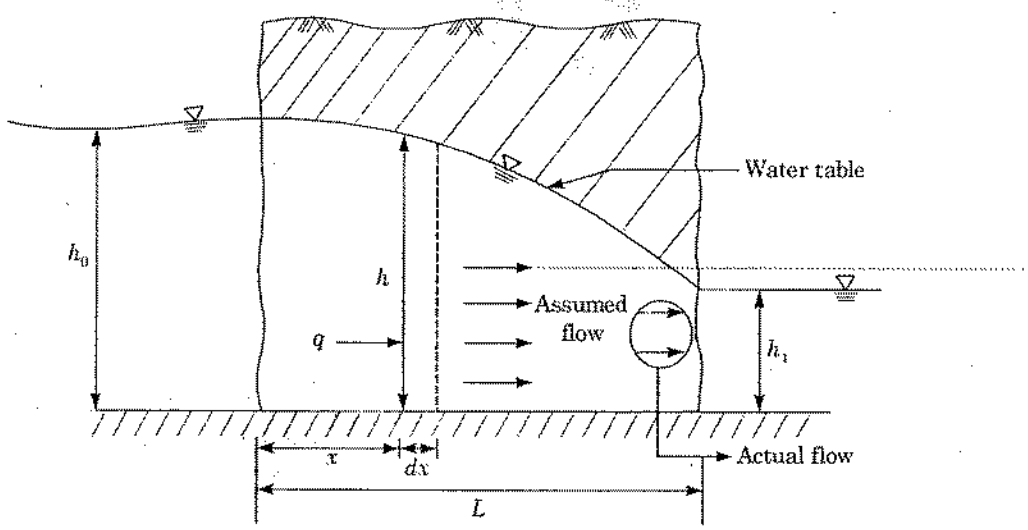


Fig. 10.9

Discharge per unit width inside = q

$$q = +K\left(-\frac{dh}{dx}\right)\times h\times 1$$

$$\int_0^L \frac{q\,dx}{K} = -\int_{h_0}^{h_1} h\,dh$$

$$\frac{qL}{K} = -\left(\frac{h_1^2-h_0^2}{2}\right)$$

$$\boxed{q = \frac{K(h_0^2-h_1^2)}{2L}}$$

For shape of water surface profile

$$\int \frac{q\,dx}{K} = -\int h\,dh$$

$$\Rightarrow \quad \frac{qx}{K} = -\frac{h^2}{2} + C$$

at $x = 0, \quad h = h_0$

at $x = L, \quad h = h_1$

$$\Rightarrow \quad \frac{qx}{K} = \frac{h_0^2 - h^2}{2} \text{ and}$$

$$\frac{qL}{K} = \frac{h_0^2 - h_1^2}{2}$$

$$\Rightarrow \quad \frac{x}{L} = \frac{h_0^2 - h^2}{h_0^2 - h_1^2}$$

$$\Rightarrow \quad \boxed{h^2 = h_0^2 - (h_0^2 - h_1^2)\left(\frac{x}{L}\right)}$$

⇒ **Water surface is a parabola in case of unconfined aquifer.**

One dimensional flow with Recharge

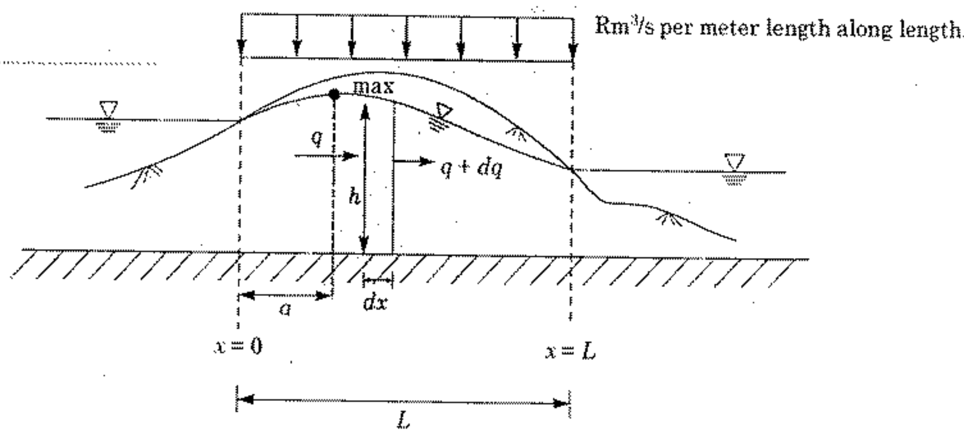
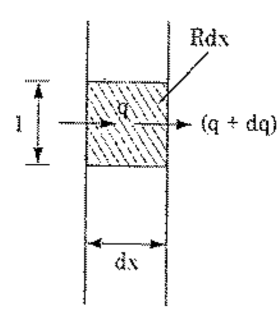


Fig. 10.10

where R = recharge at constant rate of R m³/s per unit horizontal area due to infilteration from the top of aquifer [**Note:** R for one-dimensions can be taken as R m³/s per unit length of flow, assuming unit width inside].



$$\Rightarrow \quad q + Rdx = q + dq$$

$$\Rightarrow \quad \frac{dq}{dx} = R$$

$$q = -K.h.\frac{dh}{dx} = -\frac{K}{2}\frac{d(h^2)}{dx}$$

$$\frac{dq}{dx} = -\frac{K}{2}\frac{d}{dx}\left(\frac{d(h^2)}{dx}\right) = -\frac{K}{2}\cdot\frac{d^2h^2}{dx^2}$$

$$\Rightarrow \quad R = -\frac{K}{2}\cdot\frac{d^2h^2}{dx^2}$$

$$\Rightarrow \quad \boxed{\frac{d^2(h^2)}{dx^2} = -\frac{2R}{K}}$$

by integrating twice

$$h^2 = -\frac{2R}{K}\cdot\frac{x^2}{2} + C_1x + C_2$$

The boundary condition are

$$\text{at } x = 0,\ h = h_0 \Rightarrow C_2 = h_0^2$$

$$\text{at } x = L,\ h = h_1 \Rightarrow C_1 = -\frac{\left[h_0^2 - h_1^2 - \frac{2L^2}{K}\right]}{L}$$

$$\Rightarrow \quad \boxed{h^2 = -\frac{Rx^2}{K} - \frac{\left(h_0^2 - h_1^2 - \frac{RL^2}{K}\right)}{L}x + h_0^2} \qquad (\alpha)$$

Thus water surface is in the form of ellipse.

The water table will rise above h_0, will reach a max elevation at a distance '**a**' and then falls back to h_1 at $x = L$.

The location of max water surface elevation (a) is given by $\frac{dh}{dx} = 0$

$$\Rightarrow \quad \boxed{a = \frac{L}{2} - \frac{k}{R}\left(\frac{h_0^2 - h_1^2}{2L}\right)}$$

The location $x = a$ is called *ground water divide*.

