GATEFLIX

WATER RESOURCES ENGINEERING

For
CIVIL ENGINEERING

WATER RESOURCES

SYLLABUS

Hydrology

Hydrologic cycle, rainfall, evaporation, infiltration, stage discharge relationships, unit hydrographs, flood estimation, reservoir capacity, reservoir and channel routing. Well hydraulics.

Irrigation

Duty, delta, estimation of evapo-transpiration. Crop water requirements. Design of : lined and unlined canals, waterways, head works, gravity dams, spillways. Design of weirs on permeable foundation. Types of irrigation system, irrigation methods. water logging and drainage, sodic soils.

ANALYSIS OF GATE PAPERS

Exam Year	1 Mark Ques.	2 Mark Ques.	Total
2003	6	3	12
2004	1	3	7
2005	4	5	14
2006	-	4	8
2007	2	4	10
2008	3	3	9
2009	1	3	7
2010	1	2	5
2011	1	3	7
2012	-	-	-
2013	2	3	8
2014 Set-1	1	-	1
2014 Set-2	2	1	4
2015 Set-1	1	2	5
2015 Set-2	-	2	4
2016 Set-1	2	1	4
2016 Set-2	1	1	3
2016 Set-2	1	1	3
2017	2	3	8
2018	2	3	8

CONTENTS (HYDROLOGY)

CONTENTS (IRRIGATION)

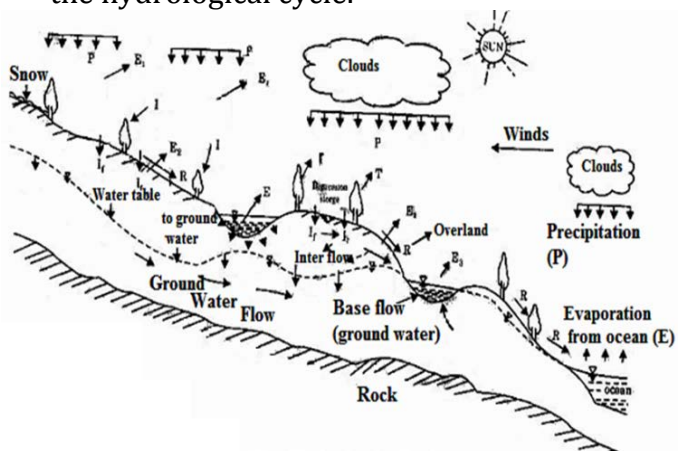
HYDROLOGY

1 WATER RESOURCE

Hydrology is the science that deals with the occurrence, circulation and distribution of water of the earth and its atmosphere.

1.1 HYDROLOGICAL CYCLE

- The hydrological cycle is a global sun-driven process whereby water is transported from the ocean to the atmosphere, from atmosphere to the land and then back to the sea.
- 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.
- 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 emerge 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 hydrology cycle is usually described in terms of six major components: Precipitation (P), Infiltration (I), Evaporation (E), and Transpiration (T). Surface Runoff(R) and Ground water flow (G). For computational purposes, evaporation and transpiration are sometimes lumped together as evaportranspiration (ET). Figure 1.1 define these component and illustrates the paths they define in the hydrological cycle.



The Hydrologic Cycle

E- Rain drop evaporation

I-Interception

T-transpiration
E-Evaporation of the land mass
E-Evaporation from water bodies
R- Surface Runoff

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, Evapo-transpiration 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 vapor 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.
- **Percolation** is the movement of water from one soil zone to a lower soil zone.
- **Transpiration** is the soil moisture taken up through the roots of a plant and discharged into the atmosphere through the foliage by evaporation.
- **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 catchment.

1.2 PRECIPITATION

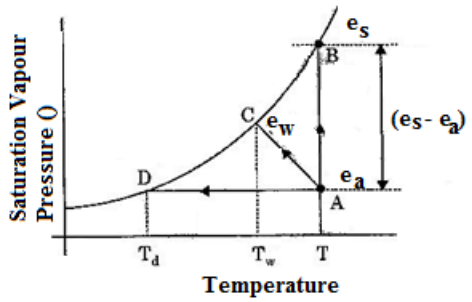
Precipitation is the fall of water in various forms on the earth from the clouds. The usual forms are rain, snow, sleet, glaze, hail, dew etc.

Before studying the phenomenon of precipitation let us consider water vapours. Air in atmosphere can easily absorb moisture in the form of water vapours. The amount of water vapours absorbed by air depends upon the temperature of air, the more is the temperature, the more water vapours it can absorb.

The water vapour exerts a partial pressure on the water surface called vapour pressure. The amount of water vapour present in air is indirectly expressed in terms of vapour pressure.

If the evaporation continues, a state of equilibrium is reached when the air is fully saturated with vapour, and therefore it cannot absorb more vapours. The vapours then exert a pressure which is known as saturation vapour pressure (e_s).e_s increases with increases in temperature.

Let us consider of parcel of air at a temperature T and a vapour pressure (e_a) indicated by pt. A. The saturation vapour pressure at that temperature is indicated by pt. B. the intercept BA = (e_s – e_a) is called saturation deflect.



If vapours are added to the parcel of air, the pt. A will move to pt. B when air is fully saturated.

If the parcel of air is cooled at constant pressure but without the addition of more vapours, the point moves horizontally towards point D and the air would be saturated when point D is reached. At that stage, the air would have a temperature called dew point temperature (T_d). Cooling of air beyond this pt. would result in condensation or formation of mist. If neither the temperature nor the pressure remains constant, the water evaporates freely and the pt. Moves to point C. In this case, water vapour rises but temperature falls. The temperature at point C. is called wet bulb temperature (T_w).

The saturation vapour pressure is indicated by e_w.

Air in atmosphere can be cooled by many processes. However, adiabatic cooling which occurs by a reduction of pressure through lifting of air masses is the main natural process.

1.2.1 CONDITIONS FOR OCCURANCE OF PRECIPITATION

Precipitation may occur only when there is some mechanism to cool the atmospheric air to bring it to saturation. Even saturation of air does not lead to precipitation. Precipitation occurs when the following conditions are satisfied.

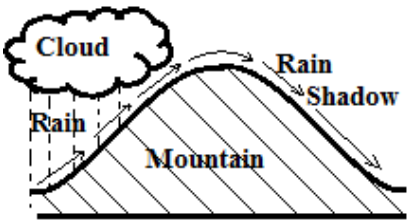
(1) Cooling of air masses
(2) Formation of clouds due to condensation
(3) Growth of water droplets
(4) Accumulation of moisture.

1.2.2 TYPES OF PRECIPITATION

Depending upon the factors responsible for lifting and cooling of air, there are following types of precipitation:

i) **Convective precipitation:** It occurs due to heating of air. The air close to the earth surface gets heated, and its density decreases. Consequently, the air rises upwards in the atmosphere and it gets cooled adiabatically to form a cloud. Precipitation caused by such clouds is called convective precipitation.

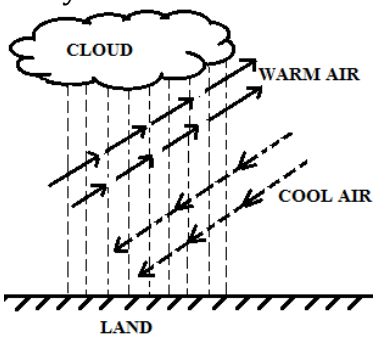
ii) **Orographic Precipitation:** Orographic rainfall occurs due to ascent of air forced by mountain barriers lying across the direction of air flow force the moisture laden air to rise along the mountain slope. It results in cooling, condensation and precipitation.



Orographic Rainfall

iii) **Cyclonic Precipitation:** A cyclone is a large zone of low pressure which is surrounded by circular wind motion. Air tend low pressure zone from and displaces low pressure cyclonic precipitation to move into the surrounding area air upwards. Thus occurs due to displacement of air in upwards direction due to pressure difference.

iv) **Frontal Precipitation:** It is a type of cyclonic precipitation. 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 settle downward.
Because the two types of front (warm and cold) have different temperature and density, frontal precipitation occurs when they clash with each other.



CYCLONIC OR FRONTAL RAINFALL

1.2.3 FORMS OF PRECIPITATION

1. **Rain:** Precipitation in form of water drops of size greater than 0.5 mm and less than 6 mm.
2. **Snowfall:** The 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. Avg. density = 0.1 gm/cc.

3. **Sleet** refers to a mixture of snow and rain but in American terminology sleet means falling of small pellets of transparent or translucent ice having a diameter of 5 mm or less.
4. **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 knows as hailstorms. Hails are very destructive and dreaded form of solid precipitation because they destroy agricultural crops and claim human and animal lives.
5. **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, intensity is usually less 0.1 cm/hr.
6. **Glaze:** It is a form of precipitation which falls as rain and freezes when comes in contact with cold ground at around 0^0C. Water drops freeze to form an ice coating also called freezing rain.

1.2.4 MEASUREMENT OF PRECIPITAION

- The total amount of precipitation on a given area is expressed as the depth of water if accumulated 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 rain-gauge.

1.2.5 TYPES OF GAUGES

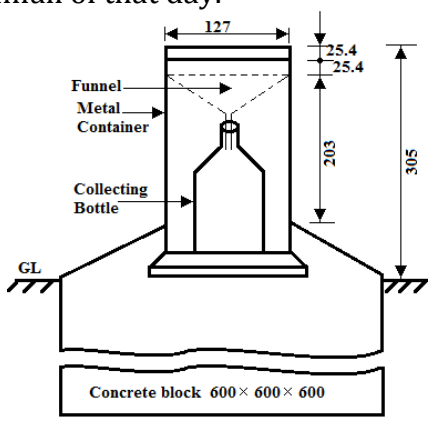
The various types of precipitation gauges used are broadly classified as:

a) Non –recording gauges, and

b) Recording gauges.

Non-Recording Gauges

- The non-recording gauge extensively used in India is the Symons' rain gauge. 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, everyday normally at 08:30 hrs which is considered as the daily rainfall. In case of heavy rainfall areas, measurements are made as often as possible. However, the last reading must be taken at 8:30 AM. So that last 24 hrs data may be added up to get the rainfall of that day.



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 into 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 rain gauge.

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 rain gauge is adopted as the **standard recording type rain gauge in India.**
This type of gauge gives a plot of the **mass curve** of rainfall.

1.3 RAIN GAUGE NETWORK

- For proper assessment of water resources, a good network of rain gauges is must. Rain gauge density is expressed as area covered per gauge. For better accuracy, catchment area per gauge should be small.
- As per the IS: 4987-1968. The recommended rain gauge network density is as follow:

In Plains;
One station per 520 km^2
Moderately elevated area;
One in 260 to 390 km^2
(av. elevation up to 1000m)
Hilly areas;
One in 130 km^2
According to WMO, at least 10 percent of the rain gauge stations should be equipped with automatic (self-recording) rain gauges.

1.4 ADEQUACY OF RAINGAUGE STATIONS

Number of rain gauge stations in a given catchment must be sufficient so that the error in precipitation measurement is not more than acceptable value.
Number of rain gauge 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

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

Where, P_1, P_2 ....., P_n are rainfall recorded at each stations and n is the total number of rain gauge stations in the catchment.

Step 2: Calculate the standard deviation, σ_{n-1}

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

$$= \sqrt{\left[\frac{\sum_1^n (P_i - P_m)^2}{n-1}\right]}$$

Step 3: Calculate co-efficient of variation, C_v, $C_v = \frac{\sigma_{n-1}}{P_m} \times 100$

Step 4: optimal number of stations, N, $N = \left(\frac{C_v}{\in}\right)^2$

Where, $\in$ is the allowable degree of error in the estimation of mean rainfall (in %)

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

In routine hydrological investigation, error of estimate should not exceed 10%

i.e. ε =0.1

Example 1

The average normal rainfall of 5 rain gauges 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 $\sigma^2 = 359.2$

$\therefore$ $\sigma = 18.95$

The coefficient of variation is calculated as:

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

$$\therefore \; N = \left(\frac{C_v}{0.10}\right)^2 = \left(\frac{0.33367}{0.1}\right)^2 = 11.13 = 12$$

Thus additional no. required $= (12 - 5) = 7$

1.5 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 January or yearly rainfall).

The 30-year normal 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 rain gauges.

1.6 AVERAGE ANNUAL RAINFALL

The amount of rain collected by a rain gauge in the last 24 hrs is called daily rainfall and the total amount collected in 1 year is called annual rainfall. Average annual rainfall is the average value of annual rainfall value for the last 35 years.

1.7 PREPARATION OF DATA

Before using the rainfall records of a station it is necessary first to 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 consistent with current record).

1.8 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.

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

Normal Ratio Method

If the 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 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.

87 → 87 × 1.1 = 95.7

87 → 87 × 0.9 = 78.3

Hence, simple arithmetic average will be used.

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

1.9 DOUBLE MASS CURVE TECHNIQUE

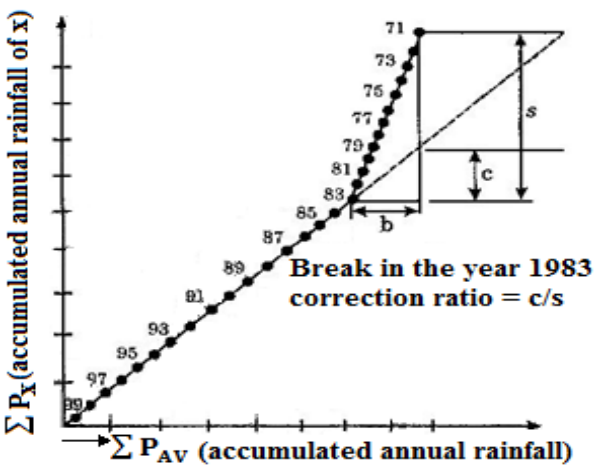
- To draw this curve, a group of stations (say 10) is taken as base station in the neighborhood of the problematic station X.
- The accumulated rainfall of station X($\sum P_x$) and accumulated values of average of group of base stations ($\sum P_{AV}$) are calculated starting from the latest record.
- The values of $\sum P_x$ as ordinate and $\sum P_{AV}$ as abscissa are plotted for available data of rainfall.
- In the plot, if a break in the slope is observed. It indicates a change in precipitation of station X.
- The values of 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.

$$P_{corr_X} = P_X \times \frac{M_c}{M_a}$$

Where,

M_c = corrected slope of double mass curve and M_a = original slope of double mass curve and Correction factor

$$= \frac{M_c}{M_a} = \frac{c}{s}.$$



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

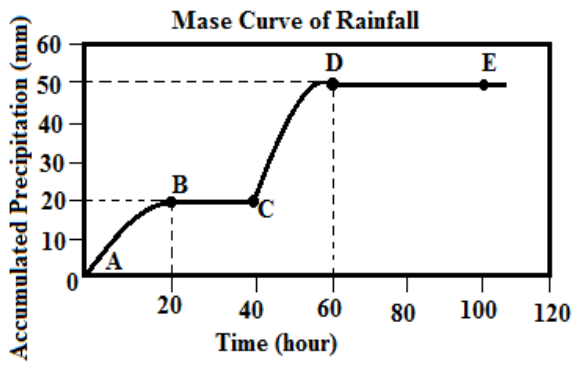
multiplied by correction ratio c/s to make it consistent with the data before the year 1983 (i.e. 1984, 1985……).

1.10 PRESENTATION OF RAINFALL DATA

Rainfall data is presented in the form of:
a) Mass curve
b) Hyetograph
c) Moving average

1.10.1 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.



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

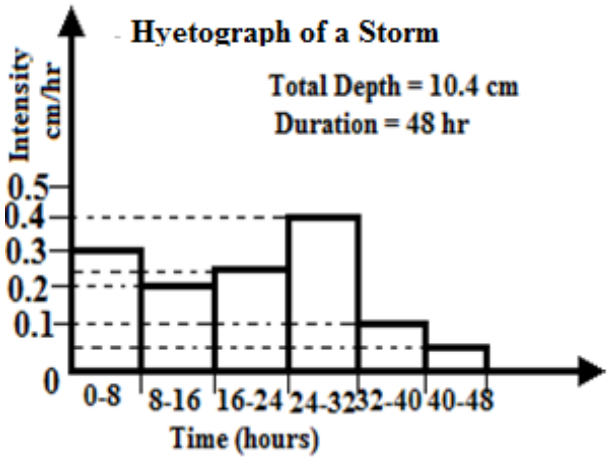
No rainfall during DE

1.10.2 HYETOGRAPH

A hyetograph is a plot of the average 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.

Hyetograph is more preferred than mass curve since it is convenient to determine the area of hyetograph.



1.10.3 MOVING AVERAGE

If we plot point rainfall (rainfall collected at rain gauge 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 of the fluctuations 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 3rdyrs is plotted against 2nd yrs. , average of 2nd, 3rd and 4thyr is plotted against 3rd yr. and so on.

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

1.11 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 (Rain gauge station).

However, hydrological analysis requires 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

1.11.1 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 rain gauges located outside the catchment.

  As per this method:

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

  P_m is the average rainfall in the catchment.

  P_i is the rainfall magnitude at the ith station inside catchment.

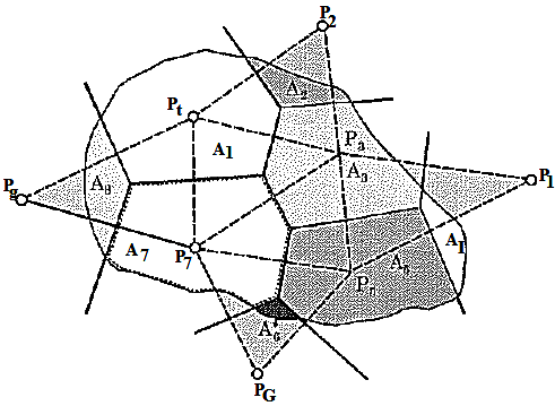
  n is the number of rain gauges in the catchment.

1.11.2 THIESSEN POLYGON METHOD

This method considers the representative area for each rain gauge also considering the rain gauge present outside the catchment. Representative area can also be thought of as the areas of influence of each rain gauge.

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 ith station

A_i is the area of influence for the ith station

A is the total area of the catchment.

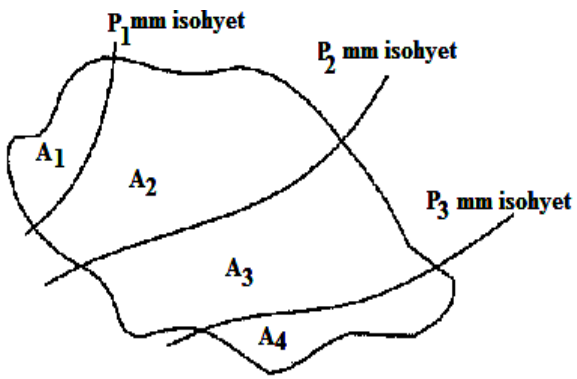
W_i is weight of station i

- Weighted average rainfall is calculated by this method.
- Polygon needs to be calculated only once for a given distribution of rain gauge 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 rain gauge.
- 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 isohyetal method is used for both plain and hilly areas.
- This method is more accurate than arithmetic-mean method and takes care of rain gauges located outside the catchment also.

1.11.3 ISOHYETAL METHOD

- Isohyetal is a line joining points of equal rainfall magnitude. The area between two adjacent isohyets 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.

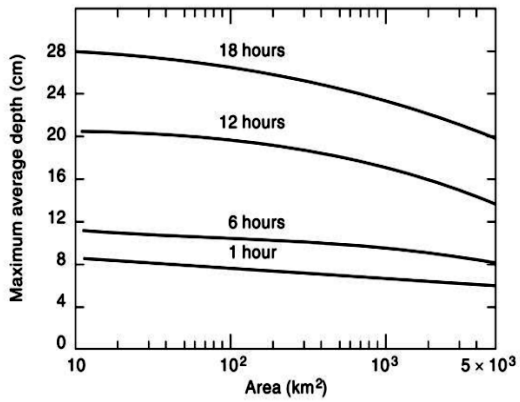


- Average rainfall is calculated as
  $P_m = (\sum P_{ij} A_{ij}) / A$
  P_m is the average rainfall in the catchment
  $P_{ij} = (P_i + P_j) / 2$
  A_{ij} is the area between two successive isohyets P_i and P_j
  A is the total area of the catchment.
  Area A_2 and A_3 falls between two isohyets each. 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 depth for area, A_1 by interpolation. Similar logic applies for area A_4.

1.12 DEPTH AREA DURATION RELATIONSHIP

- Depth of rainfall at a rain gauge station is called point rainfall. To convert the point rainfall data 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 depth 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.
  Typical depth area duration curve



DAD curve can also be obtained from empirical like

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

$\overline{P}$ = 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 rain gauge station. Hence exact determination of P_0 is not possible.

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

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

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 indicates that depth of rainfall increases as the duration increases.

MAXIMUM DEPTH AREA DURATION CURVE

In the design of hydraulic structure, we need to design the design floods.

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 maximum DAD curve.

Due to various storms of a particular duration (i. e. say various storms of 24 hr duration) depth area curve is found.

Then area is plotted w. r. to max depth of rainfall corresponding to the various storms of 24 hr duration.

This is MAX DAD curve.

It 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 rain gauge.

2 FREQUENCY OF POINT RAINFALL

2.1 FREQUENCY OF POINT RAINFALL

In many hydraulic designs, 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 descending 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 last value. Then probability p of a rainfall at position 'm' being equaled or exceeded is given by Weibull's formula as $p = \frac{m}{N+1}$ (Plotting position approach)
  Where, N= no. allotted to the last position i.e. no. yr. of record.
- Recurrence interval or return period is the average time period after which the particular rainfall value is likely to be equaled or exceeded. Recurrence interval or return period is given by

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

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

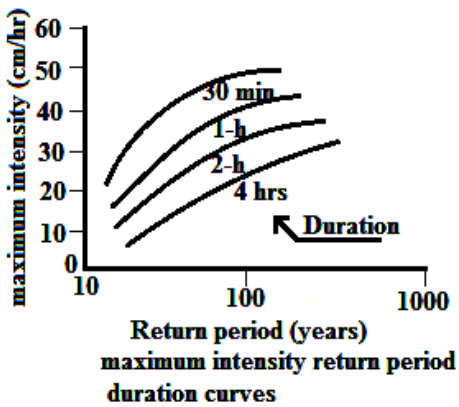
1) Probability of non-occurrence
   $= (1- p) = q$
2) Probability of occurrence of event, r, times in n successive yrs
   $= {}^nC_r p^r . q^{n-r} = \frac{n!}{r!(n-r)!} p^r q^{n-r}$
3) Probability of the event not occurring at all in all n-successive yrs
   $= {}^nC_0 p^0 . q^n = q^n = (1-p)^n$
(4) Probability of the event occurring at least once in n-successive yrs.
   $= 1 - q^n = 1 - (1-p)^n$

- % dependable flow = (100 ×p)%

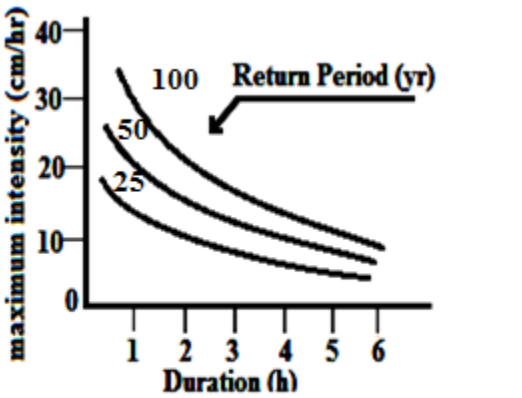
2.2 INTENSITY DURATION FREQUENCY CURVES

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 create flooding havoc in an area.

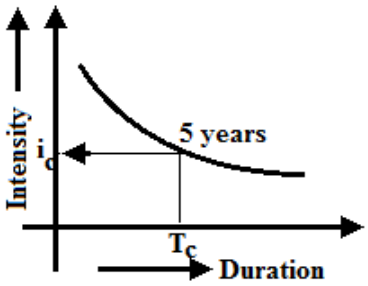


maximum intensity return period duration curves



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 over flooded 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 constant for a given catchment.

2.3 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 hydrometer logical conditions that is possible to maximize the rainfall. From this DAD of PMP is obtained.

PMP can be statistically estimated as:

$$PMP = \overline{P} + K\sigma$$

$\overline{P}$ = Mean annual rainfall series; K= Frequency factor;

σ = Standard deviation of series

PMP is used for design of <u>large hydraulic structures</u> 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 hydrometeorological 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)

3 ABSTRACTIONS FROM PRECIPITATION

3.1 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.

3.2 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.

3.2.1 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:

$E_L = C(e_w - e_a)$ Dalton's law

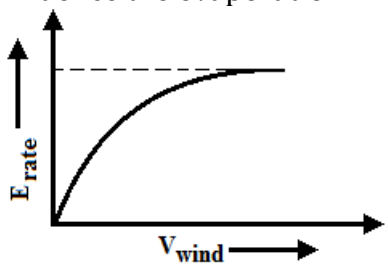
Where, E_L = rate of evaporation (mm/day), C = a constant; and e_w and e_a are in mm of mercury. Evaporation continues till $e_w = e_a$; and if $e_w < e_a$, condensation takes place.

3.2.2 TEMPERATURE

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

3.2.3 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.

3.2.4 ATMOSPHERIC PRESSURE

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

3.2.5 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

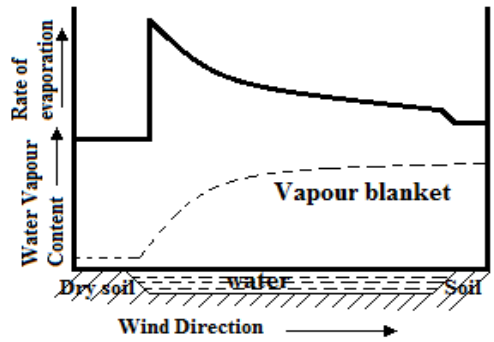
evaporation from sea water with an average salinity of about 3.5 per cent is some 2 to 3 percent less than evaporation from fresh water at the same temperature.

3.2.6 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.

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

3.3 EVAPORATION MEASUREMENT

The amount of water evaporated from a water surfaces is estimated by the following methods:
i) Using evaporimeters data,
ii) Using empirical evaporation equations,
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.

3.3.1 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:

Lake evaporation = C_p × pan evaporation

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

Sr. 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 meteorological 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 temperature 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

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).

1. **Empirical Formulae**

Empirical Formulae are based on Dalton's law.

Meyer's Formula

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

Where,

E_L = lake evaporation in mm/day;

e_w = saturated vapour pressure at the water-surface in mm of mercury;

e_a = actual vapour pressure of overlying air at a specified height in mm of mercury;

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 $u_h = Ch^{1/7}$

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 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 = 7.02mm of Hg. Wind velocity at 1 m above ground surface = 16 km/h. Estimate the average daily evaporation from the lake using Meyer's formula.

Solution

e_w = 17.54 mm of Hg

e_a = 7.02 mm of Hg

Wind speed of 9 m above ground = u_9

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

$u_9 = (9)^{1/7} \times u_1 = (9)^{1/7} \times 16 = 21.9$ km/hr

Hence using Mayer's formula

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

$E_L = 8.97$ 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.

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

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 stored in the water body over a known time interval.

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

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

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),

H_e = heat energy used up in evaporation,

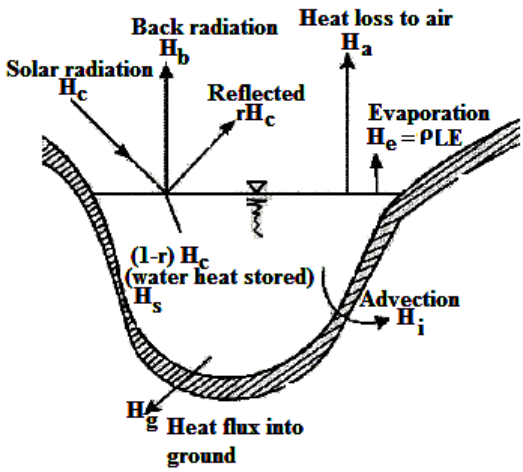
$\approx \rho L E_L$

Where,

ρ =density of water

L =latent heat of evaporation, and

E_L =evaporation in mm



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:

$$\beta = \frac{H_a}{\rho L E_L} = 6.1\times10^{-4}\times p_a \frac{(T_w - T_a)}{(e_w - e_a)}$$

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 ^0C, and

T_a = temperature of air in °C. Hence E_L can be evaluated as:

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

(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.

3.4 LAKE EVAPORATION & 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 month (m^3),

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

E_{pm} = pan evaporation loss in meters 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 (hexadecanol) 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.

3.4.1 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 vegetation, 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.

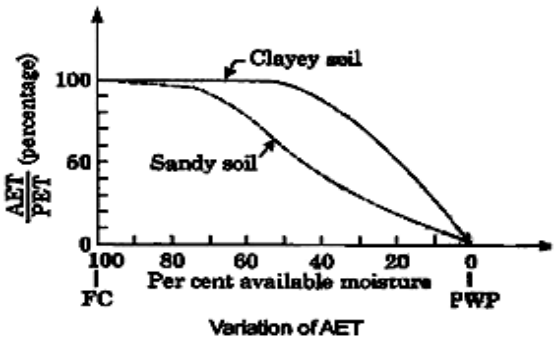
Only over those areas of earth's surface where no vegetation is present will purely evaporation occur. Evapotranspiration represents the most important aspect of water loss in the hydrologic cycle.

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 evapotranspiration occurring in prevailing/actual condition is called actual evapotranspiration (AET).
The mean annual PET (in cm) over various parts of the country is shown in the form of isopleths. Isopleths are the line on a map through places having equal depth of 'evapotranspiration'.
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 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.



Variation of AET

Index of wetness

This index is used to find the rainfall Variation for a particular year and is calculated as follows:

Index of wetness =

$$\frac{\text{rainfall in a year}}{\text{avg. annual rainfall}} \times 100$$

• The index of wetness of 25 % shows that the rainfall deficiency is 25%.

• If index of wetness 100 %; it indicates that rainfall is normal.

• Flood is not related with index of wetness.

Aridity Index (AI) is defined as

$$AI = \frac{PET - AET}{PET} \times 100$$

AI anomaly(%)	Severity class
0-25	Mild arid
26-50	moderate arid
> 50	severe arid

3.5 MEASUREMENT OF ACTUAL EVAPOTRANSPIRATION

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

3.5.1 PHYTOMETER

Measures only transpiration.

3.5.2 LYSIMETER

A lysimeter (also known as evapotranspirometer) consist 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:

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

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

3.5.3 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 is 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.

3.6 ESTIMATION OF POTENTIAL EVAPOTRANSPIRATION

Potential evapotranspiration is found out using Penman's equation & some empirical formulae:

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

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

PET =daily potential evapotranspiration, in mm per day.

A = slope of the saturation vapour pressure v/s temperature curve at the mean air temperature, in mm of mercury per ^0C

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/ ^0C .

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 temperature, which are taken as a measure of solar radiation on a given area. The potential evapotranspiration in a crop-growing season is given by:

$$E_r = 2.54K \sum \{P_h \times T_f / 100\}$$

Where,

E_r =PET in crop season, in cm,

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

$\sum$ =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 ^0F

4 INFILTRATION

4.1 INTERCEPTION DEPRESSION STORAGE AND INFILTRATION

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.
Steam 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-growth 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.

4.1.1 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 forms a part of the initial loss.

4.2 INFILTRATION

Infiltration is defined as the downward entry of water into the soil or rock surface and **percolation** is the flow of water through soil and porous or fractured rock.
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.
(infiltration + 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.
Hence
infiltration + 0 + 0 = Rainfall – 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 infiltrating into the soil gets stored as soil moisture and does not 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.

4.2.1 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.

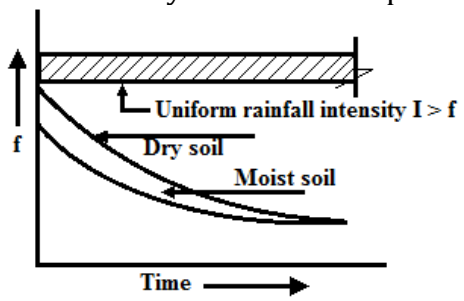
4.2.2 RAINFALL CHARACTERISTICS

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

$f = f_c$, when $i > f_c$

$f = i$, 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.



Variation of infiltration capacity with time

4.2.3 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.

4.2.4 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 effectively reduces the infiltration capacities, of various patches of ground to zero and thus contributes significantly to the frequency of flood peaks in such areas.

4.2.5 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.

4.2.6EMPIRICAL INFILTRATION EQUATIONS

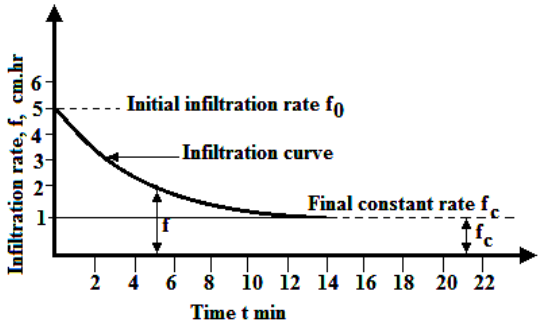
Under given soil type and antecedent moisture conditions, there will be an initial infiltration rate, f_0. This rate will decrease as more water gets infiltrated, finally achieving a constant rate, f_c, i.e., ultimate infiltration capacity. This infiltration capacity rate prevails when soils is saturated.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

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

Where, in practice, f_0, 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.



INFILTRATION CAPACITY VS TIME

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

4.2.7 RELATION OF INFILTRATION TO RUNOFF

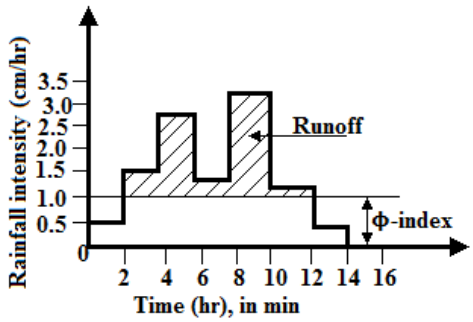
- During a major storm, capable of producing flood conditions in a river basin, evapotranspiration losses are negligible and losses by interception and depression storage are small compared to the amount of infiltration.
- Too low a rate of infiltration causes high runoff and too high a rate of infiltration 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.

4.2.8 INFILTRATION INDICES

- 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.

(1) ϕ-index

- The ϕ-index is the average rainfall above which the rainfall volume is equal to the runoff volume.
- ϕ-index is average infiltration rate during the period of rainfall excess. Rainfall excess is the rainfall contributing to runoff and the period during which such a rainfall takes place is called period of rainfall excess.
- The ϕ-index is derived from the rainfall hyetograph with the knowledge of the resulting runoff volume. The initial loss is also considered as a part of infiltration.
- If $i < \phi$-index $\Rightarrow f = i$ and if $i > \phi$-index $\Rightarrow f = \phi$-index where, i is rainfall intensity and f is infiltration rate.



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.

4.2.9 W-INDEX

The W-index is a refined version of ϕ-index. It excludes the depression storage and interception from the total losses. It is the average infiltration rate during the time rainfall intensity exceeds the capacity rate. That is

$$W = \frac{F}{t} = \frac{(P - Q - S)}{t}$$

Where F is the total infiltration, t is time during which rainfall intensity exceeds infiltration capacity, P is total precipitation corresponding to t, Q is the total runoff and S is the volume of depression storage and interception.

W-index is always less than ϕ-index (W-index $\leq \phi$-index)

5 RUNOFF

5.1 RUNOFF

Runoff may be referred to as stream flow, river discharge or catchment yield. It is normally expressed as volume per unit time.
Based on the time delay between precipitation and runoff, runoff is classified into two categories-
1) Direct runoff
2) Base flow

5.2DIRECT 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 termed as direct storm runoff or storm runoff.

