UNIT-2

Abstraction from Precipitation: Evaporation – process, measurement and estimation; Evapotranspiration-measurement and estimation; Initial Losses- Interception & Depression storage; Infiltration- process, capacities, indices, measurement & estimation.

ABSTRACTIONS FROM PRECIPITATION

All abstractions from precipitation, viz. those due to interception, evaporation, transpiration, infiltration, surface and sub-surface detention as well as storages, watershed leakages are considered as losses in the production of runoff. Evaporation from water bodies and soil masses togetherwith transpiration from vegetation is termed as Evapotranspiration.

EVAPORATION PROCESS

As is true for other bodies, water also consists of large number of molecules, which are constantly moving with different velocities in different directions.Their average velocity represents the temperature of water .Each of these molecules is attracted to every other molecule by a force directly proportional to the product of their masses and inversely proportional to the squre of the distance between them.Thus a molecule of water, which lies near the surface, will have more forces attracting it from below than from above. Therefore, this molecule is less rigidly held and can be detached from the water surface, if its own velocity is sufficient to overcome the resisting force.In the immediate vicinity of the water surface, a thin film of air does exists.The temperature of this film of air is the same as that of water.When water vapour comes out of a given body of water,they try to crowd this film of air, and thereby quickly saturating the same with water vapour.The vapour pressure of this film of air, when it is fully saturated with vapour, is known as saturation vapour pressure.If the actual pressure of the air above this thin saturated film is less than the saturation vapour pressure, the vapour already collected in this film will disperse into the air and the evaporation will continue;but if the actual vapour pressure of the air is more than the saturation vapour pressure , then there can be no evaporation.

Although there is always continuous exchange of water molecules to and from the atmosphere, the hydrologic definition of evaporation is restricted to the net rate of vapour transport to the atmosphere i.e. net escape of water molecules from the liquid state to the gaseous state.This change in state requires an exchange of about 585 cal/gram of water evaporated. If temperature of the surface is to be maintained, these large quantities of heat must be supplied by radiation and conduction from the overlying air or at the expense of energy stored below the surface.It may be noted that only fast moving molecules can escape into air as vapour and when this fast moving molecule goes out of water body, the average velocity of the remaining molecules will fall down, and thus , reducing the temperature of water and that is the reason why cooling is produced by evaporation.

FACTORS CONTROLLING EVAPORATION PROCESS

The rate of evaporation is influenced by meteorological factors like solar radiation, air and wind temperatures, vapour pressure at the water surface and air above, wind speed, humidity and minimally by atmospheric pressure and also by the nature of the evaporating surface like area of water surface,depth of water in the water body and quality of water. Since solar radiation is an important factor, evaporation also varies with latitude, season, time of day, and sky conditions.

VAPOUR PRESSURE

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 $E_L=C\ (e_w-e_a)$. This equation is known as Dalton's law of evaporation.Evaporation continues tillew=ea.If $e_w>e_a$ condensation takes place.

TEMPERATURE

Other factors remaining the same, the rate of evaporation increases with increase in the water temperature. Regarding air temperature, although there is a general increase in the evaporation rate with increasing temperature, a high correlation between evaporation rate and air temperature does not exist.Thus for the same mean monthly temperature it is possible to have evaporation to different degree in a lake in different months.

HUMIDITY

If the humidity of the atmosphere is more, the evaporation will be less. During the process of evaporation, water vapour move from the point of higher moisture content to the point of lower moisture content and the rate of this movement is governed by the differerence of their moisture gradient existing in the air. Therefore, if the humidity is more, the evaporation will be less.

WIND

Wind aids in removing the evaporated water vapour from the zone of evaporation and consequently creates greater scope for evaporation. However, if the wind velocity is large enough to remove all the evaporated water vapour, any further increase in wind velocity does not influence vaporation.Thus the rate of evaporation increases with the wind speed upto a critical speed beyond which any further increase in the wind speed has no influence on the evaporation rate. This critical speed is a function of the size of the water surface.

ATMOSPHERIC PRESSURE

Other factors remaining same, a decrease in the barometric pressure, as in high altitudes, increases evaporation.However, this is not exactly so, because of the decrease of temperature at higher altitudes evaporation rate decreases.

QUALITY OF WATER

The effect of salinity, or dissolved salts, is brought about by the reduced vapour pressure of the solution.The vapour pressure of seawater (35,000ppm dissolved salts) is about 2% less than that of pure water at the same temperature.The presence of any dissolved salts in water reduces the saturated vapour pressure, which consequently reduces the rate of evaporation. For this reason, usually, the evaporation decreases by 1% for every 1% increase in the salinity of a water body.Turbidity may also affect the rate of evaporation by affecting the heat transfer within the depth of the water body.Any foreign material which tends to eal the water surface or change its vapour pressure will affect the evaporation.

HEAT STORAGE IN WATER BODIES

Deep water bodies have more heat storage than shallow ones. A deep lake may store radiation energy received in summer and release it in winter causing less evaporation in summer and more evaporation in winter compared to a shallow lake exposed to a similar situation. However, the effect of heat storage is essentially to change the seasonal evaporation rates and the annual evaporation rate is seldom affected.

AREA OF WATER SURFACE

The amount of evaporation is directly proportional to the area of evaporation. If the exposed area is large, the evaporation will be more, and vice-versa.

MEASUREMENT AND ESTIMATION OF EVAPORATION

The rate of evaporation from large water surfaces, such as lakes, reservoirs, ponds, rivers, etc.can be measured and estimated by the following four methods:

(a) Pan measurement method

(b)Using empirical formulae

(c) Storage equation method

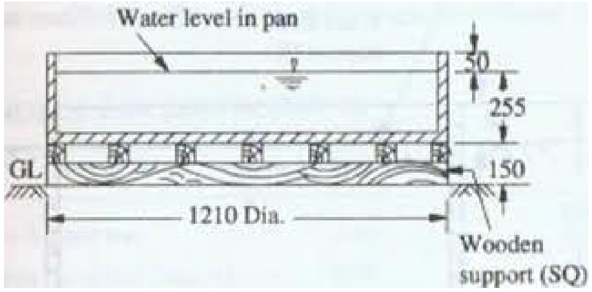
(d) Energy budget method

(a)MEASUREMENT OF EVAPORATION

Evaporimeters are water containing pans of specified sizes exposed to the atmosphere and the loss of water by evaporation is measured in them at regular intervals.The more important types of these pans are mentioned below:

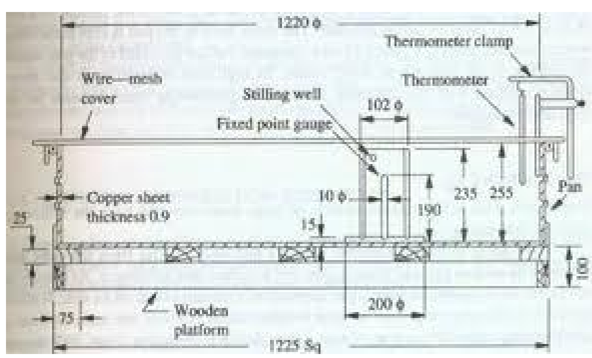
(i) **USWB CLASS ‘A’ PAN**

It is a standard pan of unpainted galvanized iron used by National Weather BureauService of United States 1210mm in diameter, 255mm deep, and it is placed 15cm above ground surface on a wooden frame to let air circulate beneath the pan .The depth of water is to be kept in a fixed range, such that the water surface is atleast 5 cm and never more than 7.5 cm below the top of pan. Water surface level is measured daily with a hook gauge in a stilling basin, and evaporation is computed as the difference between observed levels, adjusted for any precipitation measured in a standard raingauge. Alternately, water is added each day to bring the level upto a fixed point in the stilling basin. This method assures proper level at all times.Pan coefficient is about 0.6 to 0.8, generally taken as 0.7.