Flow to the left of devide will be towards left side water body and to the right of divide will be toward downstream water body. Equation for discharge at a distance 'x' from U/S water body is given by

$$q_x = -K.h.\frac{dh}{dx}$$

$$\Rightarrow \quad \boxed{q_x = R\left(x - \frac{L}{2}\right) + \frac{K}{2L}\left(h_0^2 - h_1^2\right)}$$

$$\boxed{q_0 = -\frac{RL}{2} + \frac{K}{2L}\left(h_0^2 - h_1^2\right)}$$

$$\boxed{q_L = -\frac{RL}{2} + \frac{K}{2L}\left(h_0^2 - h_1^2\right)}$$

$$\boxed{q_L = q_0 + RL}$$

Tile Drain Problem

The provision of Tile drain system is used to drain waterlogged areas, the objective being to reduce the level of the water table. Let us provide a set of tile drain as shown in the figure below and let there is a uniform recharge 'R'.

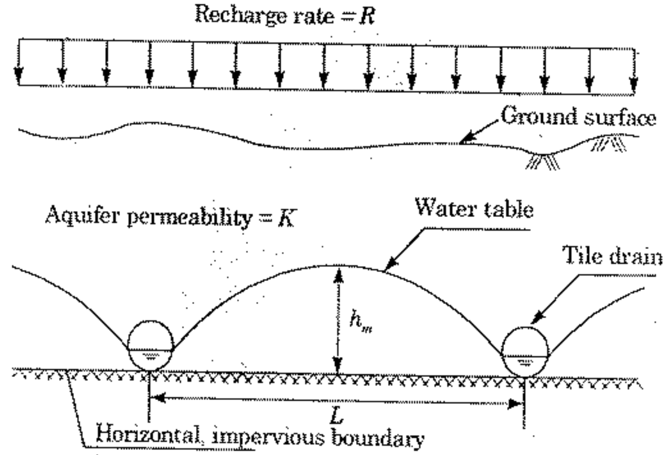


Fig. 10.11. Tile drains under a constant recharge rate.

An approximate expression to the water table profile ~~can be obtained~~ from the formula 'α' as discussed earlier by neglecting the depth of water in the drains, i.e. $h_0 = h_1 = 0$. The water table profile will then be

$$h^2 = \frac{R}{K}(L - x)x$$

The maximum height of the water table occurs at $a = L/2$ and is of magnitude

$$h_m = \frac{L}{2}\sqrt{R/K}$$

Considering a set of drains, since the flow is steady, the discharge entering a drain per unit length of the drain is

$$q = 2\left(R\frac{L}{2}\right) = RL$$

Wells

- Wells form the most important mode of groundwater extraction from an aquifer. While wells are used in a number of different applications, they find extensive use in water supply and irrigation engineering practice.
- For an unconfined aquifer, prior to the pumping, the water level in the well indicates the static water table. A lowering of this water level takes place on pumping. If the aquifer is homogeneous and isotropic and the water table horizontal initially, due to the radial flow into the well through the aquifer, the water table assumes a conical shape called *cone of depression*.
- The drop in the water table elevation at any point from its previous static level is called *drawndown*.
- The areal extent of the cone of depression is called *area of influence* and its radial extent *radius of influence*. At constant rate of pumping, the drawndown curve develops gradually with time due to the withdrawal of water from storage. This phase is called unsteady flow as the water table elevation at a given location near the well changes with time.
- On prolonged pumping, an equilibrium state is reached between the rate of pumping and the rate of inflow of groundwater from the outer edges of the zone of influence. The drawdown surface attains a constant position with respect to time when the well is known to operate under steady-flow conditions.
- As soon as the pumping is stopped, the depleted storage in the cone of depression is made good by groundwater inflow into the zone of influence. There is a gradual accumulation of storage till the original (static) level is reached. This stage is called *recuperation* or *recovery* and is an unsteady phenomenon.
- Changes similar to the above take place due to pumping of a well in a confined aquifer, but with the difference that, it is the piezometric surface instead of the water table that undergoes drawndown with the development of the cone of depression.
- In confined aquifers with considerable pizometric head, the recovery into the well takes place at a very rapid rate.

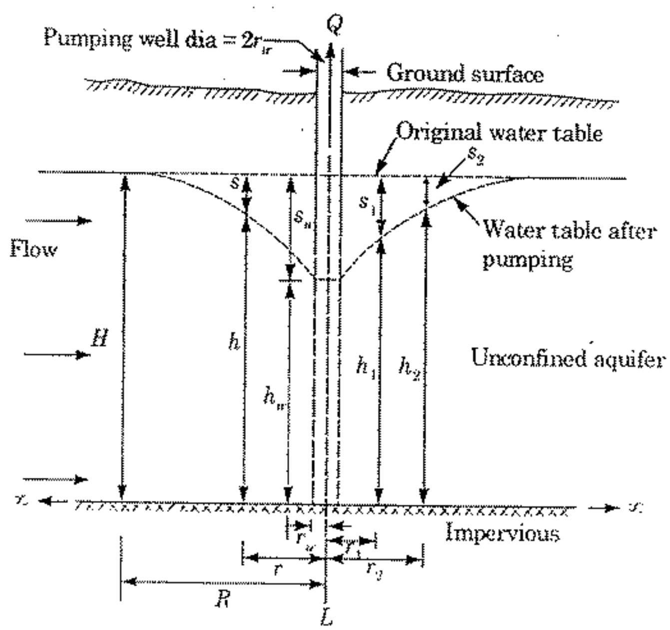


Fig. 10.12. Well operating in a unconfined aquifer.

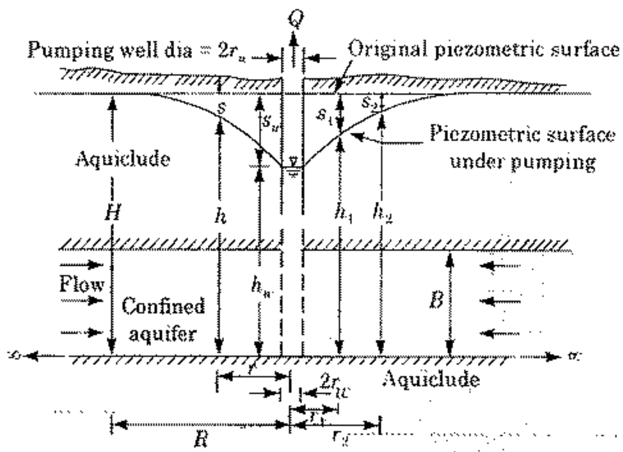


Fig. 10.13. Well operating in an confined aquifer.

Steady Confined Flow (Fully Penetrating Well)

Full penetrating well means the well which penetrates upto the bottom of the aquifer so that flow is more or less radial.