5.2.1 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 is in turbulent regime.

5.2.2 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 favorable 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 soil, the interflow is sometimes classified into prompt interflow & delayed interflow.

5.2.3 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 is runoff calculations.

5.2.4 BASE FLOW OR GROUND WATER 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.
- The infiltrated water which percolates deeply becomes groundwater and when groundwater table rise and intersects the stream channels of the basin it discharges into stream as the groundwater runoff.
- Ground water flow is sometimes referred to as base flow, dry weather flow, and effluent seepage.
  For the practical purposes of analysis total runoff in stream channels is generally classified as direct runoff and base flow.

Beginning of Water Year

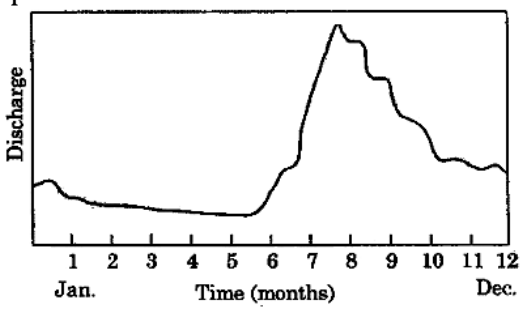
The time when precipitation exceeds the average evapotranspiration losses is called the beginning of water year.
1st June in India is considered the beginning of water year. Beginning of water year is decided such that flood season is not divided between successive years.

5.3 RUNOFF CHARACTERISTIC OF STREAM

On the basis of hydrograph studies a stream can be classified as:
a) Perennial
b) Intermittent
c) Ephemeral

5.3.1 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 stream. Thus, considerable amount of ground water flow occurs during non precipitation period.

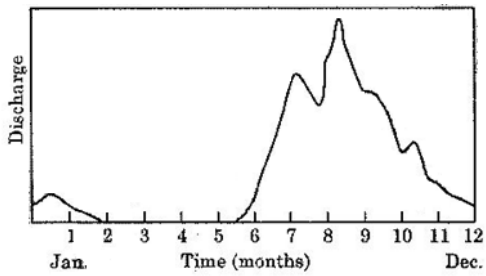


Perennial stream

- In perennial streams, 100% dependable flow has a finite value.

5.3.2 INTERMITTENT STREAM

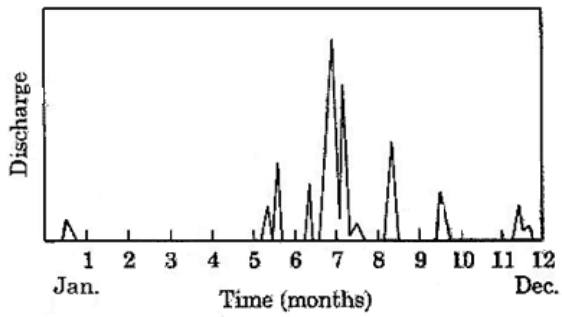
- Intermittent stream has limited contribution from the groundwater.
- Stream remains dry for most part of dry month.
- Base flow (ground water flow) occurs significantly during wet season.
- During dry sections the water table drops to a level lower than that of the stream bed.



INTERMITTENT STREAM

5.3.3 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 rivers in arid zones are of the ephemeral kind.
- In intermittent/ephemeral streams Q_{100} (100% dependable flow) = 0.(zero)



Ephemeral stream

5.4 FLOW DURATION CIRVE AND FLOW MASS CURVE

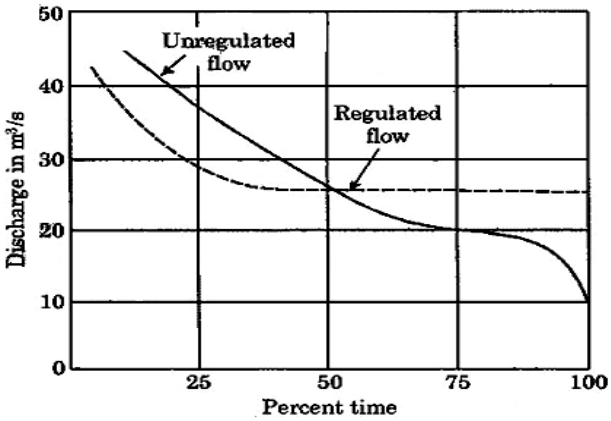
When the runoff rate observed at the catchment outlet is continuously, it can be plotted as the ordinate against time on abscissa to provide the runoff hydrograph. There are two other ways of evaluating the variability in the runoff. They are the flow-duration curve and the flow mass curve.

5.4.1 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 of m, 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 dividing 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 m^3/s and 50 m^3/s for 73 days, then the mid value of the class interval, i.e. 45 m^3/s, is plotted against percent of time given by $\frac{73}{366}\times 100 = 19.94\%$, 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.



Flow duration curve

Some of the important uses are

(1) In evaluating various dependable flows in the planning of water resources engineering projects.

(2) Evaluating the characteristics of the hydro power potential of a river.

(3) Designing of drainage system

(4) In flood control studies.

(5) Computing the sediment load and dissolved solid load of a stream.

(6) Comparing the adjacent catchments with a view to extend the stream flow data.

Flow mass curve (Rippl's mass curve)

The flow mass curve is a plot of the cumulative discharge volume against time plotted in chronological order.

The ordinate of the mass curve, V at any time t is thus

$$v = \int_{t_0}^{t} Q\, dt$$

Where,
t_0 = time at the beginning of the curve.
Q = discharge rate.

The slope of the mass curve at any point represents

$$\frac{dv}{dt} = Q = \text{rate of flow at that instant}$$

Determination of run off

(1) Using Empirical formula
(2) Using Infiltration curve
(3) Using hydrograph and unit hydrograph
(4) Using run-off coefficient method
(5) Using rainfall-runoff co-relation

Rainfall runoff relationship

It is one of the most common method to estimate the run off. The equation for straight line regression between run off R and rain fall P is

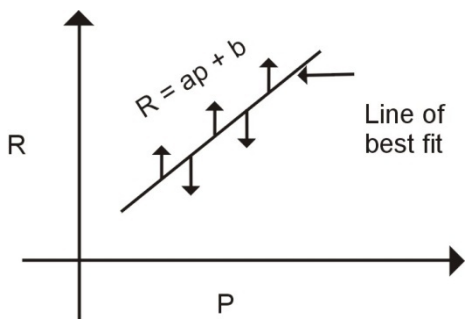
R = aP + b

The values of the co-efficients a and b are given by

$$a = \frac{n\left(\sum PR\right) - \left(\sum P\right)\left(\sum R\right)}{N\left(\sum P^2\right) - \left(\sum P\right)^2}$$

$$b = \frac{\sum R - a\left(\sum P\right)}{N}$$

Where, N = number of observation sets R and P.

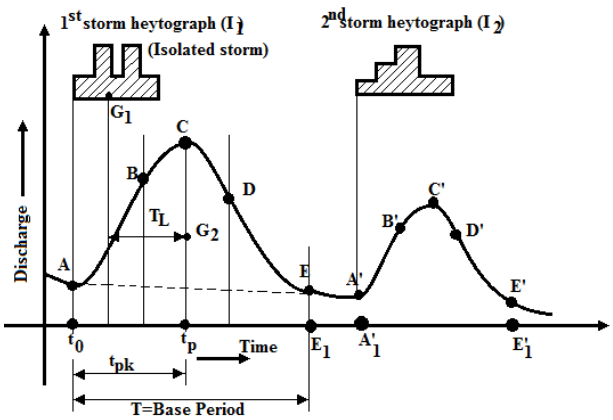


6 SURFACE WATER HYDROLOGY

6.1 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. 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.



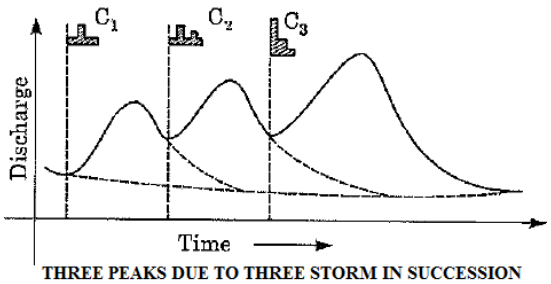
SINGLE PEAKED HYDROGRAPH

6.1.1 FEATURES OF A HYDROGRAPH

Hydrograph is a graphical variation of discharge against time. It 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 occur in a catchment such that the 2nd one does not 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 multi peaked hydrographs are obtained.



THREE PEAKS DUE TO THREE STORM IN SUCCESSION

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. It is a skewed distribution which may be skewed to left or right.
- 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 center of mass of hydrograph.
- T_L= Lag time
- t_{PK} = is the time of peak from the starting point 'A'.

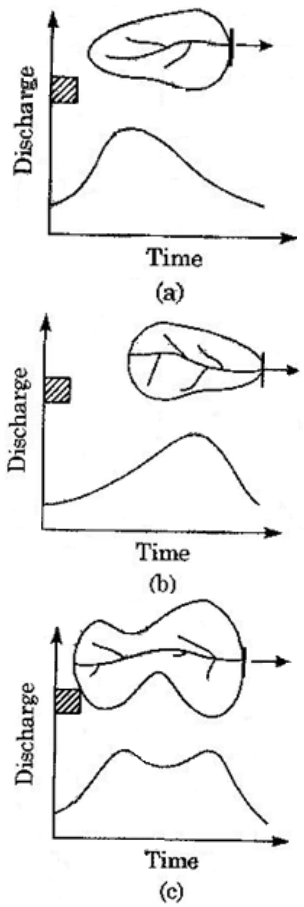
In the Figure (II) storm, C_1, C_2 and C_3 are called complex storm. A hydrograph due to complex storm (multi peaked hydrograph) can be resolved into corresponding single peaked hydrographs.
However, single peaked hydrographs resulting from isolated storms are generally preferred for hydrological analysis.

6.2 FACTORS AFFECTING FLOOD HYDROGRAPH

Shape of hydrograph depends broadly on catchment and rainfall characteristics and also on climatic factors. 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).



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 slop affects the shape of the hydrograph. Larger slopes generate more velocity than smaller slopes and hence can dispose of 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.

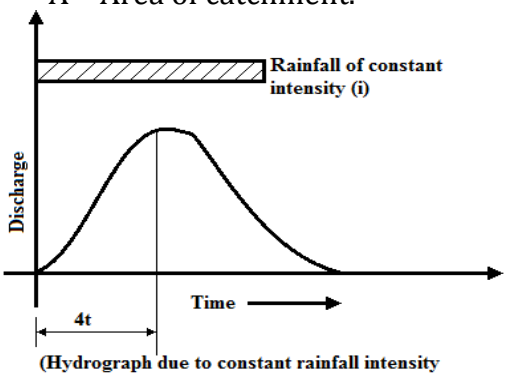
(iv)Drainage Density

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.

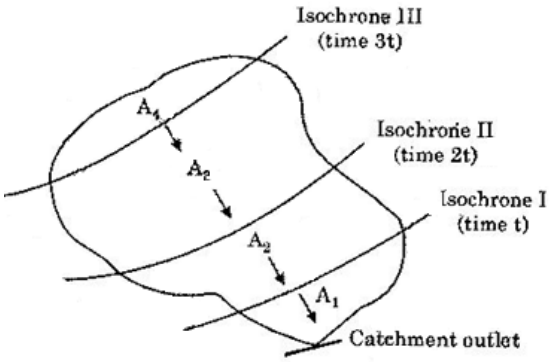
b) **Rainfall duration:** If the rainfall intensity is constant, then the rainfall duration determines the peak flow and time period of the surface runoff.
If uniform rainfall continues, the discharge will go on increasing up to the rainfall duration equal to time of concentration.
If rainfall continues 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 × A),
Where
i= intensity of uniform rainfall and
A = Area of catchment.



(Hydrograph due to constant rainfall intensity greater than the time in which emtire catchment starts contributing)

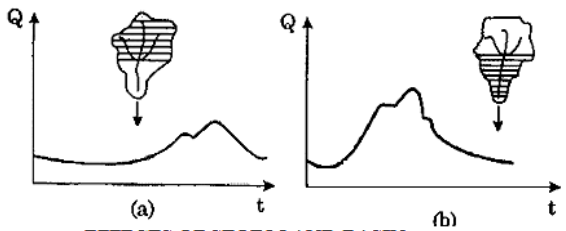
Note:
Isochrones are imaginary lines across the catchment from where water particles travelling downward take the same time to reach the catchment outlet.

c) Effect of a real distribution of rainfall over catchment.



If the only area A_1 receives rainfall but other areas do not then since the area is nearest to outlet, the resulting hydrograph immediately rise. 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 ×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 up to time 3t. Pictorially these things are shown as.



EFFECTS OF STORM AND BASIN CHARACTERISTICS ON HYDRIGRAPH 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

6.3 COMPONENTS OF A HYDROGRAPH

The essential components of a hydrograph are:

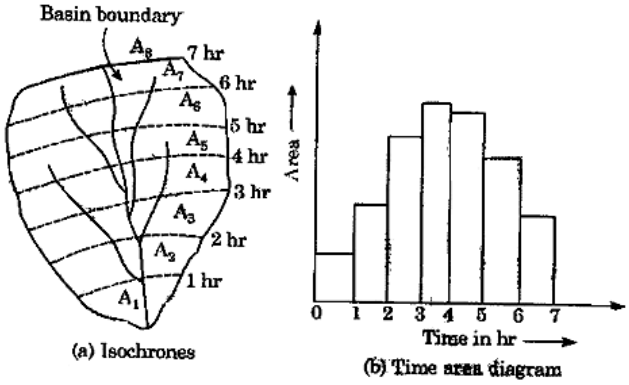
i) The rising limb,
ii) The crest segment, and
iii) The recession limb.

6.3.1 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.



6.3.2 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.

6.3.3 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

$$Q_t = Q_0 K_r^t$$

$$Q_t = Q_0 e^{-at} \text{ (where, } a = -\ln K_r)$$

Where Q_0 =initial discharge

Q_t =discharge at time t

K_r^t = 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

$$S_t = -\int Q_t.dt$$

$$S_t = -\int Q_0 e^{-at} dt$$

$$S_t = \frac{-Q_0 e^{-at}}{-a} + C$$

At $t = \infty$, $S_t = 0$ $\quad C = 0$

$$S_t = \frac{Q_0}{a} e^{-at}$$

$$S_t = \frac{Q_t}{-\ln K_r}$$

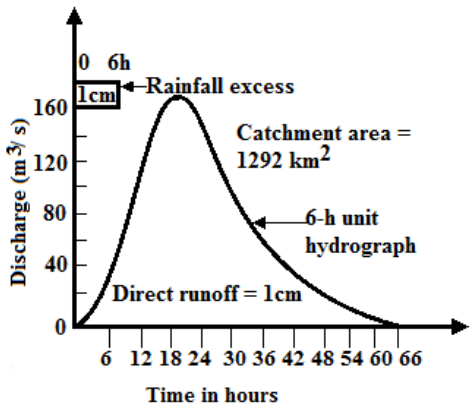
Thus discharge at any time is proportional to storage remaining at that time.

6.4 EFFECTIVE RAINFALL HYDROGRAPH

When initial losses and infiltration losses are subtracted from the rainfall hyetograph, we get effective rainfall hyetograph (ERH). It is also known as hyetograph:

(Area under ERH) × (Catchment area) = Volume of runoff = Area under direct runoff hydrograph

6.5 UNIT HYDROGRAPH



6-hr 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)

  Assumption made in the theory of unit hydrograph (As proposed by Sherman) is as follows:

6.5.1UNIT 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 affect the new rainfall.

Note:

This antecedent precipitation is otherwise important because of its effect on soil-

infiltration rate, digressional and detention storage, and hence, on the resultant hydrograph.

1. 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 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.

6.6 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:

Volume of DRH = $\sum Q \Delta t$ = 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.

6.7 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:

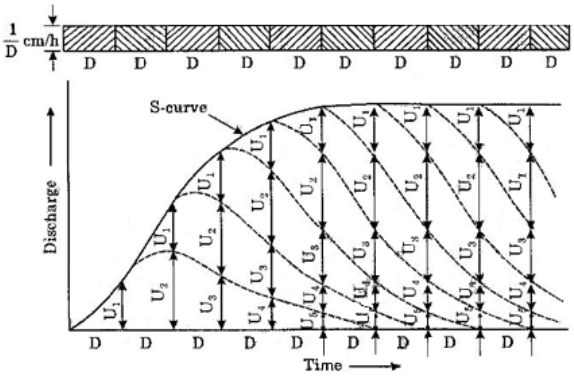
Case (i) Changing a Short Duration Unit hydrograph to Longer Duration

- 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 hydrograph, 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 3-hour unit graph, because of a lower intensity storm for a longer time.

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 hydrographs 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 hydrograph, 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 $\frac{T_B}{D}$.



S-curve hydrograph

Since the rainfall rate is equal to the runoff rate at the equilibrium state, it follows that

$$Q_e = A.\frac{1}{D}km^2 - cm/h = 2.778\frac{A}{D}m^3/s$$

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.

6.8 SYNTHETIC UNIT HYDROGRAPH

- Unit hydrograph 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 hydrographs 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.

6.9 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 $u(t) = \frac{1}{i}\frac{ds}{dt}$, 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.

7 FLOODS

FLOODS

- Any flow which is relatively high and which overtops the natural or artificial 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 food 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 increase, 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 meteorological and hydrological conditions, which are reasonably characteristic of the drainage basin being considered, but excluding extremely rare condition.

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 hydrometeorological conditions that would maximize 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 structure like spillways, bridge openings, flood banks, etc. It may be the MPF or SPF or a food 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 PMF is used in situations where the failure of the structure would result in loss of life and catastrophic damage and as such complete security from potential floods is sought. On the other hand, SPF is often used where the failure of a structure would cause less severe damages. Typically, the SPF is about 40% to 60% of the PMF for the same drainage basin. The criteria used for selecting the design flood for various hydraulic structures vary from one country to another. The following table gives a brief summary of the

guidelines adopted by CWC India to select design floods.

Table guidelines for selecting Design Floods (CWC, India)

S. No.	Structure	Recommended design flood
1	Spillways for major and medium projects with storages more than 60 Mm^3	(a) PMF determined by unit hydrograph and probable maximum precipitation (PMP) (b) If (a) is not applicable or possible flood frequency method with T = 1000 years
2	Permanent barrage and minor dams with capacity less than 60 Mm^3	(a) SPF determined by unit hydrograph and Standard project storm (SPS) which is usually the largest recorded storm in the region (b) Flood with a return period of 100 years. (a) or (b) whichever gives higher value
3	Pickup weirs	Flood with a return period of 100 or 50 years depending on the importance of the project.
4	Aqueducts (a) Waterway (b) Foundations and free board	Flood with T = 50 years Flood with T = 100 years
5	Project with very scanty or inadequate data	Empirical formulas

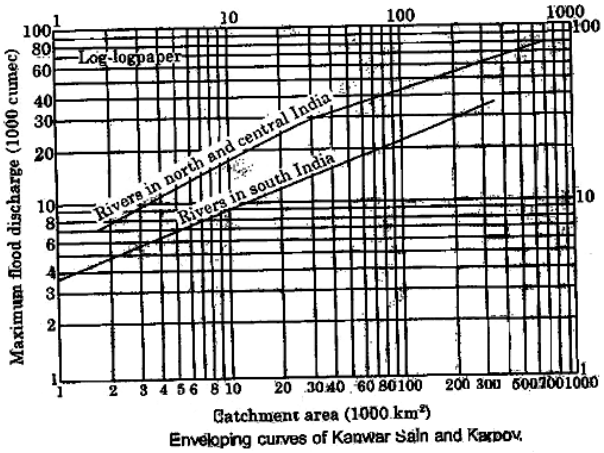
- The methods used in the estimation of the design flood can be grouped as under:-
  i) Physical indication of past flood
  ii) Envelope curves
  iii) Empirical flood formulae
  iv) Rational method
  v) Unit hydrograph application
  vi) Frequency analysis (or Statistical methods).

7.1 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.

7.1.1 ENVELOPE CURVE

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 ay return period to the peak value.



Enveloping curves of Kanwar Sain and Karpov.

Envelope curves are generally used for comparison only and the design floods got by other methods should be higher than those obtained from envelope curve.

Following equation is sometimes used for enveloping curve of maximum floods throughout the world

$$Q = \frac{3010A}{(277+A)^{0.78}}$$

Where Q is in m^3/s. A is in km^2.

7.2 EMPIRICAL FORMULA

The simplest of the empirical relationships are those which relate the flood peak to the drainage area. The various empirical 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

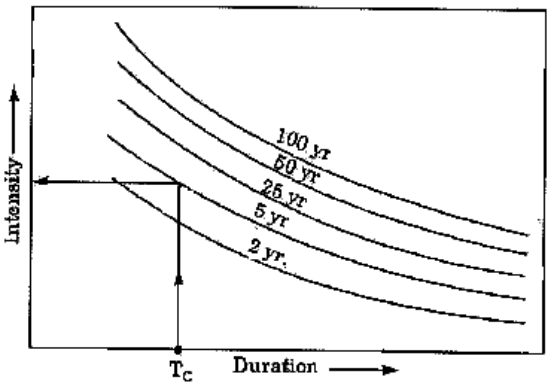
$$Q_p = \frac{124A}{\sqrt{A+10.4}}$$

Where, A is the catchment area in km^2. Q_p is peak flow in m^3/s.

7.3 RATIONAL METHOD

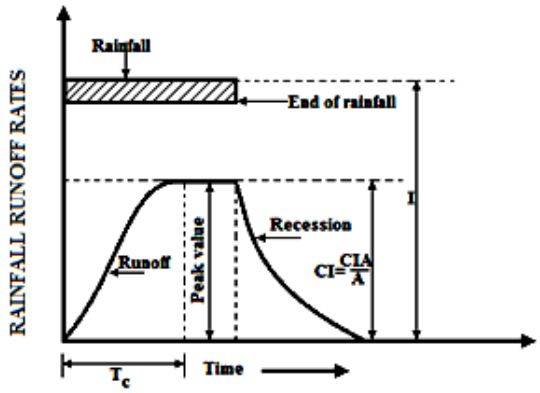
- We know that the maximum flood flow is produced by a certain rainfall intensity which lasts for a time equal to or greater than the period of concentration time.
- The concentration time is the maximum time required by the surface runoff to reach the basin outlet.
- When a storm continues beyond concentration 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

Intensity - Frequency - Duration Curves



RUNOFF HYDROGRAPH DUE TO UNIFORM RAINFALL

7.3.1 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.

7.3.2 TIME OF CONCENTRATION (t_c)

Time of concentration can be calculated using the following formula:-

7.3.3 US PRACTICE

For small drainage basins, the time of concentration is approximately equal to the lag time of the peak flow. Thus,

$$t_c = t_p = C_t L\left(\frac{LL_{ca}}{\sqrt{S}}\right)^n$$

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

$C_t L$ and n are basin constants.

7.3.4 KIRPICH EQUATION (1940)

t_c = **0.0078L**$^{0.77}$**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.

7.4 RUNOFF COEFFICIENT

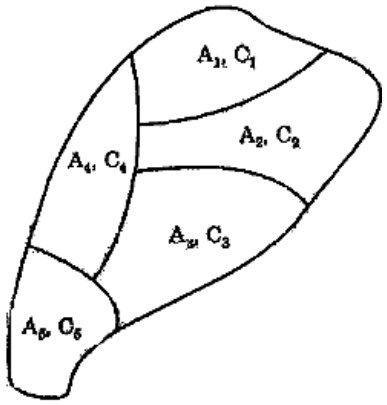
$$C = \frac{\text{runoff}}{\text{rainfall}} = \text{Runoff coefficient}$$

= impermeability factor.

If different portions of catchment have different runoff coefficient then C_{eq} is calculated as

$$C_{eq} = \frac{C_1A_1 + C_2A_2 + C_3A_3 + ...}{A_1 + A_2 + A_3 + ...}$$

Where C_i = Runoff coefficient pertaining to area A_i etc.



The rational formula is found to be suitable for peak-flow prediction in small catchments up to 50 km^2 in area. It finds considerable application in urban drainage designs and in the design of small culverts and bridges.

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 be used in frequency analysis.

Dams, diversion, urbanization and other land used changes introduce the inconsistency in data hence the data must be modified to bring it at par with the current condition of the catchment.

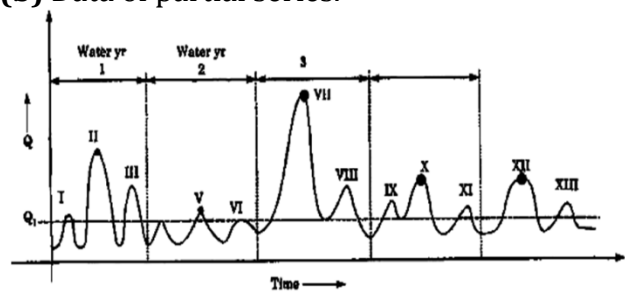
What data should be used for analysis?

If data for 50 years 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 (i.e. III) may be more than the peak of 2nd year (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 it required during the construction period of a large dam (say 4-5years), partial series are the best. While, for the spillway design flood, the annual series are preferred.

Hence normally for floods of large recurrence interval (i.e. greater than 20 yrs, or in other words flood of exceedance probability $\leq$ 0.05) annual data series is used and for small recurrence interval (4-5yrs) partial data series is used.

In our course we will concentrate only on annual data series.

7.5 ESTIMATION OF DESIGN FLOODS FOR A PARTICULAR RETURN PERIOD

Annual series are arranged in descending order of magnitude and rank no. is assigned 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 <u>Weibull's Formula</u>

$$T_r = \frac{n+1}{m}$$

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 empirical distribution.

This graph may be extrapolated or interpolated to get the design flood of any specific return period. We may also plot flood magnitude (y-axis, simple scale) and return period (x –axis, log – scale) and use it for extrapolation [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 distribution are:-

(a) Gumbel's distribution

(b) Log Pearson Type III distribution.

7.5.1 GUMBEL'S METHOD

As per Gumbel's method $X_T = \overline{X} + K.\sigma_{n-1}$

Where X_T = value of variate (i.e. flood) with a return period of T

$\overline{X}$ = Mean value of variate $\frac{\sum x}{n}$ = $\overline{X}$ (from annual series)

n = no. of yrs of record

σ_{n-1} = Standard deviation of the sample of size n

$$\sigma_{n-1} = \sqrt{\frac{\sum (X-\overline{X})^2}{n-1}}$$

K = Frequency factor

$$K = \frac{y_T - \overline{y}_n}{S_n}$$

y_T = reduced variate

$y_T = (-)\left[\text{In In}\frac{T}{T-1}\right]$

$\bar{y}_n$ = 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)

$y_n \longrightarrow 0.577$

$S_n \longrightarrow 1.2825$

[Normally for n > 50 also some time we use y_n = 0.57, S_n = 1.2825 without much error]

Hence the steps involved are:-

1. Assemble the discharge data and note the sample size n. Hence 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

SOME IMPORTANT POINTS

- For average of annual series, Gumbels method gives T = 2.33 yrs. Thus the value of flood with T = 2.33 yrs is called <u>mean annual flood.</u>
- X_T v/s T plot on Gumbel probability paper is a straight line. This property can be used for calculation of X_T for any T using Gumbel probability paper plot.

7.6 CONFIDENCE LIMIT

Due to limited data, the value of variate X_T determined using Gumbel'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

$X_{1/2} = X_T \pm f(\alpha).S_e$

Where $f(\alpha)$ = function of confidence probability 'α'. It can be found using the following table:

α in %	50	68	80	90	95	99
f(α)	0.674	1.0	1.282	1.645	1.96	2.58

S_e = Probable error = $b\frac{\sigma_{n-1}}{\sqrt{n}}$

$b = \sqrt{1+1.3K+1.1K^2}$

$K = \frac{y_T - \bar{y}_n}{S_n}$

$y_T = -\text{In In}\frac{T}{T-1}$

n = Sample size

T = Return period

σ_{n-1} = Standard deviation of sample

7.7 RISK RELIABILITYAND SAFETY FACTOR

The probability of occurrence of an event ($x \geq X_T$) 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 event $x \geq X_T$ in n years)

$\bar{R} = 1 - (1 - P)^n = \left[1 - \frac{1}{T}\right]^n$

Where

p = probability $p(x \geq X_T) = \frac{1}{T}$

T = return period

The reliability or assurance R_e, is defined as the probability that a particular event is

never equalled or exceeded in n successive years or assurance.

$$R_e = 1 - \overline{R} = \left[1 - \frac{1}{T}\right]^n$$

Safety factor and safety margin are defined as follows:

$$\text{Safety factor} = \frac{\text{design value}}{\text{estimated value}}$$

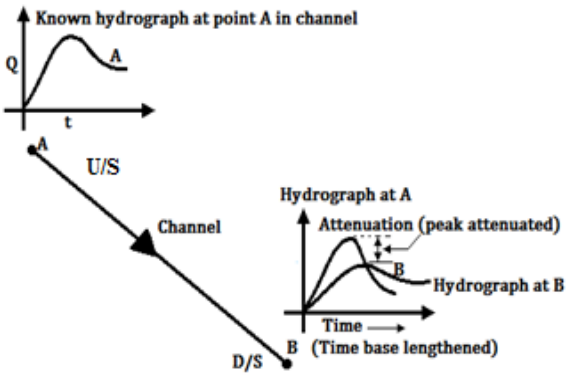
Safety margin = design value – estimated value.

8 FLOOD ROUTING

8.1 INTRODUCTION

Flood routing is a procedure whereby the shape of a flood hydrograph 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 behavior 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 (hydrological 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 continuity equation and momentum equation.

8.2 BASIC EQUATINS USED IN FLOOD ROUTING

1. Continuity equation:

$I - Q = \frac{ds}{dt}$ i.e. inflow-outflow = storage

For small time increment Δt

$$\left(\frac{I_1+I_2}{2}\right)\Delta t - \left[\frac{Q_1+Q_2}{2}\right]\Delta t = [S_2 - S_1] \quad (A)$$

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

$$\frac{\partial Q}{\partial x} + T\frac{\partial y}{\partial t} = 0 \qquad (B)$$

Where T = top surface width, y = depth of flow.

3. The momentum equation is given by

$$\left(\frac{\partial y}{\partial x} + \frac{V}{g}\frac{\partial V}{\partial x} + \frac{1}{g}\frac{\partial V}{\partial t} = S_O - S_f\right) \qquad (C)$$

V = velocity of flow,
S_O = bed slope,

S_f = energy line slope

8.3 VARIOUS METHODS OF RESERVOIR ROUTING

$$\left.\begin{array}{l}\text{a) Modified Pul's Method}\\ \text{b) Goodrich Method}\end{array}\right\}\text{(Graphical Method)}$$

c) Standard 4th order Rungle-Kutta method (Numerical method)

Modified Pul's Method

$$\frac{(I_1+I_2)}{2}\Delta t-\frac{(Q_1+Q_2)}{2}\Delta t=S_2-S_1$$

The 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 rearrange the term such that all known's are on one side and all unknown are on other side.

$$\left(\frac{I_1+I_2}{2}\right)\Delta t+\left(S_1-\frac{Q_1\Delta t}{2}\right)=\left(S_2+\frac{Q_2\Delta t}{2}\right)$$

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 $\left(S+\frac{Q\,\Delta t}{2}\right)$ = $\bar{f}$(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, $\left(S_1-\frac{Q\,\Delta t}{2}\right)$ 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.

Goodrich method: (Hydrological reservoir routing)

$$\left(\frac{I_1+I_2}{2}\right)\Delta t-\left(\frac{Q_1+Q_2}{2}\right)\Delta t=S_2-S_1$$

$$(I_1+I_2)+\left(\frac{2S_1}{\Delta t}-Q_1\right)=\left(\frac{2S_2}{\Delta t}+Q_2\right)$$

All terms is 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.

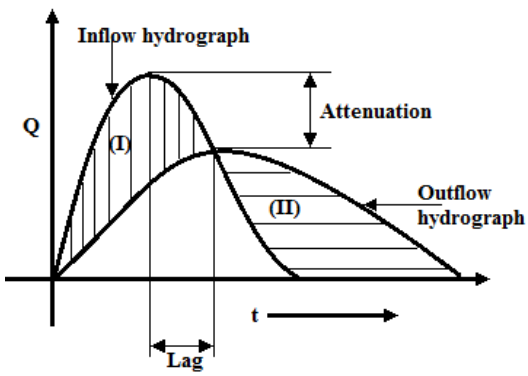
Hence discharge, elevation and storage at the end of time increment is known.

For next time increment

$\left[\left(\frac{2S}{\Delta t}+Q\right)_2-2Q_2\right]$ Of previous time increment = $\left(\frac{2S}{\Delta t}-Q\right)_1$ for use as initial value.

8.4 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

S = S(h)

$\frac{dS}{dt} = A. \frac{dH}{dt}$, A = Area of reservoir at elevation H . . . . . (α)

The outflow is also a function of H

Q = Q(H)

$\frac{dQ}{dt} = 0$ at peak flow

$\frac{dH}{dt} = 0$ and $\frac{dS}{dt} = 0$ at peak flow [from (α)]

At peak flow storage is max also

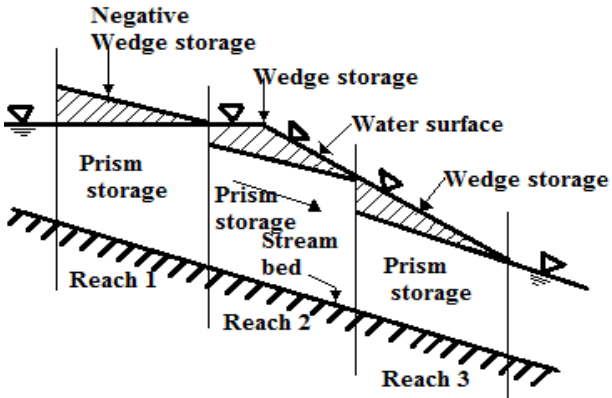
$I - Q = \frac{dS}{dt}$

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.

8.5 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 stage the wedge storage volume is considerable while during falling stages, as inflow drops more rapidly than outflow, the wedge storage becomes negatives.



Flood is in rising in reach 2 and reach 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 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 & other roughness characteristics.

Note:- By choosing arbitrary value of x between 0 – 0.5 and by finding out accumulated storage as $\sum(I-Q)\Delta t$, 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 constant.

8.5.1 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

$S_2 - S_1 = K[x(I_2 - I_1)+(1 - x)Q_2 - Q_1]$ ----(A)

Where suffixes 1 and 2 refer to the conditions before and after the time interval Δt.

The continuity equation for the reach is

$$S_2 - S_1 = \left(\frac{I_1+I_2}{2}\right)\Delta t - \left(\frac{Q_1+Q_2}{2}\right)\Delta t \text{ --- (B)}$$

From equations (A) and (B), Q_2 is evaluated as

$Q_2 = C_0 I_2 + C_1 I_1 + C_2 Q_1$ ------(C)

(Muskingum routing equation)

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 constants for a reach. In general, for the nth 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 appropriate 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.

FLOOD CONTROL

(1) Structural methods

(a) Flood control reservoirs
(i) Detention reservoir - gated hence controlled outlet
(ii) Retarding basin - ungated hence uncontrolled outlet
(b) Levees (flood embankments)
(c) Channel improvement
(d) Flood ways
(e) Soil conservation

(2)Non - structural methods

(a) Flood plain zoning
(b) Flood warning, evacuation and relocation

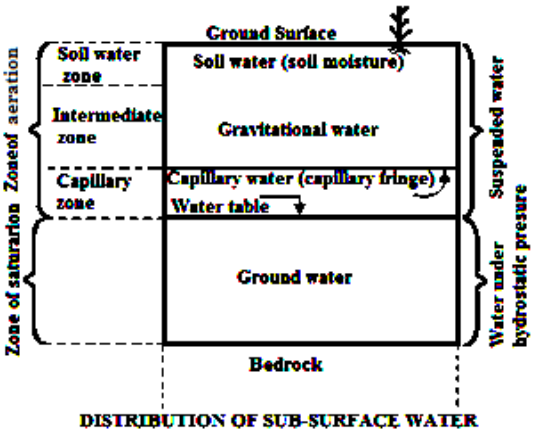
9 GROUND WATER

9.1 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.**



DISTRIBUTION OF SUB-SURFACE WATER

9.1.1 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.