(ii) ISI STANDARD (Modified Class A) PAN

Details of this type of pan, used in India, are specified in IS: 5973-1970.It consists of a pan 1220mm in diameter with 255mm of depth. The pan is made of copper sheet of 0.9mm thickness,tinned inside and painted white outside.A fixed poit gauge indicates the level of water. A calibrated cylindrical measure is used to add or remove water maintaining the water level in the pan to a fixed mark. The top of the pan is covered fully with a hexagonal wire netting of galvanized iron to protect the water in the pan from birds, and also to make the water temperature more uniform during day and night.The pan is placed over a square wooden platform of 1225mm width and 100mm height to enable circulation of air underneath the pan.The evaporation from this pan is found to be less by about 14% compared to that from unscreaned pan. The pan coefficient is around 0.8.



(iii) COLORADO SUNKEN PAN

This pan, 920 mm square and 460 mm deep is made up of unpainted galvanized iron sheet and burried into the ground within 100 mm of the top. The advantage of the sunken pan is that radiation and aerodynamic characteristics are similar to those of a lake. The disadvantages are: (i) difficult to detect leaks, (ii) extra care is neededto keep the surrounding area free from tall grass, dust,etc., and (iii) expensive to instal.

(iv) USGS FLOATING PAN

With a view to simulate the characteristics of a large body of water, this square pan(900mm side x 450mm depth) supported by drum supports in the middle of a raft (4.25m x 4.87m) is set afloat in a lake.The water level in the pan is kept at the same level as the lake leaving a rim of 75mm. Diagonal baffles provided in the pan reduce the surging in the pan due to wave action. Its high cost of installation and maintainance together with the difficulty involed in performing measurements are its main disadvantages.

PAN COEFFICINTS

Evaporatio pans are not exact models of large reservoirs and have the following principal drawbacks:

(i) They differ in the heat-storing capacity and heat transfer from the sides and bottom. The sunken pan and floating pan aim to reduce this deficiency. As a result of this this factor the evaporation from a pan depends to a certain extent on its size.While a pan of 3m diameter is known to give a value which is about the same as from a neighbouring large lake, a pan of size 1m diameter indicates about 20% excess evaporation than that of the 3m diameter pan.

(ii) The height of the rim in an evaporation pan affects the wind action over the surface. Also, it casts a shadow of variable magnitude over the water surface.

(iii) The heat-transfer characteristics of the pan material is different from that of the reservoir.

In view of the above, Lake evaporation=C_p x pan evaporation, in which C_p= pan coefficient.

The values of C_p in use for different pans are given below:

VALUES OF PAN COEFFICIENTS,C_p

Sr. No.	**Type of Pan**	**Average Value**	**Range**
1.	Class A Land Pan	0.70	0.60-0.80
2.	Madified Class A Pan(ISI Pan)	0.80	0.65-1.10
3.	Colorado Sunken Pan	0.78	0.75-0.86
4.	USGS Floating Pan	0.80	0.70-0.82

(b) USING EMPIRICALFORMULAE

Most formulae to estimate lake evaporation using meteorological data are based on the Dalton's equation $E_L=Kf(u)\ (e_w-e_a)$, where E_L=lake evaporation in mm/day,ew=saturated vapour pressure at the water-surface temperature in mm of mercury,e_a=actual vapour pressure of over-lying air at a specified height in mm of mercury, f(u)=wind-speed correction factor and K=a coefficient.The term ea is measured at the same height at which wind speed is measured.Two commonly used formulae are :

MEYER'S FORMLA (1915)

$E_L=K_M(e_w-e_a)[1+(u_9/16)]$ In this equation E_L,e_w,e_a are as defined earlier, whereas u_9=monthly mean wind velocity in km/hr at about 9m above ground and K_M=coefficient accounting for various other factors with a value of 0.36 for large deep waters and 0.50 for small shallow waters.

ROHWER'S FORMULA (1931)

Rohwer's formula considers a correction for the effect of pressure in addition to the wind-speed effect and is given by

$E_L=0.771(1.465-0.000732p_a)(0.44+0.0733u_0)(e_w-e_a)$

In which p_a=mean barometric reading in mm of mercury and u_0=mean wind velocity in km/hr at ground level, which can be taken at 0.6m above ground.In the lower part of atmosphere,upto a height of about 500m above the ground level, the wind velocity can be assumed to follow the 1/7 power lawas $u_h=C\ h^{1/7}$

(c) ANALYTICAL METHODS OF ESTIMATION OF EVAPORATION

(i) Water budget method E_L=P+(V_{is}-V_{os})+(V_{ig}-og)-T_L- ΔS where P=daily precipitation, V_{is}=daily surface inflow into the lake,V_{ig}=daily groundwater inflow, V_{os}=dailysurface outflow from the lake, V_{og}=daily surface outflow, E_L=daily lake evapoation,ΔS=increase in lake storage in a day,and T_L=daily transpiration loss.

In the above equation P,V_{is},V_{os} and ΔS can be measured.However,it is not possible to measure V_{ig},V_{og} and TL and therefore these quantities can only be estimated.Transpiration losses can be considered to be insignificant to some reservoirs.

(ii) Energy balance method $E_L=[(H_n-H_g-H_s-H_i)/p_wL(1+\beta)]$ where Hn=net incoming energy,Hg=heat lost to ground,Hs=heat stored in the water body,Hi=net heat conducted out of the system by water flow (advected energy), p_w=density of water,L=latent heat of vaorization of water,$\beta=6.1 \times 10^{-4} P_a[(T_w-T_a)/(e_s-e_a)$

EVAPOTRANSPIRATION

In studying the hydrologic balance for a catchment area,one is concerned with the evaporation from all water,soil,snow,ice,vegetation,and other surfaces plus transpiration; the total of these processes are known as Evapotranspiration.

Transpiration is the process by which water leaves the body of a living plant and reaches the atmosphere as water vapour. The difference in concentration between the sap in the root cells of a plant and the soil water causes an osmotic pressure which moves soil water throughthe root membrane into the root cells. High salinity of the soil solution and/or high moisture tension in the soil may prevent or greatly reduce the osmtic transfer.Once inside the root, the water is transferred through the plantto the intercellular space within the leaves.Air enters the leafthrough the stromata, openings in the leafe surface, and the chloroplasts within the leaf use carbon dioxide from the air and a small portion of the available water to manufacture carbohydrates for plant growth (photosynthesis). As air enters the leaf, water escapes through the open stomata; this is the process of transpiration. The ratio of the water transpired to that used in forming plant matter is very large (upto 800% or more). Consumptive Use is the total evaporation from an area plus the water used directly in building plant tissues.

While transpiration takes place, the land area in which plants stand also lose moisture by the evaporation of water from soil and water bodies. Since in the process of vegetation growth, it is generally not feasible to separate the transpiration and its connected evaporation from the plants surroundings, a combined measure is obtained and this is called Evapotranspiration.If sufficient water is always available to completely meet the needs of vegetation, fully covering the area, the resulting evapotranspiration is called Potential Evapotranspiration (PET).It essentially depends on the climatic factors. The real evapotranspiration occuring in a specific situation is called ActualEvapotranspiration.

MEASUREMENT OF EVAPOTRANSPIRATION

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

LYSIMETERS

A lysimeter ssis a special watertight tank containing a block of soil and set in a field of growing plants.Th e plants grown in the lysimeter are the same as in the surrounding field. Evapotranspiration is estimated in terms of the amount of water required to maintain constant moisture conditions within the tank measured either volumetrically or gravimetrically through an arrangement made in the lysimeter.It should be designed to accurately reproduce the soil conditions,moisture content, type and size of the vegetation of the surrounding area.They should be so buried that the soil is at the same level inside and outside the container. Lysimeter studies are time-consuming and expensive.