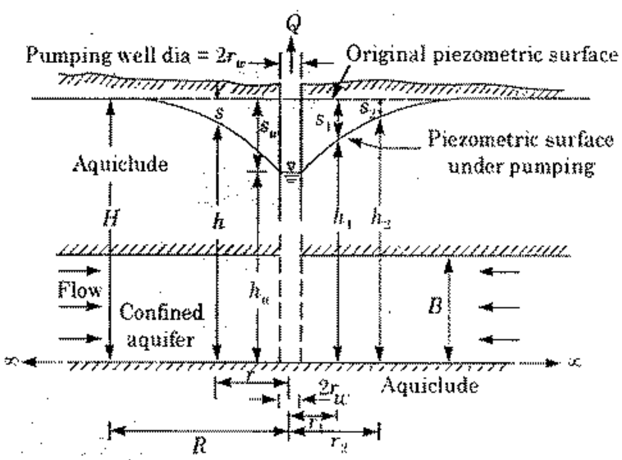


Fig. 10.14. Well operating in an confined aquifer.

At a radial distance r from the well, if h is the piezometric head, the velocity of flow by Darcy's law is

$$V_r = K\frac{dh}{dr}$$

The cylindrical surface through which this velocity occurs is $2\pi rB$. Hence

$$Q = (2\pi rB)\left(K\frac{dh}{dr}\right)$$

$$\frac{Q}{2\pi KB}\frac{dr}{r} = dh$$

Integrating between limits r_1 and r_2 with the corresponding piezometric heads being h_1 and h_2 respectively.

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

$$\boxed{Q = \frac{2\pi KB\,(h_2 - h_1)}{\ln \frac{r_2}{r_1}}}$$

This is the equilibrium equation for the steady flow in a confined aquifer. **This equation is popularly known as *Thiem's equation*.**

Further, at the edge of the zone of influence, $s = 0$, $r_2 = R$ and $h_2 = H$, at the well wall $r_1 = r_w$, $h_1 = h_w$ and $s_1 = s_w$. Hence

$$\boxed{Q = \frac{2\pi KBS_w}{\ln \frac{R}{r_w}}}$$

This is called dupits formula.

Steady Unconfined Flow

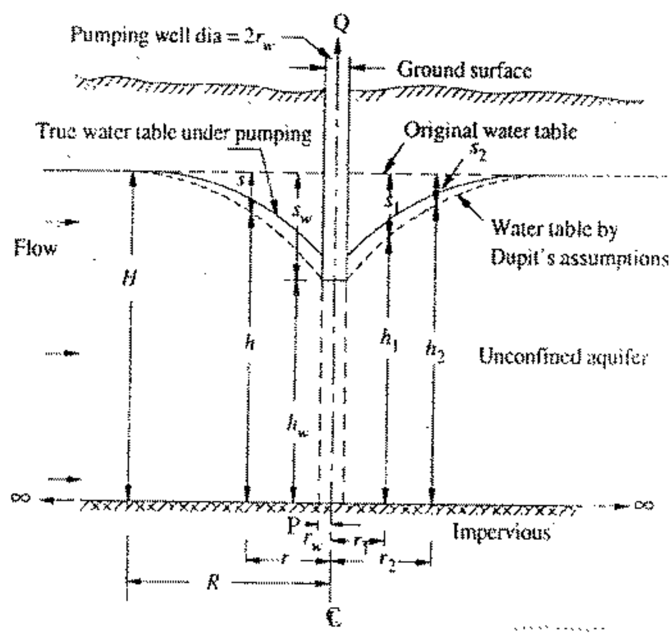


Fig. 10.15. Radial flow to a well in an unconfined aquifer.

$$V_r = K\frac{dh}{dr}$$

$$Q = (2\pi rh)V_r = 2\pi rKh\ \frac{dh}{dr}$$

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

Integrating between limits r_1 and r_2 where the water-table depths are h_1 and h_2 respectively and on rearranging

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

This is the equilibrium equation for a well in an unconfined aquifer (Thiems formula). As at the edge of the zone of influence of radius R, H = saturated thickness of the aquifer. Hence

$$Q = \frac{\pi K \left(H^2 - h_w^2\right)}{\ln \frac{R}{r_w}}$$

R is normally between 300-500 m.

where h_w = depth of water in the pumping well of radius r_w.

Non-Equilibrium Formula for Confined Aquifers (Unsteady Radial Flows)

The main drawback of the equilibrium formulas given by Thiem and Dupit, was the problem to attain equilibrium conditions, which is not an easy job to do. The pumping has to be continued at a uniform rate for a very long time, so as to achieve steady flow conditions.

Hence we adopt non-equilibrium formula. As per this

$$\boxed{s = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt}{r^2 S} - 0.5772\right]} = \frac{Q}{4\pi T} \times \textbf{[Well function]}$$

where s = Drawdown in the observation well after a time t.

T = Coefficient of transmissibility.

Q = Constant discharge pumped out from the well.

S = Coefficient of storage of measured drawdown.

r = Radial distance of the observation well from the main pumped well.

If in an observation well at a distance r, the drawdowns are respectively s_1 and s_2 at time t_1 and t_2 after the pumping was started in the main well, then

$$S_2 - S_1 = \frac{Q}{4\pi T} \log_e \frac{t_2}{t_1}$$

Example 10.4

In an artesian aquifer, the drawdown is 1.2 metres at a radial distance of 10 metres from a pumped well after two hours of pumping. On the basis of non-equilibrium equation, determine its pumping time for the same drawdown (i.e. 1.2 m) at a radial distance of 30 metres from this well.

Sol: The non-equilibrium equation is

$$s = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt}{r^2 S} - 0.5772\right]$$

In the given equation, the drawdown is the same in both the observation wells, therefore, $s_1 = s_2 = 1.2$ m.

well (1)	well (2)
$r_1 = 10$ m	$r_2 = 30$ m
$t_1 = 2$ hrs	$t_2 = ?$

Now $s_1 = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_1}{r_1^2 S} - 0.5772\right]$

$$s_2 = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_2}{r_2^2 S} - 0.5772\right]$$

But $S_1 = S_2$

$$\Rightarrow \therefore \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_1}{r_1^2 S} - 0.5772\right] = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_2}{r_2^2 S} - 0.5772\right]$$

$$\text{or} \quad \log_e \frac{4Tt_1}{r_1^2 S} = \log_e \frac{4Tt_2}{r_2^2 S}$$

$$\text{or} \quad \frac{4Tt_1}{r_1^2 S} = \frac{4Tt_2}{r_2^2 S}$$

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

Putting the respective values, we get

$$\text{or} \quad \frac{2\text{ hrs.}}{(10)^2} = \frac{t_2}{(30)^2}$$

$$t_2 = \frac{(30)^2}{(10)^2} \times 2 \text{ hrs.}$$

$$= 9 \times 2 \text{ hrs.} = 18 \text{ hrs.}$$

$\mathbf{t_2 = 18}$ **hrs. Ans.**

Interference Among Wells

When two wells, situated near to each other, are discharging, their drawdown curves intersect within their radius of zero drawdown. Thus, though the total discharge is increased, the discharge in individual well is decreased due to interference.