9.1.2 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 meters.
- All the infiltration water must pass through this region.

9.1.3 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.

9.1.4 SATURATED ZONE

In the saturated zone, groundwater fills the pore space 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.

The top surface of zone of saturation or ground water is known as phreatic surface.

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 is classified into four categories:- 1.Aquifer, 2.Aquitard, 3.Aquiclude, 4.Aquifuge.

9.1.5 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.

9.1.6 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.

9.1.7 AQUICLUDE

Formations like clay which is highly porous but not permeable due to very small size of pores.

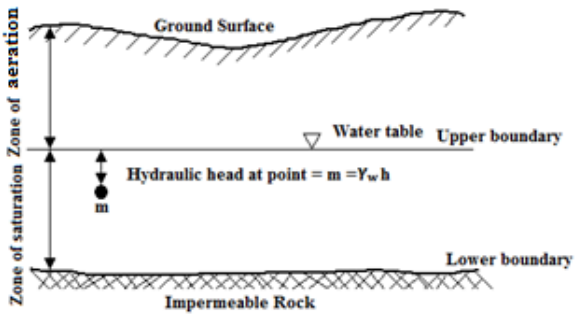
9.1.8 AQUIFUGE

It is a geological formation which is neither porous nor permeable. Massive compact rock without any fractures is an Aquifuge.

9.2 TYPES OF AQUIFERS

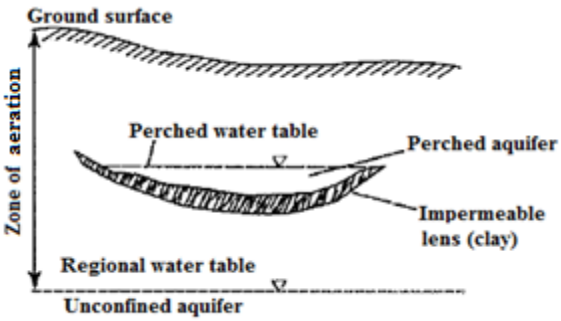
9.2.1 UNCONFINED AQUIFERS

- Unconfined aquifers are 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 atmosphere 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.



UNCONFINED AQUIFER

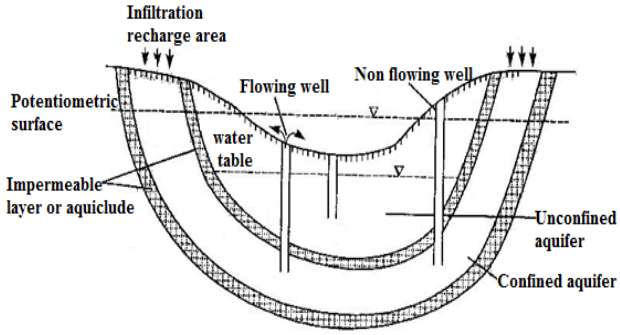
- In unconfined aquifer the water table goes down if water is withdrawn from the aquifer storage and the water table moves up if water is added into the aquifer storage.
- The water level in a large diameter dug wells tapping unconfined aquifer represents water table. This aquifer is also known as water 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.



PERCHED AQUIFER

9.2.2 CONFINED AQUIFERS

- A confined aquifer (also called artisan aquifer) is the one which is overland by an impermeable layer or an Aquiclude. Unlike the unconfined aquifer, the water in the confined aquifer is not in direct contact with the atmosphere.



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 up to 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.

9.3 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.

9.3.1 POROSITY

The amount of pore space per unit volume of the aquifer material is called porosity. It is expressed as

$$n = \frac{V\upsilon}{V_O}$$

Where n = porosity, $V\upsilon$ = volume of voids and V_O = volume of the porous medium.

9.3.2 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 – 65	1 – 10
Sand	35 – 40	10 – 30
Gravel	30 – 40	15 – 30

Note that although clay is more porous than sand, the specific yield of clay is very small compared to that of sand.

9.3.3 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 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. 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 piezometric 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.

9.3.4 DARCY'S LAW

V = K i (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's law is valid for Re < I

$$Re = \frac{Vd_a}{v}$$

Where,

Re = Reynolds 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.

v = 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

$$\upsilon_a = \frac{V}{n}$$

Where n = porosity. The actual velocity υ_a is the velocity that is obtained by tracking a tracer added to the groundwater.

9.3.5 COEFFICIENT OF PERMEABILITY

The coefficient of permeability, also designated as hydraulic conductivity reflects the combined effects of the porous medium and fluid properties. The coefficient of permeability K can be expressed as

$$K = Cd_m^2 \frac{\gamma}{\mu}$$

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.

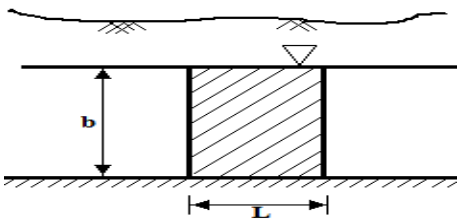
The coefficient of permeability is often considered in two components, one reflecting the properties of the medium only and the other incorporating the fluid properties.

$$K = K_O \frac{\gamma}{\mu}$$

Where,

$K_O = Cd_m^2$ The parameter K_O 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 m².

9.3.6 COEFFICIENT OF TRANSMISSIBILITY (T)



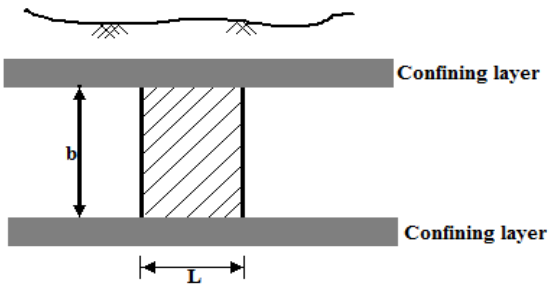
UNCONFINED AQUIFER

Area of flow = BL

Q = KiA = KibL= (Kb) iL

Q = TiL

T is called coefficient of transmissibility.



CONFINED AQUIFER

Q = TiL, T = Kb

T is called coefficient of transmissibility.

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].

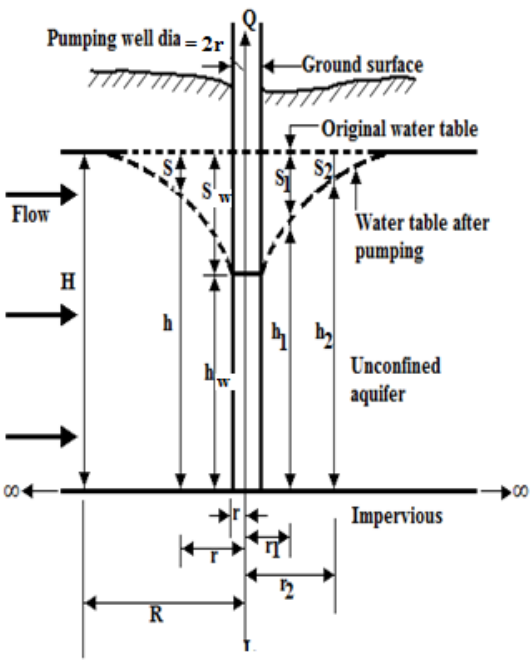
T has a unit of (length)² / time. Storage coefficient (s) and Transmissibility coefficient (t) are known as formation constants of an aquifer and plays an important role in the unsteady flow through porous media. Storage coefficient and transmissibility or transmissivity is determined in the field by carrying out pumping test on wells and measuring the discharge and lowering of water levels in the observation wells.

9.4 WELLS

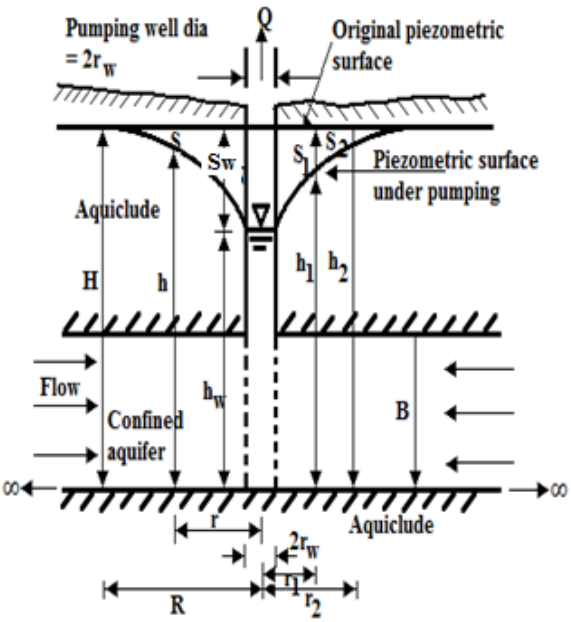
- Wells form the most important mode of groundwater extraction from an aquifer. While wells are used in a number of different applications, they find extensive use in water supply and irrigation engineering practice.
- For an unconfined aquifer, prior to the pumping, the water level in the well indicates 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 drawdown.
- 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 drawdown 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 ground water 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 ground water 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 drawdown with the development of the cone of depression.
- In confined aquifers with considerable piezometric head, the recovery into the well takes place at a very rapid rate.



WELL OPERATING IN AN UNCONFINED AQUIFER



WELL OPERATING IN A CONFINED AQUIFER

9.4.1 STEADY CONFINED FLOW (FULLY PENETRATING WELL)

Full penetrating well means the well which penetrates up to the bottom of the aquifer so that flow is more or less radial.

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. $\frac{Q}{2\pi KB}\ln\frac{r_2}{r_1}$

$=(h_2 - h_1)$

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

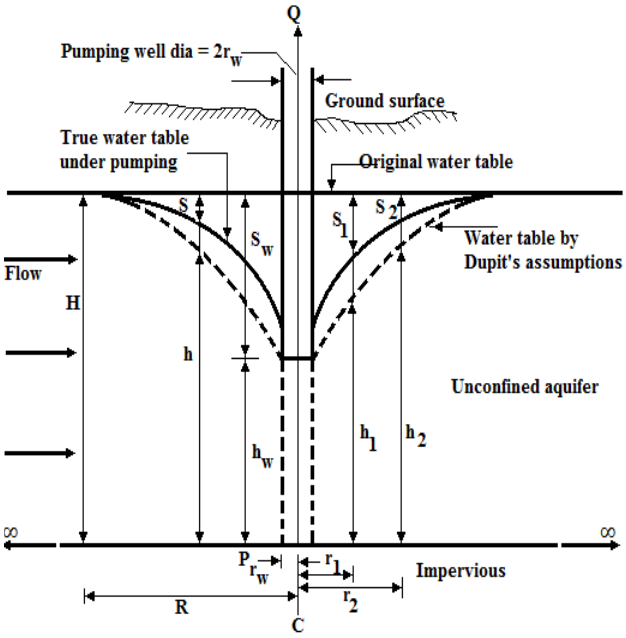
This is the equilibrium equation for the steady flow in a confined aquifer. **This equation is popularly known as Thiem's equation.**

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

$$Q = \frac{2\pi KBS_w}{\ln\frac{R}{r_w}}$$

This is called Dupit's Formula

9.4.2 STEADY UNCONFINED FLOW



RADIAL FLOW TO A WELL IN AN UNCONFINED AQUIFER

$$Q = \frac{\pi K(H^2 - h_w^2)}{\ln\frac{R}{r_w}}$$

R is normally between 300-500 m.

Where h_w = depth of water in the pumping well of radius r_w.

NON-FQUILIBRIUM 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

$$s = \frac{Q}{4\pi T}\left[\log_e\frac{4Tt}{r^2.S} - 0.5722\right] = \frac{Q}{4\pi T}\times$$ **[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

$$S_2 - S_1 = \frac{Q}{4\pi T}\log_e \frac{t_2}{t_1}$$

9.5 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.

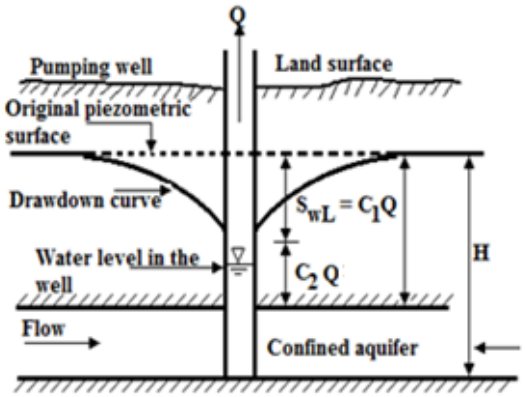
9.6 WELL LOSS

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,
   $S_{wL} \propto Q$

and (s_{wt} and s_{wc}) $\propto Q^2$
Thus, $s_w = C_1Q + C_2Q^2 = \text{total loss}$
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



DEFINATION SKETCH FOR WELL LOSS

indicates clogging of well screens etc. and requires immediate remedial action.

9.6.1 SPECIFIC CAPACITY

The specific capacity of a well is defined as the well yield per unit of drawdown. Hence, the

$$\text{Sp. Capacity} = \frac{\text{Discharge well}}{\text{Drawdown}}$$

$$= \frac{Q}{C_1Q + C_2.Q^2}$$

$$\text{Sp. Capacity} = \frac{1}{C_1 + C_2.Q}$$

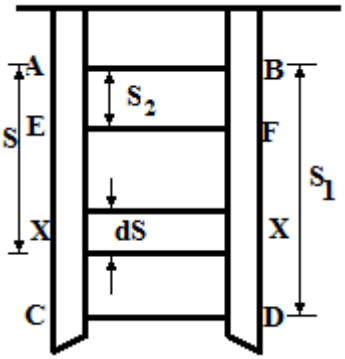
The equation clearly shows that the sp. Capacity of the well is not constant but decreases as the discharge increase.

9.6.2 SAFE YIELD

The maximum rate at which withdrawal of groundwater in a basin can be carried without producing undesirable results is termed safe yield. The "undesirable" results inside (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.

9.7 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 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.

C'' is called the specific capacity of the open well. Knowing 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

Or

$$Q = \left(\frac{C''}{A}\right)A.s$$

Or

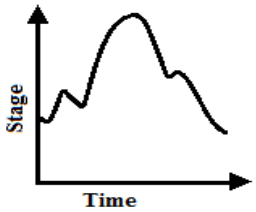
$$Q = \left(\frac{2.3'}{T}\log_{10}\frac{s_1}{s_2}\right)A.s$$

A and s are known, the discharge for any amount of drawdown (s) can be easily worked out. .

10 STREAM FLOW MEASUREMENTS

10.1 Measurement of Velocity (v)

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:-
  i) Vertical axis meters
  ii) 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.
- Numbers of revolutions are counted for a known interval of time and from this N_s and V is known.

10.2 VELOCITY MEASUREMENT BY FLOATS

When distance travelled (s) by a floating object in time (t) is noted, we can get surface velocity

$$\upsilon_s = \frac{s}{t}$$

From surface velocity average velocity is found out using correction factor

$$\bar{v} = K.v_s$$

This method is useful for:-
a) Small stream in floods
b) Preliminary survey

10.3 DISCHARGE MEASUREMENTS

1. Direct determination of stream discharge is done using:-
   a) Area-velocity methods-Moving boat method
   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.

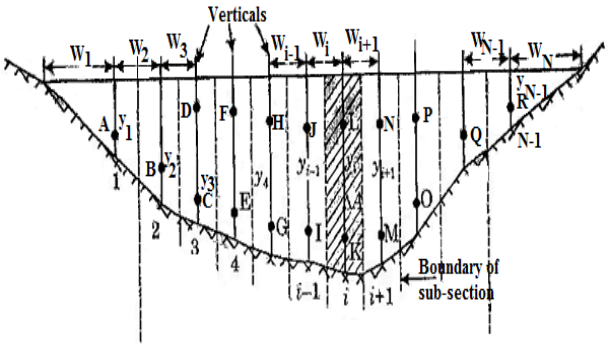
10.4 Direct Method

1) Area Velocity Method

In this method whole c/s of river is divided into number of sections and sectional discharge is calculated and total discharge is equal to summation of sectional discharges.

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

$$\bar{v} = \frac{V_{0.2} + V_{0.8}}{2} \text{ Or } V_{0.6} \text{ as the case may be.}$$



STREAM SECTION FOR AREA - VELOCITY METHOD

In the above figure discharge is calculated in three parts and added together to get total discharge.

(i) For calculating discharge the section of river or stream is divided in to number of segment according to the following condition.
(ii) Discharge in each segment must be less than 10 % of the total discharge.
(2) Difference in velocities for adjacent segment should not be more than 20 %
(3) The segmental width should not be more than $\frac{1}{15}th$ to $\frac{1}{20}th$ of the riverbed.

$\Delta Q_1,\ \Delta Q_2, \ldots\ldots$ be the segmental discharge then, total discharge for the stream is given by the summation of $\Delta Q_1,\ \Delta Q_2, \ldots\ldots$

$$\sum(\Delta Q_1 + \Delta Q_2 + \Delta Q_3 + \ldots\ldots)$$

Average width for first section,

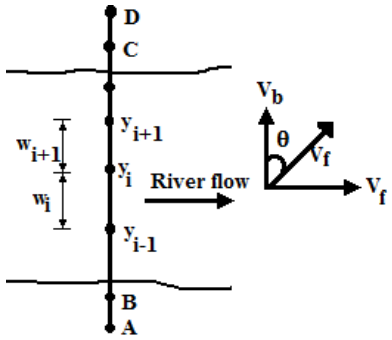
$$\overline{w_1} = \frac{\left(w_1 + \frac{w_2}{2}\right)^2}{2w_1}$$

Average width for last section,

$$\overline{w_{n-1}} = \frac{\left(w_n + \frac{w_{n-1}}{2}\right)^2}{2w_n}$$

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 up to a particular fixed depth below meter 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 arrangement. 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 calculating as $Q = \sum \Delta Q_i$

Where, $\Delta Q_i = \frac{y_i + y_{i+1}}{2} W_{i+1} \times V_f$

$$= \left(\frac{y_i + y_{i+1}}{2}\right)[(V_r\cos\theta)(\Delta t)](V_r\sin\theta)$$

$$\Delta Q_i = \left(\frac{y_i + y_{i+1}}{2}\right) V_r^2 \cos\theta\sin\theta \,.\, \Delta t$$

Moving boat method is not recommended in stream with water depth less than 3 m..

2) DILUTION METHOD

In this method a chemical compound called tracer such as sodium chloride, sodium dichromate, colour dye or some other radioactive material is introduced in the stream at the given location.

3) ULTRASONIC METHOD

It is used to measure the velocity of flow at certain depth in channel by simultaneously transmitting sound pulses through the water from transducers located in the banks on either side of the river.
It is particularly applicable to rivers up to about 300m or more in width, where
(a) There is no stable stage - discharge relation
(b) A measuring structure is unsuitable or not feasible.

4) ELECTROMAGNETIC METHOD

In this method a long conductor is buried 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 which is related to the discharge in the river.

10.5 INDIRECT METHOD

SLOPE AREA METHOD

When it is not possible to make direct measurement especially during floods, slope-area method is a most commonly used indirect method of measurement. In this method discharge is computed on the basis of a uniform flow equation involving channel characteristics, water surface profile and a roughness coefficient. The drop in water surface profile for a uniform reach of channel represents losses caused by bed roughness. The well known Manning's equation is used in this method. The discharge is given by equation,

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

10.6 STAGE DISCHARGE RELATIONSHIP

After a sufficient number of discharge measurements have been made at a gauging station along with 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 exists between discharge and stage.

10.7 PERMANENT CONTROL

For permanent control the relationship between the stage and the discharge is single-valued relation which is expressed as:-

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

In which Q = stream discharge, G = gauge height (stage), α=a constant which represent the gauge reading corresponding to zero discharge, C_r and β are rating curve constants.
By taking logarithms, we get

$$\text{Log } Q = \beta \log (G - a) + \log C_r$$

Or

$Y = \beta X + b$

For a given series of data, β and C_r are found out using regression analysis.

And $b = \frac{\sum y - \beta\left(\sum X\right)}{N}$

The coefficient of correlation r is given by

$$r = \frac{N\left(\sum XY\right) - \left(\sum X\right)\left(\sum Y\right)}{\sqrt{N\left(\sum X^2\right) - \left(\sum X\right)^2}\sqrt{N\left(\sum Y^2\right) - \left(\sum Y\right)^2}}$$

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 α representing the stage (gauge height) for zero discharge in the stream is a hypothetical parameter and cannot be measured in the field. It is calculated by some specialised method which is beyond the scope of this chapter.

GATE QUESTIONS

Topics

1. PRECIPITATION AND GENERAL ASPECTS OF HYDROLOGY
2. EVAPORATION AND ITS MEASUREMENTS
3. INFILTRATION, RUNOFF AND HYDROGRAPHS
4. FLOODS, FLOOD ROUTING AND FLOOD CONTROL

1 PRECIPITATION & GENERAL ASPECTS OF HYDROLOGY

Q.1 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 10ha-m and 20ha-m 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 12cm and 3cm respectively. The calibrated pan factor is 0.7. The seepage loss from the reservoir during this period in hecter-meters is

a) 0.0 b) 1.0
c) 2.4 d) 4.6

[GATE-2003]

Q.2 Match list-I with List-II and select the correct answer using the codes given below the list.

List-I	**List-II**
A. Rainfall intensity	1. Isohyets
B. Rainfall excess	2. Cumulative rainfall
C. Rainfall averaging	3. Hyetograph
D. mass curve	4.direct runoff hyetograph

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

[GATE-2003]

Q.3 The intensity of rainfall and time interval of typical storm are:

Time interval (min)	Intensity of rainfall (mm/min)
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 min duration of the storm is

a) 1.5 mm/min b) 1.85 mm/min
c) 2.2 mm/min d) 3.7 mm/min

[GATE-2005]

Q.4 An isohyet is a line joining points of

a) Equal temperature
b) Equal humidity
c) Equal rainfall depth
d) Equal evaporation

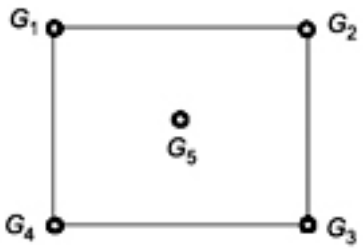
[GATE-2013]

Q.5 In a catchment, there are four rain-gauge stations, P, Q, R, and S. Normal annual precipitation values at these stations are 780 mm, 850 mm, 920 mm, and 980 mm, respectively. In the year 2013, stations Q, R, and S, were operative but P was not. Using the normal ratio method, the precipitation at station P for the year 2013 has been estimated as 860 mm. If the observed precipitation at stations Q and R for the year 2013 were 930 mm and 1010 mm, respectively; what was

the observed precipitation (in mm) at station S for that year?

[GATE-2015]

Q.6 A catchment is idealized as a 25 km x 25 km square. It has five rain gauges, one at each corner and one at the center, as shown in the figure.



During a month, the precipitation at these gauges is measured as G_1 = 300 mm, G_2 = 285 mm, G_3 = 272 mm, G_4 = 290 mm and G_5 = 288 mm. The average precipitation (in mm, up to one decimal place) over the catchment during this month by using the Thiessen polygon method is _____.

[GATE-2017]

Q.7 Rainfall depth over a watershed is monitored through six numbers of well distributed rain gauges. Gauged data are given below :

Rain Gauge Number	1	2	3	4	5	6
Rain Depth (mm)	470	465	435	525	480	510
Area of Thiessen Polygon (x $10^4 m^2$)	95	100	98	80	85	92

The Thiessen mean value (in mm. up to one decimal place) of the rainfall is _____.

[GATE-2018]

ANSWER KEY:

1	2	3	4	5	6	7							
(d)	(b)	(b)	(c)	**1076.2**	**287.375**	**479.09**							

EXPLANATIONS

Q.1 (d)

Inflow to reservoir, I = 10 ha-m
Outflow from reservoir,
O = 20 ha–m
Area A=1km^2 = 10^6 m^2
Loss due to evaporation,
E = 12 cm × pan coefficient × Area
= 12 × 10^{-2} A × 0.7
= 12 × 10^{-2} × 10^6 × 0.7
= 8.4 × 10^4 m^3 = 8.4 ha–m
Rainfall,
P = 3 cm
= 0.03 × 10^6 m^3
= 3 × 10^4 m^3 = 3 ha–m
Change in storage,
ΔS = -20 cm
= -0.20 × 10^6 m^3
= -20 ha-m
Inflow – outflow = change in storage
(I + P) – (O + E + seepage)= -20
(10 + 3) – (20 + 8.4 + seepage) = -20
Seepage = 4.6 ha-m

Q.2 (b)

Rainfall intensity = Hyetograph (intensity versus time curve)
Rainfall excess – Direct runoff hydrograph
Rainfall averaging – Isohytes (Isohytes is the line of having equal rainfall depth and used to measure the average rainfall over an catchment)
Mass cure – Cumulative Rainfall

Q.3 (b)

Time interval(min)	rainfall (mm/ min)
0 – 20	0.7×10+1.1×10=18
10 – 30	1.1×10+2.2×10=33
20 – 40	2.2×10+1.5×10=37
30 – 50	1.5×10+1.2×10=27
40 – 60	1.2×10+1.3×10=25
50 – 70	1.3×10+0.9×10=22
60 – 80	0.9×10+0.4×10=13

Maximum rainfall in 20 min = 37 mm

$$\text{Intensity} = \frac{37}{20} = 1.85\text{mm / min}$$

Q.4 (c)

Line joining the points of equal rainfall depth is called isohyets.

Q.5 (1076.2)

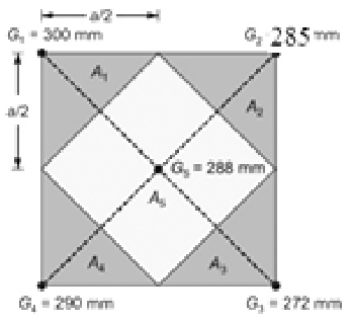
P_X = Precipitation in 2013 at station X
N_x = Normal annual precipitation at station X

$$\frac{P_s}{N_s} = \frac{1}{3}\left[\frac{P_P}{N_P} + \frac{P_Q}{N_Q} + \frac{P_R}{N_R}\right]$$

$$\Rightarrow \frac{P_s}{980_s} = \frac{1}{3}\left[\frac{860}{780} + \frac{930}{850} + \frac{1010}{920}\right]$$

$$\Rightarrow P_s = 1076.20 \text{ mm}$$

Q. 6 (287.375)



Let sides of square as, a = 25 km

Now area of the polygon is calculated as

$A_1 = A_2 = A_3 = A_4$

$= \frac{1}{2} \times \frac{a}{2} \times \frac{a}{2} = \frac{a^2}{2} = \frac{25^2}{8} = 78.125$

A_5 = Area of square of sides $\frac{\sqrt{2}a}{2} = \frac{a}{\sqrt{2}}$

$= \frac{a^2}{2} = \frac{25^2}{2} = 312.5 \text{ km}^2$

$\therefore$ Mean precipitation, $\bar{P}$

$$= \frac{G_1 A_1 + G_2 A_2 + G_3 A_3 + G_4 A_4 + G_5 A_5}{A_1 + A_3 + A_3 + A_4 + A_5}$$

$$\therefore \bar{P} = \frac{(300 + 285 + 272 + 290) \times 78.125 + 288 + 312.5}{(4 \times 78.125) + 125.5}$$

$= 287.375$ mm

Q. 7 (479.09)

Thiessen mean value of rainfall ;

$$P_{avg} = \frac{\sum_{i=1}^{6} P_i A_i}{\sum_{i=1}^{6} A_i}$$

$$\Rightarrow P_{avg} = \frac{470 \times 95 + 465 \times 100 + 435 \times 98 + 525 \times 80 + 480 \times 85 + 510 \times 92}{95 + 100 + 98 + 80 + 85 + 92}$$

$= 479.09$ mm

2 EVAPORATION AND ITS MEASUREMENTS

Q.1 Isopleths are lines on a map through points having equal depth of
a) Rainfall
b) Infiltration
c) Evapotranspiration
d) Total Runoff

Q.2 The average surface area of a reservoir in the month of June is 20 km^2. In the same month, the average rate of inflow is $10m^3/s$, outflow rate is $15m^3/s$, monthly rainfall is 10 cm, monthly seepage loss is 1.8 cm and the storage change is 16 million m^3. The evaporation (in cm) in that month is
a) 46.8 b) 136.0
c) 13.6 d) 23.4
[GATE-15]

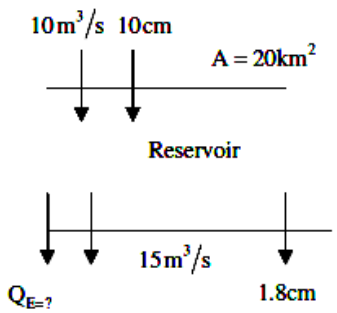
ANSWER KEY:

1	2												
(c)	(d)												

EXPLANATIONS

Q.1 **(c)**
The mean annual PET (in cm) over various parts of the country is shown in the from of isopleths. Isopleths are the lines on a map though places having equal depth of "evapotransipiration"

Q.2 **(d)**



Let 'x' evaporation takes takes place in month of June.

Total inflow

$$\frac{10 \times 30 \times 24 \times 60 \times 60}{20 \times 10^6} \times 100 + 10$$

$$= 139.6\,\text{cm}$$

Total outflow

$$\frac{15 \times 30 \times 24 \times 60 \times 60}{20 \times 10^6} \times 100 + 1.8 + x$$

$$= 196.2 + \text{cm}$$

As total outflow is more than total inflow therefore depression in storage takes place.
Depression in storage

$$= \frac{16 \times 10^6}{20 \times 10^6} \times 100 = -80 \text{ cm}$$

$$\Rightarrow \quad 139.6 - (196.2 + x) = -80 \text{ cm}$$

$$-x = -80 + 56.6$$

$$\therefore \quad x = 23.4 \text{ cm}$$

3 INFILTRATION, RUNOFF AND HYDROGRAPHS

Common data for Questions 1 and 2.

An average rainfall of 16 cm occurs over a catchment during a period of 12 h with a uniform intensity.

The unit hydrograph (Unit depth = 1 cm, duration = 6h) of the catchment rises linearly from 0 to 30 cumec in 6 h and then falls linearly from 30 to 0 cumecs in the next 12 h. ϕ index of the catchment is known to be 0.5 cm/h. Base flow in the river is known to be 5 cumec.

Q.1 Peak discharge of the resulting direct runoff hydrograph shall be

a) 150 cumec b) 225 cumec
c) 230 cumec d) 360 cumec

[GATE-2003]

Q.2 Area of the catchment in hectare is

a) 97.20 b) 270
c) 9720 d) 27000

[GATE-2003]

Q.3 While applying the rational formula for computing the design discharge, the rainfall duration is stipulated as the time of concentration because

a) this lead to the largest possible rainfall intensity
b) this lead to the smallest possible rainfall intensity
c) this time of concentration is the smallest rainfall duration for which the rational formula is applicable.
d) this time of concentration is the largest rainfall duration for which the rational formula is applicable.

[GATE-2003]

Q.4 The vertical hydraulic conductivity of the top soil at certain stage is 0.2. cm/ h. A storm of intensity 0.5 cm/h 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/h
b) 0.2 cm/h
c) between 0.2 and 0.5 cm/h
d) 0.5 cm/h

[GATE-2003]

Q.5 The average rainfall of a 3 h duration storm is 2.7 cm and the loss rate is 0.3 cm/h. 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

[GATE-2004]

Q.6 The rainfall during three successive 2 h 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/h b)0.28 cm/h
c)0.30 cm/h d)0.80 cm/h

[GATE-2004]

Statements for Linked Answer Q.7 & Q.8

A 4h unit hydrograph of a catchment is triangular in shape with base of 80 h. The area of the catchment is 720 km^2. The base flow and ϕ index are 30m^3/s and 1mm/h, respectively. A storm of 4 cm occurs uniformly in 4 h over the catchment.

Q.7 The peak discharge of 4 h unit hydrograph is

a) 40 m^3/s b) 50 m^3/s

c) 60 m^3/s d) 70 m^3/s

[GATE-2005]

Q.8 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

[GATE-2005]

Q.9 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 hydrograph
b) a point, after the intersection of the inflow & outflow hydrographs
c) the tail of inflow hydrograph
d) a point, before the intersection of the inflow & outflow hydrographs

[GATE-2005]

Common data for Questions 10 and 11.

For a catchment, the S curve (or S hydrograph) due to a rainfall of intensity 1 cm/h is given by Q = 1 – (1 + t) exp (-t) (t is in hour and Q is in m^3/s).

Q.10 What is the area of the catchment?

a)0.01 km^2 b)0.36 km^2
c)1.00 km^2 d)1.28 km^2

[GATE-2006]

Q.11 What will be the ordinate of a 2 hour unit hydrograph for his catchment at t = 3 hour?

a)0.13 m^3/s b)0.20 m^3/s
c)0.27 m^3/s d)0.54 m^3/s

[GATE-2006]

Q.12 During 3 h storm event, it was observed that all abstractions other than infiltration are negligible. The rainfall was idealized as three 1 h storms of intensities 10 mm/h, 20 mm/h and 10 mm/h respectively and the infiltration was idealized as a Horton curve, f = 6.8 + 8.7 exp (-t) (t is in mm/h and t is in h). What is the effective rainfall?

a)10.00 mm b)11.33 mm
c)12.43 mm d)13.63 mm

[GATE-2006]

Q.13 An isolated 4 hour storm occurred over a catchment as follows:

Time	1st hr	2nd hr	3rd hr	4th hr
Rainf all (mm)	9	28	12	7

The Ø-index for the catchment is 10mm/hr. The estimated runoff depth from the catchment due to the abovestorm is

a)10 mm b)16 mm
c)20 mm d)23 mm

[GATE-2007]

Common Data for questions 14 and 15.

Ordinates of a 1-hour unit hydrograph at 1 hour intervals, starting from time t = 0, Are 0, 2, 6, 4, 2, 1 and 0 m^3/s.

Q.14 Catchment area represented 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

[GATE-2007]

Q.15 Ordinate of 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

[GATE-2007]

Common Data for Question 16 and 17

1h triangular unit hydrograph of a watershed has the peak discharge of 60 m^3/s at 10 h and time base of 30 h. The ϕ index is 0.4 cm/h and base flow is 15 m^3/s.

Q.16 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

[GATE-2009]

Q.17 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/s b) 240 m^3/s
c) 249 m^3/s d) 258 m^3/s

[GATE-2009]

Q.18 Match List-I with List-II and select the correct answer using the codes given below the lists.

List – I

A. Evapo-transpiration
B. Infiltration
C. Synthetic unit hydrograph
D. Channel routing

List – II

1. Penman method
2. Synder's method
3. Muskingum method
4. Horton's method

Codes:

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

[GATE-2010]

Common Data for Questions 19 and 20.

The ordinates of a 2 h unit hydrograph at 1 h intervals starting from time = 0 are 0, 3, 8, 6, 3, 2 and 0 m^3/s. Use trapezoidal rule for numerical integration, if required.