FIELD PLOTS

In special plots all the elements of the water budget in a known interval of time are measured and the evapotranspiration determined as:

Evapotranspiration=[Precipitation+Irrigation input-Runoff-Increase in soil storage-Ground water loss]

Measurements are usually confined to precipitation, irrigation input, surface runoff,and soil moisture. Ground water loss due to deep percolation is difficult to measure and can be minimised by keeping the moisture condition of the plot at the field capacity. This method provides fairly raliable results.

ESTIMATION OF EVAPOTRANSPIRATION

Most simple and commonly used methods of estimation of evapotranspiration are:

(i) Blaney- Criddle Equation

(ii) Thornthwaite Formula

(iii) Penman's equation

(i) Blaney- Criddle Equation

The potential evapotranspiration in a crop-growing season is given by E_T=2.54KF and F=$P_h T_f$/100

Where E_T=PET in a crop season in cm, K=an empirical coefficient depending on the type of the crop and stage of growth, F=sum of monthly cosumptive factors for the period, P_h=monthly percent of annual day-time hours dpending on the latitude of the place,and Tf= mean monthly temperature ^{0}F.

(ii) THORNTHWAITE FORMULA

This formula was developed from data of eastern USA and uses only the mean monthly temperature together with an adjustment for day-lengths. The PET is given bythis formula as

$E_T=1.6L_a[10T/I_t]^a$

Where E_T=mean PET in cm, L_a=adjustment for the number of hours of daylight and days in the month, related to the latitude of the place, T=mean monthly air temperature 0 C, I_t=the total of 12 monthly values of heat index=$E_1^{12}i$, where $i=(T/5)^{1.514}$, a= an empirical constant=$6.75x\ 10^{-7} I_t^3-7.71x10^{-5}I_t^2+1.792x\ 10^{-2}\ I_t+0.49239$

(iii) PENMAN'S EQUATION

Penman's equation is based on sound theoretical reasoning and is obtained by a combination of the energy balance and mass transfer approach.Penman's equation, incorporating some of the modifications suggested by other investigators is

PET=$\frac{AHn+Ea\gamma}{A+\gamma}$ where PET=daily potential evapotranspiration in mm per day,A=slope of the saturation vapour pressure vs temperature curve at the mean air temperature,in mm of mercury per ^{0}C, H_n=net radiation in mm of evaporable water per day,E_a=parameter including wind velocity and saturation deficit,γ=psychrometric constant=0.49mm of mercury/^{0}C.

The net radiation is the same as used in the energy budget and is estimated by the following equation:

$Hn=Ha(1-r)[a+b\frac{n}{N}]-бT_a^4(0.56-0.092)\sqrt{ea})[0.10+0.90\frac{n}{N}]$ where H_a= incident solar radiation outside the atmosphere on a horrizontal surface, expressed in mm of evaporable water per day, a=a constant depending upon the latitude φ and is given by a=0.29cosφ, b=a constant with an average value of 0.52, n=actual duration of bright sunshine in hours, N=maximum possible hours of bright sunshine,r=reflection coefficient whose values are given below:

Surface	Range of r values
Close ground crops	0.15-0.25
Bare lands	0.05-0.45
Water surface	0.05
Snow	0.45-0.95

Б=Stefan-Boltzman constant=2.01x10-9 mm/day,T_a=mean air temperature in degrees kelvin=273+^{0}C,e_a=actual mean vapour pressure in the air in mmof mercury.

The parameterEa is estimated as Ea=0.35[1+$\frac{u2}{160}$] (ew-ea) in which u_2=mean wind speed at 2 m above ground in km/day,e_w=saturation vapour pressure at mean air temperature in mm of mercury,e_a=actual vapour pressure.

For the computation of PET, data on n,e_a,u_2, mean air temperature and nature of surface (r) are needed which can be obtained from actual observations or through available meteorological data of the region.

INITIAL LOSSES:

INTERCEPTION

Interception is that amount of precipitation water which is intercepted by vegetative foliage, buildings, and other objects lying over the land surface. Various factors that affect interception are (i) storm factor, (ii) plant factor, (iii) season of the year, and (iv) prevailing wind.

DEPRESSION STORAGE

A catchment area generally has many depressions of shallow depth and of varying size and shape. When precipitation takes place, water runs towards these depressions and fill them before actual overland flow or runoff towards a stream takes place. The water retained in these depressions is called depression storage and these depressions form miniature reservoirs detaining water temporarily. The water stored in these depressions partly evaporates and partly infiltrates into the ground. The following relationship may be used for computing the depression storage:

$\mathbf{V_{ds}=K\ [1-e^{-Pe/K}]}$ where V_{ds}=Volume of water stored in surface depression, P_e= Rainfall excess (volume of water in excess of infiltration and interception), K=Depression storage capacity of the basin.

Depression storage depends on following factors: (i) Land form, (ii) Soil characteristics, (iii) Topography, (iv) Antecedent precipitation index, and (v) man made disturbance like terace forming etc.

WATERSHED LEAKAGE

It is the flow of water from one basin to another, or from one basin to sea through major faults, fissures or other geographical and geological features. It is different than the man-made inter-basin transfer of surface water.

INFILTRATION-Process:

Infiltration is the movement of water through the soil surface into the soil as distinguished from percolation, the movement of water through the soil. Although a distinction is made between infiltration and percolation, these two phenomena are closely related since infiltration can not continue unimpeded unless percolation removes infiltered water from the surface soil. When water is first applied to the soil surface, gravity water moves down through the larger soil openings following the path of least rsistance, while the smaller surface pores take in water by capillarity. The down-ward moving gravity water is also taken in by capillary pores. The capillary forces continuously divert gravity water into capillary-pore space, so that the quantity of gravity water passing successively lower horizons is steadily diminished. As capillary pores at the surface are filled and intake capacity is reduced, the infiltration rate decreases. In homogeneous soil, infiltration decreases gradually untill the zone of aeration is saturated. Normally, the soil is stratified and sub-soil layers are often less permeable than the surface soil. In this case, the infiltration rate is eventually limited to the rate of percolation through the least pervious subsoil stratum.

When water is applied at the surface of the soil, following four moisture zones in the soil are identified: (i) saturation zone, (ii) transition zone, (iii) transmission zone, and (iv) wetting zone.

INFILTRATION CAPACITY:

The maximum rate at which water can enter the soil at a point in a given set of conditions is called the infiltration capacity; f_p.The actual infiltration rate f_i equals the infiltration capacityf_p only when the supply i_s (rainfall intensity less rate of retention) equals or exceedsfp. The value of f_p is at a maximum f_o at the beginning of a storm and approaches a low, constant rate f_c as the soil profile becomes saturated. The limiting value is controlled by subsoil permeability.Horton found the infiltration-capacity curves approximate the form:

$f_p=f_c+ (f_o-f_c)\ e^{-kt}$,where k is an empirical constant, and t is time from beginning of rainfall.The equation is applicable only when is>fp through out the storm. Philip suggested the equation

$f_p=[(bt^{-1/2})/2] +a$ Integrating this equation with respect to time gives the cummulative infiltrationF at time t as : $F=b\ t^{1/2} +at$

Infiltration capacity depends on many factors such as soil type including compaction due to rain, soil moisture content, organic matter, vegetative cover, and temperature. Of the soil characteristics affecting infiltration, non-capillary porosity is perhaps the most important.

INFILTRATION INDICES

The infiltration approach to runoff estimates assumes that the surface runoff from a given storm is equal to that portion of the rainfall which is not disposed of through (i) interception and depression storage, (ii) evaporation during storm, and (iii) infiltration. If items (i) and (ii) are invariable or insignificant or can be assigned reasonable values, one need be concerned only with rainfall, infiltration, and runoff.Therefore in hydrological calculations involving floods it is found convenient to use a constant value of infiltration rate for the duration of the storm. The defined average infiltration rate is called infiltration index. Two types of indices are in common use, namely **φ** index and **W**-index.