Figure blow shows interference between two wells. If the two wells are a distance B apart, and have the same diameter and drawdown and discharge over the same period of time, it can be shown with the help of method of complex variables, that the discharge through each well is given by

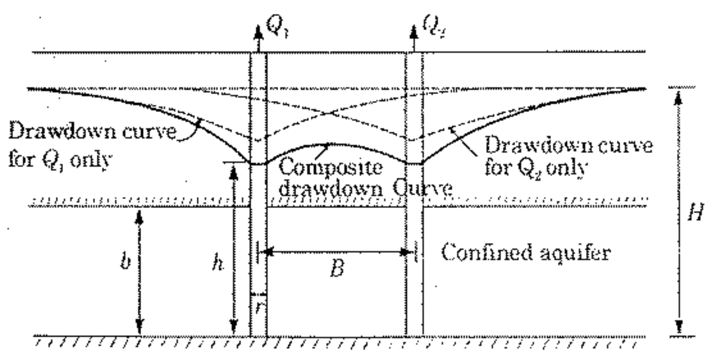


Fig. 10.16. Interference between two wells.

$$Q_1 = Q_2 - \frac{2\pi kb\,(H-h)}{\log_e \frac{R^2}{rB}}$$

where R is the radius of area of influence ($R >> B$).

Similarly, if there are three wells forming an equilateral triangle a distance B on a side, and if all the three wells have the same characteristics

$$\text{then,} \qquad Q_1 = Q_2 = Q_3 = \frac{2\pi kb\,(H-h)}{\log_e \frac{R^3}{rB^2}}$$

Well Loss

In a pumping artesian well, the total draw down at the well s_w, can be considered to be made up of three parts:

1. Head drop required to cause laminar porous media flow, called *formation loss*, w_{sL};
2. drop of piezometric head required to sustain turbulent flow in the region nearest to the well where the Reynolds number may be larger than unity, s_{wt}; and
3. head loss through the well screen and casing, s_{wc}.

Of these three,

$$S_{wL} \propto Q$$

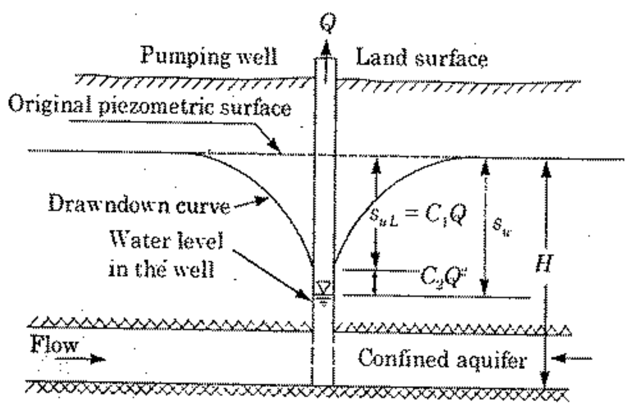


Fig. 10.17. Definition sketch for well loss.

and (s_{wt} and s_{wc}) $\propto Q^2$

$$\text{thus,} \qquad s_w = C_1Q + C_2Q^2 = \text{total loss}$$

While the first term C_1Q is the formation loss the second terms C_2Q^2 is termed *well loss*.

The magnitude of a well loss has an important bearing on the pump efficiency. Abnormally high value of well loss indicates clogging of well screens etc. and requires immediate remedial action.

Specific Capacity

The specific capacity of a well is defined as the well yield per unit of drawdown. Hence, the

$$\text{Sp. capacity} = \frac{\text{Discharge of well}}{\text{Drawdown}}$$

$$= \frac{Q}{C_1 Q + C_2 Q^2}$$

$$\therefore \quad \text{Sp. capacity} = \left[\frac{1}{C_1 + C_2 Q}\right]$$

The equation clearly shows that the sp. capacity of the well is not constant but decreases as the discharge increases.

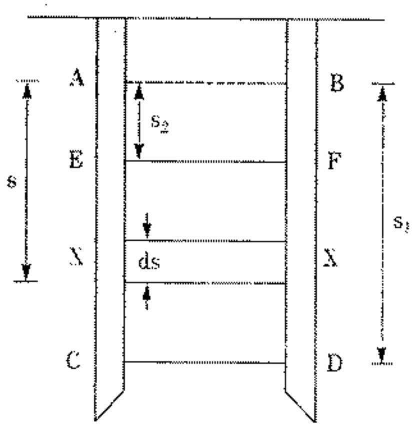
Safe Yield

The maximum rate at which the withdrawal of groundwater in a basin can be carried without producing undesirable results is termed *safe yield*. The "undesirable" results include (i) permanent lowering of the groundwater table or piezometric head, (ii) maximum drawdown exceeding a preset limit leading to inefficient operation of wells and (iii) salt-water encroachment in a coastal aquifer. Depending upon what undesirable effect is to be avoided, a safe yield for a basin can be identified.

Recuperating Test

Although the pumping test gives accurate value of safe yield, it sometimes becomes very difficult to adjust the rate of pumping, so as to keep the well water level constant. In such circumstances, recuperation test is adopted.

In this method, the water is first of all drained from the well at a fast rate so as to cause sufficient drawdown. The pumping is then stopped. The water level in the well will start rising. The time taken by the water to come back to its normal level or some other measured level is then noted. The discharge can then be worked out as below:



AB = Static water level in the well before the pumping was started.

CD = Water level in the well when the pumping was stopped.

s_1 = Depression head in the well at the time the pumping was stopped

EF = Water level in the well at the noted time (say after a time T from when the pumping is stopped).

s_2 = Depression head in the well at time T after the pumping is stopped.

Let X-X be the position of water level at any time t after the pumping was stopped, and let the corresponding depression head be s. Let ds be the decrease in depression head in a time dt after the time t. Hence, in a time

t after the pumping is stopped, the water level recuperates by $(s_1 - s)$. It again recuperates by ds in a time dt after this.

∴ Volume of water entering the well in the small interval of time (dt)

$$= dV = A.dt \quad (1)$$

where A is the cross-sectional area of the well at the bottom.

Also, if Q is the rate of recharge into the well at the time t under a depression head s, then the volume of water entering the well in this small time interval is

$$= dV = Q.dt$$

But

$$Q \propto s$$

$$\therefore \quad Q = C'.s \quad (2)$$

where C' is a constant depending on the soil through which the water enters the well.

$$\therefore \quad dV = C'.s.dt \quad (3)$$

Equating (1) and (3), we get

$$-A.ds = C'.s.dt$$

(The –ve sign indicates that s decreases as t increases)

$$\text{or} \quad \frac{C'.dt}{A} = -\left(\frac{ds}{s}\right)$$

Integrating between the limits

$t = 0, \quad s = s_1$

$t = T, \quad s = s_2$

we get,

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

$$\text{or} \quad \frac{C'}{A}|t|_0^T = -|\log_e s|_{s_1}^{s_2}$$

$$\text{or} \quad \frac{C'}{A}(T) = -\log_e \frac{s_2}{s_1}$$

$$= -2.3 \log_{10} \frac{s_2}{s_1}$$

$$= 2.3 \log_{10} \frac{s_1}{s_2}$$

$$\therefore \quad \frac{C'}{A} = \frac{2.3}{T}\log_{10}\frac{s_1}{s_2}$$

Knowing the values of s_1, s_2 and T from the above test, the value of $\frac{C'}{A}$ can be calculated. C' is called the *specific capacity* of the open well. Knwoing the value of $\frac{C'}{A}$, the discharge Q for a well under a constant depression head H can be calculated as follows:

$$Q = C'.s$$

$$\text{or} \quad Q = \left(\frac{C'}{A}\right) A.s$$

$$\text{or} \quad \boxed{Q = \left(\frac{2.3}{T}\log_{10}\frac{s_1}{s_2}\right) A.s}$$

A and s are known, the discharge for any amount of drawdown (s) can be easily worked out.