Q.19 What is the catchment area represented by the unit hydrograph?

a) 1.00 km^2 b) 2.00 km^2
c) 7.92 km^2 d) 8.64 km^2

[GATE-2011]

Q.20 A storm of 6.6 cm occurs uniformly over the **Catchment in 3h. If** Ø index is equal to 2mm/h and base flow is 5m^3/s, what is the peak flow due to the storm?

a) 41.0 m^3/s b) 43.4 m^3/s
c) 53.0 m^3/s d) 56.2 m^3/s

[GATE-2011]

Q.21 A watershed got transformed from rural to urban over a period of time. The effect of Urbanization on storm runoff hydrograph from the watershed is to

a) decrease the volume of runoff
b) increase the time to peak discharge
c) decrease the time base
d) decrease the peak discharge

[GATE-2011]

Statement for Linked Answer Question 22 and 23

The drainage area of a watershed is 50 km^2. The Ø index is 0.5 cm/hour and the base flow at the outlet is 10m^3/s. One hour unit hydrograph (unit depth=1 cm) of the Watershed is triangular in shape with a time base of 15 hours. The peak ordinate occurs at 5 hours.

Q.22 The peak ordinate (in $m^3/s/cm$) of the unit hydrograph is

a) 10.00 b) 18.52
c) 37.03 d) 185.20

[GATE-2012]

Q.23 For a storm of depth of 5.5 cm and duration of 1 hour, the peak ordinate (in m^3/s) of the hydrograph is

a) 55.00 b) 82.60
c) 92.60 d) 102.60

[GATE-2012]

Q.24 The ratio of actual evapotranspiration to potential evapotranspiration is in the range

a) 0.0 to 0.4 b) 0.6 to 0.9
c) 0.0 to 1.0 d) 1.0 to 2.0

[GATE-2012]

Q.25 **Group I** contains parameters and **Group II** lists method/instruments.

Group I

P. Stream flow velocity
Q. Evapo-transpiration rate
R. Infiltration rate
S. Wind velocity

Group II

1. Anemometer
2. Penman's method
3. Horton's method
4. Current meter

The **CORRECT** match of **Group I** with **Group II** is

a) P – 1, Q – 2, R – 3, S – 4
b) P – 4, Q – 3, R – 2, S – 1
c) P – 4, Q – 2, R – 3, S – 1
d) P – 1, Q – 3, R – 2, S – 4

[GATE-2012]

Statement for Linked answer Q.26 & Q.27.
At a station, storm I and 5 hour duration with intensity 2 cm/h resulted in a runoff of 4 cm and storm II of 8 hour duration resulted in a runoff of 8.4 cm. Assume that the ϕ-index is the same for both the storms.

Q.26 The ϕ-index (in cm/hr) is
a) 1.2 b) 1.4
c) 1.0 d) 1.6

[GATE-2013]

Q.27 The intensity of storm II (in cm/h) is:
a) 2.00 b) 1.75
c) 1.50 d) 2.25

[GATE-2013]

Q.28 In reservoirs with an uncontrolled spillway, the peak of the plotted outflow hydrograph
a) lies outside the plotted inflow hydrograph
b) lies on the recession limb of the inflow hydrograph
c) lies on the peak of the inflow hydrograph
d) is higher than the peak of the plotted inflow hydrograph

[GATE-2014]

Q.29 An isolated 3-h rainfall event on a small catchment produces a hydrograph peak and point of inflection on the falling limb of the hydrograph at 7 hours and 8.5 hours respectively, after the start of the rainfall. Assuming, no losses and no base flow contribution, the time of concentration (in hours) for this catchment is approximately
a) 8.5 b) 7.0
c) 6.5 d) 5.5

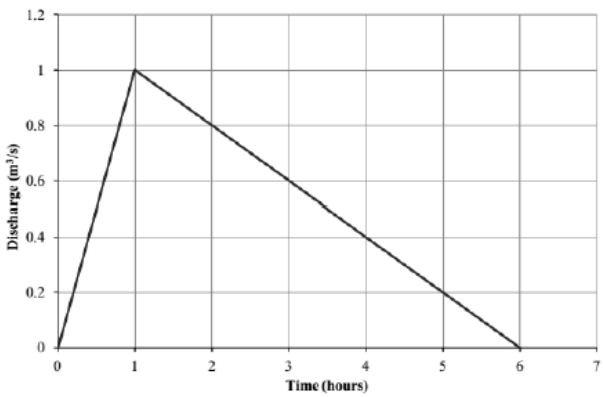
[GATE-2014]

Q.30 The 4-hr unit hydrograph for a catchment is given in the table below. What would be the maximum ordinate of the S-curve (in m^3 / s) derived from this hydrograph?

Time (hr)	Unit hydrograph ordinate (m^3/s)
0	0
2	0.6
4	3.1
6	10
8	13
10	9
12	5
14	2
16	0.7
18	0.3
20	0.2
22	0.1
24	0

[GATE-2015]

Q.31 The direct runoff hydrograph in response to 5 cm rainfall excess in a catchment is shown in the figure. The area of the catchment (expressed in hectares) is_____ .



[GATE-2016]

Q.32 The ordinates of a one-hour unit hydrograph at sixty minute interval are 0, 3, 12, 8, 6, 3 and 0 m^3/s. A two-hour storm of 4 cm excess rainfall occurred in the basin from 10 AM. Considering constant base flow of $20m^3/s$, the flow of the river (expressed in m^3/s) at 1 PM is _____.

[GATE-2016]

Q. 33

The ordinates of 2-hour unit hydrograph for a catchment are given as

Time (h)	0	1	2	3
Ordinates (m^2/s)	0	5	12	25

The ordinates (in m^3/s) of a 4-hour unit hydrograph for this catchment at the time of 3h would be_____.

Q. 34 An effective rainfall of 2-hour duration produced a flood hydrograph peak of 200 m^3/s. The flood hydrograph has a base flow of 20 m^3/s. If the spatial average rainfall in the watershed for the duration of storm is 2 cm and the average loss rate is 0.4 cm/hour, the peak of 2-hour unit hydrograph (in m^3/s-cm, up to one decimal place) is _____.

[GATE-2017]

Q. 35 The infiltration capacity of a soil follows the Horton's exponential model, $f = c_1 + c_2 e^{-kt}$. During an experiment, the initial infiltration capacity was observed to be 200 mm/h. After a long time, the infiltration capacity was reduced to 25 mm/h. If the infiltration capacity after 1 hour was 90 mm/h, the value of the decay rate constant, k (in h^{-1}, up to two decimal places) is _____ .

[GATE-2017]

Q. 36 During a storm event in a certain period. the rainfall intensity is 3.5 cm/hour and the - ϕ index is 1.5 cm/hour. The intensity of effective rainfall (in cm/hour, up to one decimal place) for this period is _____.

[GATE-2017]

Q.37 The infiltration rate f in a basin under ponding condition is given by $f = 30 + 10e^{-2t}$, where, f is in mm/h and t is time in hour. Total depth of infiltration (in mm, up to one decimal place) during the last 20 minutes of a storm of 30 minutes duration is _____.

[GATE-2018]

Q.38 The total rainfall in a catchment of area 1000 km^2, during a 6 h storm, is 19 cm. The surface runoff due to this storm computed from triangular direct runoff hydrograph is $1 \times 10^8\ m^3$. The ϕ_{index} for this storm (in cm/h, up to one decimal place) is _____.

[GATE-2018]

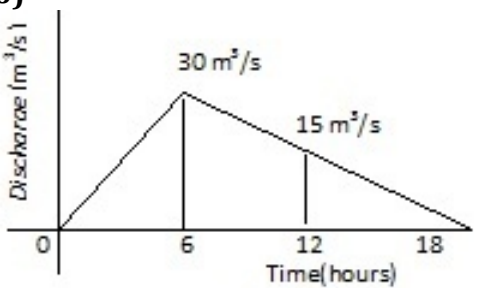
[GATE-2017]

ANSWER KEY:

1	2	3	4	5	6	7	8	9	10	11	12	13	14
(b)	(c)	(c)	(c)	(b)	(c)	(b)	(a)	(a)	(b)	(c)	(d)	(c)	(d)
15	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**	**25**	**26**	**27**	**28**
(c)	(c)	(b)	(b)	(c)	(a)	(c)	(b)	(d)	(c)	(c)	(a)	(d)	(b)
29	**30**	**31**	**32**	**33**	**34**	**35**	**36**	**37**	**38**				
(d)	22	21.6	60	15	150	.9904	2.0	11.7	1.5				

EXPLANATIONS

Q.1 (b)



(1) Time	(2) Ordinate of 6h UH	(3) Lagged by 6h	(4) Addition col(2) + col(3)	(5) Ordinate of 12h UH col(4)÷ 2
0	0	-	0	0
6	30	0	30	15
12	15	30	45	22.5
18	0	15	15	7.5
		0	0	0

zPeak discharge of 12 h UH
= 22.5 m^3/s
Average rainfall = 16 cm
Infiltration = 12 × 0.5 = 6 cm
Rainfall excess = 16 – 6 = 10 cm
Hence peak discharge of DRH = 10 × peak discharge of 12 h UH
= 10 × 22.5 = 225 m^3/s

Q.2 (c)

Depth of rainfall 1 cm × Area of catchment = Area under UH

$$0.01\times A=\frac{1}{2}(18\times3600)\times30$$

A = 9720 × 10^4 m^2
= 9720 hectare

Q.3 (c)

Time of concentration is the time when whole catchment starts contributing to the runoff at the outlet and it is the smallest duration for which the rational formula is applicable.

Q.4 (c)

Permeability = Hydraulic Conductivity

$\left(\frac{\mu}{\rho g}\right)$ A storm of i = 0.5 cm/hr occurs over the soil for an indefinite period.

What is the meaning of indefinite period'?

In order to achieve steady state condition, continuous application of water is necessary so that the water level is maintained constant. **'Surface drainage adequate'** means at state

condition some water flows as surface runoff.

'Storm has lasted for a very long time' means the hydraulic conductivity approaches saturated hydraulic conductivity (so that Darcy's law can be obeyed) and at steady state condition infiltration rate is also constant.

The constant infiltration rate which is achieved after very long time, shall be more than 0.2 cm/hr (as hydraulic conductivity has increased) but should be less than storm intensity 0.5 cm/hr due to adequate drainage.

Q.5 (b)

Average rainfall = 2.7 cm
Loss in 3 hours = 0.3 × 3 = 0.9 cm
Run off depth = 2.7 – 0.9 = 1.8 cm
Peak flow of flood hydrograph
= 210 m^3/s
Peak flow of DRH = 210 – base flow
= 210 – 20 = 190 m^3/s
Peak of 3h unit hydrograph

$$= \frac{\text{Peak discharge of DRH}}{\text{Run off depth}}$$

$$= \frac{190}{1.8} = 105.50\ m^3/s$$

Q.6 (c)

Total rainfall = 0.5+2.8+ 1.6 = 4.9 cm
Surface runoff = 3.2 cm
Infiltration = 4.9 – 3.2 = 1.7 cm
Time of rainfall = 6 hr
If infiltration is assumed constant, infiltration rate will be 1.7/6 = 0.28 cm/hr.
But this infiltration rate cannot be possible throughout the time period of 6 hrs because from 0-2 hr rainfall rate = 0.5/2 = 0 cm/hr is lesser than infiltration rate and infiltration rate can never be more than rainfall rate.
Now, taking time period from 2 to 6 hrs.
Excess rainfall duration = 6 – 2
= 4 hrs
Total rainfall = 2.8 + 1.6
= 4.4 cm
Surface runoff = 3.2 cm (because from 0-2 hrs, storm will not contribute in surface runoff)
Infiltration = 4.4 – 3.2 = 1.2 cm

$$\text{Ø index} = \frac{\text{Infilteration}}{t_e}$$

$$= \frac{1.2}{4} = 0.30\ cm/h$$

For t = 2 to 4 hrs,

$$\text{Rainfall intensity} = \frac{2.8}{2}$$

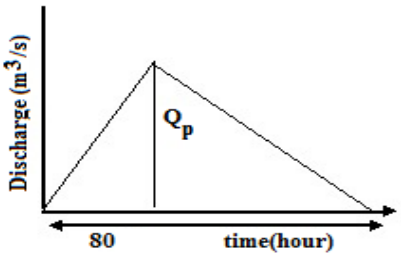
= 1.4 cm/hr > Ø
For t = 4 to 6 hrs,

$$\text{Rainfall intensity} = \frac{1.6}{2}$$

= 0.8 cm/hr > Ø
Hence, Ø index = 0.3 cm/h

Q.7 (b)

Catchment area,
A = 720 km^2 = 720 × 10^6 m^2



Excess rainfall = 1 cm
Peak discharge = Q_p
Run off volume = Area under UH

$$A \times 0.01 = \frac{1}{2}(80\times3600)\times Q_p$$

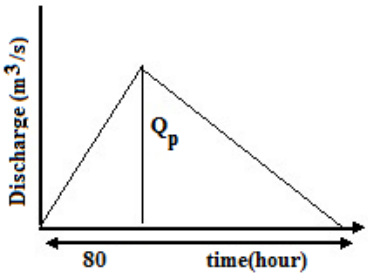
$$= 720 \times 10^6 \times 0.01$$

$$= \frac{1}{2}(80\times3600)Q_p$$

$$\Longrightarrow Q_p = 50\ m^3/s$$

Q.8 (a)

Depth of precipitation = 4 cm



Infiltration in 4 hours = 4 × 1mm
= 4 mm = 0.4 cm
Run off depth = 4 – 0.4 = 3.6 cm
Peak of DRH of 4 hours = 3.6 × peak of UH of 4 hrs
= 3.6 × 50 = 180 m^3/s
Peak of flood discharge = Peak of DRH + base of flow
= 180 + 30 = 210 m^3/s
(base flow= 30 m^3 / s)

Q.9 (a)

When outflow from a reservoir is uncontrolled as in freely operating spillway, then the peak of outflow hydrograph will occur at a point of intersection of inflow and outflow curves, whereas if outflow from a reservoir is controlled, then peak will occur after the intersection of the curve.

Q.10 (b)

$Q = 1 - (1 + t)e^{-t}$
Saturation discharge for S curve, Qs

$$= \lim_{t\to\alpha}\{1-(1+t)e^{-t}\}$$

$$= \lim_{t\to\alpha}\left[1-\frac{(1+t)}{e^{t}}\right]$$

Using L-Hospital's Rule,

$$Qs = \lim_{t\to\alpha}\left[1-\frac{1}{e^{+t}}\right] = 1m^3/s$$

Rainfall intensity, i = 1 cm/hr

$$= \frac{1}{360000} m/s$$

Catchment area × Rainfall intensity = Qs

$$A \times \frac{1}{360000} = 1$$

$\Longrightarrow$ A = 360000 m^2 = 0.36 km^2

Q.11 (c)

The duration of S-curve = 1 hr
The ordinate of 2-hr UH is obtained by the following procedure:

Step 1 : The ordinate of S-curve at t = 3 hr

$$S_3 = 1-(1+3)e^{-3} = 0.8\ m^3/s$$

Step 2 : The ordinate of S-curve at

$$t = 3-2 = 1\ hr$$

$$S_1 = 1-(1+1)e^{-1} = 0.26\ m^3/s$$

Step 3 : $S_3 - S_1 = 0.8 = 0.26 = 0.54\ m^3/s$

Step 4 : The ordinate of UH at t = 3 hr is

$$\frac{S_3 - S_1}{2} = 0.27\, m^3/s$$

Q.12 (d)

For the First Hour,
Rainfall = 10 mm

$$\text{Infiltration} = \int_0^1 f.dt = \int_0^1 (6.8+8.7e^{-t})dt$$

= 12.299 mm
Effective rainfall in first hour p_1 = 10 – 12.299 = –ve value
Hence, p_1 = 0 effective rainfall cannot be (–) ve

For the Second Hour,
Effective rainfall, p_2 = Rainfall infiltration

$$= 20 - \int_1^2 (6.8+8.7e^{-t})dt$$

= 20 – 8.823 = 11.177 mm

For the Third Hour
Effective rainfall, p_3

$$= 30 - \int_2^3 (6.8+8.7e^{-t})dt$$

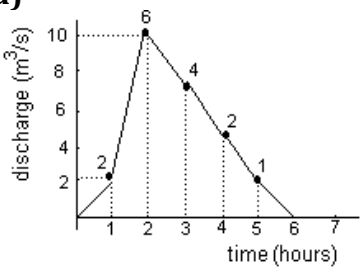
= 10 – 7.544 = 2.456 mm
Total effective rainfall in 3h,
$p = p_1 + p_2 + p_3$
= 0 + 11.177 + 2.456 = 13.633 mm

Q.13 (c)

Time (hour)	Rainfall (mm)	Ø index (mm/hr)	Effective rainfall (mm)
0	0	10	0
1	9	10	0
2	28	10	18
3	12	10	2
4	7	10	0

Run off depth = effective rainfall = 20 mm

Q.14 (d)



Excess rainfall × catchment area = Area under unit hydrograph

$0.01 \times A = 15 \times 3600 m^3$

$\Rightarrow A = \frac{15 \times 3600}{0.01} m^2$

$= 5.4 \times 10^6 m^2 = 5.4 km^2$

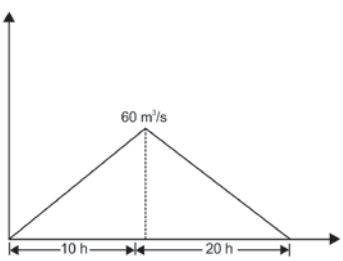
Q.15 (c)

(1) Time	(2) Ordinate of 1h UH	(3) UH lagged by 1h	(4) UH lagged by 2h	(5) Ordinate of DRH of 3 cm in 3h col(2)+col(3)+col(4)	(6) Ordinate of 3h UH col(5)÷3
0	0	-	-	0	0
1	2	0	-	2	2/3
2	6	2	0	8	8/3
3	4	6	2	12	4
4	2	4	6	12	4
5	1	2	4	7	7/3
6	0	1	2	3	1
7			1	1	1/3
8			0	0	0

Thus at time t = 3h ordinate of 3h UH = 4 m^3/s

Q.16 (c)

For UH depth of effective rainfall = 1cm



Volume of runoff = $A \times 10^{-2} m^3$

Volume of runoff = Area of UH

$A \times 10^{-2} = \frac{1}{2} \times 60 \times (30 \times 3600)$

$A = 324 \times 10^6 m^2 = 324 km^2$

Q.17 (b)

Total rainfall = 5.4 cm

Infiltration in 1 hour = ϕ – index × 1

= 0.4 × 1 = 0.4 cm

Runoff depth = 5.4 – 0.4 = 5 cm

The discharge in the unit hydrograph decreases from 60 m^3 / sec to 0 in a period of 20 hours.

∴ Discharge at the 15th hour

$= 60 - \frac{60}{20} \times 5 = 45 m^3/sec$

Ordinate of DRH of 1h = 45 × 5 = 225 m^3/sec

Ordinate of flood hydrograph,

Q = 225 + Base flow

= 225 + 15

= 240 m^3 / sec

Q.18 (b)

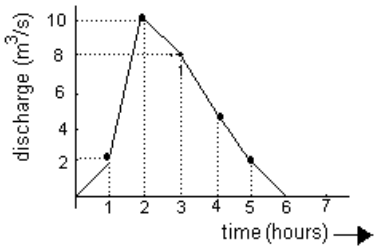
Evapotrpiration – Penman method

Infiltration – Horton's method

Synthetic unit hydrograph – Snyder's method

Channel routing – Muskingum method

Q.19 (c)



Area of unit hydrograph = 1 cm × catchment area

$$\frac{1}{2}(0+3)\times1+\frac{1}{2}(3+8)\times1+\frac{1}{2}(8+6)\times1$$
$$+\frac{1}{2}(6+3)\times1+\frac{1}{2}(3+2)\times1+\frac{1}{2}(2+0)\times1$$

= 0.01 × A

22 × 3600 m^3 = 0.01 A

$\Rightarrow$ A = 7.92 × 10^6 m^2 = 7.92 km^2

Q.20 (a)

Duration of storm is different from duration of unit hydrograph (UH)

So, we have to convert 2h UH into 3 h UH (duration of UH equal to storm duration)

Storm duration = m × duration of UH

3 hour = m 2hour

As m is not an integer so using S curve method.

(1) Time hr	(2) Ordinate of 2h UH	(3) S curve addition	(4) S curve ordinate	(5) S curve lagged by 3h	(6) col(4) – col(5)	(7) Ordinate of 3h UH col(6)×2/3
0	0		0	-	0	0
1	3		3	-	3	2
2	8	0	8	-	8	5.33
3	6	3	9	0	9	6 peak value
4	3	8	11	3	8	5.33
5	2	9	11	8	3	2
6	0	11	11	9	2	1.33
7	0	11	11	11	0	0
8	0	11	11	11	0	0

Peak of 3h UH = 6m^3/s

Effective rainfall of storm

= 6.6 – 3 × 0.2 = 6 cm

Peak flow due to storm

= 6 × peak of 3hUH + base flow

= 6 × 6 + 5= 41 m^3/s

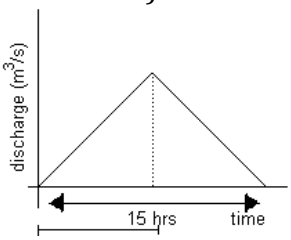
Q.21 (c)

Due to urbanization, there will be proper drainage system in the catchment so peak of hydrograph will come earlier and time base will get reduced.

Q.22 (b)

Unit hydrograph is a hydrograph of direct runoff resulting from 1 cm of effective rainfall.

So volume of water contained inside the unit hydrograph (i.e., area of unit hydrograph is equal to 1 cm × catchment area)



$$\frac{1}{2}\times\alpha\times15\text{hrs}=1\text{cm}\times A$$

$$\frac{1}{2}\times\alpha\times15\times3600=0.01\times50\times10^6$$

α=18.52 m^3/s/cm of effective rainfall

Q.23 (d)

For a storm of 5.5 cm and duration of 1 hour

Effective rainfall = storm depth - Ø index

= 5.5 – 0.5 = 5 cm

Peak ordinate of hydrograph due to 5 cm effective rainfall

= 5 × peak ordinate of UH + base flow

= 5 × 18.52 + 10 = 102.60 m^3/s

Q.24 (c)

Ratio of actual evapotranspiration to potential evapotranspiration is in the range of 0 to 1.

Q.25 (c)

Q.26 (a)

Both the storms having same infiltration index

$\Rightarrow$ (P - Ø) × t_R = Runoff

$\Rightarrow$ (2.0 - Ø) × 5 = 4

$\Rightarrow$ Ø = $-\left(\frac{4}{5}\right)+2.0$ = 1.2 cm/hr

Q.27 (d)

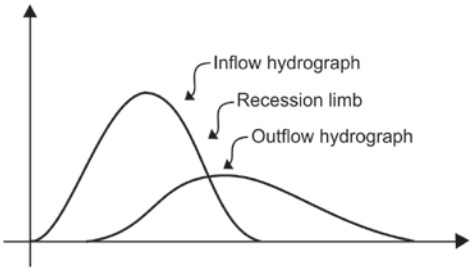
Let the rainfall intensity of 2nd storm be I cm/hr

$\Rightarrow$ (I - ϕ) × t_R = Runoff

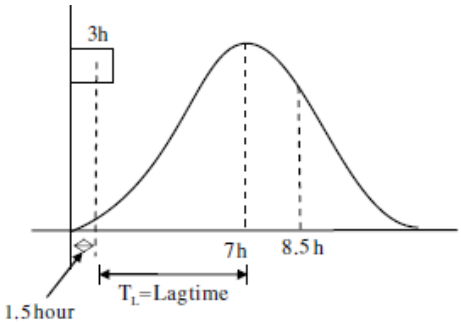
$\Rightarrow$ (I – 1.2) × 8 = 8.4

$$\Rightarrow I = \left(1.2 + \frac{8.4}{8}\right) = 2.25 \text{ cm/hr}$$

Q.28 (d)



Q.29 (b)



For small catchment, time of concentration is equal to lag time of peak flow. $T_c = 7 - 1.5 = 5.5h$

Q.30 (22)

Time	UHO	S-cure Addition	S_A
0	0		0
2	0.6		0.6
4	3.1	0	3.1
6	10	0.6	10.6
8	13	3.1	16.1
10	9	10.6	19.6
12	5	16.1	21.1
14	2	19.6	21.6
16	0.7	21.1	21.8
18	0.3	21.6	21.9
20	0.2	21.8	22
22	0.1	21.9	22
24	0	22	22

Maximum S-curve ordinate is 22.

Q.31 (21.6)

Area under hydrograph = direct runoff volume

$$\frac{1}{2} \times 1 \times 6 \times 60 \times 60 = 5 \times \frac{1}{100} \times A$$

$$A = \frac{1}{2} \times \frac{6 \times 60 \times 60 \times 100}{5} = \frac{2160000}{10}$$

$$= 216000 m^2 = 21.6 \times 10^4 \times m^2$$

$$\boxed{A = 21.6 \text{ hectares}}$$

Q.32 (60)

Time	Ordinate of 1 ho UH	Lag	Ordinate of 2h DRH	Ordinate of 2h UH
10.00	0		0	0
11.00	3	0	3	1.5
12.00	12	3	15	7.5
01.00	8	12	20	10
02.00	6	8	14	7
03.00	3	6	9	4.5
03.00	3	6	9	4.5
04.00	0	3	3	1.5
		0	0	0

Flow of river = rainfall excess × ordinate of 2-h UH +Base flow

$$= 4 \times 10 + 20 = 60 m^3/s$$

Q.33 (15)

Time (hr)	Ordinate of 2 hr Unit Hydrograph	Ordinate of 2 hr Unit Hydrograph lag by 2 hr	Ordinate of 4hr DRH	Ordinate of 4 hr Unit Hydrograph = Ordinate of given DRH / 2 cm
0	0	-	-	-
1	5		5	2.5
2	12	0	12	60
3	25	5	30	15
4	41	12	53	26.5
		25	25	12.5
		41	41	20.5

∴ Ordinate of 4 hr U.H. at 3 hr duration = 15.0 m^3/sec

Q.34 (150)

Peak of 2 hr direct runoff hydrograph
= Peak of 2 hr flood hydrograph - Base flow
= 200 - 20
= 180m^3/s
The rainfall excess depth for a given duration 2 hr effective storm/rainfall is given by

Rainfall excess depth = 2 cm $-0.4\ \frac{\text{cm}}{\text{hr}} \times 2$ hr

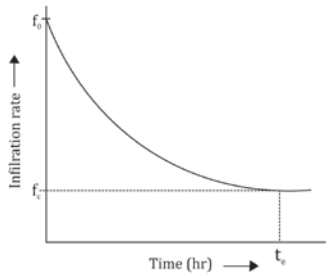
= 1.2 cm

$\therefore$ Peak of 2 hr UH = $\frac{180}{1.2} = 150 m^3/s-cm$

Q.35 (.9904)

Horton's expression for infiltration capacity of a soil is given as

$$f = C_1 + C_2 \cdot e^{-kt}$$



Here,

C_1 = Constant or steady stage infiltration capacity

C_2 = Difference between initial and steady stage intittration capacity

$= (f_0 - f_c)$

f = Infiltration capacity of any time 't

$\therefore \quad f_p = f_c + (f_0 - f_c)e^{-k_h t}$

$\Rightarrow \quad 90 = 25 + (200 - 25)e^{-k_h 1}$

$\Rightarrow \quad K_h = 0.9904\ hr^{-1}$

Q.36 (2.0)

Let the rainfall duration is 't' hours

$\therefore$ Total rainfall = 3.5 cm/hr × t hr = 3.5 t

Total infiltration = ϕ – index × t = 1.5 t

Intensity of rainfall excess $= \frac{\text{Total rainfall excess}}{\text{Time period}}$

$= \frac{3.5t - 1.5t}{t}$ cm/hr = 2.0 cm/hr

Q. 37 (11.7)

Infiltration rate $f(t) = 30 + 10e^{-2t}$

Total infiltration depth in time 10 min. to 30 min . i.e. 0.166 hour to 0.5 hour

$= \int_{0.166}^{0.5} \left(30 + 10e^{-2t}\right) dt$

=11.74mm

Q. 38 (1.5)

$\text{Surface runoff} = \frac{1 \times 10^8 \text{m}^3}{1000 \times 10^6 \text{m}^2}$

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

Total rainfall = 19 cm

$\text{Rainfall intensity} = \frac{19}{6} = 3.167\text{cm/hr}$

$\text{w} - \text{index} = \frac{\text{P} - \text{Q}}{\text{t}}$

$= \frac{\text{total infiltration}}{\text{total duration of storm}}$

$\therefore \text{w} - \text{index} = \frac{19 - 10}{6} = 1.5\text{ cm/hr}$

As intensity of rainfall > w – index.

And rainfall intensity is uniform

therefore ϕ – index = w – index = 1.5 cm/hr.

4 FLOODS, FLOOD ROUTING AND FLOOD CONTROL

Q.1 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

[GATE-2008]

Q.2 A 1-h rainfall of 10 cm magnitude at a station has a return period of 50 years. The probability that a 1-h rainfall of magnitude 10 cm or more will occur in each of two successive years is:

a) 0.04 b) 0.2
c) 0.02 d) 0.0004

[GATE-2013]

Q.3 The Muskingum model of routing aflood through a stream reach is expressed as $O_2 = K_0 I_2 + K_1 I_1 + K_2 O_1$, where K_0, K_1 and K_2 are the routing coefficients for the concerned reach, I_l and I_2 are the inflows to reach, and O_1 and O_2 are the are the outflows from the reach corresponding to time steps 1 and 2 respectively. The sum of K_0, K_1 and K_2 of the model is

a) -1 b) -0.5
c) 0.5 d) 1

[GATE-2014]

Q.4 The type of flood routing (Group I) and the equation(s) used for the purpose (Group II) are given below.

Group I	Group II
P. Hydrologic flood routing	1. Continuity equation
Q. Hydraulic flood routing	2. Momentum equation
	3. Energy equation

The correct match is

a) P-1; Q-1, 2 & 3 b) P-1; Q-1 & 2
c) P-1 & 2; Q-1 d) P-1 & 2; Q-1 & 2

[GATE-2016]

Q. 5 A culvert is designed for a flood frequency of 100 years and a useful life of 20 years. The risk involved in the design of the culvert (in percentage, up to two decimal places) is_____.

[GATE-2018]

Q. 6 For routing of flood in a given channel using the Muskingum method, two of the routing coefficients are estimated as C_0 = -0.25 and C_1 = 0.55. The value of the third coefficient C_2 would be_____.

ANSWER KEY:

1	2	3	4
(b)	(d)	(d)	(b)

EXPLANATIONS

Q.1 (b)

Main function of any reservoir in the flood routing is to reduce the peak and increase the time base so that there will be less damage on the downstream of the flow.

Q.2 (d)

Return period of rainfall

$T = 50$ years

$\therefore$ Probability of occurrence once in 50 years,

$$p = \frac{1}{50} = 0.02$$

Probability of occurrence in each of 2 successive year $= p^2 = (0.02)^2 = 0.0004$

Q.3 (d)

$K_0 + K_1 + K_2 = 1$

Q.4 (b)

In Muskingum flood routing method

$$C_0 + C_1 + C_2 = 1$$

$$\Rightarrow \quad C2 = 1 - (-0.25) - 0.55 = 0.7$$

Q. 5 (18.20)

Risk = The probability of a flood to occur at least once in n-successive years.

$$\therefore \quad \text{Risk} = 1 - q^n$$

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

$$= 1 - \left(1 - \frac{1}{T}\right)^n = 1 - \left(1 - \frac{1}{100}\right)^{20}$$

$$= 1 - (0.99)^{20}$$

$$= 0.18209 = 18.209\%$$

Q. 6 (0.7)

IRRIGATION

1 WATER REQUIREMENT OF CROPS

1.1 INTRODUCTION

Crop water requirements

The term water requirements of a crop means the total quality of all water and the way in which a crop requires water, from the time it is sown to the time it is harvested. The water requirements of crop vary with the crop as well as with the place. It depends upon climate, types of soil, method of cultivation and a useful rainfall.

Advantages of Irrigation:

- Increase in crop yield
- Protection from famine
- cultivation of superior crops
- Elimination of mixed cropping
- Hydropower generation
- Domestic and industrial water supply.

Disadvantages of Irrigation:

- Water logging
- Water losses
- Diseases
- Pollution problem

1.1.1 SATURATION CAPACITY

It is the total water content of a soil when all the pores of the soil are filled with water.

1.1.2 FIELD CAPACITY (FC)

It is the maximum amount of water content which can be held by soil against gravity. It is the upper limit of capillary water or the water content available to the plant roots.

1.1.3 PERMANENT WILTING POINT (PWP)

It is the water content in a soil when plants become permanently wilted. This value is 2% for sand soil, and 30% for clayey soils.

1.1.4 ULTIMATE WILTING POINT (UWP)

It occurs when plants are completely wilted i.e., die away. It is similar to hygroscopic coefficient.

It is the difference in water content of the soil between the field capacity and the permanent wilting point.

1.1.6 READILY AVAILABLE MOISTURE

It is that portion of the available moisture, which is most easily extracted by plants roots. About 75% of the available moisture is usually readily available.

1.1.7 EQUIVALENT DEPTH

Equivalent depth of water held by soil at field capacity = $s \times d \times FC$
Equivalent depth of water held by soil at PWP = $s \times d \times PWP$
Available water or moisture depth, $y = S \times d$ (FC – PWP)
It is also called as storage capacity of the soil.

1.1.8 READILY AVAILABLE MOISTURE DEPTH

$d_w = S \times d$ (Field Capacity – Water content at lower limit of readily available moisture)
or $d_w = S \times d$ (Field Capacity – optimum moisture)
Where s = Specific gravity = γ_d/γ_w, d = depth of root zone

1.1.9 EVAPORTRANSPIRATION LOSS OR CONSUMPTIVE USE (DEPTH/TIME)

If Cu is evapotranspiration loss or consumptive use (depth/time)
Then frequency of irrigation, f=dw/Cu

1.1.10 CROP PERIOD

Total time elapses between the sowing of the crop and its harvesting.

1.1.12 BASE PERIOD

Total time between the first watering done for the preparation of the land for sowing of a crop and the last watering done before its harvesting. Thus crop period is slightly more than the base period for any crop.

1.1.13 DUTY (D)

It is defined as the area of land in hectares which can be irrigated for growing any crop if one cumec of water is supplied continuously to the land for the entire base period of crop.

1.1.14 DELTA (Δ)

It is the total depth of water over the irrigated land required by a crop grown on it during the entire base period of the crop.

Crop	Average Delta (cm)
Rice	120
Wheat	37.5
Cotton	45
Tobacco	60
Sugarcane	90

1.1.15 DUTY AND DELTA

Relation between Duty and Delta :
$D = 8.64\ B/\Delta$
Where, D = Duty in Hectares/cu×mec,
B = Base period in days,
Δ = Delta in 'm'
Duty of water increases as one moves from the head of the canal system to the field and hence the place at which duty of water is measured must be specified. In order to state
i)The base of duty of water and
ii) Place of measurement of duty of water

1.2 Base of duty of water

It is the period to which duty of water has reference

1.2.1 PALEO IRRIGATION (PALEO)

It is watering done prior to the sowing of a crop.

1.2.2 KOR WATERING

The first watering after the plants have grown a few cm high.

1.2.3 OUTLET FACTOR

Duty of water at the canal outlet.

1.2.4 ONE CUMEC-DAY

8.64 hectare meters, it is a volumetric unit. It is the total volume of water supplied at 1 cumec in a day.

1.2.5 CONSUMPTIVE USE OR EVAPOTRANSPIRATION (C_U)

It is the total loss of water due to plants transpiration and evaporation from the land. Lysimeter is used to measure C_u.

1.3 IRRIGATION EFFICIENCIES

1.3.1 WATER CONVEYANCE EFFICIENCY (η_C)

It is the ratio of quantity of water delivered to the field to the quantity of water diverted into the canal system from reservoir.