Φ index:

It is defined as that rate of rainfall above which the rainfall volume equals runoff volume. It is derived from the rainfall hyetograph with knowledge of the resulting runoff volume. The initial loss is also considered as infiltration. If the infiltration intensity is less than φ, then the infiltration rate is equal to the rainfall intensity; however, if the rainfall intensity is larger than φ the difference the rainfall and the infiltration in an interval of time represent the runoff volume. The amount of rainfall in excess of the index is called rainfall excess or effective rainfall.

On the basis of rainfall and runoff correlations, CWC has found the following relationships for estimation of φ index for flood producing storms and soil conditions in India:

$R=\alpha I^{1.2}$ and $\varphi = (I-R)/24$, where R =runoff in cm from a 24-hour rainfall of intensity of I cm/hour and α=a coefficient which depends upon soil type as detailed below:

Sr. No.	Type of soil	Coefficient α
1	Sandy soil and sandy loam	0.17 to 0.25
2	Coastal alluvium and silty loam	0.25 to 0.34
3	Red soils, clayey loam, grey and brown alluvium	0.42
4	Blacl-cotton and clayey soils	0.42 to 0.46
5	Hilly soils	0.46 to 0.50

In absence of any other data, a φ – index value of 0.10 cm/hour can be assumed.

W-index:

The W index is the average infiltration rate during the time rainfall intensity exceeds capacity rate.It is essentially equal to the φ index minus the average rate of retention by interception and depression storage.

MEASUREMENT OF INFILTRATION

Infiltration characteristics of a soil at a given location can be estimated by (i) using flooding type infiltrometers, (ii) measurement of subsidence of free water in a large basin or pond, (iii) rainfall simulator, and (iv) hydrograph analysis.

DOUBLE-RING INFILTROMETER

Two sets of concentrating rings with diameters of 30cm and 60cm and of a minimum length of 25cm are used. The two rings are inserted into the ground and water is aplied into both the rings to maintain a constant depth of about 5cm.The outer ring provides water jacket to the infiltering water from the inner ring and hence preventsthe spreading out of the infiltering water of the inner ring. The water depths in the inner and outer rings are kept the same during the observation period.The measurement of water volume is done on the inner ring only. The experiment is carried out till a constant infiltration rate is obtained. A perforated disc is provided on the surface of the soil in the inner ring as well as in the annular space to prevent formation of turbidity and settling of fine silt on the soil surface.

The chief disadvantages of flooding- type infiltrometers are: (i) the raindrop impact effect is not simulated, (ii) the driving of the tube or rings disturbs the soil structure, and (iv) the results of infiltrometers depend to some extent on their size with the larger meters giving less rates than the smaller ones due to border effects.

RAINFALL SIMULATORS

In this a small plot of land, of about 2mx4m size, is provided with a series of nozzles on the longer side with arrangements to collect and measure the surface runoff rate. The specially designed nozzles produce raindrops falling from a height of 2m and are capable of producing various intensities of rainfall. Experiments are conducted under controlled conditions with various combinations of intensities and durations and the surface runoff rates and volumes are measured in each case. Using the water budget equation involving the volume of rainfall, infiltration and runoff, the infiltration rate and its variation with time are estimated. If the rainfall intensity is higher than the infiltration rate, infiltration capacity values are obtained.

Rainfall simulator type infiltrometers give lower values than flooding type infiltrometers due to effect of rainfall impact and turbidity of the surface water present in the former.

HYDROGRAPH ANALYSIS

Reasonable estimation of the infiltration capacity of a small watershed can be obtained by analysing measured runoff hydrographs and corresponding rainfall records.Water budget equations can be used to estimate the abstraction by infiltation. Evaporation losses are estimated by knowing the land cover/use of the watershed.

UNIT-3

Runoff and Hydrographs: Hydrograph, runoff characteristics of stream, Yield, Rainfall-runoff correlations, flow duration curve, mass curve, droughts and floods. Factors affecting flood hydrographs, unit hydrograph and its analysis, S-curve hydrograph, synthetic and instantaneous unit hydrographs.

RUNOFF AND HYDROGRAPH

Runoff means the draining or flowing off of precipitation water from a catchment area through a surface channel. Engineering Hydrology is concerned primarily with three characteristics of streamflow:monthly and annual volumes available for storage and use,ten-daily low flow rates for in-stream uses of water through canal diversion works, hourly and daily flows and river water levels for flood mitigation.

COMPONENTS OF RUNOFF

The route followed by a water particle from the time it reaches the ground surface until it enters a stream channel is devious. It is convenient to visualize three main routes of travel: **overland flow or surface runoff, interflow or subsurface storm flow, and groundwater flow or base flow or dry-weather flow.**

Overland flow is that water which travels over the ground surface to a channel. **Surface ruoff** includes precipitation falling on the stream system whereas overland flow does not.

A part of the precipitation water that infiltrates the soil surface moves laterally through the upper soil layers and returns to the surface at some location away from the point of entry into the soil.This component of runoff is known as **inter flow or storm seepage or subsurface storm flow or quick return flow.** The proportion of total runoff which occurs as interflow depends on the physical features of the catchment.A thin soil cover overlying rock, hardpan, or plowbed a short distance below the soil surface favours substantial quantities of interflow, whereas uniformly permeable soil encourages downward percolation to groundwater.Depending upon the time delay between the infiltration and the outflow, the interflow is sometimes classified into **prompt interflow**, i.e. the interflow with least timelag and **delayed interflow**.

Some part of precipitation water may percolate downward until it reaches the water table. This groundwater accretion may eventually discharge into the streams as **groundwater flow (also called base flow and dry-weather flow)** if the water table intersects the stream channels of the basin.

The distinctions drawn between the above three components of runoff are arbitrary. Water may start out as overland flow, infiltrate, and complete its trip to the stream as subsurface storm flow; or infiltrated water may surface where a relatively impervious stratum intersects a hillside, and finish its journey to the stream as overland flow.

For convenience it has been customary to consider the total flow to be divided into only two parts: **storm, or direct, runoff and base flow**. The distinction is actually on the basis of time of arrival of water in the stream rather than on the path followed. **Direct runoff** is presumed to consist of surface runoff and a substantial portion of the interflow; whereas **base flow** is considered to be largely groundwater.The direct runoff includes rainfall on the surface of the stream and the snow-melt also.

NATURAL FLOW

Runoff representing the response of a catchment to precipitation reflects the integrated effects of a wide range of catchment, climate and rainfall characteristics.True runoff is therefore streamflow in its natural condition, i.e. without human intervention. Such a stream flow unaffected by works of men, such as reservoirs and diversion structures on a stream, is called natural flow or virgin flow.The natural flow volume in time Δt at the terminal point of a catchment is expressed by water balance equation as

$$R_N= (R_o-V_r) +V_d+E+E_X+\Delta S$$

Where R_N=Natural runoff volume in time Δt

R_o=Observed flow volume in time Δt at the terminal site

V_r=Volume of return flow from irrigation, domesticwater supply and industrial use

V_d=Volume diverted out of the stream for irrigation, domestic water supply and industrial use

E=net evaporation losses from reservoirs on the stream

E_X=Net export of water from the basin

ΔS=Change in storage volumes of water storage bodies on the stream

The natural flows have to be derived based on observed flows and data on abstractions from the stream.Always, it is the natural flow that is used in all hydrological correlations.

HYDROGRAPH

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

(i) Annual hydrographs showing the variation of daily or weekly or 10 daily mean flows over a year.

(ii) Monthly hydrographs showing the variation of daily mean flows over a month.