Example 10.5

During a recuperation test, the water level in an open well was depressed by pumping by 2.5 metres and is recuperated by an amount of 1.6 metres in 70 minutes.

(a) Determine the yield from a well of 3 m diameter under a depression head of 3.5 metres.

(b) Also determine the diameter of the well to yield 10 litres/second under a depression head of 2.5 metres.

Sol:

$$\frac{C'}{A} = \frac{2.3}{T}\log_{10}\frac{s_1}{s_2}$$

where

$$s_1 = \text{Initial drawdown} = 2.5 \text{ m}$$

$$s_2 = \text{Final drawdown}$$

$$= 2.5 - 1.6 = 0.9 \text{ m}$$

$$T = \text{Time} = 70 \text{ minutes}$$

$$= \frac{70}{60} \text{ hr}$$

$$= 1.167 \text{ hr}$$

$$\therefore \quad \frac{C'}{A} = \frac{2.3}{1.167}\log_{10}\frac{2.5}{0.9}$$

$$= 0.875 \text{ m}^3\text{/hr/m}^2\text{/m of depression head}$$

(a) Yield from a well of 3 m diameter, under a depression head of 3.5 m is given by

$$Q = \left(\frac{C'}{A}\right) A.s$$

$$= 0.875 \times \left(\frac{\pi}{4} \times 3^2\right) \times 3.5$$

$$= 21.65 \text{ m}^3\text{/hr}$$

$$= 6.02 \text{ litres/sec.} \quad \textbf{Ans.}$$

(b) If

$$Q = 10 \text{ litres/sec,}$$

$$= \frac{10 \times 60 \times 60}{1000} \text{ m}^3\text{/hr}$$

$$= 36 \text{ m}^3\text{/hr}$$

$$s = 2.5 \text{ m}$$

$$Q = \left(\frac{C}{A}\right) As$$

$$\therefore \quad 36 = (0.875) \times A \times 2.5$$

$$\text{or} \quad A = \frac{36}{0.875 \times 2.5} = 16.46 \text{ m}^2$$

$$\text{or} \quad \frac{\pi}{4} d^2 = 16.46$$

$$\text{or} \quad d = 4.58 \text{ m}; \quad \text{Say } 4.6 \text{ m}$$

Hence, the diameter of the required well = 4.6 m. **Ans.**

OBJECTIVE QUESTIONS

1. The yield of a well depends upon
   (a) permeability of soil
   (b) area of aquifer opening into the wells
   (c) actual flow velocity
   (d) all of the above

2. Match **List-I** with **List-II** and select the correct answer using the codes given below the lists:

   List-I
   A. Specific yield
   B. Specific capacity
   C. Specific retention
   D. Specific storage

   List-II
   1. Volume of water retained per unit volume of aquifer
   2. Volume of water drained by gravity per unit volume of aquifer
   3. Difference of porosity and specific storage
   4. Well yield per unit drawdown
   5. Volume of water released from unit volume of aquifer for unit decline in piezometric head

   Codes:

	A	B	C	D
(a)	2	4	1	5
(b)	4	2	3	5
(c)	2	5	1	4
(d)	4	2	3	1

3. Water present in an artesian aquifer is usually
   (a) at sub atmospheric pressure
   (b) at atmospheric pressure
   (c) at 0.5 times of the atmospheric pressure
   (d) above atmospheric pressure

4. An aquifer confined at top and bottom by impermeable layers is stratified into three layers as follows:

Layer	Thicknes (m)	Permeability (m/day)
Top layer	4	30
Middle layer	2	10
Bottom layer	6	20

The transmissivity (m^2/day) of the aquifer is
(a) 260 (b) 227
(c) 80 (d) 23

5. Specific capacity of a well is the
   (a) volume of water that can be extracted by the force of gravity from a unit volume of aquifer
   (b) discharge per unit drawdown of the well
   (c) drawdown per unit discharge of the well
   (d) rate of flow through a unit wdith and entire thickness of aquifer

6. Match **List-I** (Equation) with **List-II** (Applicability or principle of equation) and select the correct answer using the codes given below the lists:

List-I
A. Theim's equation
B. Dupit's assumption
C. Bernoulli's equation
D. Continuity equation

List-II
1. is based on energy conservation principle
2. is based on mass conservation principle
3. is applicable to steady flow towards a well in a confined aquifer
4. is applicable to steady flow in an unconfined aquifer

Codes:

	A	B	C	D
(a)	4	3	2	1
(b)	3	4	2	1
(c)	4	3	1	2
(d)	3	4	1	2

7. The performance of a well is measured by its
   (a) Specific capacity (b) Specific field
   (c) Storage coefficient (d) Permeability coefficient

8. Consider the following statements:

A well development

1. involves resersal of flow through the well screen
2. increases permeability towards the well
3. decreases permeability towards the well
4. is continued till sand/silt free water is pumped out

Which of these statements is/are correct?

(a) 1, 3 and 4 (b) 1, 2 and 4

(c) 3 only (d) 1 and 2

9. Match **List-I** (Well hydraulics parameters) with **List-II** (Definition) and select the correct answer using the codes given below the lists:

List-I

A. Specific yield

B. Safe yield

C. Specific capacity

D. Field capacity

List-II

1. Discharge per unit drawdown of well

2. Same as specific retention

3. Measure of water that can be removed by pumping

4. Limit of withdrawal from well without depletion of the aquifer

5. Water-bearing capacity of aquifer

Codes:

	A	B	C	D
(a)	4	3	2	5
(b)	3	4	1	2
(c)	4	3	1	2
(d)	3	4	2	5

10. Consider the following statements:

Assertion (A): The yeild of a well varied from 10 m^3/day to 20 m^3/day when the aquifer area changed from 50 m^2 to 75 m^2.

Reason (R): The yield is found to be directly proportional to the area of an aquifer opening into a well.

Of these statements:

(a) both A and R are true and R is the correct explanation of A

(b) both A and R are true but R is not a correct explanation of A

(c) A is true but R is false

(d) A is false but R is true

11. Which one of the following correctly defines aquiclude?

(a) A saturated formation of earth material which not only stores water but also yields it in sufficient quantiy.

(b) A formation through which only seepage is possible and thus the yield is insignificant compared to an aquifer.