1.3.2 WATER APPLICATION EFFICIENCY (η_A)

It is the ratio of quantity of water stored in the root zone of the plants to the quantity of water delivered.

1.3.3 WATER USE EFFICIENCY (η_U)

It is the ratio of quantity of water used beneficially including the water required for leaching to the quantity of water delivered.

1.3.4 WATER STORAGE EFFICIENCY (η_s)

Ratio of quantity of water stored in the root zone during irrigation to the quantity of water needed to bring content of the soil to field capacity.

1.3.5 WATER DISTRIBUTION EFFICIENCY (η_d)

The effectiveness of irrigation may also by measured by its water distribution efficiency. (η_d)It indicates the uniformity in distribution of water over the entire root zone.

$$\eta d = \left(1 - \frac{y}{d}\right) \times 100$$

Where, y is the average numerical deviation in the depth of water from average depth (d).

1.3.6 CONSUMPTIVE USE EFFICIENCY

Ratio of normal consumptive use of water to the net amount of water depleted from the root zone.

1.4 IRRIGATION REQUIREMENTS OF CROPS

Effective rainfall (Re):

Effective rainfall is defined as that a part of the rainfall which is effectively used by the crop after rainfall losses due to surface run off and deep percolation have been accounted for. The effective rainfall is the rainfall ultimately used to determine the crop irrigation requirements.

1.4.1 CONSUMPTIVE IRRIGATION REQUIREMENT (CIR)

It is the amount of water required to meet the evapotranspiration needs of a crop.
CIR = C_U – Re
Where Re = Effective Rainfall

1.4.2 NET IRRIGATION REQUIREMENT (NIR)

Amount of irrigation water required to be delivered at the field to meet evapotranspiration and other needs such as leaching
NIR = Cu – Re +Le
Where Le = Leaching

1.4.3 FIELD IRRIGATION REQUIREMENT (FIR)

Field irrigation requirement (FIR) = NIR / η_a

1.4.4 GROSS IRRIGATION REQUIREMENT (GIR)

Gross irrigation requirement (GIR) = FIR/ η_c

1.5 Gross Command Area (GCA)

Total area which can be irrigated by a canal system if unlimited, quantity of water is available.

1.5.1 CULTURABLE COMMAND AREA (CCA)

That portion of the GCA which is culturable or cultivable.
CCA = GCA – Uncultivable area

1.5.2 CULTURABLE CULTIVATED AREA

That portion of the CCA which is actually cultivated during a crop season.

Cutturable uncultivated Area: It is the area in which crop is not sown in a particular season.

Water logged Area: An agricultural land is said to be water logged when its productivity or fertility is affected by high water table.

1.5.2.1 INTENSITY OF IRRIGATION

The percentage of the CCA proposed to be irrigated annually.

1.5.2.2 CAPACITY FACTOR

Ratio of mean supply discharge of canal for a certain duration to its maximum discharge capacity.

1.5.2.3 TIME FACTOR

Ratio of number of days the canal has actually run during a watering period to the total number of days of the watering period.

Kor Depth:

It is the depth of water applying in the First watering which is given to a crop.

Kor Period:

The portion of the base period in which kor watering is needed is known as kor period.

Nominal duty:

$$\text{Nominal duty} = \frac{A\,(ha)}{\left(\frac{Xn}{m}\right) cumec}$$

Where, A = Area actually irrigated

x = water released daily from outlet For n days.

m = total crop period in days.

1.6 Crop seasons

i) Kharif season ii) Rabi season

Kharif crops (also known as monsoon crops): Sown in t month of April and harvested in September Ex: Rice, Maize.
Rabi crop (also called winter crops): Sown in October and harvested in March.
Ex: Wheat, Tobacco.

1.6.1 PERENNIAL CROP

Sugar cane (Water is to be supplied through the year)

1.6.2 HOT WEATHER CROPS

February to June (Crops grown between kharif season and Rabi season)

1.6.3 SUMMER CROPS

Hot weather crops and kharif crops are together called summer crops.

1.6.4 DRY CROPS

Crops grown without irrigation. Depend on the rainfall, for survival

1.6.5 WET CROPS

Crops which require irrigation.

Crop ratio:

It is the ratio of area irrigated in the Rabi season to area irrigated in Kharif season.

Root zone depth:

It is the maximum depth of soil strata in which the crop spreads its root system and derives water from the soil.

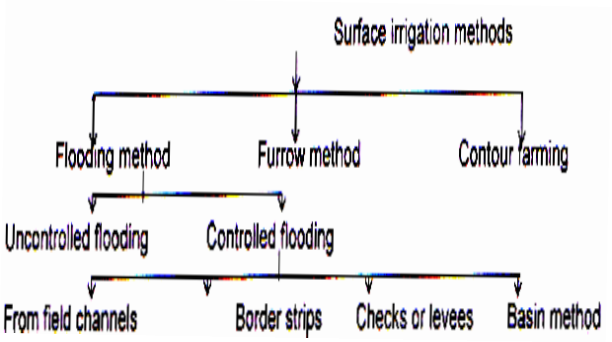
2 METHODS OF IRRIGATION

2.1 INTRODUCTION

Broadly, classified into 3 groups
i) surface irrigation methods
ii) Sub – surface irrigation methods
ii) Sprinkler irrigation methods

2.2 SURFACE IRRIGATION METHODS

Water is supplied by spreading on the land



Surface irrigation method:

Surface irrigation is the oldest and the most common method of irrigation. In this method, water is either ponded on the soil or allowed to flow continuously over the surface of the soil for the duration of irrigation.

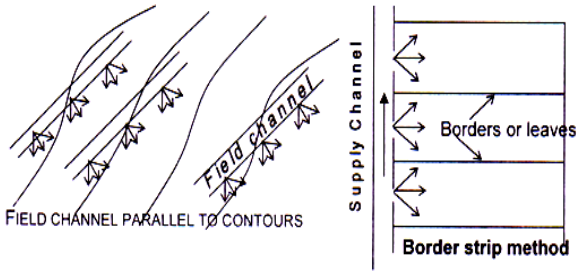
2.2.1 FLOODING METHOD

Uncontrolled or free or wild flooding : Water is applied over the land without any prior
Preparation of the land and without enforcing any control over the water by levees. Suitable for smooth and flat land. There is more wastage water in this method.

Controlled flooding : Also known as free flooding. Water is applied on the land with control over the quantity as well as direction of flow, prior preparation of land is done.

2.2.2 FLOODING FROM FIELD CHANNELS

Land is divided into strips by series of field channels or laterals and the strips are supplied water from these field channels. The supply channel is located at higher edge of field. Irrigation is possible on one side of field channel. Suitable for flat as well as relatively steep slopes.

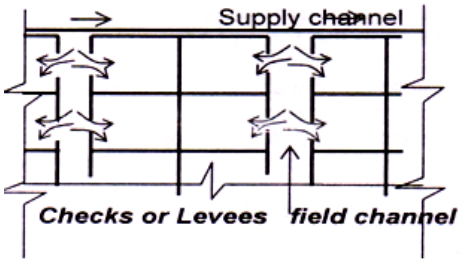


2.2.3 BORDER STRIP METHOD

Land to be irrigated is divided into a series of long narrow strips about 10 to 20 m wide and 100 to 400 m long separated from each other by low levels or bunds and each strip is supplied water by the field channel. The strips will have slope along the direction of flow. Water flows in the form of a sheet to the lower end of the field.

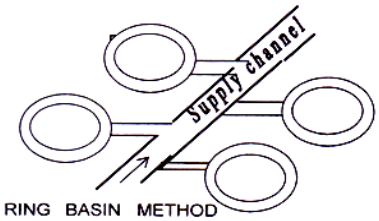
2.2.4 CHECKS OR LEVEES METHOD

Most common method also called method of irrigation by plots of land. Land is divided into small plots surrounded by cheeks or levees. Each plot has nearly level surface. Area of each plot is between 400 to 500 m^2



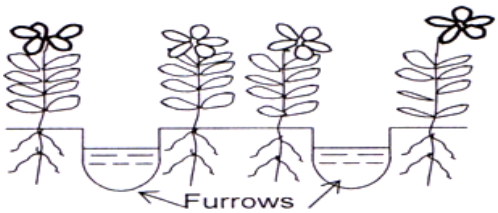
2.2.5 BASIN METHOD

Suitable for fruit trees. A basin is created surrounding each tree and field channel supplies water to the basin. Used for irrigation of orchards.



2.2.6 FURROW METHOD

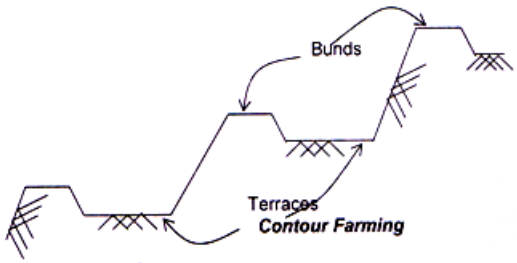
Water is applied to the land to be irrigated by a series of long, narrow field channel called furrows reduces evaporation losses. Suitable for row crops such as maize etc.



Water is not spread over the land. Water flowing in the furrows infiltrates into the soil and spreads laterally to the roots of plants. It has high water application efficiency.

2.2.7 CONTOUR FARMING

Practiced in hilly regions where the land will be having steep slopes Also controls erosion due to rain.



2.3 Sprinkler irrigation method

Water is applied in the form of spray by pipes and nozzle system. Erosion of soil is eliminated. Uniform application of water is possible. No land preparation is required. Wind may distort the application of water. Power requirement is more for constant pumping of water.

2.4 Sub surface irrigation method

Water is applied below the ground surface directly to the root zone of the plants, by pipe network. Evaporation losses are reduced Eg : Drip or trickle irrigation

2.5 Drip or Trickle irrigation

- Latest method of irrigation, getting popular.
- Water is applied to the land surface near the base of the plants by a network of plastic pipes (PVC pipes) and drip nozzles called emitters or drippers.
- Require a pumping unit, pipe lines and emitters. has highest water application efficiency (as high as 90%)
- Suitable for areas where there is water scarcity and salt problems(saline lands) Evaporation loss is reduced and deep percolation is avoided.
- Quite suitable for small trees and widely spaced plants, for fruit plants.
- Can be used for any topography. Land need not be level.
- Not suitable for closely planted crops such as wheat, rice etc.

Border strip Method of Irrigation

Time required for water to cover an area,

$$T = \frac{y}{f}\log_e\left(\frac{q}{q - f\,A}\right)$$

y = average depth of sheet of flowing water;

q = discharge of irrigation stream;

A = area of the strip or land covered at any time;
f = rate of infiltration
Max area that can be irrigated with a stream of discharge (q) can be found as follows

$$A_{max} = \frac{q}{l}$$

3 GRAVITY DAMS

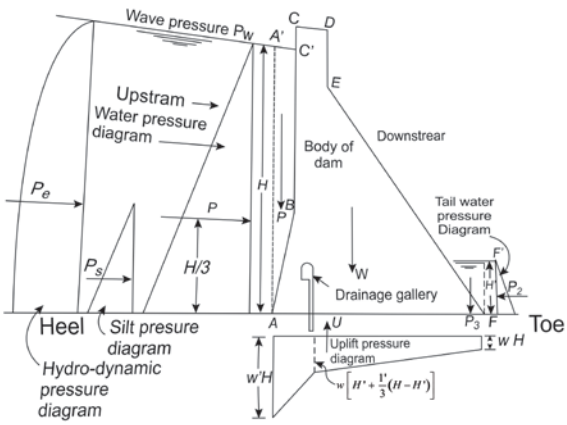
3.1 Gravity Dam

A dam which resists the external forces acting on it by its self weight. Generally made of masonary or concrete.

3.1.1 AXIS OF DAM OR BASE LINE OF DAM

In plan, it is the horizontal trace of upstream edge of the top of the dam.
In the cross section of the dam, the vertical line passing through the upstream edge of the top of the dam

3.1.2 FORCES ACTING ON A GRAVITY DAM



Water pressure, weight of dam, uplift pressure, earthquake forces, ice pressure, wave pressure, silt pressure, vertical acceleration ,horizontal acceleration, horizontal inertia force etc.

3.2 Modes OF Failure OF Gravity Dam

a) Over turning :

$$F.S. = \frac{\text{Stabilizing moment}}{\text{Overturning moment}}$$

b) Sliding : At any horizontal section of the dam, the factor of safety (FS) against sliding,

$$F.S = \frac{\mu.\sum V}{\sum P_H}$$

where μ= coefficient of friction

$\sum V$ = Total net vertical force

$\sum P_H$ = Sum of horizontal forces causing sliding

Sliding factor,

$$\tan\theta = \frac{\sum P_H}{\sum V}$$

Factor of safety against sliding ,

$$FS = \frac{\mu}{\tan\theta}$$

$$\text{Shear friction factor} = \frac{\mu.\sum V + b.q}{\sum P_H}$$

Where b = width of joint or section-area = b x 1

q = shear strength of joint (generally taken 14Kg / cm2 for rock foundation)

c) Overstressing in the dam material

The stresses in the dam material should be within the allowable limits.

d) Compression (or) Crushing at base

The maximum pressure on the foundation soil due the dam construction shall be less than safe bearing capacity of foundation soil. The pressure at base on the soil is found as follows:

$$P_n = \frac{\sum V}{b}[1 \pm 6(e/b)]$$

where $\sum V$ = algebraic sum of vertical forces
e = eccentricity
b = base width of the dam

e) Tension: No tension shall be permitted at any point. For no tension to develop, the eccentricity should be less than (b/6). In other words the resultant should always lie within middle third of the base.

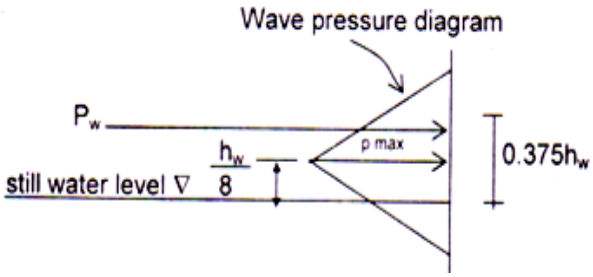
3.2.1 Wave pressure, Molitor's formula.

Height of wave, $h_w = 0.0322\sqrt{F.V}$..... for F > 32 Km h_w in meters
F = the fetch of reservoir.....in Km
Fetch: It is the straight length of water expanse measured normal to the axis of dam
V = wind velocity in km/hr.
Wave pressure force, $P_w = 2.4\ wh_w^2$ per metre length of dam.
W = unit weight of water
Pw acts at a height of 0.375 hw above the still water level
Maximum Pressure intensity occurs at a height of hw/8 above the still water level.
The pressure distribution is curvilinear.
Pressure distribution may be assumed to be triangular height of $\left(\frac{5}{3}\right)$ hw. Hence, the total pressure P_w is given by

$$P_w = (2.4)\ whw \times \frac{1}{2}\left(\frac{5}{3}\ hw\right)$$

$$= 2\ whw\ (t/m)$$

$$= 2000\ hw\ (Kg./m)$$

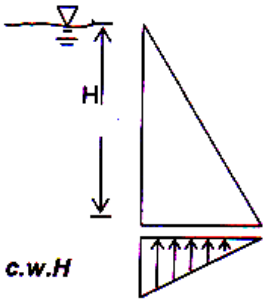


3.3 SILT PRESSURE

It is assumed that mixture of silt and water behaves as a liquid with the following unit weight.
For horizontal force component, the specific weight of silt liquid is taken as 1360 kg/m³
For Vertical component it is taken as 1925 kg/m³

3.3.1 ELEMENTARY PROFILE OF GRAVITY DAM

A right angled triangle for no tension condition, the base width $B = \frac{H}{\sqrt{(S-C)}}$



If uplift is not considered
$B = (H/\sqrt{S})$
For no sliding condition the base width
$b = \frac{H}{\mu(S-C)}$
If uplift is not considered
$b = \frac{H}{\mu . S}$

3.4 LIMITING HEIGHT OF ELEMENTARY PROFILE (HC)

Principal stress at the toe will be σ_1 = w. H (S – C +1)
If f = allowable crushing stress of dam material

$$Hc = \frac{f}{w(S-C+1)}$$

w = unit weight of water

Hc = The maximum height which may be provided for a dam having elementary profile without exceeding allowable stress for the dam material To be on safer side, the lower value of Hc is obtained when C = 0 in the above equation limiting height of a dam having elementary profile

$$Hc = \frac{f}{w(S+1)}$$

If height of a dam is less than or equal to the Hc , the dam is called ' Low gravity dam

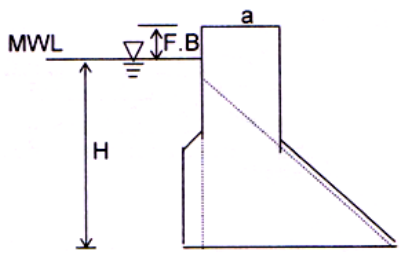
If height of a dam is more than the Hc. It is known as the 'high gravity dam'

3.4.1 PRACTICAL PROFILE OF A GRAVITY DAM

Free board : As per IS, free board shall be more than 1.5 time the height of wave or 0.9 m whichever is greater

Economical top width, a = 14% of H

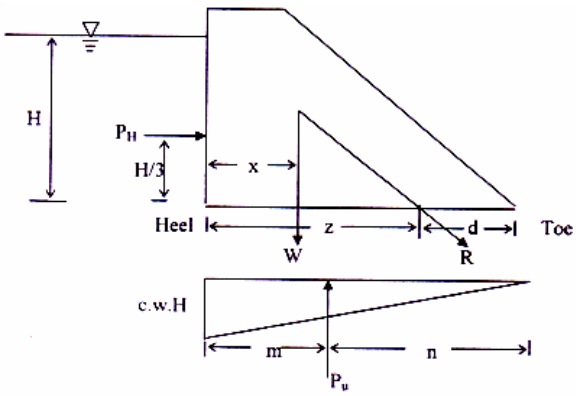
Bligh's empirical formula, a = $0.552\sqrt{H}$



3.4.2 DRAINAGE GALLERY

- An opening left in the dam.
- To provide drainage of the dam (i.e., to drain off water which seeps through the upstream face of dam), for drilling and grouting of foundation , for post cooling of concrete, to lay mechanical equipment for operation of outlet gates and spill way gates etc.

3.5 STABILITY CACULATIONS FOR A GRAVITY DAM



CONSIDER 1 M LENGTH OF DAM

Water pressure PH = ($wH^2/2$)

w = unit weight of water = 1 ton/m3

W = Total weight of dam

Pu = Total uplift pressure = ½ c. w H…b

c = uplift coefficient (varies from 0 to 1)

X = C.G distance of W from heel

z = Distance of resultant from heel

d = distance of resultant from toe

3.5.1 TO FIND RESULTANT DISTANCE FROM HEEL (I.E.., TO FIND Z)

$$z = \frac{P_{H.}(H/3) + W.x - P_u.m}{(W - P_u)}$$

$$= \frac{\text{Algebric sum of moments about heel}}{\text{net weight of dam}}$$

∴ Eccentricity , e = z – (b/2)

3.5.2 TO FIND RESULTANT DISTANCE FROM TOE (i.e.., to find d)

$$d = \frac{P_{H.}(H/3) + P_u.n - W(b-X)}{(W - P_u)}$$

$$= \frac{\text{Algebric sum of moments about heel}}{\text{net weight of dam}}$$

∴ Eccentricity , e = (b/2) – d

For no tension condition, the e ≤ (b / 6)

3.5.3 PRESSURE ON THE BASE SOIL

$$P = \frac{\sum V}{b}[1 \pm (6e/b)]$$

$\sum V$ = net vertical force = W - Pu

$$P_{max} \text{ at toe} = \frac{\sum V}{b}[1 + (6e/b)]$$

$$P_{min} \text{ at heel } = \frac{\sum V}{b}[1-(6e/b)]$$

3.5.4

For safety against foundation failure, the $P_{max} \leq$ safe bearing capacity of soil

3.5.5

$$\text{F.S against overturning} = \frac{W[b-x]}{P_H.(H/3)+P_{u.}N}$$

3.5.6

$$\text{F.O safety against sliding} = \frac{\mu.\sum V}{\sum P_H}$$

3.5.7

$$\text{Shear friction factor} = \frac{\mu.\sum V + b.q}{\sum P_H}$$

3.5.8

Principal stresses in the dam material near the toe will be the maximum principal stress (compressive), that is,

$$\sigma_1 = P_n.\sec^2\beta - p.\tan^2\beta$$

3.5.9

Shear stress (τ) at the toe will be

$\tau = (P_n - P)\tan^2\beta$

P_n = Max. stress on the soil at the toe

P = Water pressure at the toe = w h

B = slope of d/s face of dam with vertical.

For no tail water, P = 0

$$\sigma_1 = P_n.\sec^2\beta$$

For safety, σ_1 should be less than allowable compressive stress in concrete

4 SPILL WAYS

4.1 INTRODUCTION:

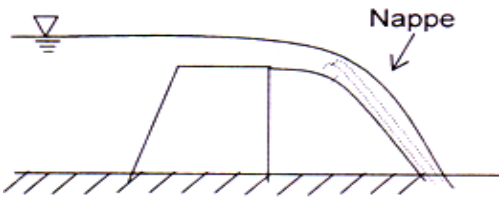
Necessity of spill ways

The spill ways are openings provided at the body of the dam to discharge safely the excess water or flood water when the water level rises above the normal pool level. A spillway is provided to discharge the excess flow of water from reservoir to downstream. A spillwayis thus, the safety valve for a dam.

4.2 TYPES OF SPILL WAYS

4.2.1 FREE OVERFALL OR STRAIGHT DROP SPILL WAY

A low height narrow crested weir having its downstream face vertical or nearly vertical.



Commonly used for low earth dams or weirs.
d/s apron will be subjected to large impact pressure.

4.2.2 OGEE SPILL WAY OR OVERFLOW SPILL WAY

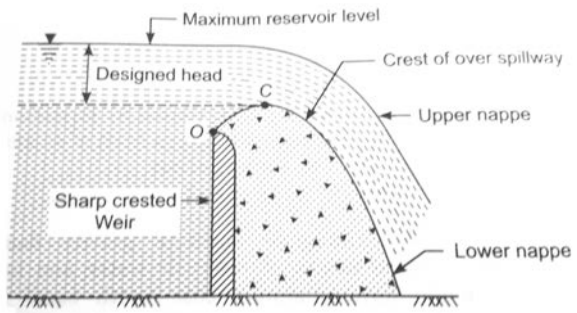
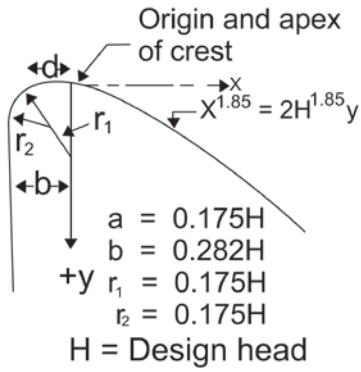


Fig. 18 Ogee spillway

- Widely used for gravity arch dams.
- The over flowing water is guided smoothly over the spillway and is made to glide over the downstream face of the spill way.
  The profile of a spillway is ogee or S-shaped.
- For an ideal profile of an ogee spillway, if discharge is occurring at the design head, the pressure on the spillway will be atmospheric only (i.e., no hydrostatic pressure on the spillway)
- For discharges at a head less than the design head, positive hydrostatic pressure will be exerted on the spillway by the flowing water.
- For discharges at a head greater than the design head, there will be separation of flowing water from the spillway and thus negative pressure (suction) will be created. The negative pressure will increase the effective head and there by the discharge will be increased.
- Discharge over an ogee spillway is proportional to $H^{3/2}$ (H is head of flow on the crest).
- Ogee spillway has relatively high coefficient of discharge of 2.2



(d) Elements of nappe-shaped crest profiles

The details of the crest profile are shown in the fig. The downstream curve of the ogee has the following equation :

$$x^{1.85} = 2\ H^{1.85}\ y$$

Where, x and y are the co-ordinates of the crest profile measured from the apex of the crest.

H is the design head.

4.2.3 SIPHON SPILL WAY

A siphon spillway is the one which utilises A siphonic action to discharge the surplus water.

Saddle Siphon spillway:

It is closed conduit of inverted U Shape. Commonly used types are

(1) Hood type

(2) Titled outlet type

4.2.4 VOLUTE SIPHON SPILL WAY

It consists of a vertical shaft having a funnel at the top end and the bottom end is connected to a bend pipe. When the water rises above the full reservoir level, it spills over the circumference of the funnel and flows with a spiral motion through it.

4.2.5 CHUTE OR TROUGH SPILL WAY

This spill way is simply a rectangular open channel or trough (known as chute) provided on the dam to discharge the surplus water from the reservoir to the same river on the downstream side. The spill way may be provided along the abutment of the dam or along the edge of the reservoir at the full supply level.

Shaft Spillway:

→ The shape is just like a funnel.

→ Also called as glory hole spillway.

→ When water drops through a vertical shaft in a foundation material to a horizontal conduit that conveys the water past the dam.

→ Lower end of shaft is turned at right angle and then water taken out below the dam horizontally.

Side - channel spillway:

A Side channel spillway is the one in which the flow, after passing over a weir or ogee crest, is carried away by a channel running essentially parallel to the crest.

Conduit or tunnel spillway:

Conduit or tunnel spillway is the one in which a closed channel is used to convey the discharge around or under a dam. The closed channel may be in the form of a vertical or inclined shaft, a horizontal tunnel through earth dam, a conduit constructed with open cut and back filled with earth materials.

4.2.6 CHANNEL SPILLWAY

It is completely separated from the main body of the dam. The spillway is constructed at right angles to the dam and at any side.

4.2.7 ENERGY DISSIPATION

When water spills and flows over the spillways, then it acquires a very high velocity, as the whole potential energy (due to potential head) is transformed into kinetic energy. The process of destruction of this kinetic energy is known as energy dissipation.

4.2.8 PURPOSE OF ENERGY DISSIPATION

To destroy the kinetic energy and to resist the erosion of the river bed. Some devices are adopted which are known as energy dissipaters.

4.3 TYPES OF ENERGY DISSIPATOR

1) Hydraulic jump with stilling basin
2) Buckets : buckets are generally Provided on the downstream for energy dissipation of spillways.

Hydraulic Jump :

• A hydraulic jump is defined as a rise in the level of water in an open channel.

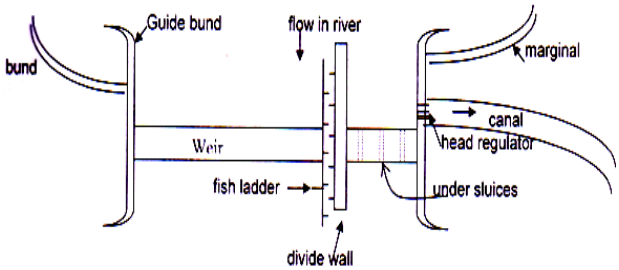
• It occurs when a liquid at a high velocity discharges in to a zone that has a lower velocity.

5 DIVERSION HEAD WORKS

5.1 INTRODUCTION

5.1.1 DIVERSION HEAD WORKS

Works constructed across the river to raise water level and to divert in to the canal and also other purposes



5.1.2 WEIR

Structure constructed across a river to raise its water level and divert into the canal. Usually shutters are provided to increase the storage

5.1.3 BARRAGE

The raising up of water level (or ponding) is accomplished mainly by means of gates. A barrage provides better control on the water level.

5.1.4 UNDER SLUICES

Opening provided in the weir wall, located on the same side of the off taking canal. The discharge capacity of the under sluices is greater of

i) 2 times maximum discharge of off taking canal
ii) Maximum winter discharge
iii) 10 to 15% of maximum flood discharge.

If no special device to control slit entering into the canal is provided, the crest of the under sluices should be at least 1.20 m lower than the crest of the head regulator.

If the excluder is provided to reduce entry of slit into the canal, the crest of under sluices should be about 1.80 to 2.0 m below the crest of the head regulator.

5.1.5 DIVIDE WALL

Constructed at right angles to the axis of the weir to separate under sluices from the rest of the weir to the canal.

5.1.6 BREAST WALL

Provided in a canal head regulator above the gates to prevent spilling of water over the gates into the canal during high floods.

5.1.7 SILT EXCLUDER

It excludes (prevents) silt from entering the canal. It is provided on the river bed in front of head regulator.

5.1.8 SILT EXTRACTORS OR SILT EJECTORS

They remove the silt which has already entered the canal from the head. It is provided in the canal.

5.1.9 FISH LADDER

To allow migration of fish from U/s to D/s side Guide bank and marginal bund are river protection works.

CANAL HEAD REGULATOR

It is the structure constructed at the head of the canal where it takes off from the river.

5.1.10 CAUSES OF FAILURES OF WEIRS

5.1.10.1 PIPING OR UNDER MINING

Progressive erosion in the backward direction (i.e.., towards u/s)

5.1.10.2 MEASURES TO PREVENT PIPING

Providing sufficient length of impervious floor so that path of percolation is increased and exit gradient reduced. Also by providing sheet piles at U/s and D/s ends of impervious floor.

5.1.10.3 UPLIFT PRESSURE

Uplift pressure is resisted by
1. Providing sufficient thickness of floor
2. Providing sheet pile at U/s end

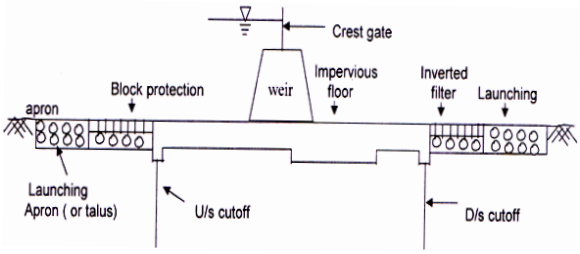
5.1.10.4 RUPTURE OF FLOOR DUE TO SUCTION CAUSED BY HYDRAULIC JUMP

Remedies : Providing additional thickness of flow

5.1.10.5 SCOUR ON U/S AND D/S OF WEIR DUE TO HEAVY VELOCITY OF RIVER FLOW

Remedies:
Providing sheet piles on U/s and D/s sides much below the calculated scour level. By providing launching aprons on U/s and D/s sides so that stone of aprons may settle into the scour holes.

5.2 CROSS SECTION OF WEIR



5.2.1 IMPERVIOUS FLOOR

To provide required creep length and to resist uplift pressure on D/s side
Small thickness and more length on U/s side. More thickness on D/s side

5.2.3 CUT OFF

To increase creep length and thereby reducing the hydraulic gradient.

5.2.4 INVERTED FILTER

Consists of layer of materials of increasing permeability from bottom to top. It is provided on d/s, after the impervious floor to relieve uplift pressure i.e., it allows seepage water to escape without dislocating the soil particles.

5.2.5 LAUNCHING APRON OR PERVIOUS APRON

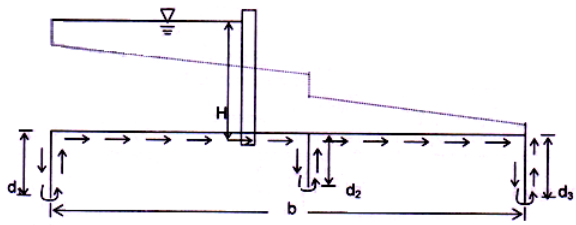
It is loosely packed stones, to protect the impervious floor and sheet piles from scour holes (piping) progressing towards the impervious floor and sheet piles. provided on U/s and D/s side.

5.2.6 BLOCK PROTECTION

Provided on U/s side, immediately at the upstream end of impervious floor, to protect impervious floor from the effect of scouring. This is made of concrete blocks or stone blocks over a bed of loose stone packing. The joints are finished with cement mortar.

5.3 DESIGN OF WEIRS ON PERMEABLE FOUNDATION

Bligh 's creep theory



- Bligh assumed that the percolating water creeps along the base profile of the structure which is in contact with the soil
- Creep length (L) : The length of path traversed by percolating water
  Total creep length, $L = b + 2d_1 + 2d_2 + 2d_3$
- Bligh assumed that head loss per unit length of creep (i.e., hydraulic gradient) is constant throughout the percolating passage.
  hydraulic gradient , I = (H/L)
- Bligh's creep coefficient (C) : it is the reciprocal of hydraulic gradient

$$C = \frac{1}{i} = \frac{L}{H}$$

$$\Rightarrow L = CH$$

5.3.1 Safety against piping

As per Bligh, to avoid piping the minimum length of creep.
L = CH or the hydraulic gradient , $i \leq (1/C)$
H = seepage Head (the difference between Water levels on U/s and D/s)
Worst condition is that when the water level on U/s side is maximum and no water on D/s side.
The value of C varies from 5 to18 depending upon the type of soil. For fine sand and mud.
C = 18, For Coarse Sand C = 12, for gravel C = 5 to 9

5.3.2 SAFETY AGAINST UPLIFT PRESSURE

As per Bligh's theory, thickness to be provided by taking a factor of safety of 4/3 is, $t = \frac{4}{3}\frac{h}{(G-1)} = h$ = ordinate of Hydraulic Gradient line from top of floor
It would be more economical to provide more creep length on U/s side instead of D/s side. According to Bligh a vertical cut off at U/s end of the floor is more useful than the one at D/s end of the floor.

5.4 LIMITATIONS OF BLIGH'S CREEP THEORY

- No distinction between vertical and horizontal creep
- No distinction between effectiveness of outer and inner faces of sheet pile
- Significance of exit gradient is not considered

5.5 Lanes weighed creep theory

- Lane proposed that horizontal creep is less effective in reducing uplift or causing loss of head, than the vertical creep. A weightage factor of 1/3 is proposed for horizontal creep and 1.0 for vertical creep.
- Weighted creep length , L = 1/3 B + V
  B = sum of horizontal contacts and sloping contacts less than 45° to the horizontal
  V = sum of vertical contacts and sloping contacts greater than 45° to the horizontal
  For safety against piping, $\frac{H}{L} \leq \frac{1}{C}$
  C_1 = Lane's coefficient of creep

5.6 Khosla 's theory

5.6.1
Outer faces of sheet piles are more effective than inner ones

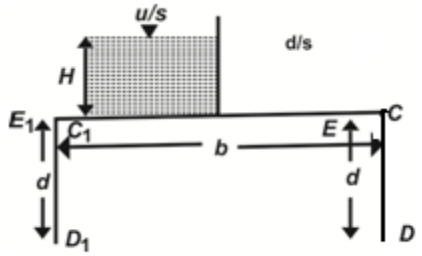
5.6.2
Intermediate sheet piles if smaller in length than outer ones, are ineffective.