(iii) Seasonal hydrographs depicting the variation of the discharge in a particular season such as the monsoon season or dry season.

(iv) Hourly Flood hydrographs or hydrographs due to a storm representing stream flow due to a storm over a catchment.

COMPONENTS OF HYDROGRAPH

(i) Overland flow or Surface Runoff: It is that water which travels over the ground surface to a channel or depression which may carry a small rivulet of water in turbulent flow during a rain and after a short while after. Such channels are numerous, and the distance water must as overland flow is relatively short, rarely more than a hundred meters. Therefore overland flow soon reaches a channel, and if it occurs in sufficient quantity, is an important element in the formation of flood peaks.

(ii) Interflow or Sub-surface flow: It is the water that infiltrates the soil surface and moves laterally through the upper soil layers until it enters a stream channel.It moves more slowly than surface runoff and reaches the streams later.The proportion of total runoff which occurs as interflow depends upon on the physical features of the basin.

(iii) Baseflow or Groundwater flow or Dry-weather flow: It is the water that percolates downward until it reaches the water table leading to rise in groundwater table which may eventually discharge into the streams if the water table intersects the stream channels of the basin.

Hydrograph Separation

WATER YEAR

In annual runoff studies it is advantageous to consider a water year beginning from time when precipitation exceeds the average evaporation losses. In India,June 1st is the beginning of a water year which ends on May 31st of the followingcalender year.In a water year a complete cycle of climate changes is expected and hence the water budget will have the least amount of carry over.

RUNOFF CHARACTERISTICS OF STREAMS

The flow characteristics of a stream depend upon:

(i) Rainfall characteristics, such as magnitude, intensity, distribution according to time and space, and its variabilty

(ii) Catchment characteristics such as soil, land use/cover, slope, geology, shape and drainage density.

(iii) Climatic factors which influence evapotranspiration.

The interrelationship of these factors is quite difficult. In India the streams can be classified as:

(i) Perennial, (ii) Intermittent, and (iii) Ephemeral streams

YIELD

The total quantity of surface water that can be expected in a given period from a stream at the outlet of its catchment is known as **yield** of that catchment in that period. Depending upon the period chosen we have annual yield and seasonal yield signifying yield of the catchment in a year and in a specified season respectively.A list of values of annual yield in a number of years constitutes an annual time series which can be analyzed to assign probabilities of occurances of various events.A common practice is to assign a dependibility value (say 75% dependable yield) to the yield. Thus, 75% dependable annual yield is the value that can be expected to be equalled to or exceeded 75% times (i.e. on an average 15 times in a span of 20 years).Similarly, 50% dependable yield is the annual yield value that is likely to be equalled or exceeded 50% of times (i.e. on an average 10 times in 20 years).

It should be remembered that the yield of a stream is always related to the natural flow in the river. The annual yield of a basin at a site is thus taken as the annual natural water flow in the river at the site plus the return flow to the stream from different uses such as irrigation, domestic water supply and industries.

RAINFALL-RUNOFF CORRELATION

The relationship between rainfall in a period and the corresponding runoff is quite complex and is influenced by a host of factors relating to catchment and climate. The most common method is to fit a linear regression line between R and P and to accept the result if the correlation coefficient is nearer unity. For large catchments, sometimes it is found advantageous to have exponential relationship as $R=\beta P^m$ where β and m are constants.

Many improvements of the above basic rainfall-runoff correlation by considering additional parameters such as soil moisture and antecedent rainfall have been attempted.For calculation of the annual runoff from the annual rainfall a commonly used antecedent precipitation index P_a is given by

$P_a=a P_i+b P_{i-1}+c P_{i-2}$ where Pi,PI-1 AND Pi-2 are the annual precipitation in the ith,(i-1) th (i-2)th year and i=current year, a,b and c are the coefficients with their sum equal to unity.

EMPIRICAL EQUATIONS :(Binnie's percentages, Barlow's tables, Strange's tables, Inglis and Desuza formula, Khosla's formula) and Watershed simulation, SCS-CN method of estimating runoff volume.

FLOW-DURATION CURVE

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.The streamflow data is arranged in descending order of discharges, using class intervals if the number of individual values is very large.If N number of data points are used in this listing, the plotting position of any discharge (or class value)Q is: $P_p=m/ (N+1) \times 100$

Where, m is the order number of the discharge (or class value), P_p=percentage probability of the flow magnitude being equalled or exceeded.The plot of discharge Q against Pp is the flow-duration curve. Arithmatic,semi-log,log-log scale paper is used depending uponthe range of data and use of the plot.The flow-duration curve represents the cummulative frequency distribution and can be considered to represent the streamflow variation of an average year.The following characteristics of the flow-duration curve are of interest:

(i) The slope of a flow-duration curve depends upon the interval of the data selected.For example, a daily streamflow data gives a steeper curve than a curve based on monthly data for the same stream.This is due to the smoothening of small peaks in the monthly data.

(ii) The presence of a reservoir in a stream considerably modifies the virgin flow duration curve depending on the nature of flow regulation.

(iii) The virgin-flow duration curve when plotted on a log probability paper plots as a straight line atleast over the central region.From this property, various coefficients expressing the variability of the flow in a stream can be developed for the description and comparison of different streams.

(iv) The chronological sequence of occurrence of the flow is masked in the floe-duration curve.A discharge of say 1000cumec in a stream will have the same percentage Pp whether it has occurred in January or June.This aspect, a serious handicap, must be kept in mind while interpreting a flow-duration curve.

(v) The flow-duration curve plotted on alog-log paper is useful in comparing the flow characteristics of different streams.A steep slope of curve indicates a stream with a highly variable discharge. On the other hand, a flat slope indicates a slow response of the catchment to the rainfall and also indicates small variability.At lower end of the curve,a flat portion indicates considerable base flow. A flat curve on the upper portion is typical of river basins having large flood plains and also of rivers having large snowfall during a wet season.

USES OF FLOW-DURATION CURVES

(i) Evaluating various dependable flows in the planning of water resources engineering projects.

(ii) Evaluating the characteristics of the hydropower potential of a river.

(iii) Designing of drainage system

(iv) Conducting flood control studies

(v) Computing the sediment load and dissolved solids load of a stream.

(vi) Comparison of the adjacent catchments with a view to extend the stream flow data.

FLOW-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 $V=\int_{to}^{t} Q\, dt$

where t_o is the time at the beginning of the curve and Q is the discharge rate.

DROUGHTS

Drought is a climatic anomaly characterized by deficit supply of moisture resulting from either sub-normal rainfall, irratic rainfall distibution or more demand of water from crops.Further, during droughts the quality of available water is generally highly degraded resulting in serious environmental and health problems.Drought is classified as (i) Meteorological drought, (ii) Hydrological drought, (iii)Agricultural drought.

The causes of drought are essentially due to temporal and spatial aberrationsin the rainfall, improper management of available water and lack of conservation of runoff in surface and sub-surface storages.Strategies for making drought prone areas less vulnerable to drought are as follows:

(i) Creation of storages throughminor, medium and major projects.

(ii) Inter-basin transfer of waters from surplus water areas to drought prone areas.

(iii)Development of ground water potential.

(iv) Development of appropriate water harvesting practices.

(v) Soil moisture conservation measures.

(vi) Minimization of evaporation losses from water bodies.

(vii) Encouraging pastures, forestry and other modes of development which is less water demanding.