(c) A geological formation which is neither porous nor permeable.

(d) A geological formation which is essentially impermeable to the flow of water.

12. The discharge per unit drawdown at the well is known as
    (a) Specific yield
    (b) Specific storage
    (c) Specific retention
    (d) Specific capacity

13. How is the determination of aquifer parameters S (Storage Coefficient) and T (Transmissivity Coefficient) done?
    1. By recording the drawdown in a pumped well at different time intervals.
    2. By recording the drawdown in installed observation wells at different time intervals.

    Select the correct answer using the codes given below:
    (a) 1 only
    (b) 2 only
    (c) Both 1 and 2
    (d) Neither 1 nor 2

14. An unconfined aquifer of porosity 30%, permeability 35 m/day and specific yield of 0.20 has an area of 100 km^2. If the water table falls by 0.25 m during a drought, the volume of water lost from storage, in million cubic metres, is
    (a) 2.0
    (b) 5.0
    (c) 1.0
    (d) 4.0

ANSWERS

1. (d)	5. (b)	9. (b)	13. (c)
2. (a)	6. (d)	10. (d)	14. (b)
3. (d)	7. (a)	11. (d)	
4. (a)	8. (b)	12. (d)	

PRACTICE SET

PRECIPITATION AND ITS MEASUREMENTS

Q.1. Explain how the average depth of precipitation over an area due to a storm is computed from isohyetal map of storm.

Q.2 The mass curve of precipitation resulted from the storm of 14th August 1983 gave the following values:

Hour	Accumulated depth at the end of periods in mm
22:00	0.0 (Beginning of storm)
22:05	10.2
22:10	20.8
22:15	33.0
22:20	47.2
22:25	55.8
22:30	64.0
22:35	71.6
22:40	78.8
22:45	85.4
22:50	91.4 (End of storm)

For the above storm, construct hyetograph and draw maximum intensity-duration curve.

• **Q.3** Estimate the average depth of precipitation, from depth-area curve, that may be expected over an area of 2400 km^2, due to the storm of 27th September, 1978 lasting 24 hrs, assuming the storm centre to be located at the centre of the area. The isohyetal map for the storm gave the areas enclosed between different isohyets as follows:

Isohyets in mm	Enclosed area in km^2
21	543
20	1345
19	2030
18	2545
17	2955
16	3280
15	3535
14	3710
13	3880
12	3915

Hence determine the depth of rainfall, due to the storm, that may be expected to be recorded by a rain gauge placed at the storm centre.

Q.4 Explain how maximum intensity-duration-frequency curve is obtained.

Q.5 A precipitation station X was inoperative for some time during which a storm occurred. At three stations A, B and C surrounding X the total precipitation recorded during this storm are 75, 58 and 47 mm respectively. The normal annual precipitation amounts at stations X, A, B and C are respectively 757, 826, 618 and 482 mm. Estimate the storm precipitation for station X.

Q.6 The information available from a isohyetal map of 1100 km^2 basin is as follows:

Zone	Area (km^2)	Rain gauge stations	Normal annual rainfall (cm)
I	85	A	120
II	290	B, C	95, 96
III	395	D, E, F	60, 65, 70
IV	230	G	45
V	65	H	21
VI	24	–	–

How many additional rain gauge stations will be required if the desired limit of error in the man value of rainfall is not to exceed 10 cm? Suggest how you propose to distribute these stations.
What factors will you consider in locating the additional raingauge stations between different isohyetals?

Q.7 Compute and draw the storm hyetograph and the intensity duration curve for the following storm (of a given frequency) on a drainage basin.

Duration (min)	Accumulated precipitation (cm)
0	–
30	5.0
60	7.5
90	8.5
120	9.0

Q.8 For a drainage basin of 640 km^2, isohyetals based on a storm event yield the following data:

Isohyetal interval (cm)	Inter-isohyetal area (km^2)
14 – 12	90
12 – 10	140
10 – 8	125
8 – 6	140
6 – 4	85
4 – 2	40
2 – 0	20

Estimate the average depth of precipitation over the basin.

Q.9 A watershed has five non-recording rain gauges, installed in its area. The amount of rainfall recorded for one of the years is given below:

Rain gauge station	Annual rainfall in (cm)
I	1000
II	120
III	190
IV	95
V	125

Find the required optimum number of non-recording and recording rain gauges for this watershed. Assume an error of 10% in the estimation of mean rainfall.

Q.10 What are the limitations of Theissen polygon method when compared to isohyetal method?

Q.11 There are five rain gauge sections viz. P, Q, R, S, T. Theissen polygon network details are given in table below. Calculate the quivalent uniform depth (EUD)

Rain gauge	Area (%)	Rainfall (mm)
P	24	45
Q	21	57
R	37	65
S	08	67
T	10	78

Q.12 The annual rainfall at station X and the average annual rainfall at 18 surrounding stations during 1952 to 1970 are as follows:

Annual rainfall in cm at X:

305, 38.9, 43.7, 32.2, 27.4, 32.0, 49.3. 28.4, 24.6, 21.8, 28.2, 17.3, 22.3, 28.4, 24.1, 26.9, 20.6, 29.5 and 28.4. 22.8, 35.0, 30.2, 27.4, 25.2, 28.2, 36.1, 18.4, 25.1, 23.6, 33.3, 23.4, 36.0, 31.2, 23.1, 33.2 and 26.4.

Explain how the consistency of the record at station X can be verified and how to determine the year in which a change in regime has occurred.

Q.13 The recorded annual rainfall from five raingauge stations in a catchment and the corresponding Thiessen polygon areas are as follows:

Thiessen polygon areas (cm^2)	Rainfall (cm)
25	125
30	175
30	225
10	275
5	325

The scale of the map is 1 : 50,000. Estimate the volume and the mean depth of the rainfall. Estimate the average annual discharge at the outlet, if the runoff coefficient of the catchment is 0.3.

Q.14 Following observations were made for conducting a water budget of a reservoir over a period of one month:

Average surface are = 10 km^2, Mean surface inflow rate = 10 cumec, Mean surface outflow rate = 15 cumec, Rainfall = 10 cm, Fall in the reservoir level = 1.5 m, Pan evaporation = 20 cm. Assuming the pan-factor as 0.7, estimate the average seepage discharge from the reservoir during the month.

EVAPORATION, TRANSPIRATION AND STREAM FLOW MEASUREMENT

Q.15. What are the factor affecting evaporation from water surfaces? Describe briefly a method of estimating the evaporation from weather data.

Q.16. In order to compute the flood discharge in a stream by the slope area method, the following data have been obtained:

	Upstream section	Middle section	Downstream section
Area (m^2) →	108.6	103.1	99.80
Wetted perimeter (m) →	65.3	60.7	59.40
Gauge reading (m) →	316.8	—	316.55

Determine the flood discharge assuming Manning's n = 0.029 and length between downstream section and upstream section as 250 m.