5.6.3
Undermining of floors starts from D/s and if hydraulic gradient at D/s point is more than the critical hydraulic gradient

5.6.4
A Vertical cut off at the D/s end of floor is essential to prevent undermining
The floor length is fixed such that at the D/s end the exit gradient is less than the permissible gradient.

exit gradient, $G_e = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}$

The allowable exit gradient should be within 1/5 to 1/6



$$\phi C_1 = 100 - \phi_E, \quad \phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$$

$$\phi D_1 = 100 - \phi_D, \; \phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right)$$

$$where \qquad \lambda = \frac{1 + \sqrt{1 + a^2}}{2} \qquad \left(\alpha = \frac{b}{d}\right)$$

6 DESIGN OF UNLINED CANALS

6.1 INTRODUCTION

KENNEDY'S THEORY

Silt is kept in suspension by the eddies formed over the width of the channel.
Critical Velocity (Vo): Velocity which will keep the channel free from silting or scouring.
Vo = 0.55 $D^{0.64}$ -- applicable for Punjab region (now in Pakistan)
Vo = critical velocity in m/sec d = depth of flow, in m
bed width has no effect on the Critical Velocity
Critical Velocity Ratio , m = V_k/V_o
For other regions, Velocity of flow to avoid silting and scouring is, V_k= 0.55 m $D^{0.64}$
For determining mean velocity of flow, Kennedy recommended to use Kutter' Equation, which is as follows :

$$V = \frac{23 + \frac{1}{N} + \frac{0.00155}{s}}{1 + (23 + \frac{0.00155}{s})\frac{N}{\sqrt{R}}}\sqrt{RS}$$

V = Mean velocity of flow in m/sec.
R = Hydraulic radius, m
S = bed slope,
N = Rugosity coefficient
Design of channel by Kennedy : * Equations required
1) Q = A.V
2) Kutter's Equation, for V
3) V_k= 0.55m$D^{0.64}$
....Data required: Q, N, m and S or B/D ratio
Procedure (1) Given Q, N, m and S steps
Assume a trial value of D in meters
Calculate velocity V from equation,
V_k = 0.55m $D^{0.64}$
Calculate cross sectional area,
A = Q/V
Assuming side slope of channel as (1/2 Horizontal) to (1 Vertical) and find the bed width.
Calculate the actual mean velocity (v) of flow from kutter's Equation.
If the velocity (v) calculated by kutter's equation is nearly equal to velocity calculated in step no.2, the assumed depth is correct. If not, assume another trial value D and repeat the above procedure.
The above design procedure can be carried out with the help of Garret's Diagram.

6.2 Drawbacks in Kennedy's Theory

Limitations of Kutter's Equation are incorporated in Kennedy's theory
No equation for bed slope (s) by Kennedy
Complex Phenomenon of still transportation is incorporated in a single factor called 'm' involves trial and error.
Lacey's Theory: Lacy developed the regime theory (But the regime theory concept was initially put forwarded by Lindley)
Regime Channel: A stable channel whose width, depth and bed slope have undergone modification by silting and scouring and are so adjusted that they have attained equilibrium.
Silt is kept in suspension by eddies generated form wetted perimeter.

Lacey's regime theory

For a channel to be in regime, conditions to be established are :
Regime conditions (true regime conditions)

i) Channel should be flowing uniformly in unlimited incoherent alluvium of the same character as that transported by channel.

ii) Silt grade and silt charge should be constant.
iii) Discharge should be constant.
To measure silt grade, Lacey introduced silt factor (f) :
Incoherent Alluvium : A loose granular material which can be scoured as easily as it can be deposited.
An artificial channel will undergo two stages of regime.
1) Initial regime
2) final regime
Initial Regime: Longitudinal slope and depth attain equilibrium
Final Regime: Longitudinal slope, depth and width of the channel reach equilibrium.
Lacey's equations are applicable for a channel which has attained final or true regime.
Cross Section of a regime Channel : Semi – elliptical section

6.3 LACEY'S REGIME EQUATIONS

Design procedure using Lacey's Theory

Step-1 : Calculate the velocity by

$$V = \left(\frac{Qf^2}{140}\right)^{\frac{1}{6}} m/s$$

Where f is silt factor = $1.75\sqrt{dmm}$
V = velocity in m/s
Q= discharge in cumec

Step – 2 : Find the hydraulic mean depth

$$R = \frac{5}{2}\left(\frac{v^2}{f}\right)$$

Step - 3 : Find the area of channel section by

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

Step - 4 : Find the wetted perimeter by

$$P = 4.75\sqrt{Q}$$

Step - 5 : Find the bed slope by

$$S = \frac{f^{\frac{5}{3}}}{3340Q^{\frac{1}{6}}}$$

6.4 DRAWBACKS IN LACEY'S THEORY

Regime conditions are only theoretical, may not be achieved in practice
Equations are derived based on single factor 'f' Silt charge and silt grade have not been properly defined Lacey's equations are empirical.

The concentration of silt is not taken in to account.

The characteristics of regime of channel may not be same for all cases.

7 CROSS DRAINAGE WORKS

Cross Drainage work :

A cross drainage work is a structure carrying the discharge from a natural stream across a canal intercepting the stream. Canal comes across obstructions like rivers, natural drains and other canals.

TYPE I: CD works carrying the canal over the natural drain.

7.1 AQUEDUCT

- To carry a canal over a natural drain.
- It is adopted where the bed of the canal is well above the H.F.L of the drain.
- The drain flows under atmospheric pressure.

7.1.1 SIPHON AQUEDUCT

- Siphon aqueduct is constructed where water surface level of drain at high flood is higher than the canal bed. The bed of the drain is depressed and provided with pucca floor the drain flows under pressure.
  Uplift pressure occurs on the under side of the trough (or the barrel) i.e on the bottom of the structure which is crossing over the natural drain :
- The max .uplift pressure occurs on the under side of the trough at the upstream end of the barrel.
- The uplift pressure would be max. when the highest flood is passing in drain and when there is no water in the trough at that time.

TYPE II : CD works carrying the natural drain over the canal.

7.2 SUPER PASSAGE

- To carry natural drain over the canal.
- Constructed where the bed of the drain is well above the canal F.S.L.
- Syphon is constructed where the F.S.L of canal is higher than the bed of the drain.
- The max uplift pressure on the underside of the drain bed occurs when there is no water flowing in the drain and the water table has risen up to drain bed.

TYPE II : CD works admitting the drain water into the canal.

7.3 LEVEL CROSSING

It is provide when the beds of the canal and the drain are at the same level.

7.3.1 INLET AND OUTLET

- Inlet is provide in a canal bank to admit drain water into the canal.
- Outlet allows the admitted drain water to discharge out from the canal.

Section of type of cross drainage works

- Relative bed levels
- Availability of suitable foundation
- Economical consideration
- Discharge of the drainage
- Construction problems.

8 CANAL OUTLETS

8.1 INTRODUCTION

8.1.1 OUTLET

It is a device through which water is released from a channel into a water course or field channel.

8.2 TYPES OF OUTLETS

8.2.1 MODULAR OUTLET

- In this type, the discharge is independent of water levels in the distributing channel and water course. i.e., Constant discharge is obtained.
- The examples for modular outlets are a) Gibb's module b) Khanna 's Module.

8.2.2 NON MODULAR OUTLET

- Discharge depends on difference in water levels in the distributing channel and water course e.g Submerged pipe outlet
- The main advantage is that they can work with very small available heads.

8.2.3 SEMI MODULAR OUTLET

- Discharge varies only with water level in the distributing channel but it is independent of water levels of the water course.
- The semi modular outlets are pipe outlet, Kennedy's gauge outlet, open flume outlet etc.

8.3 SENSITIVITY

It is the ratio of rate of change of discharge of an outlet to the rate of change of water level of distributing channel.
[(dq/q)/(dy/y)].

8.3.1 FLEXIBILITY

- It is the ratio of rate of change of an outlet to the rate of change of discharge of the distributing channel [(dq/q)/(dy/y)].
- For a modular outlet flexibility and sensitivity is equal to zero and hence it is also called as rigid module.

8.3.2 PROPORTIONALITY

If the flexibility is equal to one, then the outlet is termed as proportional outlet.

8.3.3 EFFICIENCY

It is defined as the ratio of the head recovered to head put in.

8.4 DROWNING RATIO

It is defined as the ratio between the depths of water levels over the crest on the D/S and U/S of the outlet.

9 CANAL REGULATION WORKS

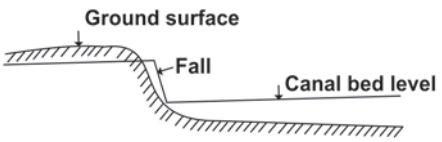
9.1 INTRODUCTION

These are structures constructed to regulate the discharge, full supply level or velocity of flow in a channel. The various canal regulation works are as follows.

Necessity of canal falls:

→When the slope of the ground suddenly changes to steeper slope, the permissible bed slope cannot be maintained. It requires excessive earth work in filling to maintain the slope.

→ In such case falls are provided to avoid excessive earth work in filling.



→ When the slope of the ground is more or / less uniform and the slope is greater than the permissible bed slope of canal.

9.2 Types of falls

Ogee fall

→ In this type of fall, the gradual convex and concave surface is provided with an aim to provide smooth transition and to reduce disturbance and impact.

→A hydraulic jump is formed which dissipates a part of kinetic energy.

→Upstream and downstream of the fall is provided by stone pitching.

Stepped fall

→It consists of a series of vertical drops in the form of steps.

→ It is provided where sloping ground is very long and requires a long glacis to connect higher bed level u/s with lower bed level d/s.

→It is a modification of rapid fall

→Vertical drops are provided to protect the canal bed and sides from damage by erosion.

→Brick walls are provided at each drop.

Rapid fall

→ When the natural ground level is even and rapid, this rapid fall is suitable.

→ It consists of long sloping glacis.

→ Curtain walls are provided on both u/s and d/s sides.

→ Rubble masonary with cement grouting is provided from u/s curtain wall to d/s curtain wall.

Vertical drop fall

- It is a structure constructed across a channel to lower down the water level and dissipate surplus energy. It is required when natural slope of the ground is greater than designed bed slope of the channel. The difference in slopes is adjusted by constructing a vertical drop.

Trapezoidal notch fall

- If a trapezoidal notch fall is provided , there is neither draw down nor heading up of water on u/s of the fall. The depth – discharge relationship of the channel remains undisturbed.

9.2 STRAIGHT GLACIS TYPE OF FALL

It utilizes hydraulic jump for the dissipation of energy.

It consists of straight glacis provided with a crest wall.

Curtain walls are provided at toe and heel.

Stone pitching is required at upstream on & downstream of the fall.

Well or Cylinder Notch fall

In this type, water of canal from higher level is thrown in a well or a cylinder from where it escapes from bottom.

Baffle or Inglis fall

Here, glacis is straight and sloping, but baffle wall provided on the downstream floor, dissipate the energy.

Main body of glacis is made of concrete.

Curtain walls both at toe and heel are provided.

9.2.1 SIPHON FALLS

Siphon falls are adopted to maintain a constant level on u/s side

9.2.2

Roughening devices are provided in the cisterns for dissipation of energy.

9.3 Montague type fall

Hydraulic jump falls and it is always flumed (contracted). It's profile is parabolic.

9.3.1

King's vanes and Gibb's Groyne wall are devices to control silt entry into the off taking channel.

9.3.2 Bed bars

Constructed in the bed of unlined channel to serve as a permanent mark of reference for indicating alignment.

9.3.3 Escapes

- Structured in the bed of unlined channel for the disposal of surplus water from the channel (it is called **Surplus Escapes**).
- Sometimes, escapes are provided in the head reaches of canal to scour out bed silt deposited in the canal. They are called canal **Scouring Escapes**.
- Sometimes, at the tail end of the channel when it meets a drain, an escapes is provided to maintain the required FSL in the canal. Such escapes are called **Tail escapes**.

Head Regulator

Regulators constructed at the off taking point are called head regulators.

When it is constructed at the head of main canal it is known as canal head regulator.

When it is constructed at the head of distributary, it is called distributary head regulator.

Function:

To control the entry of water either from the reservoir or from the main canal.

To control the entry of silt in to off taking or main canal.

To serve as a meter for measuring discharge of water.

Cross Regulator

A regulator constructed in the main canal or parent canal downstream of an off take canal is called cross-regulator.

10 WATER LOGGING

10.1 WATER LOGGING

- Water logging is a condition in which there is excessive moisture in the soil making the land less productive of circulation of air.
- The water logging affects the fertility of the land and leads to a reduction in the crop yield. Water logging is usually caused by a rise of sub soil water table.
- The depth of water table at which it tends to make the land water logged, depends on the (i) height of capillary fringe and (ii) type of crop. The crop yield is adversely affected when the capillary meniscus surface rises to within 0.6 m of ground surface.
- For most of the agricultural soils, the height of capillary fringe varies 0.9 m to 1.5 m. The land will, therefore, be water logged when the water table is within 1.50 m to 2.1 m below the ground.

10.2 CAUSES OF WATER LOGGING: MAIN CAUSES OF WATER LOGGING ARE

1. Excessive rainfall in the area
2. Flat ground profile
3. Seepage of water from canals and the adjoining lands.
4. Improper drainage of surface runoff
5. High rate of infiltration of water
6. Excessive irrigation
7. Submergence due to flood

10.3 EFFECTS OF WATER LOGGING: LOGGING HAS THE FOLLOWING EFFECTS

1. It causes anaerobic conditions near the roots of plants. Excess water prevents circulation of air and hence destroys bacteria which require aerobic condition and other chemicals to live and produce nitrates required by plants. This reduces the yield of crop.
2. It makes cultivation difficult as the water logged areas cannot be easily cultivated.
3. It causes growth of wild aquatic plants
4. It causes salinity of soil
5. It lowers soil temperature which affects the activities of the bacteria
6. Results in reduction of yield.
7. Root growth is restricted.
8. Plants will be attacked by certain diseases.

10.4 WATER LOGGING CONTROL

1. By providing adequate surface drainage with open drains through which rain water could be quickly disposed off.
2. By providing efficient under drainage.
3. By controlling seepage of water from canals.
   - By Lining the canals
   - Lowering F .S . L
   - Using irrigation water economically
4. By preventing seepage from reservoirs.
5. Pumping surplus ground water from wells.
6. By introducing crop rotation
7. By improving the natural drainage of the area
8. By introducing lift irrigation

11 LAND DRAINAGE

11.1 INTRODUCTION

A properly designed drainage system is an effective means to prevent land from getting water logged as well as to relive the land already water logged.

11.2 TYPES OF DRAINS FOR DRAINAGE

1) Open drains
2) Closed drains

Open drains are further classified into two types :

11.2.1 SHALLOW SURFACE DRAIN

Used to drain irrigation water supplied to fields and also to quickly dispose of storm water.

11.2.2 Deep open drains:

Used to drain out the subsoil. It is useful for prevention and also for relieving the land which is already water logged.
The open drains are provided with trapezoidal cross- section and designed on the same principles of irrigation channels.

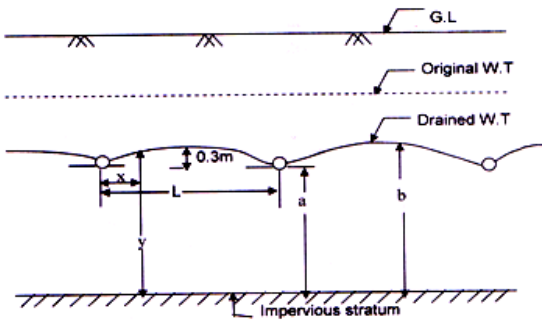
11.3 DRAW BACKS OF OPEN DRAINS

i) For construction of these drains, valuable agricultural land is wasted.
ii) Obstruct the farming operations.
iii) Bridges are required on the drains.
iv) Valuable plant nutrients are washed down in the open drains.

11.3.1 CLOSED DRAINS (TILE DRAINS)

Useful for prevention of water logging of land as well as for relieving the land already water logged. These drains are provided at suitable depth below the ground level with open joints and covered with filter of coarse sand. The usual diameter of tile drain is 10 cm. The drains are located about 0.3 m lower than desired water table. The usual spacing of drains is from 15 to 45 m. These are laid at a steeper than 1 in 500. The closed drains have their outlets in natural or artificial channels.
Closed drains are designed to carry only seepage water and their capacity is determined by the rate of infiltration.

11.4 FLOW OF GROUND WATER TO THE DRAINS AND SPACING OF DRAINS



Let 'L' = the spacing of drains
'a'=height of the drain above the impervious stratum
'b' = max height of the drained W.T above the impervious stratum

11.4.1 ASSUMPTIONS

i) The hydraulic gradient at a distance x from the centre of drain is dy/dx
ii) Flow lines are parallel and area of flow section at a distance x from the per unit Length of the drain is y × 1

iii) Discharge towards the drain is inversely proportional to the distance from the drain

iv) Darcy's law is valid.

$$Q_y = k\left(\frac{dy}{dx}\right)\cdot(y\times 1)$$

Where Q_y is the discharge passing through the section at a distance 'X' from the drain.

Let Q_d be the total discharge of each drain per unit length.

*When x=L/2, the discharge is zero

*When x = 0, the discharge from each side is $\frac{Q_d}{2}$

It can be derived that, $L = \frac{4k(b^2 - a^2)}{Q_D}$

The above equation shows that, the spacing of drains is independent of drain size.

The Q_D will depend upon infiltration into the ground.

*Commonly the Q_D is adopted as 1% of the average annual rainfall drained of in 24 hours.

12 QUALITY OF IRRIGATION WATER

12.1 INTRODUCTION

The quality of irrigation water is determined by the concentration of salts (salts of chlorides, sulphates and borates of sodium, potassium , calcium , magnesium) in water. The concentration of these salts is determined by measuring electrical conductivity of water which is usually expressed in unit of millimhos per cm.
If sodium ions predominate in irrigation water, they will tend to replace calcium & magnesium ions from the clay. As a result, the soil clods crumble and fine silica grains are released which clog the soil resulting in the reduction of its permeability and destruction of the structure of soil. Therefore, the quality of water should be determined by assessing the exchangeable sodium ions present in water.

12.2 EXCHANGEABLE SODIUM RATIO (ESR):

- It is defined as the concentration of exchangeable sodium ions divided by the sum of concentrations of exchangeable calcium, magnesium , sodium and potassium ions.
- Exchangeable Sodium Ratio = $\frac{Na^{+}}{Ca^{++} + Mg^{++} + Na^{+} + K^{+}}$
- The ESR is often expressed in % which is then termed as Exchangeable Sodium Percentage (ESP).
  *The concentration of all the elements are expressed in mill equivalents per litre (meq/l) which is equal to the concentrations is ppm divided by the equivalent weight of the element.

12.2.1 SODIUM ADSORPTION RATIO (SAR)

The concentration of exchangeable sodium ions in water may also be determined by another more rational parameter termed as SAR.

$$\text{Sodium Adsorption Ratio} = \frac{N_a^{+}}{\sqrt{\frac{C_a^{++} + M_g^{++}}{2}}}$$

SAR	Type of water
0-18	low sodium water (S1)
10-18	medium – sodium water (S_2)
18-26	high- sodium water (S_3)
26	very high – sodium water (S_4)

12.2.3 SALTS IN THE SOIL

Some of the mineral salts present in the agricultural soil are injurious to the plant. These salts are also called alkali salts and common examples are Nacl, $Na2SO_4$ and NA2 CO_3. Out of these, NA_2CO_3 is most harmful and NaCl is least harmful. The Na_2CO_3 is sometimes called 'black alkali'.

12.2.4 SALT EFFLORESCENCE

- When the ground W.T is closed the water from the W.T rises up by capillary action and brings with it the alkali salts in solution.
- Water evaporates from the surface of land, leaving behind an accumulation of these salts, in patches, on the surface of land.
- This phenomenon of the salts coming in solution and forming a thin crust (5 to 7.5 cm) on the surface after evaporating of water is called "salt efflorescence".

12.2.5 SALINE SOIL

- The excess concentration of salts, when present in the root zone of any plant, has corroding effect on the roots and

the growth of the plant gets checked and the plant ultimately dies. Such salt affected soil is known as 'Saline soil' and is unproductive.

- The electrical conductivity of saline soils is greater than 4 millimhos/cm (at 25^0c) and the exchangeable sodium percentage is less than 15. The pH value is < 8.5. The salts present in saline soil usually appear in the form of a white efflorescent crust on the surface of soil and hence the saline soils are also commonly termed as "white alkali" soils. The saline soils can be reclaimed by leaching process.

12.2.6 ALKALINE SOIL OR SODIUMIZED SOIL

- If the salt efflorescence is allowed to be on land for some time and the soil is clayey, a Base Exchange reaction may take place, thus 'sodiumising' the clayey. Because of sodiumization, the soil becomes impermeable, ill aerated and highly unproductive. Such a 'sodiumzied soil' is 'Alkaline soil' . Reclamation of alkaline soil is more difficult than a saline soil.
- For Alkali soil electrical conductivity is less than 4 millimhos/cm and ESP < 15 and pH value lies between 8.5 and 10. These soils are also termed as "black alkali" soils because a black crust forms on the surface of these soils. These soils can be reclaimed by reducing the exchangeable sodium percentage and removing the released sodium salts. Gypsum and sulphur are added for reclamation leaching process.

12.2.7 RECLAMATION OF SALT AFFECTED LAND

a) By providing adequate drainage system so that the W.T will be lowered to the required limit.

b) By leaching technique: Excess water is applied so that the salts from the upper soil get washed out.

c) By growing certain salt resisting crops : Rice and gram can tolerate alkali salts to a certain extent.

d) By chemical treatment: When $Na_2CO_3$is present in a salt affected soil ,chemical called gypsum ($CaSO_4$) is mixed with soil, to reclaim.

GATE QUESTIONS

Topics

1. WATER REQUIRMENTS OF CROPS
2. DESIGN OF STABLE CHANNELS
3. THEORIES OF SEEPAGE, SPILLWAYS
4. DESIGN AND CONSTRUCTION OF GRAVITY DAMS

1 WATER REQUIRMENTS OF CROPS

Q.1 The total irrigation depth of water, required by a certain crop in its entire growing period (150 days), is 25.92 cm. The culturable command area for a distributary channel is 100000 hectare. The distributary channel shall be designed for a discharge.
a) less than 2 cumecs
b) 2 cumecs
c) 20 cumecs
d) more than 20 cumecs
[GATE-2003]

Q.2 The moisture content of soil in the root zone of an agricultural crop at certain stage is found to be 0.05.The field capacity of the soil is 0.15. The root zone depth is 1.1 m. The consumptive use of crop at this stage is 2.5 mm/day and there is no precipitation during this period. Irrigation efficiency is 65%. It is intended to raise the moisture content to the field capacity in 8 days through irrigation. The necessary depth of irrigation is
a) 115 mm b) 169 mm
c) 200mm d) 285 mm
[GATE-2003]

Q.3 A canal irrigates a portion of a culturable command area to grow sugarcane and wheat. The average discharges required to grow sugarcane and wheat are, respectively, 0.36 and 0.27 cumec. The time factor is 0.9. The required design capacity of the canal is
a) 0.36 cumec b) 0.40 cumec
c) 0.63 cumec d) 0.70 cumec
[GATE-2004]

Q.4 A sprinkler irrigation system is suitable when
a) the land gradient is steep and the soil is easily erodible
b) the soil is having low permeability
c) the water table is low
d) the crop to be grown have deep roots
[GATE-2004]

Q.5 The culturable commanded area for a distributary is $2\times10^8 m^2$. The intensity of irrigation for a crop is 40%. If kor water depth and kor period for the crop are 14 cm and 4 weeks, respectively, the peak demand discharge is

a) 2.63 m^3/s b) 4.63 m^3/s
c) 8.58 m^3/s d) 11.58 m^3/s
[GATE-2005]

Q.6 In a cultivated area, the soil has porosity of 45% and field capacity of 38%. For a particular crop, the root zone depth is 1.0 m, the permanent wilting point is 10% and the consumptive use is 15 mm/day. If the irrigation efficiency is 60%, what should be the frequency of irrigation such that the moisture content does not fall below 50% of the maximum available moisture?
a) 5 day b) 6 day
c) 9 day d) 15 day
[GATE-2006]

Q.7 The culturable command area for a distributed channel is 20,000 hectares. Wheat is grown in the entire area and the intensity of irrigation is 50%. The kor period for wheat is 30 days and the kor water

depth is 120 mm. The outlet discharge for the distributary should be

a) 2.85 m^3/s b) 3.21 m^3/s
c) 4.63 m^3/s d) 5.23 m^3/s

[GATE-2007]

Q.8 An outlet irrigates an area of 20 ha. Discharge (L/s) required at this outlet to meet the evapotranspiration requirement of 20 mm occurring uniformly in 20 days neglecting other field losses

a) 2.52 b) 2.31
c) 2.01 d) 1.52

[GATE-2008]

Q.9 The consumptive use of water for a crop during a particular stage of growth is 2.0 mm/day. The maximum depth of available water in the root zone is 60 mm. Irrigation is required when the amount of available water is 50% of the maximum available water in the root zone. Frequency of irrigation should be

a) 10 days b) 15 days
c) 20 days d) 25 days

[GATE-2007]

Q.10 An agricultural land of 437 hectare is to be irrigated for a particular crop. The base period of the crop is 90 days and the total depth of water required by the crop is 105 cm. If a rainfall of 15 cm occurs during the base period, the duty of irrigation water is

a) 437 hectare/cumec
b) 486 hectare/cumec
c) 741 hectare/cumec
d) 864 hectare/cumec

[GATE-2009]

Common Data for Questions 11 and 12.

The moisture holding capacity of the soil in a 100 hectare farm is 18 cm/m. The field is to be irrigated when 50% of the available moisture in the root zone is depleted. The irrigation water is to be supplied by a pump working for 10 hours a day, and water application efficiency is 75%. Details of crops planned for cultivation are as follows:

Crop	Root zone depth (m)	Peak rate of moisture use (mm/day)
X	1.0	5.0
Y	0.8	4.0

Q.11 The capacity of irrigation system required to irrigate crop X in 36 hectares is

a) 83 L/s b) 67 L/s
c) 57 L/s d) 53 L/s

[GATE-2010]

Q.12 The area of crop Y that can be irrigated when the available capacity of irrigation System is 40 L/s

a) 40 hectare b) 36 hectare
c) 30 hectare d) 27 hectare

[GATE-2010]

Q.13 Wheat crop requires 55 cm of water during 120 days of base period. The total rainfall during this period is 100 mm. Assume the Irrigation efficiency to be 60%. The area (in ha) of the land which can be irrigated with a canal flow of $0.01 m^3/s$ is

a) 13.82 b) 18.85
c) 23.04 d) 230.40

[GATE-2012]

Q.14 The translation of rice requires 10 days and total depth of water required during transplantation is 48 cm. During transplantation, there is an effective rainfall (useful for irrigation) of 8 cm. The duty of irrigation water (in hectares/ cumec) is:

a) 612 b) 216
c) 300 d) 108

[GATE-2013]

Q.15 Irrigation water is to be provided to a crop in a field to bring the moisture content of the soil from the existing 18% to the field capacity of the soil at 28%. The effective root zone of the crop is 70 cm. If the densities of the soil and water are 1.3g/cm^3 and 1.0g/cm^3 respectively, the depth of irrigation water (in mm) required for irrigating the crop is _____.

[GATE-2014]

Q.16 The two columns below show some parameters and their possible values.

Parameter	Value
P-Gross Command Area	I-100 hectares/cumec
Q-Permanent Wilting Point	II-6°C
R-Duty of canal water	III-1000 hectares
S-Delta of wheat	IV-1000 cm
	V-40 cm
	VI-0.12

Which of the following options matches the parameters and the values correctly?

a) P-I, Q-II, R-III, S-IV
b) P-III, Q-VI, R-I, S-V
c) P-I, Q-V, R-VI, S-II
d) P-III, Q-II, R-V, S-IV

[GATE-2015]

Q.17 A field channel has cultivable commanded area of 2000 hectares. The intensities of irrigation for gram and wheat are 30% and 50% respectively. Gram has a kor period of 18 days, kor depth of 12 cm, while wheat has a kor period of 18 days and a kor depth of 15 cm. The discharge (in m^3/s) required in the field channel to supply water to the commanded area during the kor period is _____ .

[GATE-2015]

Q. 18 The culturable command area of a canal is 1000ha. The area grows only two crops – rice in the kharif season and wheat in the rabi season. The design discharge of the canal is based on the rice requirements, which has an irrigated area of 2500ha, base period of 150 days and delta of 130 cm. The maximum permissible irrigated area (in ha) for wheat, with a base period of 120 days and delta of 50 cm, is _____.

[GATE-2017]

Q. 19 The intensity of irrigation for the kharif season is 50% for an irrigation project with culturable command area of 50,000 hectares. The duty for the kharif season is 1000 hactare cumec. Assuming transmission loss of 10%, the required discharge (in cumec, up to two decimal places) at the head of the canal is_____.

[GATE-2018]

ANSWER KEY:

1	2	3	4	5	6	7	8	9	10	11	12	13	14
(d)	(c)	(d)	(a)	(b)	(d)	(c)	(b)	(b)	(d)	(b)	(d)	(a)	(b)
15	**16**	**17**	**18**	**19**									
91	(b)	1.427	5200	27.78									

EXPLANATIONS

Q.1 (d)

Depth of water required by crop,
h = 25.92 cm = 0.2592 m
Growing period,
B = 150 days
= 150 × 86400 seconds
Area, A = 100000 hectare
= 10^9 m^2
Water supplied by distributing = Water required by crop
Design discharge × B = A × h
Q × 150 × 86400 = 10^9 × 0.2592
⟹ Q = 20m^3/s
Demand of water is not uniform in whole base period and canal may also remain close for some time for maintenance purpose, so canal should be designed for discharge more than 20 m^3/s.

Q.2 (c)

Moisture content, w = 0.05
Field capacity, F_c = 0.15
Root zone depth,
d = 1.1 m = 1100 mm
Consumptive use,
C_u = 2.5 mm/day
Efficiency, 65%
η = 0.65
It is intended to raise the moisture content to field capacity in 8 days.
So, depth of water required to meet the consumptive use demand in 8 days,
h_1 = 2.5 × 8 = 20 mm
So, necessary depth of irrigation = $\frac{(F_c\text{-}w).d+h_1}{\eta}$

$$= \frac{(0.15-0.05)\times 1100+20}{0.65}$$

= 200 mm

Note: As γ_d is not given neither we can calculate it due to insufficient data so we have to assume that given moisture content is volumetric moisture content.

i.e. moisture content w = $\frac{V_w}{V}$

where V_w → Volume of water
V → Volume of soil
∴ depth of water = (FC – w) × depth of root zone.

Q.3 (d)

Sugarcane requires irrigation throughout the year and wheat during the season of winter (Rabi). So, design capacity will be sum of two discharges divided by time factor.

$$Q_{design} = \frac{Q_S + Q_w}{\text{time factor}}$$

$$= \frac{0.36+0.27}{0.9} = 0.70\ m^3/s$$

= 0.70 cumec

Q.4 (a)

The conditions favouring the use of sprinkler irrigation method are:

1. When the land topography is irregular.
2. When the land gradient is steeper & soil is easily erodible.
3. When the soil is excessively permeable.
4. When water table is high.
5. When water availability is difficult and scarce.

Q.5 (b)

Area to be irrigated = culturable command area × intensity of irrigation

$= 2 \times 10^8 \times 0.4 = 8 \times 10^7\ m^2$
Kor depth = 0.14
Kor period = 4 weeks
Water discharged in Kor period = Volume of water required by crop
$Q \times 4 \times 7 \times 86400 = 8 \times 10^7 \times 0.14$
$\Rightarrow Q = 4.63\ m^3/s$

Q.6 (d)
Depth of root zone,
H = 1m = 1000 mm
Field capacity, $w_f = 38\%$
Permanent wilting point $w_p = 10\%$
Moisture content can fall upto 50% of maximum available moisture, so the moisture content which plant can use is,
w = 50% of (38% - 10%)
= 14%
Porosity, n = 45%

$\therefore$ Void ratio, $e = \frac{n}{1-n} = 0.818$

Assuming that at field capacity soil is fully saturated (S = 1)

$\therefore Se = w_f G_s \quad \Rightarrow G_s = \frac{Se}{w_f}$

$= \frac{1 \times 0.818}{0.38} = 2.153$

Dry unit wt. of soil,

$\gamma_d = \frac{G_s \gamma_w}{1+e} = \frac{2.153\gamma_w}{1.818} = 1.184\ \gamma_w$ Depth of water within root zone which plant can use

$h = \frac{\gamma_d}{\gamma_w} \times (\text{depth of root zone}) \times w$

$= 1.184 \times 1000 \times 0.14$
= 165.8 mm
Consumptive use, C_u = 15 mm/day

Frequency of irrigation $= \frac{h}{C_u}$

$= \frac{165.8}{15} = 11.05 \simeq 11$ days

Note: Irrigation efficiency has no use in calculating frequency of irrigation. However if it has been asked to calculate the depth of water required for irrigation then irrigation efficiency is useful parameter.

Depth of water to be applied $= \frac{165.8}{\eta_a}$

$= \frac{165.8}{0.6} = 276.33$ mm

(Where f is Lacey's silt factor)

Q.7 (c)
Area to be irrigated,
A = culturable command area × intensity of irrigation
$= 20000 \times 0.5 = 10000$ hectare
Kor period,
t = 30 days = 30×86400 s
Kor water depth,
d = 120 mm = 0.12 m
Water supplied by distributary in kor period = Water required by crop
$Q \times t = A \times d$
$Q \times 30 \times 86400 = (10000 \times 10^4) \times 0.12$
$\Rightarrow Q = 4.63\ m^3/s$

Q.8 (b)
Volume of water required for evapotranspiration = Area × depth
$= (20 \times 10^4)(20 \times 10^{-3})$
$= 4000\ m^3$
$V = 4 \times 10^6$ litres
This volume has to be supplied in 20 days

$\therefore$ Discharge $= \frac{V}{t}$

$= \frac{4 \times 10^6}{20 \times 86400} = 2.315$ l/s

Q.9 (b)
Consumptive use of water = 2 mm/day
Maximum depth of water in root zone = 60 mm
Irrigation is required at 50% deficit

$= \frac{50}{100} \times 60 = 30$ mm

Frequency of irrigation

$= \frac{\text{Allowable deficit}}{\text{Consumptive use}} = \frac{30}{2}$

= 15 days

Q.10 (d)

Base period, B = 90 days

Total depth of water required by crop = 105 cm

Rainfall = 15 cm

So, depth of water provided by irrigation system,

Δ = 105 cm – 15 cm = 90 cm

$$\text{Duty, } D = \frac{8.64B}{\Delta}$$

$$= \frac{8.64 \times 90}{0.9}$$

= 864 hectare/cumec

Note: Duty is defined as the area which can be irrigated by 1 m^3/s discharge flowing continuously over the base period.

Q × t = A × depth

1 × 90 × 86400 = A × 0.9

A = 864 × 10^4 m^2

= 864 hectare

Q.11 (b)

For crop X, peak rate of moisture use = 5 mm/day

Area = 36 hectare

= 36 × 10^4 m^2

Discharge of pump = Q m^3/s

Water supplied by pump in a day when it works for 10 h in a day with 75% efficiency

V = Q × (10 × 3600) × 0.75 m^3

This volume should be equal to volume of moisture use per day

V = Q × 10 × 3600 × 0.75

= 5 × 10^{-3} × 36 × 10^4

⟹ Q = 0.0666 m^3/s

= 66.66 L/S

= 67 L/S

Q.12 (d)

For crop Y, rate of moisture use = 4 mm/day

= 4 × 10^{-3} m/day

Capacity of pump = 40 L/S

= 40 × 10^{-3} m^3/s

Water supplied by pump in a day = Water used by crop over area A in a day

$$40 \times 10^{-3} \times (10 \times 3600) \times \frac{75}{100}$$

= 4 × 10^{-3} × A

⟹ A = 27 × 10^4 m^2

= 27 hectare

Q.13 (a)

Net water required by wheat crop during base period = crop requirement of water – rainfall

= 55 cm – 100 mm

= 45 cm = 0.45 m

Volume of water required,

V_1 = Area × 0.45 m

volume of water supplied by canal during base period of 120 days with an efficiency of 60%

V_2 = discharge × base period × efficiency

= 0.01 × (120 × 86400) × 0.6

= 62208 m^3

V_1 = V

⟹ 0.45 A = 6220

⟹ A = 138240 m^2 = 13.82 ha

Q.14 (b)

Days = 10 day

Effective depth for transplantation = 48 – 8 = 40 cm

⟹ Duty is area in hectare which is irrigated with per m^3/s of discharge.