FACTORS AFFECTING HYDROGRAPH:

Physiographic Factors	**Climatic Factors**
1. Basin characteristics :Shape, Size,Slope, Nature of valley, Elevation,Drainage density	1.Storm characteristics:Precipitation,Intensity,Duration, Magnitude and movement of storm

2.Infiltration characteristics: Land use and cover,soil type and geologicalconditions,lakes,swamp and other storages	2.Initial losses
3.Channel characteristic:cross –section,roughness and storage capacity	3.Evapotranspiration

UNIT HYDROGRAPH & ITS ANALYSES

The hydrograph of outflows from a small basin is the sum of the elemental hydrographs from all the subareas of the basin modified by the effect of transit time through the basin and storage in the stream channel.Since the physical characteristics of the basin-shape, size, slope, etc.-are constant, one might expect considerable similarity in the shape of hydrographs from storms of similar rainfall characteristics. This is the essence and concept of unit hydrograph introduced by Sherman in 1932 as a tool for estimating hydrograph shape.The unit hydrograph is a typical hydrograph for the basin.It is called unit hydrograph because, for convenience, the runoff volume under the hydrograph is commonly adjusted to 1 cm equivalent depth over the catchment.

It would be wrong to imply that one typical hydrograph would suffice for any basin.Although the physical characteristics of the basin remain comparatively constant,the variable characteristics of storms such as duration of rainfall,time-intensity pattern,areal distribution of rainfall, and amount of rainfall cause variations in the shape of the resulting hydrograph.

A unit hydrograph is defined as the hydrograph of direct runoff resulting from one unit depth (1 cm, or 1 mm, or 1 inch) of rainfall excess occuring uniformly over the basin and at a uniform rate for a specified duration**.** The duration assigned to a unit hydrograph should be the duration of rainfall producing significant runoff, determined by inspection of hourly rainfall data.

Basic Assumptions:

(i) **Time invariance:** The direct –runoff response to a given effective rainfall in a catchment is a time-invariant.It implies that the DRH for a given effective rainfall in a catchment is always the same irrespective of when it occurs.

(ii) Linear response: The direct-runoff response to the rainfall excess is assumed to be linear i.e. ordinates of flow are proportional to volume of runoff for all storms of a given duration.

(iii) Time Base: For a storm of same duration but with a different amount of runoff, the hydrograph of direct runoff is assumed to have the same time base as the unit hydrograph.

Derivation of Unit Hydrograph:

The unit hydrograph is best derived from the hydrograph of a storm of reasonably uniform intensity, duration of desired length, and a relatively large runoff volume. The first step is to separate the base flow from direct runoff.The volume of direct runoff is the determined, and the ordinates of the direct runoff hydrograph are divided by observed runoff depth.The adjusted ordinates form a unit hydrograph.

A unit hydrograph derived from a single storm may not be representative, and it is therefore desirable to average unit hydrographs from several storms of about the same duration.Compute the average peak flow and time to peak and unit hydrograph is then sketched to conform to the shape of other graphs, passing through the computed average peak, and having the required unit volume.

CONVERSION OF UNIT HYDROGRAPH DURATION

Method of Superposition

There is frequently a need to convert an existing unit hydrograph for one storm duration to another-shorter to better cope with spatial and intensity variations, or longer to reduce required computations and possibly in recognition of the coarseness of the available data.If two t_R- hr (duration) unit hydrographs, one lagged by t_R hr with respect to the other, are added, the result is the characteristic hydrograph for 2 units of rainfall excess and 2.t_R-hr duration. Dividing the ordinates by 2 yields the 2.t_R-hr unit hydrographs.In other words, the n. t_R-hr unit hydrograph is the average of n t_R-hr unit hydrograph, each lagged t_R hr with respect to the previous one.

S-Curve or Summation Curve Method

The S- curve is the hydrograph that would result from an infinite series of unit runoff increments.Each S-curve applies to a specific duration within which each unit of runoff is generated.The S-curve is constructed by adding together a series of unit hydrographs, each lagged tR hr with respect to the preceding one. If the time base of the unit hydrograph is T hr, then a continuous rainfall producing one unit of runoff every period would develop a constant outflow at the end of T hr.Thus only T/t_R unit hydrographs need be combined to produce an S curve which should reach equilibrium at flow

$qe=2.78A/t_R$ where qe is in cumec, A is drainage area in sq. km., the runoff in cm and t_R is again the unit duration.Commonly, the S curve tends to fluctuate about equilibrium flow.This means that the initial unit hydrograph does not actually represent runoff at a uniform rate over time tR. Thus Scurve serves as an approximate check on the assumed duration of effective rainfall for the unit hydrograph.

The difference between two S curves with initial points displaced by t'_R hr gives a hydrograph for the new duration t'_R. Since the S curve represents runoff production at a rate of one unit in t_R hr, the runoff volume represented by this new hydrograph will be t'_R/t_R units.Thus the ordinates of the unit hydrograph for t'_R hr are computed by multiplying the S curve differences by the ratio t_R/t'_R.

SYNTHETIC UNIT HYDROGRAPH

Synthetic unit hydrographs are derived for ungauged catchements.This rquires a relation between the physical geometry of the area and the resulting hydrographs.Three approaches have been used:

(i)Formulas relating hydrograph features to basin characteristics (time of peak, peak flow, and time base of the unit hydrograph), (ii) Transposition of unit hydrographs, (iii) Storage Routing.

SNYDER'S METHOD

Snyder developed a set of empirical equations for synthetic unit hydrographs in 1938 based on a study of a large number of catchments in the Appalanchian Highlands in eastern USA.Basin lag (time interval from mid-point of unit rainfall excess to the peak of the unit hydrograph),$t_p=C_t(LL_{ca})^{0.3}$ where t_p is in hours, L=basin length measured along the river from the basin divide to the gauging stationin km, L_{ca}= distance along the main water course from the gauging station to a point opposite to the watershed centroid in km,C_t=a regional constant representing watershed slope and storage.According to Snyder its value ranged from 1.35 to 1.65 but studies elsewhere indicate the variation as 0.3 to 6.0.Linsley et al found that the basin lag tp is better correlated with the catchment parameter $[LL_{ca}/\sqrt{S}]$ where S= basin slope.Hence the modified formula is $tp=C_{tL}[LL_{ca}/\sqrt{S}]^n$ where C_{tL} and n are basin constants. Value of n was found to be equal to 0.38.Values of CtL were 1.715 for mountainous drainage areas, 1.03 for foot-hill drainage areas and 0.50 for valley drainage areas.

Snyder adopted a standard duration t_r hours of effective rainfall given by $t_r=t_p/5.5$

The peak discharge Q_{ps} in cumec of a unit hydrograph of standard duration t_r hour is given by Snyder as $Q_{ps}=2.78C_p\ A/t_p$

Where A=catchment area in sq.km. and C_p=a regional constant whose value ranges from 0.56 to 0.69 for Snyder's study area and is considered as an indicator of the retention and storage capacity of the watershed.Like C_t, the value of C_p also vary quite considerably depending on the characteristics of the region and the vary from 0.31 to 0.93.

Snyder adopted as the time base of the unit hydrograph (days), $T=3+3t_p/24 = (72+3t_p)$ hours. It gives reasonable estimates of time base for large catchments, but may give excessively large values for small catchments.

The constants adopted here are dependant on the procedure used to separate base flow from direct runoff.The above equations define the three factors necessary to construct the unit hydrograph for duration t_r, which Snyder took as $0.18t_p$. For any other duration t_R he used an adjusted lag

$t_{pR}=t_p+(t_R-t_r)/4=(21/22)t_p+t_R/4$

The best way to apply such methods is to derive coefficients from gaged streams in the vicinity of the problem basin and use these for the ungaged stream, i.e. transposition of unit hydrographs.

To assist in the sketching of unit hydrographs, the widths of unit hydrographs in hours at 50% and 75% of peak have been found for US catchments by US Army Corps of Engineers. These are correlated to the peak discharge intensity and are given by $W_{50}=5.87/q^{1.08}$ and $W_{75}= W_{50}/1.75$.