Q.17. During the passage of a flood, the following data were estimated at two sections; 500 m apart:

Section	Water surface elevation (m)	Area of flow section (m^2)	Hydraulic mean depth (m)
Upstream, P	85.233	91.746	2.835
Downstream, R	85.176	84.354	2.917

The eddy loss coefficient for gradual contraction is to be taken as 0.1 and for gradual expansion as 0.35. Estimate the flood discharge passing through the reach, Manning's n = 0.022.

Q.18. Using the equation V(m/s) = 0.65 N + 0.03, obtain the velocity at 0.6 times the depth from the free surface. Here N stands for revolutions/s. Data on current meter observations are given below in Tabular form.

Also compute the discharge through the section:

Distance from one bank (m)	Depth (m) (y)	Current meter observation at 0.6 y	
		No. of revolutions	Time in seconds
3.0	0.4	30	150
6.0	0.8	50	130
9.0	1.2	70	100
12.0	2.0	100	80
15.0	3.0	150	60
18.0	2.5	200	50
21.0	2.2	130	40
24.0	1.0	90	130

Q.19. A 25 g/l solution of a flourescent tracer was discharged into a stream at a constant rate of 10 cm^3/s. The background concentration of the day in the stream water was found to be zero. At a downstream section sufficiently far away, the dye was found to reach an equilibrium concentration of 5 parts per billion. Estimate the stream discharge.

INFILTRATION, RUNOFF AND HYDROGRAPHS

Q.20. One a catchment area of 20 hectares rainfalls of 7.5 cm, 2.0 cm and 5.0 cm occurred on three consecutive days. The average ϕ-index was 2.5 cm/day. The distribution graph percentages of surface runoff which extended over six days for every rainfall of such magnitudes are 5, 15, 40, 25, 10 and 5. Determine the ordinates of the discharge hydrograph and determine the peak discharge. Neglect base flow.

Q.21. Explain briefly the concept of a unit hydrograph.

Q.22. In a typical 4 hours storm producing 100 mm of runoff from a basin the stream flows observed are as follows:

Time in hrs	0	2	4	6	8	12	16	20
Flow in cumecs	0.00	2.44	8.10	13.50	11.34	6.75	2.30	0.00

Estimate the peak flow in the stream and the time of its occurrence in a flood created by an 8 hours storm which produces 50 mm of runoff during the first four hours and 75 mm of runoff during the second four hours. Assume base flow to be negligible.

Q.23. What are the basic propositions of unit hydrograph theory as propounded by L.K. Sherman?

Q.24. Explain briefly ***'infiltration indices'***.

Q.25. How do you determine storage capacity of a reservoir with the help of a mass curve of runoff if a constant or a variable demand is known?

Q.26. Define unit hydrograph.

Ordinates of 8-hour unit hydrograph for a drinage basin are given below. Obtain a 24-hour unit hydrograph by tabulation method. Neatly sketch it.

Time (hrs)	Ordinate of 8-hr unit hydrograph	Time (hrs) continued	Ordinate of 8-hr unit hydrograph continued
0	0.00	48	57.00
4	5.50	52	42.00
8	13.50	56	31.00
12	26.540	60	22.00
16	45.00	65	14.00
20	82.00	68	9.50
24	162.00	72	6.60
28	240.00	76	4.00
32	231.00	80	2.00
36	165.00	84	1.00
40	112.00	88	0.00
44	79.00		

Q.27. A 2-hour unit hydrogaph in a rather steep catchment is given below:

Time in hrs	0	2	4	6	8	10	12
Discharge ($\times$100 m^3/sec)	0	0.54	0.175	1.27	0.58	0.25	0

Compute the 1-hr unit hydrograph for the catchment.

Q.28. The average storm rainfall values over a catchment in three successive 2-hour intervals are 3, 5 and 2 cm, respectively. The ϕ-index for the catchment is taken as 0.2 cm/hr. The 2-hour unit hydrograph ordinates are given in the table. Base flow can be taken as 7 m³/sec at the beginning of the storm, linearly increasing to 9 m³/sec at 2 hours after the direct runoff peak discharge and later linearly decreasing to 8 m³/sec at 4 hours after the end of the direct runoff. Compute the resulting flood hydrograph.

Time (hrs)	0	2	4	6	8	10	12	14	16
Discharge (m^3/sec)	0	10	40	70 peak	50	20	10	6	0

Q.29. The following is the set of observed data for successive 15 minutes period of 105 minutes storm in a catchment:

Duration (min)	15	30	44	60	75	90	105
Rainfall (cm/hr)	2.0	2.0	8.0	7.0	1.25	1.25	4.5

If the value of ϕ-index is 3 cm/hr, estimate the net runoff, the total rainfall and the value of W-index.

Q.30. The ordinates of a flood hydrograph, resulting from two successive storms each of 1 cm rainfall excess and 6 hour duration, are tabulated below. Find a 6-hour unit hydrograph.

Upto which point we have unit ordinate

Time (hr)	0	6	12	18	24	30	36	42	48	54	60	66	72
Ordinate of 6 hr flood hydrograph (m^3/sec)	10	30	90	220	280	220	166	126	92	62	40	20	10

Q.31. What are the assumptions and limitations of Sherman's Unit Hydrograph Theory?

Q.32. The amounts of water flowing from a certain catchment area at the proposed dam site are as tabulated below:

Month	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.
Inflow ($10^5 m^3$)	2.83	4.25	5.66	18.40	22.64	22.64	19.81	8.49	7.10	7.10	5.66	5.66

Determine:

(i) the minimum capacity of reservoir if water is to be used to feed the turbines of hydropower plant at an uniform rate and no water is to be spilled over.

(ii) the initial storage required to maintain the uniform demand as above.

Q.33. The following are the ordinates for a flood hydrograph resulting from an isolated storm of 6 hours duration.

Time (hr)	Ordinates of flood hydrograph (cumec)
0	5
12	15
24	40
36	80
48	60
60	50
72	25
81	15
96	5

Determine the ordinates of 1 cm-6 hr unit hydrograph if the catchment area is 450 km².

Q.34. The daily flows in a river for three consecutive years are given in the table by class intervals along with the number of days the flow belonged to this class. What are the 50% and 75% dependable flows for the river?

Daily mean discharge (m^3/s)	Number of days in each class interval		
	1981	1982	1983
100-90.1	0	6	10
90-80.1	16	19	16
80-70.1	27	95	38
70-60.1	21	60	67
60-50.1	43	51	58
50-40.1	59	38	38
40-30.1	64	29	70
30-20.1	22	48	29
20-10.1	59	63	26
10 negligible	54	26	13

Q.35. Given below are the monthly rainfall, P, and the corresponding runoff values, R, for a period of 10 months for a catchment. Develop a correlation between R and P.

Month No.	P (cms)	R (cms)
1	4	0.2
2	22	7.1
3	28	10.9
4	15	4.0
5	12	3.0
6	8	1.3
7	4	0.4
8	15	4.1
9	10	2.0
10	5	0.3

Q.36. In a catchment, the average rainfall for a storm at two successive 6 hour intervals was 3.0 cm and 6.0 cm respectively. The abstraction losses ϕ index were estimated to be 0.20 cm/hr. For the same catchment, the calculated data for a 6 hr unit hydrograph is available and is given below. Find the direct runoff hydrograph due to the storm.