We know that,

$$\text{Duty} = \frac{8.64 \times \text{base period}}{\Delta}$$

$$\Longrightarrow \text{Duty} = \frac{8.64 \times 10}{0.4}$$

= 216 hec/ cumec

Q.15 (91 to 91)

Depth of irrigation water

$$D = \frac{\gamma_d}{\gamma_w} d(F.C - W.P)$$

$$= \frac{1.3}{1} \times 70 \times (0.28 - 0.18)$$

$= 9.1 \text{ cm} \Longrightarrow 91\text{mm}$

Q.16 (b)

P-Gross Command Area=1000 ha
Q-Permanent Wilting Point=0.12
R-Duty of canal water=100 ha/cumec
S-Delta of wheat=40 cm

Q.17 (1.427)

For Rabi crops Gram and wheat

Discharge required for gram,

$$Q_1 = \frac{A_1}{D_1} = \frac{200 \times 0.3}{8.64 \times \frac{18}{0.12}} = 0.463 \text{ m}^3/\text{s};$$

$$\text{Duty} = 8.64 \frac{B}{\Delta}$$

Discharge required for wheat,

$$Q_2 = \frac{A_2}{D_2} = \frac{2000 \times 0.5}{8.64 \times \frac{18}{0.15}} = 0.964 \text{m}^3/\text{s};$$

$Q_1 + Q_2$ is required

$= 0.964 + 0.463 = 1.427 \text{m}^3/\text{s}$

Q. 18 (5200)

CCA = 10000 ha

For rice, $\Delta_r = 130 \text{ cm} = 1.3 \text{ m}$

$A_r = 2500 \text{ ha}$

$B_r = 150 \text{ days}$

$$\text{Duty, } D_r = \frac{8.64 B_r}{\Delta_r} = \frac{8.64 \times 150}{1.30}$$

$D_r = 996.923$ ha/cumecs

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

$$Q = \frac{2500}{996.923} \text{ m}^3/\text{s} = 2.5077 \text{ m}^3/\text{s}$$

For wheat also, discharge is 2.5077 m^3/s

$B_w = 120$ days

$\Delta = 50$ cm

$A_w = Q \times D_w$

$$D_w = \frac{8.64 \times B_w}{\Delta_w}$$

$$= \frac{8.64 \times 120}{0.5} = 2073.6$$

$\therefore \quad A_w = 2.5077 \times 2073.6$

$= 5199.966 \text{ ha} \simeq 5200 \text{ ha}$

Q.19 (27.78)

Culturable command area = 50000 ha

Intensity of irrigation for kharif season = 50%

So, Area under kharif = 25000 ha

Duty for kharif season = 1000 ha/cumec

$$\text{Duty} = \frac{\text{Area}}{\text{Discharge}}$$

$\therefore$ Discharge at the head of field

$$Q = \frac{25000 \text{ ha}}{1000 \text{ ha/cumec}} = 25 \text{ cumec}$$

Transmission/conveyance loss = 10%

$\eta_{\text{conveyance}} = 90\ \%$

$$\text{Discharge at the head of canal} = \frac{25}{0.9} \text{ cumec}$$

$= 27.78$ cumec

2 DESIGN OF STABLE CHANNELS

Q.1 A launching apron is to be designed at downstream of a weir for discharge intensity of 6.5 $m^3/s/m$. For the design of launching aprons, the scour depth is taken two time of Lacey scour depth. The silt factor of the bed material is unity. If the tail water depth is 4.4 m, the length of launching apron in the launched position is

a) $\sqrt{5}$m b) 4.7 m
c) 5 m d) $5\sqrt{5}$m

[GATE-2005]

Q.2 On which of the canal systems, R.G. Kennedy, executive engineer in the Punjab Irrigation Department made his observations for proposing his theory on stable channels?

a) Krishna western Delta canals
b) Lower Bari Doab canals
c) Lower Chenab canals
d) Upper Bari Doab canals

[GATE-2005]

Q.3 As per the Lacey's method for design of alluvial channels, identify the true statement from the following:

a) Wetted perimeter increase with an increase in design discharge
b) Hydraulic radius increase with an increase in silt factor
c) Wetted perimeter decreases an increase in design discharge
d) Wetted perimeter increase with increase in silt factor

[GATE-2007]

Q.4 A stable channel is to be designed for a discharge of Q m^3/s with silt factor f as per Lacey's method. The mean flow velocity (m/s) in the channel is obtained by

a) $\left[\frac{Qf^2}{140}\right]^{1/6}$ b) $\left[\frac{Qf}{140}\right]^{1/3}$
c) $\left[\frac{Q^2f^2}{140}\right]^{1/6}$ d) $0.48\left[\frac{Q}{F}\right]^{1/3}$

[GATE-2008]

Q.5 The depth of flow in an alluvial channel is 1.5 m. If critical velocity ratio is 1.1 and Manning's n is 0.018, the critical velocity of the channel as per Kennedy's method is

a) 0.713 m/s b) 0.784 m/s
c) 0.879 m/s d) 1.108 m/s

[GATE-2009]

ANSWER KEY:

1	2	3	4	5
(c)	(d)	(a)	(a)	(b)

EXPLANATIONS

Q.1 (c)

Lacey's scour depth,

$$R = 1.35\left(\frac{q^2}{f}\right)^{1/3}$$

Here, q = 6.5 $m^3/s/m$

Silt factor, f = 1

$$R = 1.35\left\{\frac{(6.5)^2}{1}\right\}^{1/3} = 4.702 \text{ m}$$

So, scour depth = 2R

= 2 × 4.702 = 9.404 m

Tail water depth = 4.4 m

Length of launching apron

= 9.404 – 4.4 = 5 m

Q.2 (d)

Q.3 (a)

As per Lacey's method of design for alluvial channels,

Wetted perimeter,

$$P = 4.75\sqrt{Q}$$

$$\Rightarrow P \propto \sqrt{Q}$$

Hence, P increases with increase in discharge in root zone

Q.4 (a)

Mean flow velocity

$$v = \left[\frac{Qf^2}{140}\right]^{1/6}$$

Q.5 (b)

Critical velocity ratio,

m = 1.1

Depth, y = 1.5 m

As per Kennedy's equation,

$v = 0.55\ my^{0.64}$

$= 0.55 \times 1.1 \times (1.5)^{0.64}$

= 0.784 m/s

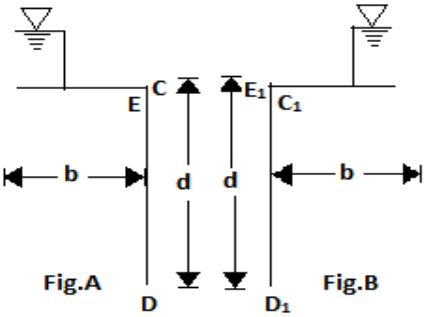
3 THEORIES OF SEEPAGE, SPILLWAYS

Q.1 While designing a hydraulic structure, the piezometric head at bottom of the floor is computed as 10 m. The datum is 3 m below floor bottom. The assured standing water depth above the floor is 2.0. The specific gravity of the floor material is 2.5. The floor thickness should be

a) 2.00 m b) 3.33 m
c) 4.40 m d) 6.00 m

[GATE-2003]

Q.2 Uplift pressures at points E and D (Fig. A) of a straight horizontal floor of negligible thickness with a sheet pile at downstream end are 28% and 20%, respectively. If the sheet pile is at upstream end of the floor (Fig.B) uplift pressures at points D_1 and C_1 are



a) 68% and 60% respectively
b) 80% and 72% respectively
c) 88% and 70% respectively
d) 100% and zero respectively

[GATE-2005]

Q.3 Which one of the following equations represents the downstream profile of Ogee spillway with vertical upstream face? [(x, y) are the coordinates of the point on the downstream profile with origin at the crest of the spillway and H_d is the design head].

a) $\frac{y}{H_d} = -0.5\left(\frac{x}{H_d}\right)^{1.85}$

b) $\frac{y}{H_d} = -0.5\left(\frac{x}{H_d}\right)^{1/1.85}$

c) $\frac{y}{H_d} = -2.0\left(\frac{x}{H_d}\right)^{1.85}$

d) $\frac{y}{H_d} = -2.0\left(\frac{x}{H_d}\right)^{1/1.85}$

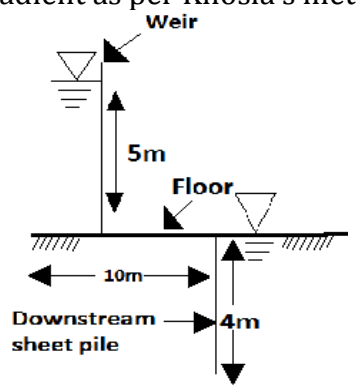
[GATE-2005]

Q.4 Water emerges from an ogee spillway with velocity = 13.72 m/s and depth = 0.3m at its toe. The tail water depth required to from a hydraulic jump at the toe is

a) 6.48 m b) 5.24 m
c) 3.24 m d) 2.24 m

[GATE-2008]

Q.5 A weir on a permeable foundation with downstream sheet pile is shown in the figure below. The exit gradient as per Khosla's method is



a) 1 in 6.0 b) 1 in 5.0
c) 1 in 3.4 d) 1 in 2.5

[GATE-2008]

Q.6 Profile of a weir on permeable foundation is shown in figure I and an elementary profile of 'upstream pile only case' according to Khosla's

theory is shown in figure II. The uplift pressure heads at key points Q, R and S are 3.14 m, 2.75 m and 0 m, respectively (refer figure II).

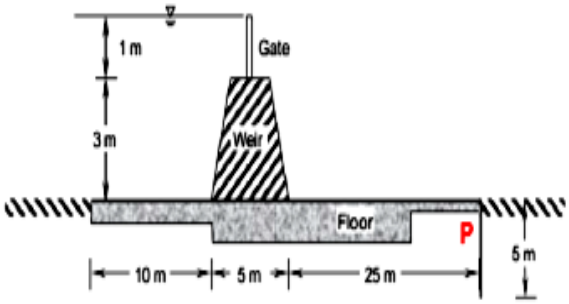


Figure I

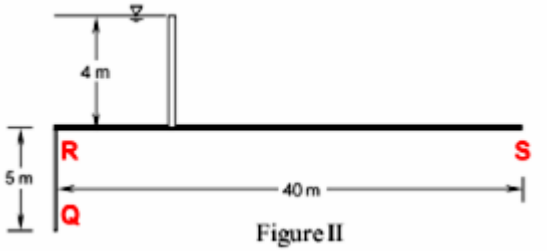


Figure II

What is the uplift pressure head at point P downstream of the weir (junction of floor and pile as shown in the figure I)?

a) 2.75 m
b) 1.25 m
c) 0.8 m
d) Data not sufficient

[GATE-2016]

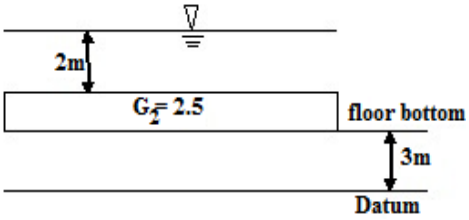
ANSWER KEY:

1	2	3	4	5	6
(a)	(b)	(a)	(c)	(c)	(b)

EXPLANATIONS

Q.1 (a)

Piezometric head at the bottom of floor = 10 m



Pressure head + Datum head = 10
Pressure head + 3 = 1
Pressure head = 7 m
Weight of floor and weight of water above it will counteract this upward pressure head.

$2\gamma_w + t.G_s\gamma_w = 7\gamma_w$

2 + 2.5 t = 7

$\Rightarrow$ t = 2.00 m

Q.2 (b)

Uplift pressure when sheet pile is at upstream = 100 – uplift pressure when sheet pile is at downstream

Uplift pressure at D_1 = 100 – Uplift pressure at

D = 100% - 20% = 80%

Uplift pressure at C1 = 100 – Uplift pressure at

E = 100% - 28% = 72%

Q.3 (a)

For a spillway having a vertical upstream face, the downstream crest is given by the equation

$$x^{1.85} = -2H_d^{0.85}y$$

$$x^{1.85} = -2\frac{H_d^{1.85}}{H_d}.y$$

$$\left(\frac{y}{H_d}\right) = -0.5\left(\frac{x}{H_d}\right)^{1.85}$$

Q.4 (c)

Depth at the toe,

y_1 = 0.3 m

Depth of tail water required to form a hydraulic jump = y_2

$$\frac{y_2}{y_1} = \frac{1}{2}\left[\sqrt{1+8F_1^2} - 1\right]$$

$$\frac{y_2}{0.3} = \frac{1}{2}\left[\sqrt{1+\frac{8\times v_1^2}{gy_1}} - 1\right]$$

$$= \frac{1}{2}\left[\sqrt{1+8\times\frac{13.72^2}{9.81\times 0.3}} - 1\right]\times 0.3$$

y_2 = 3.24 m

Q.5 (c)

As per Khosla's Method,

Exit gradient,

$$G_E = \frac{H}{d}.\frac{1}{\pi\sqrt{\lambda}}$$

where, $\lambda = \frac{1}{2}\left[1+\sqrt{1+\alpha^2}\right]$

and, $\alpha = \frac{b}{d}$

Here, b = 10 m

Depth of sheet pile, d = 4 m

Head loss, H = 5 m

$$\alpha = \frac{10}{4} = 2.5$$

$$\lambda = \frac{1}{2}\left[1+\sqrt{1+2.5^2}\right] = 1.846$$

$$G_E = \frac{H}{d}.\frac{1}{\pi\sqrt{\lambda}}$$

$$= \frac{5}{4}\frac{1}{\pi\sqrt{1.846}} = 0.293 = \frac{1}{3.41}$$

Q.6 (b)

$$\phi_R = \frac{2.75}{4}\times 100 = 68.75\%$$

$$\phi_p = 100 - \phi_R = 31.25\%$$

Now,

$$\phi_p = \frac{\text{Pressure head at point}}{\text{Total head}} \times 100$$

$$\Rightarrow 31.25 = \frac{h}{4} \times 100 \Rightarrow h = 1.25\text{m}$$

4 DESIGN AND CONSTRUCTION OF GRAVITY DAMS

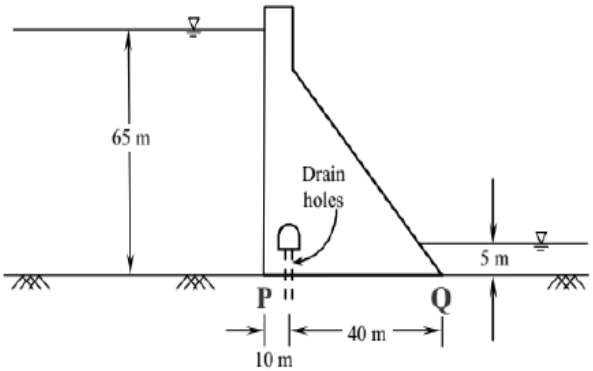
Q.1 The base width of an elementary profile of a gravity dam of height H is b. The specific gravity of the material of the dam is G and upfilt pressure coefficient is K. The correct relationship for no tension at the heel is given by

a) $\frac{b}{H} = \frac{1}{\sqrt{G-K}}$ b) $\frac{b}{H} = \sqrt{G-K}$

c) $\frac{b}{H} = \frac{1}{G-K}$ d) $\frac{b}{H} = \frac{1}{K\sqrt{G-K}}$

[GATE-2008]

Q.2 A concrete gravity dam section is shown in the figure. Assuming unit weight of water as 10kN/m³ and unit weight of concrete as 24kN/m³, the uplift force per unit length of the dam (expressed in kN/m) at PQ is___



[GATE-2016]

Q.3 **Group I** contains three broad classes of irrigation supply canal outlets. **Group II** lists hydraulic performance attributes.

Group I

P. Non-modular outlet

Q. Semi-modular outlet

R. Modular outlet

Group II

1. Outlet discharge depends on the water levels in both the supply canal as well as the receiving water course.

2. Outlet discharge is fixed and is independent of the water levels in both the supply canal as well as the receiving water course.

3. Outlet discharge depends only on the water level in the supply canal.

The **CORRECT** match of **Group I** with **Group II** is

a) P – 1, Q – 2, R – 3

b) P – 3, Q – 1, R – 2

c) P – 2, Q – 3, R – 1

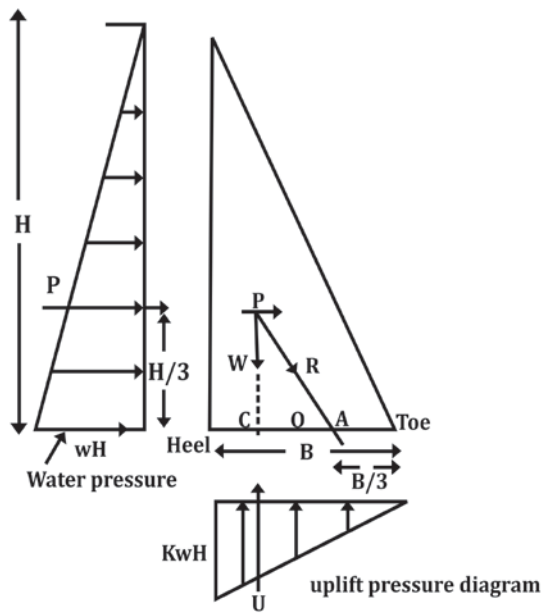
d) P – 1, Q – 3, R – 2

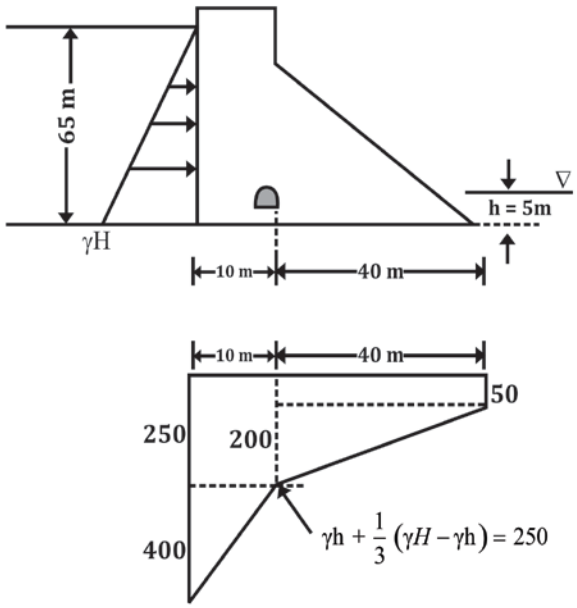
ANSWER KEY:

1	2	3
(a)	10500	(d)

EXPLANATIONS

Q.1 (a)





For no tension at heal, the resultant R must pass through middle third.

$\therefore$ OA $= \frac{b}{6}$ and AC $= \frac{b}{3}$

Also OC $= \frac{b}{c}$

Let W = Self weight of dam = wG

Now taking moments of all forces about A

$$w\left(\frac{b}{3}\right) - U\left(\frac{b}{3}\right) - P\left(\frac{H}{3}\right).$$

$$-\frac{1}{2}\,wH.H.\frac{H}{3} = 0$$

Solving, we get

$$b^2(G-k) = H^2$$

$$\frac{b}{H} = \frac{1}{\sqrt{G-k}}$$

Q.2 (10500)

Total uplift pressre

$$= 250\times10 + 40\times50 + \frac{1}{2}\times200\times40 + \frac{1}{2}\times400\times10$$

$$= 2500 + 2000 + 4000 + 2000$$

$$= 10500 \text{ kN/m}$$

Q. 3 (d)

Non-modular outlets are those whose discharge is a function of the difference in levels between the water surface in the distributing channel and the water course.

Semi-Modular Outlets: The discharge through a **semi-modular** canal **outlet** (or **semi-module** or flexible **outlet**) depends only on the water level in the distributary, and is unaffected by the water level in the watercourse provided a minimum working head required for its working is available.

Modular Irrigation Outlets: As the **outlet** discharge of this type is

independent of the difference of water levels of the parent channel and field channel it is also called rigid **module**.

ASSIGNMENT QUESTIONS

Q.1 The percentage of total quantity of water in the world that is saline is about
a) 71% b) 33%
c) 67% d) 97%

Q.2 The percentage of total quantity of fresh water in the world available in the liquid form is about
a) 30% b) 70%
c) 11% d) 51%

Q.3 A catchment has an area of 150 ha and a runoff/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, the average stream flow in the period is
a) 1.33 m^3/s
b) 16.7 m^3/s
c) 100 m^3/minute
d) 6×10^4 m^3/hour

Q.4 A catchment of area 120 km^2 has three distinct zones as below: Zone Area (km^2) Annual runoff (cm) A 61 52 B 39 42 C 20 32 the annual runoff from the catchment, in cms, is
a) 126.0 b) 42.0
c) 45.4 d) 47.3

Q.5 If the average annual rainfall and evaporation over land masses and oceans of the earth are considered it would be found that
a) Over the land mass the annual evaporation is the same as the annual precipitation.
b) About 9% more water evaporates from the oceans than what falls back on them as precipitation.
c) Over the oceans about 19% more rain falls than what is evaporated.
d) Over the oceans about 19% more water evaporates than what falls back on them as precipitation.

Q.6 In the hydrological cycle the average residence time of water in the global
a) Atmospheric moisture is larger than that in the global rivers
b) Oceans is smaller than that of the global groundwater
c) River is larger than that of the global groundwater
d) Oceans is larger than that of the global groundwater

Q.7 The hydrologic cycle states that
a) Total inflow - total outflow = constant
b) Subsurface inflow = subsurface outflow
c) Inflow into basin = outflow from the basin
d) Mass inflow - mass outflow = change in mass storage

Q.8 A tropical cyclone is a
a) Low-pressure zone that occurs in the northern hemisphere only
b) High-pressure zone with high winds
c) Zone of low pressure with clockwise winds in the northern hemisphere
d) Zone of low pressure with anticlockwise winds in the northern hemisphere

Q.9 Orographic precipitation occurs due to air masses being lifted to higher altitudes by
a) The density difference of air masses
b) A frontal action
c) The presence of mountain barriers
d) Extra tropical cyclones

Q.10 The standard recording raingauge adopted in India is of
a) Weighing bucket type
b) natural syphon type
c) Tipping bucket type
d) telemetry type

Q.11 The following recording raingauges does not produce the mass curve of precipitation as record:
a) Symon's raingauge
b) tipping-bucket type gauge
c) weighing-bucket type gauge
d) natural syphon gauge.

Q.12 The Double mass curve technique is adopted to
a) Check the consistency of raingauge records
b) To find the average rainfall over a number of years
c) To find the number of raingauges required
d) To estimate the missing rainfall data

Q.13 For a given storm the highest rainfall P_o and the average rainfall depth $\overline{P}$, are Related as $\frac{\overline{P}}{P_o}$
a) K exp (A^n)
b) exp ($-KA^n$)
c) K^{-A}
d) constant

Q.14 By DAD analysis, the maximum average depth over an area of 10^4 km^2 due to one-day storm is found to be 47 cm. For the same area the maximum average depth for a three day storm can be expected to be
a) < 47 cm
b) > 47 cm
c) = 47 cm
d) inadequate information to conclude

Q.15 The probable maximum depth of precipitation over a catchment is given by the relation PMP =
a) $\overline{P} + KA^n$
b) $\overline{P} + K\sigma$
c) $\overline{P}\exp(-KA^n)$
d) $m\overline{P}$

Q.16 A rainfall with an intensity of 5 mm/h is classified as
a) Traces
b) light rain
c) Moderate rain
d) heavy rain

Q.17 A precipitation in the form of water droplets of size less than 0.5 mm and intensity less than 1 mm/h is known as
a) Rain
b) Sleet
c) Hail
d) Drizzle

Q.18 The Indian Meteorological department has changed over from Symon's gauge to fibreglass raingauges of two sizes. The collector areas of these gauges are:
a) (1000 cm^2 and 500 cm^2)
b) (400 cm^2 and 20 cm^2)
c) (200 cm^2 and 100 cm^2)
d) (100 cm^2 and 50 cm^2)

Q.19 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 mean depth of precipitation is
a) $\frac{\Sigma ap^3}{\Sigma a^2}$
b) $\frac{\Sigma ap}{n}$
c) $\frac{\Sigma ap}{\Sigma a}$
d) $\frac{\Sigma ap^5}{\Sigma a^3}$

Q.20 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

Q.21 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)

Q.22 Coefficient of variation is given by Standard deviation

a) $\frac{\text{Standard deviation}}{\text{Mean}} \times 100$

b) $\frac{\text{Variance}}{\text{Mean}} \times 100$

c) $\frac{\text{Mean}}{\text{Standerd deviation}} \times 100$

d) $\frac{\text{Mean}}{\text{Variance}} \times 100$

Q.23 According to Indian Standards, the number of raingauge stations for an area of 5200 km^2 in plains should be

a) 10 b) 15
c) 20 d) 40

Q.24 The normal annual rainfall at stations A, B and C situated in meteorologically homogeneous region are 175 cm, 180 cm and 150 cm respectively. In the year 2000, station B was inoperative and stations A and C recorded annual precipitations of 150 cm and 135 cm respectively. The annual rainfall at station B in that year could be estimated to be nearly

a) 150 cm b) 143 cm
c) 158 cm d) 168 cm

Q.25 Match List-I (Type of rain gauge) with List-II (Characteristic) and select the correct answer using the codes given below the lists:

List-I
A. Tipping bucket type
B. Weighing bucket type
C. Symon's gauge
D. Natural syphon type gauge

List-II
1. Standard recording type gauge adopted in India
2. Gives intensity of the rainfall
3. Gives mass curve of rainfall
4. Non recording gauge

Codes:

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

Q.26 If e_w and e_a are the saturated vapour pressures of the water surface and air respectively, the Dalton's law for evaporation E_L in unit time is given by E_L =

a) $(e_w - e_a)$ b) $K e_w - e$.
c) $K (e_w - e_a)$ d) $K (e_w + e_a)$

Q.27 A canal is 80 km long and has an average surface width of 15 m. If the evaporation measured in a class a pan is 0.5 cm/day, the volume of water evaporated in a month of 30 days is (in m^3)

a) 12600 b) 18000
c) 180000 d) 126000

Q.28 The ISI standard pan evaporimeter

a) is same as the US class a pan
b) has an average pan coefficient value of 0.60
c) has less evaporation than a US class a pan

d) has more evaporation than a US class a pan

Q.29 Evapotranspiration is confined to
a) Daylight hours
b) Night-time only
c) Land surfaces only
d) none of these

Q.30 Interception losses
a) Include evaporation, through flow and stem flow
b) Consists of only evaporation loss
c) includes evaporation and transpiration losses
d) Consists of only stem flow

Q.31 The ISI standard pan evaporimeter
a) Is the same as the US class an evaporation pan
b) Has a pan coefficient which is about 14% less than US class a pan
c) Has a pan coefficient which is about 14% more than US class a pan
d) Is a sunken pan

Q.32 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

Q.33 A 6-h storm had 6 cm of rainfall and the resulting runoff was 3 cm. If the Ø-index remains at the same value the runoff due to 12 cm of rainfall in 9 h in the catchment is
a) 9.0 cm b) 4.5 cm
c) 6 cm d) 7.5 cm

Q.34 A 6-hour storm with hourly intensities of 7, 18, 25, 12, 10 and 3 mm per hour produced a run-off of 33 mm., then the Ø-index is
a) 7 mm/h b) 3 mm/h
c) 10 mm/h d) 8 mm/h

Q.35 During a 6-hour storm, the rainfall intensity was 0.8 cm/hour on a catchment of area 8.6 km^2. The measured runoff volume during this period was 2, 56,000 m^3. The total rainfall was lost due to infiltration, evaporation, and transpiration (in cm/hour) is
a) 0.80
b) 0.304
c) 0.496
d) sufficient information riot available

Q.36 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

Q.37 If the numbers of rain gauge stations are six, average rainfall is 118.6 cm and standard deviation is 35.04 for a 10 % error in the estimation of mean rainfall, what will be the optimum number of additional stations in the catchment?

(a) 3 (b) 15 (c) 9 (d) 6

Q.38 The following is not a direct stream flow determination technique:
a) Dilution method
b) Ultrasonic method
c) Area-velocity method
d) Slope-area method

Q.39 A stilling well is required when the stage measurement is made by employing a
a) Bubble gauge
b) Float gauge recorder
c) Vertical staff gauge
d) inclined staff gauge

Q.40 In a triangular channel the top width and depth of flow were 2.0 m and 0.9 m respectively. Velocity measurements on the centre line at 18 cm and 72 cm below water surface indicated velocities of 0.60 m/s and 0.40 m/s respectively. The discharge in the channel (in m^3/s) is
a) 0.90 b) 1.80
c) 0.45 d) None of these

Q.41 The stage discharge relation in a river during the passage of a flood wave is measured.
If Q_R = discharge at a stage when the water surface was rising and Q_F = discharge at the same stage when the water surface was falling, then
a) $Q_F = Q_R$
b) $Q_R > Q_F$
c) $Q_R < Q_F$
d) Q_R/Q_F = constant at all stages

Q.42 The dilution method of stream gauging is ideally suited for measuring discharges in
a) A large alluvial river
b) Flood flow in a mountain stream
c) Steady flow in a small turbulent stream
d) A stretch of a river having heavy industrial pollution loads

Q.43 If V_s = surface velocity of a stream at a vertical, the average velocity V in the vertical will be about
a) $V_s/0.9$ b) $0.6\ V_s$
c) $0.9\ V_s$ d) V_s

Q.44 In the case of large rivers, a number of equidistant vertical sections of the total width of flow are identified, for the purpose of finding by numerical integration, the total discharge on any day. On each section, the mean velocity is taken as the arithmetic average of two typical depths on that section. Then the mean velocity is worked out for that section. Usually, the mean velocity of any section, corresponds to which one of the following? (V represents the point velocity at the given section and the depth such as 0.1d, 0.2d.... etc.)
a) $\frac{V_{0.1d} + V_{0.9d}}{2}$ b) $\frac{V_{0.2d} + V_{0.8d}}{2}$
c) $\frac{V_{0.3d} + V_{0.7d}}{2}$ d) $\frac{V_{0.4d} + V_{0.6d}}{2}$

Q.45 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

Q.46 During a 6-hr storm, the rainfall intensity was 0.9 cm/hr on a catchment of area 8.7 km^2. The measured run off volume during this period was 257000m^3. The total rainfall that was lost due to infiltration, evaporation and transpiration (in cm/hr) is

(a) 0.8 (b) 0.408

(c) 0.596 (d) sufficient information not available

Q.47 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

Q.48 Direct runoff is made up of
a) Surface runoff, prompt interflow and channel precipitation
b) Surface runoff, infiltration and evapotranspiration
c) Overland flow only
d) Rainfall and evaporation

Q.49 A hydrograph is a plot of
a) Rainfall intensity against time
b) Stream discharge against time
c) Cumulative rainfall against time
d) Cumulative runoff against time

Q.50 The term base flow denotes
a) Delayed groundwater flow reaching a stream
b) Delayed groundwater and snowmelt reaching a stream
c) Delayed groundwater & interflow
d) The annual minimum flow in a stream

Q.51 Virgin flow is
a) The flow in the river downstream of a gauging station
b) The flow in the river upstream of a gauging station
c) The flow unaffected by works of man
d) The flow that would exist in the stream if there were no abstractions to the precipitation

Q.52 An ephemeral stream
a) Is one which always carries some flow
b) Does not have any base flow contribution
c) Is one which has limited contribution of groundwater in wet season
d) Is one which carries only snowmelt water

Q.53 An intermittent stream
a) Has water table above the stream bed throughout the year
b) Has only flash flows in response to storms
c) Has flows in the stream during wet season due to contribution of ground water
d) Does not have any contribution of ground water at any time

Q.54 The flow-duration curve is a plot of
a) Accumulated flow against time
b) Discharge against time in chronological order
c) The base flow against the percentage of times the flow is exceeded
d) The stream discharge against the percentage of times the flow is equalled or exceeded

Q.55 The flow-mass curve is an integral curve of
a) The hydrograph
b) The hyetograph
c) The flow duration curve
d) The S-curve

Q.56 The average rainfall for 3 hour duration storm is 2.75 cm and the loss rate is 0.35 cm/hr. The flood hydrograph has a base flow of 20m^3/s and produces a peak flow of 200 m^3/s. The peak of a 3-h unit hydrograph is

(a) 130.5 m^3/s (b) 105.88 m^3/s

(c) 87.5 m^3/s (d) 70 m^3/s

Q.57 In an influent stream
a) There is a contribution from the groundwater to the stream due to seepage
b) There is a contribution from the stream to the groundwater due to percolation
c) There is a seepage of groundwater into the stream at

the sides and percolation of stream flow water at the bed

d) There will be neither seepage into the stream nor percolation from the stream to the groundwater

Q.58 The rainfall is 10 mm/hr on an area of one hectare. The runoff value will be equal to

a) 100 m^3/hr b) 10 m^3/hr
c) 1000 m^3/hr d) 1 m^3/hr

Q.59 Khosla's formula for monthly runoff Rm due to a monthly rainfall P_m is $R_m = P_m - L_m$ where L_m is

a) A constant
b) Monthly loss and depends on the mean monthly catchment temperature
c) A monthly loss Coefficient depending On the antecedent precipitation index
d) A monthly loss depending on the infiltration characteristics of the catchment

Q.60 For a given storm, other factors remaining same,

a) Basins having low drainage density give smaller peaks in flood hydrographs
b) Basins with larger drainage densities give smaller flood peaks
c) Low drainage density basins give shorter time bases of hydrographs
d) The flood peak is independent of the drainage density

Q.61 A 4 hour unit hydrograph of a basin with area of 1728 km^2 can be approximated as a triangle with base period of 48 hours. The peak ordinate of hydrograph is

a) 400 m^3/s b) 300 m^3/s
c) 200 m^3/s d) 100 m^3/s

Q.62 For a catchment area of 36 km^2, the equilibrium discharge of an S-curve obtained by 2-hour unit hydrograph summation is

a) 25 m^3/sec b) 50 m^3/sec
c) 100 m^3/sec d) 72 m^3/sec

Q.63 The term 'mean annual flood' denotes

a) Mean of floods in partial-duration series
b) Mean annual flow
c) A flood with a recurrence interval of 2.33 years
d) A flood with a recurrence interval of N/2 years, where N = number of years of record

Q.64 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

Q.65 The design flood commonly adopted in India for barrages and minor dams is

a) Probable maximum flood
b) A flood of 50-100 years return period
c) Peak flood
d) Standard project flood or a 100-year flood, whichever is higher

Q.66 The Muskingum method of flood routing is a

a) Form of reservoir routing method
b) Hydraulic routing method
c) Complete numerical solution of St. Venant equations
d) Hydrologic channel-routing method

Q.67 The Muskingum method of flood routing assumes the storage S is

related to inflow rate I and outflow rate Q of a reach as S=
a) K [x I + (1 - x) Q]
b) K [x (1 - x) I]
c) K[x l (1 - x) Q]
d) K x [I - (1 - x) Q]

Q.68 The Muskingum method of flood routing gives $Q_2 = C_0 I_2 + C_1 I_1 + C_2 Q_1$. The coefficients in this equation will have values such that
a) $C_o + C_1 = C_2$
b) $C_o - C_1 - C_2 = 1$
c) $C_o + C_1 + C_2 = 0$
d) $C_o + C_1 + C_2 = 1$

Q.69 In the Muskingum method of channel routing the routing equation is written as $Q_2 = C_0 I_2 + C_1 I_1 + C_2 Q_1$. If the coefficients K = 12 and x = 0.15 and the time step for routing Δt = 4 hr, the coefficient C_o is
a) 0.016 b) 0.048
c) 0.328 d) 0.656

Q.70 In case of channel routing, the storage is a function of
a) Inflow discharge only
b) Outflow discharge only
c) Both (a) and (b)
d) None of the above

Q.71 The table given below the details for a particular crop, with a crop factor 0.5. The consumptive water requirement of the crop will be,

Month	**Jan.**	**Feb.**
Monthly Temp. Average in °C	20	8
Monthly % of day time hours of the year	20	8

a) 27.2 cm b) 13.6 cm
c) 32 cm d) 1 cm

Q.72 Net irrigation requirement of a crop is equal to
a) Consumptive use
b) Consumptive use — effective rainfall
c) Consumptive use — effective rainfall + leaching and other requirements
d) Percolation loss+ effective rainfall

Q.73 In an irrigation system, the land was divided into a large number of smaller size unit areas, having fairly level surface, by bunds and cross ridges. The basins thus created were filled with water to the desired depth and the water was retained for some time. This method of irrigation is known as
a) border method
b) check basin method
c) sub-irrigation
d) contour irrigation

Q.74 Which of the statements given below is not correct? In a trickle irrigation system
a) deep percolation and runoff are practically eliminated
b) the water application efficiency is very high
c) the evapotranspiration is practically eliminated
d) the fertilizer can be applied economically along with the irrigation water

Q.75 Given that the base period is 100 days and the duty of the canal is 1000 hectares per cumecs, the depth of water will be
a) 0.864 cm b) 8.64 cm
c) 86.4 cm d) 864 cm

Q.76 The delta for a crop having a base of period 120 days is 70 cm. What is the duty?
a) 2480 hectare/ cumec
b) 1481 hectare/ cumec
c) 148 hectare/ cumec
d) 1.481 hectare/ cumec

Q.77 The total depth of water required by a crop grown in the field during the entire period is called
a) duty b) delta
c) base period d)crop period

Q.78 The following data were recorded from an irrigated field:
1. Field capacity : 20%
2. Permanent wilting point : 10%
3. Permissible depletion of available soil moisture: 50%
4. Dry unit weight of soil : 1500 kgf/m^3
5. Effective rainfall : 25 mm

Based on these data, the net irrigation requirement per metre depth of soil will be
a) 75 mm b) 125 mm
c) 50 mm d) 25 mm

Q.79 For a culturable command area of 1000 hectare with intensity of irrigation of 50%, the duty on field for a certain crop is 2000 hectare/cumec. What is the discharge required at head of water course with 25% losses of water?
a) 3/16 cumec b) 1/4 cumec
c) 1/3 cumec d) 1/2 cumec

Q.80 If the discharge required for different crops is 0.4 cumecs in the field and the capacity factor and time factors are 0.8 and 0.5 respectively, then what is the design discharge of the distributary at its head?
a) 0.80 cumecs b) 0.16cumecs
c) 1.0 cumecs d) 1.24 cumecs

Q.81 The sodium adsorption ratio (SAR) of irrigation water used to express relative activity of sodium ions in exchange reactions with soil is defined as
a) $Na^+ /[(Ca^{++} + Mg^{++})/2]^{1/2}$
b) $Ca^{++} /[(Na^+ + Mg^{++})/3]^{1/2}$
c) $[(Ca^{++} + Mg^{++})/2]^{1/2}/ Na^+$
d) $2Na^+/ [(Ca^{++}+ Mg^{++})]^{1/2}$

Q.82 An irrigation water having an SAR value of 20 is called as
a) very high sodium water
b) high sodium water
c) medium sodium water
d) low sodium water

Q.83 The leaching requirement of a soil is 10%. If the consumptive use requirement of the crop is 90 mm, then the depth of water required to be applied to the field is
a) 80 mm b) 90 mm
c) 100 mm d) 110 mm

Q.84 The area, on which crops are grown in a particular season, is called
a) culturable commanded area (CCA)
b) gross sown area
c) net sown area
d) None of the above

Q.85 The following structure serves the purpose of a 'safety valve' for a canal
a) head regulator b)cross regulator
c) canal escape d) canal fall

Q.86 In modern practice the free board provided for gravity dam is
a) 3 to 4% of dam height
b) 2 hw (hw = height of wave)
c) 2 to 3% of dam height
d) 3 hw

Q.87 The most preferred soil for the central impervious core of a zoned embankment type of an earthen dam, is
a) highly impervious clay
b) highly pervious gravel
c) coarse sand
d) clay mixed with fine sand

Q.88 On moderate foundations, and particularly in seismic areas, the type of dam which can preferably be considered for construction, is
a) masonry gravity dam

b) earthen dam
c) rock fill dam
d) arch dam

Q.89 The depth of flow in an alluvial channel is 1.55m. If critical velocity ratio is 1.12 and manning's co-efficient is 0.018, the critical velocity of the channel as per Kennedy's method is

(a) 0.817 m/s (b) 0.815 m/s
(c) 0.784 m/s (d) 1.12 m/s.