After obtaining the values of Q_p, t_R, t'_p, W_{75}, W_{50} and T from Snyder's equations, a tentative unit hydrograph is sketched. An S-curve is then developed and plotted.As ordinates of the unit hydrograph are tentative, the S-curve thus obtained will have kinks.These are then smoothened and a logical pattern of the S-curve is sketched.Using this S-curve the tR hour unit hydrograph is then derived back.Further, the area under the unit hydrograph is checked to see that it represents 1 cm of runoff.The procedure of adjustment through the S-curve is repeated till satisfactory results are obtained.It may be noted that out of the various parameters of the synthetic unit hydrograph the least accurate will be the time base and this can be changed to meet other requirements.

TRANSPOSITION OF UNIT HYDROGRAPHS

One technique for transposition of unit hydrographs makes use of a dimensionless unit hydrograph which masks the effect of basin size and essentially eliminates the effect of shape,except as they are reflected in the estimate of basin lag and runoff volume.In this the ordinate is the discharge expressed as a ratio to peak discharge (Q/Qp) and the abscissa is the time expressed as a ratio of time to peak (t/tpk).

INDIAN PRACTICE

The Central Water Commission after a study of a large number of catchments in India of varying sizes in the range of 25 to 500 sq.km. has recommended the following relations for developing synthetic unit hydrographs.The peak discharge of a D-hour unit hydrograph Q_{pd} in cumec is

Q_{pd}=4.44A $^{3/4}$ for S_m>0.0028

And Q_{pd}=222A $^{3/4}$ S_m $^{2/3}$ for Sm<0.0028

Where A=catchment area in sq. km. and S_m=weighted mean slope given by

$S_m=[L_{ca}/(L_1/\sqrt{S_1}+L_2/\sqrt{S2}+\ldots\ldots.+L_n/\sqrt{Sn})]^2$ in whichLca =distance along the river from the gaging station to a point opposite the centre of gravity of the area.L1,L2…….Ln=lengthof main channel having slopes S1,S2……Sn respectively obtained from topographic maps.

The lag time in hours (i.e. time interval from mid-point of the rainfall excess to the peak of 1 –hour unit hydrograph is given by

$T_{p1}=3.95/[Q_{pd}/A]^{0.9}$

For design purposes the duration of rainfall excess in hours is taken as D=1.1 t_{p1}

The time to peak is determined by Snyder's equation, $t_p=C_t(LL_{ca})^{0.3}$

INSTANTANEOUS UNIT HYDROGRAPH

Any unit hydrograph has a particular D-hour duration.The shape of different unit hydrographs depend upon the particular value of D.As D is reduced, the intensity of rainfall excess being equal to 1/D increases and the unit hydrograph becomes more skewed.

As the duration of the unit hydrograph approaches zero, the instantaneous unit hydrograph by definition, the flow sequence represents the outflow from the instantaneous application of the unit rainfall excess over the catchment.IUH is a fictitious , coceptual unit hydrograph which represents the surface runoff from the catchment due to an instantaneous precipitation of the rainfall excess volume of 1 cm.

Mathematically, the rate of direct runoff at time t is given by

$q_t=\int_0^t f(\tau)i_e(t-\tau)d\tau$

in which $f(\tau)$is the IUH ordinate at time τ, i_e is the intensity of rainfall excess at time $(t-\tau)$, and τ is time in the past. In other words, the flow is determined by weighing the antecedent rainfall excess, where the weight applied to the rainfall occuring τ hr ago is IUH ordinate τ hr after the beginning of rainfall.

IUH can be derived from an S-curve or by routing the time –area diagram of the catchment.

UNIT-4

Flood: Rational method, empirical formulae, unit hydrograph method, flood frequency studies, statistical analysis, regional flood frequency analysis, design storm & design flood, risk/reliability and safety factor; Flood Routing: Basic equation, hydrologic storage routing & attenuation, hydrologic channel routing, flood forecasting & control, hydraulic method of flood routing.

FLOOD

A flood is an unusually high stage in a river – normally the level at which the river overflows its banks and inundates the adjoining areas.At a given location in a stream, flood peaks vary from year to year and their magnitude constitutes hydrologic series which enable one to assign a frequency to a given flood-peak value.The magnitude of flood peaks can be estimated by following alternative methods: (i)Rational method, (ii) Empirical fomulae, (iii) Unit hudrograph method, (iv) Flood-Frequency studies.

RATIONAL METHOD

The Rational formula is applicable to small-sized (<50 sq.km.) catchments only and is generally used for the design of storm water drains and other structures like small culverts and bridges handling runoff from small areas.The peak value of runoff is given by the equation:

Q_p=(1/3.6)CiA for $t>t_c$

Where Qp=peak discharge in cumec,C=coefficient of runoff Runoff/Rainfall, i=mean intensity of precipitation (mm/hour) for a duration equal to t_c and an exceedence probability P, A=area of the catchment in sq.km. t_c=time of concentration i.e. time taken for a drop of water from the farthest part of the catchment to reach the outlet.

It may be noted that the word Rational is rather a misnomer as the method involves determination of parameters t_c and C in a subjective manner.The three parameters:t_c,,I, and C are described below:

Time of Concentration,t_c

For small drainage basins, the practice in USA is to consider the time of concentration approximately equal to the lag time of peak flow,$t_c= t_p=C_{tL}[LL_{ca}/\sqrt{S}]^n$

For small plots having no defined flow channels, and from which runoff occurs as laminar overland flow,Izzard has suggested overland flow time or inlet time ,$T_o=[111.b.(L_o)^{1/3}]/(K.p)^{2/3}$ whereL_o=Length of overland flow in meters,p=Rainfall intensity in cm/hour, K=Runoff coefficient, b= a coefficient, whose value is given by $b=(0.000275p+C_r)/(S_o)^{1/3}$ whereSo=slope of the surface,Cr=Retardance coefficient(0.007 for smooth asphalt surface,0.012 for concrete pavement, 0.017 for tar and gravel pavement,0.046 for closely clipped soil, and 0.060 for dense blue grass turf).

However, for the design of hydraulic structures, Kirpich devised an equation in 1940 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 in meters,S= slope of the catchment=ΔH/L in which ΔH= difference in elevation between the most remote point on the catchment and the outlet.

For a small drainage basin having flow channels in it, would be equal to the longest combination of overland flow time called Inlet time and Channel flow time which exists anywhere in the basin.Channel flow time is generally taken as the length of the longest channel divided by the average flow velocity in the channel at about bankfull stage.

Critical Rainfall Intensity, i

The critical rainfall intensity in cm/hour is generally taken as the average rate duringthe time of concentration tc and the desired probability of exceedence P (i.e. return period T=1/P) and is determined from the rainfall- frequency–duration relationship for the given catchment atrea. This will usually be a relationship in the form of equation $i=KT^x/(t_c+a)^m$ in which K,a,x and m are constants.Published rainfall maps exibiting this form of relationship are consulted to evaluate i. In USA P is generally taken as 0.05 to 0.10.

Runoff Coefficient, C

The coefficient, C represents the integrated effect of the catchment losses and hence depends upon the nature of the surface, surface slope and rainfall intensity. Some typical values of C are given below:for

Sr. No.	Type of Area	Value of C
A.	Urban area (P=0.05 to 0.10)	
	Lawns: Sandy soil, flat,2%	0.05- 0.10
	Sandy soil,steep,7%	0.15-0.20

	Heavy soil, average, 2.7%	0.18-0.22
	Residential areas:	
	Single family areas	0.30-0.50
	Multy units, attached	0.60-0.75
	Industrial areas:	
	Light	0.50-0.80
	Heavy	0.60-0.90
	Streets	0.70-0.95
B.	Agricultural Areas	
	Flat: Tight clay;cultivated	0.50
	Woodland	0.40
	Sandy loam;cultivated	0.20
	Woodland	0.10
	Hilly: Tight clay;cultivated	0.70
	Woodland	0.60
	Sandy loam;cultivated	0.40
	Woodland	0.30

In determining peak flow rates, most applications of the Rational formula utilize the following steps:

(i) Estimate the time of concentration of the drainage area.