Time (hr)	Unit hydrograph ordinate (m^3/s)
0	0
6	10
12	25
18	40
24	100
30	150
36	100
42	75
48	25
54	15
60	0

Q.37. A 30 minute unit hydrograph for a catchment is given in Table 2. The ϕ-index is 4 mm/hr. The storm details are given in Table 1. Obtain the runoff hydrograph for the storm.

Table-1

Time (min)	Rainfall (cm)
0-30	3.3
30-60	2.7
60-90	1.9

Table-2

Time (min)	0	30	60	90	120	150	180	210	240	270	300	330	360	390
Runoff (m^3/s)	0	1.2	2.8	1.7	1.4	1.2	1.1	0.91	0.74	0.61	0.5	0.28	0.17	0

Q.38. What are the assumptions made in unit hydrograph theory?

Q.39. The catchment area of a drainage basin is 2100 km^2. The length of main stream (L) is equal to 80 km. The distance along the main stream from the basin outlet to a point on the stream which is nearer to the centroid of the basin (L_C) is equal to 50 km. Compute the widths of the 3-hr synthetic unit hydrograph at 50% and 75% of peak discharge using Snyder's method.

Take the coefficients

$C_t = 1.85\ C_p = 0.45$

C_t = coefficient accounting catchment slope

C_p = dimensionless coefficient

Q.40. The run-off data a stream gauging station for a flood in m^3/sec at three hourly intervals are as follows:

50, 50, 75, 125, 225, 290, 270, 145, 110, 90, 80, 70, 60, 55, 51, 50

The drainage area is 40 km^2. The duration of the rainfall is 3 hr. Derive the three hour unit hydrograph for the basin. Assume a constant base flow of 50 m^3/sec throughout the duration.

Q.41. Define unit hydrograph. What are its limitations? When do you recommend to use triangular unit hydrograph? Write down the expressions for peak discharge, base width and period of rise for triangular unit hydrograph.

Q.42. The ordinates of 6 hour unit hydrograph of a catchment are as follows.

Time (hours)	0	6	12	18	24	30	36	42
Discharge (m^3/sec)	0	10	40	55	45	30	7	0

The unit depth of the unit hydrograph is 1 cm. Arrive at the direct runoff hydrograph resulting from the following excess rainfall hyetograph occurring over the catchment.

Time (hours)	0-6	6-12
Rainfall intensity (cm/hr)	1	0.5

FLOODS, FLOOD ROUTING AND FLOOD CONTROL

Q.43. A flood of a certain magnitude has a return period of 25 years. What is its probability of exceedence? What is the probability that this flood may occur in the next 12 years?

Q.44. In estimating the peak discharge of a river at X, the catchment area was divided into four parts A, B, C and D. The time of concentration and the area for different parts are as follows:

Part	Time of concentration	Area in square meters
A	One hour	600×10^4
B	Two hours	750×10^4
C	Three hours	1000×10^4
D	Four hours	1200×10^4

Records for an air storm lasting for four hours, as observed, and the runoff factor during different hours are as follows:

Time in hrs		Rainfall in mm	Runoff factor
From	To		
0	1	2.50	0.50
1	2	50.00	0.70
2	3	50.00	0.80
3	4	23.50	0.85

Calculate the maximum flow to be expected at X in cumecs; assuming a constant base flow of 42.5 cumecs.

Q.45. Define: (i) Maximum probable flood; (ii) Standard project flood

Q.46. For a data of maximum recorded flood of a river, the mean and standard deviation are 4200 m^3/sec and 705 m^3/s respectively. Using Gumbel's extreme value distribution, estimate the return period of a design flood of 9550 m^3/s. Assume an infinite sample size.

Q.47. A large sample of peak floods data was available for a river. Flood frequency computations, using Gumbel's method, yield the following results

Return period, T(years)	Peak period (m^3/sec)
50	30,800
100	36,300

Estimate the magnitude of a flood for this river with return period of 200 years.

Q.48. The regression analysis of a 30 year flood data at a point on a river yielded sample mean $\bar{x} = 1200$ m^3/sec and standard deviation $s_x = 650$ m^3sec. For what discharge would you design the structure at this point to provide 95% assurance that the structure would not fail in the next 50 years? Use Gumbel's method. The value of mean and standard deviation of the reduced variate for n = 30 are 0.53622 and 1.11238 respectively.

Q.49. Flood frequency computations for a flashy river at a point 50 km upstream of a bund site indicated the following:

Return period, T(years)	Peak flood (m/sec)
50	20,600
100	22,150

Estimate the flood magnitude in the river with a return period of 500 years through use of Gumbel's method.

Q.50. A hydraulic structure is sized for a 50 year recurrence interval design discharge. What is the risk that the flow capacity will be exceeded during any future 20 year period? What is the probability that the 50 year recurrence interval peak flow rate will be exceeded in the next 50 years?

Q.51. Define recurrent interval and return period.

Q.52. Determine the design flood discharge (allowaing an increase of one third) for a bridge site with the following data:

Catchment area = 2×10^5 hectares, duration of storm = 8 hr. Storm precipitation = 3 m, time of concentration = 2 hr. gauged discharge for a past flood with average maximum daily rainfall of 18 cm was 3400 m^3/sec.

Q.53. The inflow hydrograph reading for the stream reach in m^3/sec at 12 hourly intervals are as follows:

42, 45, 88, 272, 342, 288, 240, 198, 162, 133, 110, 90, 79, 68, 61, 56, 54, 51, 45, 45 and 42.

The Muskingum coefficient for the stream reach are K = 36 hr and x = 0.15. Determine the attenuation in peak flow discharge and the time of peak outflow.

Q.54. A flood series have mean equal to 300 m^3/s and standard deviation as 50 m^3/s. Compute the magnitude of 50 years flood using Gumbel and Chaw methods.

Q.55. Flood frequency computation yields the following results:

Returns period (years)	Peak flood (m^3/s)
50.0	20,500
100.0	25,400

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Using Gumbel's method, estimate the flood for a return period of 150 years.

outflow Discharge just before flood

Q.56. A reservoir has the following elevation, discharge and storage relationships:

Elevation (m)	100.00	100.50	101.00	101.50	102.00	102.50	102.75	103.00
Storage ($10^6 m^3$)	3.350	3.472	3.380	4.383	4.882	5.370	5.527	5.856
Outflow discharge (m^3/s)	0	10	26	46	72	100	116	130

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What the reservoir level was at 100.50 m, the following flood hydrograph entered the reservoir.

Time (h)	0	6	12	18	24	30	36	42	48	54	60	66	72
Discharge (m^3/s)	10	20	55	80	73	58	46	36	55	20	15	13	11

Q.57. Show that in the level pool routing the peak of the outflow hydrograph must intersect the inflow hydrograph.