Q.90 The Garret's diagrams are based on

a) Lacey's theory
b) Khosla's theory
c) Bligh's theory
d) Kennedy's theory

Q.91 The wetted perimeter P of a stable channel is proportional to
a) Q b) 4.75 $\sqrt{Q}$
c) Q^2 d) $Q^{1/3}$
where, Q is the discharge

Q.92 Lacey's regime theory is not applicable to a channel in
a) true regime b) initial regime
c) final regime d) None of these

Q.93 Counter berms are provided in an irrigation canal
a) at the outer sides of the canal banks to allow movement of cattle
b) at the inner sides of the canal banks at the ground level, to make bank line and bed line parallel
c) at the outer sides of the canal banks to keep the saturation gradient line well within the bank section
d) None of the above

Q.94 Lacey assumed that the slit is kept in suspension because of normal components of eddies generated from
a) bed only
b) sides only
c) whole perimeter
d) None of these

Q.95 A stable channel has three degrees of freedom in the sense that the depth, width and slope can adjust to the given discharge and sediment conditions. In this sense which of the following methods of canal design gives a true regime channel for silt laden flow in fine alluvium?
a) Kennedy's method
b) Tractive force method
c) Lacey's method
d) Manning's formula and Kennedy's equation

Q.96 The total number of independent equations that form the Lacey's regime theory is
a) 6 b) 10
c) 3 d) 4

Q.97 A regime channel has a discharge of 100 m^3/ s. It will have a perimeter of
a) 4.8 m b) 10.0 m
c) 47.5 m d) 22.0 m

Q.98 For medium silt whose average grain size is 0.16 mm, Lacey's silt factor is likely to be
a) 0.30 b) 0.45
c) 0.70 d) 1.32

Q.99 Consider the following statements: Garret's diagram for the design of irrigation channel is based on:
1. Kennedy's theory
2. Lacey's theory
3. Kutter's formula
4. Manning's formula
Which of these statements are correct?

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

Q.100 In Lacey's regime theory, the velocity of flow is proportional to
a) Qf^2 b) Q/f^2
c) $(Qf^2)^{1/6}$ d) $(Q/f^2)1/6$

Q.101 If the discharge of a river is 3.0 cumec per meter width and the silt factor is 1.2, Lacey's scour depth will be
a) 1.50 m b) 4.00 m
c) 2.64 m d) 4,50 m

Q.102 A regime channel carries 45 cm of water depth and type of bed material has critical tractive stress of 0.566 N/m^2. The bed material will begin to move if bed has a minimum slope of
a) zero b) 1/17332
c) 1/7800 d) 1/790

Q.103 Lining of irrigation channels
a) increases water logging
b) increases channel cross section
c) increases command area
d) increases chances of breaching

Q.104 Safety ladders are provided in large irrigation canals, to
a) enable the fish to pass from one place to another
b) enable the cattle to cross the canal
c) enable the swimmers to get out of the canal
d) provide safe exit to avoid accidental drowning

Q.105 In the alignment of an irrigation channel wherefrom offsets have to be provided at regular intervals, changes in the given channel parameters are made use of. The correct sequence of the decreasing order of preference of these parameters is
a) width, slope, depth
b) width, depth, slope
c) depth, slope, width
d) depth, width, slope

Q.106 A land is said to be water-logged, when
a) the land is necessarily submerged under standing water
b) there is a flowing water over the land
c) the pH value of the soil becomes as high as 8.5
d) the soil pores in the root zone get saturated with water, either by the actual water table or by its capillary fringe

Q.107 The method, which uses dead furrows on cropped farms for drainage of excess irrigation or rain water, is called:
a) surface inlet b)tile drainage
c) bedding d) French drain

Q.108 The construction of impounding reservoir is required when
a) average annual flow in the stream is lower than average demand
b) the rate flow in the stream, in dry season is more than the demand
c) the rate of flow in the stream, in dry season is less than the demand
d) the rate of flow in the stream is equal to the demand

Q.109 Aggrading river are :
a) silting rivers
b) scouring rivers
c) rivers in regime
d) meandering rivers

Q.110 An attracting groyne is one which is
a) inclined upstream
b) inclined downstream
c) normal to the bank
d) same as a repelling groyne

Q.111 Canal modules help in

a) modulating and varying the canal discharge
b) releasing desired quantity of water into the water courses
c) releasing desired quantity of water into the minors
d) all of the above

Q.112 A fully modular canal outlet has its

a) sensitivity = 1, and flexibility =1
b) sensitivity =1, and flexibility = 0
c) sensitivity = 0, and flexibility =1
d) sensitivity = 0, and flexibility = 0

ANSWER KEY:

1	2	3	4	5	6	7	8	9	10	11	12	13	14
(d)	(a)	(c)	(c)	(b)	(d)	(d)	(d)	(c)	(b)	(b)	(a)	(b)	(b)
15	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**	**25**	**26**	**27**	**28**
(b)	(c)	(d)	(c)	(c)	(d)	(a)	(a)	(a)	(c)	(b)	(c)	(d)	(c)
29	**30**	**31**	**32**	**33**	**34**	**35**	**36**	**37**	**38**	**39**	**40**	**41**	**42**
(d)	(b)	(c)	(d)	(d)	(d)	(b)	(d)	(a)	(d)	(b)	(c)	(b)	(c)
43	**44**	**45**	**46**	**47**	**48**	**49**	**50**	**51**	**52**	**53**	**54**	**55**	**56**
(c)	(b)	(c)	(b)	(b)	(a)	(b)	(a)	(c)	(b)	(c)	(d)	(a)	(b)
57	**58**	**59**	**60**	**61**	**62**	**63**	**64**	**65**	**66**	**67**	**68**	**69**	**70**
(b)	(a)	(b)	(a)	(c)	(b)	(c)	(c)	(d)	(d)	(a)	(d)	(a)	(c)
71	**72**	**73**	**74**	**75**	**76**	**77**	**78**	**79**	**80**	**81**	**82**	**83**	**84**
(b)	(c)	(b)	(c)	(C)	(b)	(b)	(c)	(c)	(c)	(a)	(b)	(c)	(d)
85	**86**	**87**	**88**	**89**	**90**	**91**	**92**	**93**	**94**	**95**	**96**	**97**	**98**
(c)	(a)	(d)	(c)	(b)	(d)	(b)	(b)	(c)	(c)	(c)	(c)	(c)	(c)
99	**100**	**101**	**102**	**103**	**104**	**105**	**106**	**107**	**108**	**109**	**110**	**111**	**112**
(a)	(c)	(c)	(c)	(c)	(d)	(c)	(d)	(c)	(c)	(a)	(b)	(b)	(d)

EXPLANATIONS

Q.1 (d)

Q.2 (a)

Q.3 (c)

The average stream flow

$$= \frac{10\times10^{-2}\times0.4\times150\times10^{4}}{10\times60}\frac{m^3}{min}$$

= 100 m^3 / minute

Q.4 (c)

Annual runoff

$$= (\frac{61\times52+39\times42+20\times32}{120})m$$

= 45.4 cms

Q.5 (b)

Q.6 (d)

Q.7 (d)

Q.8 (d)

A cyclone is a large low pressure region with circular wind motion. Two types of cyclones are recognized viz., tropical cyclones and extra tropical cyclones. A tropical cyclone is a wind system with an intensely strong depression. The normal areal extent of a cyclone is about 100-200 km in diameter. The isobars are closely spaced and the winds are anticlockwise in the Northern hemisphere.

Q.9 (c)

The moist air masses may get lifted-up to higher altitudes due to the presence of mountain barriers and consequently undergo cooling, condensation and precipitation, such a precipitation is .known as orographic precipitation.

Q.10 (b)

Natural syphon type is adopted as the standard recording type rain guage in India. It is also known as float-type gauge.

Q.11 (b)

The Tipping-bucket type gauge gives data on the intensity of rainfall.

Q.12 (a)

If the conditions relevant to the recording of a rain gauge station have undergone a significant change during the period of record, inconsistency would arise in the rainfall data of that station. The checking for inconsistency of record is done by the double mass curve technique. This technique is based on the principle that when each recorded data comes from the same parent population, they are consistent.

Q.13 (b)

For a rainfall of a given duration, the average depth decreases with the area in an exponential fashion given by

$$\overline{P} = P_0 \exp(-KA^n)$$

Where, $\overline{P}$ = average depth in cms over an area A km^2

P_o = highest amount of rainfall in cm at the storm

K and n are constants for a given region.

Q.14 (b)

The maximum depth for a given storm decreases with the area. For a

given area the maximum depth increases with the duration.

Q.15 (b)

The Probable Maximum Precipitation (PMP) is defined as the greatest or extreme rainfall for a given duration that is physically possible over a station or basin. Statistical studies indicate that PMP can be estimated as

$PMP = \overline{P} + K\sigma$

Where, $\overline{P}$ = mean of annual maximum rainfall series, a = standard deviation of the series and K=a frequency factor which depends upon the statistical distribution of the series, number of years of record and the return period.

Q.16 (c)

On the basis of its intensity, rainfall is classified as:

Type	Intensity
1. Light rain	Trace to 2.5 mm/hr
2. Moderate rain	2.5 mm/hr to 7.5mm/hr
3.Heavy rain	>7.5mm/hr

Q.17 (d)

The rainfall is used to describe precipitations in the form of water drops of sizes larger than 0.5 mm. Sleet is frozen raindrops of transparent grain which form when rain falls through air at subfreezing temperature. Hail is a showery precipitation in the form of irregular pallets or lumps of size more than 8 mm. Drizzle is a fine sprinkle of numerous water droplets of size less than 0.5 mm and intensity less than 1 mm/h.

Q.18 (c)

The Indian Meteorological department (IMO) has changed over to the use of fibreglass reinforced polyester raingauges, which is an improvement over the system gauge. These come in different combinations of collector and bottle. The collector in two sizes having areas of 200 and 100 cm2 respectively.

Q.19 (c)

Q.20 (d)

Isohyets are contours of equal rainfall. The orographically weighted isohyets are prepared by tracing paper for mountainous areas and therefore they are more accurate than linearly interpolated isohyets

Q.21 (a)

The departure of Al from its corresponding normal value is known as AI anomaly, represents moisture shortage. Based on Al anomaly, the intensity of agricultural drought is classified as follows:

Al anomaly	**Severity class**
1-25	mild arid
26-50	moderate arid
>-50	severe arid

Q.22 (a)

Q.23 (a)

In plains 1 station per 520 km^2 is recommended.

Q.24 (c)

$$P_x = \frac{N_x}{2}\left[\frac{P_1}{N_1} + \frac{P_2}{N_2}\right]$$

$$= \frac{180}{2}\left[\frac{150}{175} + \frac{135}{150}\right]$$

$$= \frac{90}{25}\left[\frac{150}{7} + \frac{135}{6}\right]$$

$$= 5\times\frac{90}{25}\left[\frac{30}{7}+\frac{27}{6}\right]$$

$$= 18\left[\frac{180+189}{42}\right] = \frac{15\times 369}{42}$$

$$= \frac{3\times 369}{7} = 158.14 \text{ cm } 158 \text{ cm}$$

Q.25 (b)

Tipping Bucket Type: Such gauges are generally installed in hilly and inaccessible area, from where they can supply their measurements directly to the control room at meteorogical station. It gives intensity of rainfall.

Weighing Type: Weighs the rain which falls into a bucket placed on the platform of a spring or lever balance. The increasing weight of the bucket helps in recording the increasing quantity of collected rain, with time, by moving a pen on a revolving drum. It gives mass curve of rainfall.

Symon's Gauge: A non recording raingauge.

Natural Syphon Type: A float type gauge, provided with a self starting syphonic arrangement is most widely used in India; and is population known as Natural syphon recording rain gauge. It is standard recording type gauge adopted in India.

Q.26 (c)

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

$E_L = k(e_w - e_a)$

Where,

E_L = rate of evaporation (mm/day)

k = a constant

e_w and e_a are in mm of mercury

Q.27 (d)

Volume of water evaporated in a day

$= (80\times 10^3\times 15\times 0.5\times 10^{-2})\ m^3$

$= 6000\ m^3$

Pan constant for class A pan = 0.7

∴Volume of water evaporated in a month

$= (6000\times 30\times 0.7)m^3$

$= 126{,}000\ m^3$

Q.28 (c)

An ISI standard pan evaporimeter has about 14% less evaporation than a US Class A pan.

Q.29 (d)

Transpiration is essentially confined to day light hours and rate of transpiration depends upon the growth period of the plant. Evaporation on the other hand, continues all through the day and night although the rates are different.

Q.30 (b)

A part of rain may be caught by vegetation and subsequently evaporated. The volume of water so caught is called interception. Interception loss is solely due to evaporation and does not include transpiration, through fall or stemflow.

Q.31 (c)

The pan coefficient for US class A pan is 0.7 while it is 0.8 for ISI standard Pan.

Q.32 (d)

Certain chemicals such as cetyl alcohol (hexadecanol) and stearyl alcohol (octadecanol) form monomolecular layers on a water surface. These layers act as evaporation inhibitors by preventing the water molecules to escape past them,

Q.33 (d)

$\emptyset$-Index = $\frac{6-3}{6}$ = 0.5 cm/hr

∴Runoff due to 12 cm of rainfall

= 12 – 0.5 × 9

= 12 - 4.5

= 7.5 cm

Q.34 (d)

Total rainfall

= 7 + 18 + 25 + 12 + 10 + 3

= 75 mm

Surface runoff = 33 mm

Assuming t_e = 6 hrs

$$\phi-\text{index} = \frac{75-33}{6} = \frac{42}{6} = 7 \text{ mm/hr}$$

Since, first and last hours are not the period of rainfall excess as rainfall intensity $\leq \phi-\text{index}$

$$\phi-\text{index} = \frac{75-7-3-33}{4} = \frac{32}{4} = 8$$

mm/hr

Q.35 (b)

Rainfall lost = Rainfall - Runoff

$$= 0.8\times 6 - \frac{256000}{8.6\times 10^6}\times 100$$

= 4.8 - 2.976 =1.823 cm

Total rainfall lost in 6 hours

= 1.823 cm

∴Rainfall lost per hour

$$= \frac{1.823}{6} = 0.304 \text{ cm/hr}$$

Q.36 (d)

Q_{peak} = Kp_cA

$$= 0.4\times\frac{4.5\times 10^{-2}}{60\times 60}\times 90\times 10^4$$

= 4.5 m^3/sec

Q.37 (a)

$$\text{Co-efficient of variation } Cv = \frac{100\times 6m-1}{\overline{P}}$$

$$= \frac{100\times 35.04}{118.6}$$

= 29.54

So, optimum number of stations

$$N = \left(\frac{29.54}{10}\right)^2$$

= 8.7

= 9

Therefore, additional stations required

=9-6

=3

Q.38 (d)

Indirect methods are those methods which make use of relationship between the flow discharger and the depths at specified locations. Two types of these indirect method are flow measuring structures and slope area methods.

Q.39 (b)

The float-operated stage recorder 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 counter weight over the pulley of a recorder. To protect the float from debris and to reduce the water surface wave effects on the recording, stilling wells are provided in all float-type stage recorder installations.

Q.40 (c)

Discharge in the channel

$= \overline{V}\times$ area of channel

Area of channel

$$= \frac{1}{2} \times 0.9 \times 2 = 0.9\,m^2$$

$$\overline{V} = \frac{V_{0.2y} + V_{0.8y}}{2}$$

$$= \frac{0.6 + 0.4}{2} = 0.5 m/s$$

$\therefore$ Discharge, $\phi = 0.9 \times 0.5$

$= 0.45\ m^3/s$

Q.41 (b)

Q.42 (c)

The dilution method has the major disadvantage that the discharge is estimated directly in an absolute way. It is a particularly attractive method for small turbulent streams, such as these in mountainous areas.

Q.43 (c)

Q.44 (b)

In shallow stream of depth up to about 3.0 m, the average velocity

$\overline{V} = V_{0.6d}$

This is called single point observation. In moderately deep streams the velocity is observed at two point.

$$\therefore \overline{V} = \frac{V_{0.2d} + V_{0.8d}}{2}$$

In rivers having large flood flows. Only surface velocity (V_s) is measured within a depth of about 0.5 m from the surface. The average velocity $\bar{V}$ = KV_s where K is in the range of 0.85 to 0.95.

Q.45 (c)

The relation between the stream velocity and revolutions per second of the current meter is called the calibration equation. The calibration is unique to each instrument and is determined by towing the instrument in a special tank called towing tank. Towing tank is a long channel containing still water with arrangements for moving a carriage longitudinally over its surface at constant speed. The current meter to be calibrated is mounted on the carriage with the rotating element immersed to a specified depth in the water body in the tank. The carriage is then towed at a predetermined constant speed (v) and the corresponding average value of revolution per second (N_s) of the instrument determined. This experiment is repeated over the complete range of velocities and a best-fit linear relation in the form of the equation $v = aN_s + b$ where a, b are constants of the current meter.

Q.46 (b)

Infiltration (per 6 hr) = rainfall - Run off

$$= 0.9 \times 6 - \frac{257000}{8.7 \times 10^6} \times 100$$

$= 2.45\ cm$

$$\text{Infiltration (per hr)} = \frac{2.45}{6}$$

$= 0.408\ cm/hr.$

Q.47 (b)

Discharge is directly proportional to $\sqrt{S}$.

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

$$= 142 \times \sqrt{\frac{1}{3000} \times 6000}$$

$= 200.8\ m^3/s$

Q.48 (a)

Direct runoff 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.

Q.49 (b)

Q.50 (a)
The delayed flow that reaches a stream essentially as groundwater flow is called base flow.

Q.51 (c)
A stream flow unaffected by works of man, such as structures for storage and diversion on a stream is called virgin flow.

Q.52 (b)

Q.53 (c)
An intermittent stream has limited contribution from the groundwater. During the wet season the water table is above the stream bed and there is a contribution of the base flow to the stream flow.

Q.54 (d)
A flow-duration curve of a stream is a plot of discharge against the per cent of time the flow was equalled or exceeded. This curve is also known as Discharge frequency curve.

Q.55 (a)
The flow mass curve is a plot of the cumulative discharge volume against time plotted in chronological order. The ordinate of the mass curve, V at any time t is,

$$V = \int_{t_0}^{t} Q.dt$$

Since the hydrograph is plot of Q v/s t, it is easy that the flow mass curve is an integral curve of the hydrograph.

Q.56 (b)
Rainfall excess = 2.75 - 0.35 × 3

= 1.7 cm

Peak discharge of DRH = 200 - 20

= 180 m^3/s

Peak of 3-hr UH = $\frac{180}{1.7}$ = 105.88 m^3/s

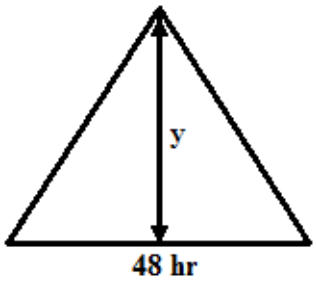
Q.57 (b)
If the bed of the stream is below the groundwater table, during period of low flows in the stream, the water surface may go down below the general water table elevation and the groundwater contributes to the flow in the stream such stream which receive groundwater flow are called effluent stream water percolates to the groundwater storage and a hump is formed in the groundwater table. Such streams which contribute to the groundwater are known as influent streams.

Q.58 (a)
Runoff = 10 mm/hr × 1 × 10^4 m^2
= 10 × 10^{-3} × 10^4
= 100 m^3/hr

Q.59 (b)
According to Khosla's formula
$R_m = P_m - L_m$
Where, $L_m = 0.48\ T_m$
For $T_m > 4.5^0C$
Where
R_m = Monthly runoff in cm
P_m= Monthly rainfall in cm
L_m = Monthly losses in cm
T_m= Mean monthly temp. of the catchment in °C.

Q.60 (a)
The drainage density is defined as the ratio of the total channel length to the total drainage area. A large drainage density creates situation conductive for quick disposal of runoff drawn the channels.

Q.61 (c)



$$\text{Discharge} = \frac{1}{2} \times 48 \times y$$

$$1728 \times \frac{1}{100} = 24\,y$$

$$y = \frac{1728 \times 10^6}{100 \times 24 \times 60 \times 60}$$

$$= 200\ m^3/s$$

Q.62 (b)

Equilibrium discharge of S-curve

$$= \frac{\text{Total discharge}}{\text{Unit hour duration}}$$

$$= \frac{36 \times 10^6 \times 0.01}{2 \times 3600} = 50\ m^3/sec$$

Q.63 (c)

Gumbel's distribution has the property which gives T = 2.33 years for the average of the annual series when N is very large. Thus the value of a flood with T = 2.33 years is called the mean annual flood.

Q.64 (c)

Kirpich equation is an empirical relation used for the estimation of the time of concentration. It is given as

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

Where, t_c = time of concentration in minutes

L = maximum length of travel of water (m)

S = slope of the catchment = ΔH/L

ΔH = difference in elevation between the most remote point on the catchment outlet.

Q.65 (d)

For permanent barrages and minor dams with capacity less than 60 Mm^3 design flood recommended is SPF or flood with a return period of 100 years whichever gives higher value.

Q.66 (d)

In reservoir routing, the storage is a unique function of outflow discharge, S = f (Q).

However, in channel routing the storage is a function of both outflow and inflow discharge and hence a different routing method is needed from muskingum method of hydrological channel routing storage is given by

$S = k\,[XI^m + (1 - X)\,Q^m]$

Q.67 (a)

Muskingum method for channel routing is given by

$S = k\,[xI + (1 - x)Q]$

Q.68 (d)

Muskingum routing equation,

$Q_2 = C_0I_2 + C_1I_1 + C_2Q_1$

$$\text{Where, } C_0 = \frac{-kx + 0.5\Delta t}{k(1-x) + 0.5\Delta t}$$

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

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

Which gives $C_0 + C_1 + C_2 = 1.0$

Q.69 (a)

$$C_0 = \frac{-kx + 0.5\Delta t}{k(1-x) + 0.5\Delta t}$$

$$= \frac{-12 \times 0.15 + 0.5 \times 4}{12(12 - 0.15) + 0.5 \times 4}$$

$$= \frac{0.2}{12.2} = 0.016$$

Q.70 (c)

In channel routing, storage is a function of both inflow and outflow.

Q.71 (b)

$C_u = k\sum f$

$= k\sum \frac{p}{40}(1.8t + 32)$

$= 0.5\left[\left\{\frac{8}{40}(1.8\times 20+32)\right\}+\left\{\frac{8}{40}(1.8\times 20+32)\right\}\right]$

= 13.6 cm

Q.72 (c)

Net Irrigation Requirement (NIR) is the amount of irrigation water required in order to meet the evapotranspiration need of the crop as well another needs such as leaching. Therefore,

NIR = C_u-R_e+ Water lost as percolation in satisfying other needs such as leaching

Q.73 (b)

Check flooding is similar to ordinary flooding except that the water is controlled by surrounding the check area with low and flat levels. Levels are generally constructed along the continuous having vertical interval of about 5 to 10 cm. This method is suitable for more permeable soils as well as, for less permeable soils. The water can also be held on the surface for a longer time in case of less permeable soils.

Q.74 (c)

In Drip irrigation method, water is slowly and directly applied to the root zone of the plants, thereby minimizing the losses by evaporation and percolation. It is the latest field irrigation technique and is meant for add at places where there exist acute scarcity of irrigation water.

Q.75 (c)

$\Delta \times D = 864 \times B$

D = 1000 ha/cumec

B = 100 days

$\Delta = \frac{864\times 100}{1000} = 86.4\,\text{cm}$

Q.76 (b)

$\text{Duty} = \frac{864\times 120}{70} = 1481\,\text{ha/cumec}$

Q.77 (b)

The value of Delta represent the total depth of water requirement of the crops.

Q.78 (c)

The readily available moisture

= 0.5 × (20 - 10) = 5%

Deficiency due to the fall of moisture is

$= \frac{\gamma d}{\gamma\omega}\times d \times \text{readily available moisture}$

$= \frac{1500}{1000}\times 1000\times 0.05 = 75\,\text{mm}$

25 mm depth of water is available from precipitation so net irrigation needed is 75 – 25 = 50 mm

Q.79 (c)

The area to be irrigated = 0.5 × 1000 = 500 ha

The discharge needed on the field

$= \frac{500}{2000} = \frac{1}{4} = 0.25\,\text{cumec}$

The discharge required at the head of water course, for 25% loss of water

$Q = \frac{0.25}{0.75} = \frac{1}{3}\,\text{cumec}$

Q.80 (c)

Design discharge

$= \frac{\text{Discharge required in the field}}{\text{Time factor}\times\text{capacity factor}}$

$= \frac{0.4}{0.5\times 0.8} = 1.0\,\text{cumecs}$

Q.81 (a)

Sodium adsorption ratio (SAR) is given by

$$SAR = \frac{Na^{+}}{\left[\frac{Ca^{++} + Mg^{++}}{2}\right]^{1/2}}$$

Q.82 (b)

SAR	Type of Water
0 - 10	Low sodium
10 - 18	Medium
18 - 26	High sodium water
> 26	Very high sodium water

Q.83 (c)

Leaching requirement in %

$$= \frac{D_i - C_u}{D_i} \times 100$$

Where,

D_i = Total irrigation water depth to be applied

C_u= consumptive use

$$10 = \frac{D_i - 90}{D_i} \times 100$$

$D_i = 100mm$

Q.84 (d)

The area, on which crops are grown in particular season, is called as a seasonal cropped area or seasonal sown area, such as rabi area or kharif area. Net sown area is the total cropped area (which is only cropped once) in the entire year, i.e. in both season.

Q.85 (c)

Canal escapes are the safety valves of canal and must be provided at regular intervals depending upon the importance of the canal and availability of suitable drainage. Minimum capacity of escape is generally kept as half of the channel capacity at the point of escape.

Q.86 (a)

Fine board is provided in gravity dam is normally 3/2 hw or 4-5% of the dam material.

Q.87 (d)

Zoned embankments are usually provided with a central impervious core, covered by a comparatively pervious transition zone, which is finally surrounded by a much more pervious outer zone. Clay inspite of it being highly impervious, may not make the best case, if it shrinks and swells too much. Due to this reason, clay is mixed with fine sand or fine gravel so as to use it as the most suitable material for the central impervious core.

Q.88 (c)

Rockfill dams are very useful in seismic regions, as they provide high resistance to seismic forces because of their flexible character

Q.89 (b)

The critical velocity as per Kennedy's method is given by

$$V_o = 0.55\, my^{0.64}$$

$$= 0.55 \times 1.12 \times (1.55)^{0.64}$$

$$= 0.815\ m/s$$

Q.90 (d)

A lot of mathematical calculations are required in designing irrigation channels by the use of Kennedy's method. To save mathematical calculations, graphical solution of Kennedy's and Kutter's equation, was evolved by Garret.

Q.91 (b)

The wetted perimeter P of a channel is given by,

$P = 4.75\sqrt{Q}$

Q.92 (b)

Lacey said that even a channel showing no silting no scouring may actually not be in regime. He differentiated between three regime conditions.

i) True regime

ii) Initial regime and

iii) Final regime

According to him, a channel which is under 'initial' regime is not a channel in regime, as there is no silting or scouring and hence regime theory is not applicable to such channels. His theory is therefore applicable only to those channels, which are either in true regime or in final regime.

Q.93 (c)

If after providing sufficient section for bank embankment, the saturated gradient cuts the downstream end of the bank the saturation line is covered by atleast 0.5 metre with the help of counter berms at the outer side of the canal banks.

Q.94 (c)

Lacey states that silt is kept in suspension due to force of vertical eddies. According to him, the eddies are generated from bed and sides, both normal to surface of generation. Hence vertical component of eddies generated from sides will also support the silt.

Q.95 (c)

According to Lacey's theory the dimensions of bed width, depth and slope of canal attain a state of equilibrium with time which is called true regime state. Lacey defined a regime channel as a stable channel transporting a minimum bed load consistent with fully active bed.

Q.96 (c)

Lacey gives three equations independent of each other representing depth, width and gradient of channel.

Q.97 (c)

Perimeter, $P = 4.75\sqrt{Q}$

$= 4.75 \times \sqrt{100} = 47.5$ m

Q.98 (c)

Lacey's silt factor,

$f = 1.761\sqrt{d_m}$

$= 1.76 \times \sqrt{0.16} = 0.704$

Q.99 (a)

Garret's diagram is graphical solution of Kennedy's and Kutter's equations to save enormous mathematical calculations in the design of irrigation channels. These have been drawn for trapezoidal channel with side slopes as $\frac{1}{2}$ H : 1 V on the assumption that irrigation channels adopt approximately this shape even though they were constructed on different side slopes.

Q.100 (c)

Velocity of flow is given by $\left(\frac{Qf^2}{140}\right)^{1/6}$

Q.101 (c)

Lacey's scour depth

$= 1.35\left(\frac{q^2}{f}\right)^{1/3}$

q = 3 cumec

f = 1.2

$R = 1.35\left(\frac{3^2}{1.2}\right)^{1/3} = 2.64$ m

Q.102 (c)

R = 0.45 m

$\gamma_w = 9.81 k/N\ m^3$

$\tau_0 = 0.566\ N/m^2$

$\tau_0 = \gamma_w RS$

$$S = \frac{\tau_0}{\gamma_w R} = \frac{0.566}{9.81 \times 10^3 \times 0.45}$$

$S = 1.2821 \times 10^{-4} = 1/7800$

Q.103 (c)

A lined canal can be designed smaller in cross-section and also in length. The steeper gradients can be provided because higher velocities are permissible. It can, therefore, help to bring high areas under command.

Q.104 (d)

In large canals, safety ladders are generally provided on side slopes, at suitable intervals along the canal length. These ladders can be used by persons or cattle, that may be swept away with the flowing water in the canal. Such a person, who may otherwise get drowned, may catch hold of one such ladder and can climb it to get out of the canal easily.

Q.105 (c)

The best alignment of off-taking channel is that in which the off-taking channel makes zero angle with the parent channel initially and then separates out in transition. The depth of water should always be such that off take channel runs full. The transitions should be properly designed, so as to avoid accumulation of silt. The transition can also be used as metering flume. Thus depth, slope and width will be the correct order of preference of parameters.

Q.106 (d)

A land is said to be water-logged when its productivity gets affected by the high water table. The productivity of land gets affected when the root zone of the plants gets flooded with water.

Q.107 (c)

Bedding is a method of surface drainage which makes use of dead furrows. They are between the two adjacent ferrous is known as bed. The depth of bed depends on soil characteristics and till age practices.

Q.108 (c)

Impounding reservoirs can store water during high flow and utilize the same during lean slow period (i.e. when flow in stream is less than demand).

Q.109 (a)

If the river is collecting sediment and is building up its bed, it is called an aggrading or of an accreting type. If the bed is getting scoured year to year it is called a degrading river. If there is no silting or scouring it is called a stable river. An aggrading river is a silting river.

Q.110 (b)

A Groyne painting downstream has the property of attracting the flow towards it and is called an attracting groyne. In attracting groynes scour holes are developed nearer to the bank, as compared to those in a repelling groyne.

Q.111 (b)

A canal outlet or a module is a small structure built at the head of the watercourse so as to connect it with a minor or a distributary channel.

Q.112 (d)

Rigid modules or modular outlets are those through which the

discharge is constant and fixed within limits irrespective of the fluctuations of the water levels of either the distributary or of the water course or of both sensitivity = 0, and flexibility = 0.