(ii) Estimate the runoff coefficient.

(iii) Select a return period,Tr and find the intensity of rain that will be equaled or exceeded, on an average, over every Tr years.

(iv)Determine the peak flood flow from the equation Q_p=CiA.

Some design situations produce larger peak flows if design storm intensities for durations less than tc are used.Substituting intensities for durations lesstc is justified only if the contributing area term in the equation is also reduced to accommodate the shortened storm duration.

The above mentioned Rational formula assumes a homogeneous catchment surface. If however, the catchment area is non-homogeneous but can be divided into distict sub areas each having a different runoff coefficient, then the runoff from each sub area is calculated separately and merged in proper time sequence. Sometimes, a non-homogeneous catchment is so cmplex that distinct subareas can not be separated. In such cases a weighted equivalent runoff coefficient $C_e=\Sigma_1^N C_i A_i /A$ is used where A_i is the areal extent of the sub area i having a runoff coefficient C_i and N is number of sub areas in the catchment.

EMPIRICAL FORMULAE

These are basically regional formule based on statistical correlation of the observed peak and important catchment properties like area.These are applicable only in the region from which they were developed and when applied to other areas they can at best give approximate values.

DICKENS FORMULA (1865)

$Q_p=C_D A^{3/4}$ where Qp = maximum flood discharge in cumec, A= catchment area in sq.km., CD=Dickens constant (6 for north Indian plains, 11-14 for north Indian hilly regions,14-28 for central India, and 22-28 for Coastal Andhra and Orissa).

RYVES FORMULA (1884)

$Q_p=C_R\ A^{2/3}$ where Qp= maximum flood discharge in cumec, A=catchment area in sq.km. and C_R=Ryves constant(6.8 for areas within 80 km from east coast,8.5 for areas which are 80-160 km from east coast,and 10.2 for limited areas near hills)

This formula was originally developed for the Tamil Nadu region and is in use in Tamil Nadu, parts of Karnataka and Andhra Pradesh.

INGLIS FORMULA (1930)

$Q_p=124A/\sqrt{A+10.4}$ where Q_p is flood peak in cumec and A is catchment area in sq.km.This formula is based on flood data of catchments in western Ghats in Maharastra.

FULLER'S FORMULA (1914)

$Q_{Tp}=C_f\ A^{0.8}\ (1+0.8 \log T)$ where Q_{Tp} is maximum 24-hr flood with a frequency of T years in cumec, A is catchment area in sq. km. and Cf is a constant varying from 0.18 to 1.88.

ENVELOPE CURVES

Kanwarsain and Karpove (1967) have developed enveloping curves on log-log graph paper representing the relationship between the peak- flood discharge and the catchment area for north as well as southIndian conditions for large catchment areas in the range of 10^3 to 10^6 sq. km. Based on the maximum recorded floods throughout the world, Baird and McIllwraith (1951) have recommended following equation:

$Q_{mp}=3025A/\ (278+A)^{0.78}$ where Discharge is in cumecs and area in sq. km.

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 appropriate unit hydrograph are available.For design purposes, extreme rainfall situations are used to obtain the design storm.The known or derived unit hydrograph of the catchment is then operated upon by the design storm to generate the desired flood hydrograph.

FLOOD-FREQUENCY STUDIES

The values of the annual maximum flood from a given catchment area for large number of successive years constitute a hydrological data series called the annual series.The data are then arranged in decreasing order of magnitude and the probability P of each event being equalled to or exceeded is calculated by the plotting-position formula

P=m/N+1 where m=order number of the event and N=total number of events in the data.The recurrence interval, T (also called the return period or frequency) is calculated as T=1/P.

Chow (1951) has shown that most frequency distribution functions applicable in hydrologic studies can be expressed by the following equation known as general equation of hydrologic frequency analysis: $x_T = x' + K\sigma$ where x_T=value of the variate x of a randum hydrological series with a return period T, x' =mean of the variate, σ=standard deviation, K= frequency factor which depends upon the return period, T and the assumed frequency distribution.

STATISTICAL ANALYSIS

Statistical distributions are usually demonstrated by use of samples numbering in thousands.No such samples are available for streamflow and it is not possible to state with certainty that a specific distribution applies to flood peaks.Numerous distributions have been suggested on the basis of their ability to fit the plotted data from the streams.Despite much efforts, tests that there is no best distribution for floods. Intuitively atleast, there is no reason to expect that a single distribution will apply to all streams worldwide. Commonly used frequency distribution functions for the prediction of extreme flood values are: (i) Gumbel's extreme-value distribution, (ii) Log-Pearson type III distribution, (iii) Log normal distribution.

GUMBEL'S METHOD

Gumbel was the first to suggest the use of the extreme-value distribution for floods. His argument was that each year of record constituted a smple of a sample of 365 values and the annual flood was the maximum value from the sample.He defined the flood as the largest of the 365 daily flows and the annual series of flood flows constituting a series of the largest values of flows.According to his theory of extreme events, the probability of occurrence of an event equal to or larger than a value x_o is

$P(X\geq x_o) = 1-e^{-[e]^{-y}}$ in which y is a dimensionless variable given by $y=\alpha(x-a)$, $a=x'-0.45005\sigma_x$ and $\alpha=1.2825/\sigma_x$

Thus $y=[1.285(x-x') / \sigma_x] +0.577$ where x'=mean and σ_x=standard deviation of the variate X.In practice it is the value of X for a given P that is required and the probability equation is transposed as

$Y=-\ln[-\ln(1-P)]$

Noting that the return period T=1/P and designating y_T=the value of y, commonly called the reduced variate, for a given T

y_T=-[ln.lnT/T-1]=-[0.834+2.303log log T/ (T-1)].The value of the variate X with a return period T is

$x_T = x' + K\sigma_x$ where $K=(y_T-0.577)/1.2825$

For practical use y_T=-[0.834+2.303 log log T/ (T-1)] and $K=y_T-y_n/S_n$ where yn=reduced mean for $N\to\infty$ $y_{n\to}0.577$ and S_n=reduced standard deviation for $N\to\infty$ $S_{n\to}1.2825$.

These equations are used under the following procedure to estimate the flood magnitude corresponding to a given return period based on an annual flood series.

1. Assemble the discharge data and note the sample size N. Here the annual flood value is the variance X. Find x' and σ_{n-1} for the given data.

2. Determine y_n and S_n.

3. Find y_T for a given T

4. Find $K= y_T-y_n/S_n$

5. Determine the required x_T by equation $x_T=x'+K\sigma_{n-1}$

To verify whether the given data follow the assumed Gumbel's distribution, the following procedure may be adopted:

The value of x_T for some return periods T<N are calculated by using Gumbel's formula and plotted as a semi-log, log-log, or Gumbel probability paper. The use of Gumbel probability paper results in a

straight line for x_T vs T plot. 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.In graphical plot this gives a mandatory point through which the line showing variation of x_T with T must pass. For the given data, values of return periods for various recorded values, x of the variate are obtained by the relation T= (N+1)/m and plotted on the graph described above.

LOG-PEARSON TYPEIII DISTRIBUTION

This distribution is extensively used in USA. In this the variate is first transformed into logarithmic base 10 and the transformed data is then analysed.If X is the variate of a random hydrologic series, then the series of Z variates where z=log x are first obtained.For this Zseries, for any recurrence interval T the general equation of hydrologic frequency analysis : $x_T = x' + K\sigma$ gives $z_T = z' + K_z\ \sigma_z$ where K_z =a frequency factor which is a function of recurrence interval T and the coefficient of skew C_s, and σ_z =standard deviation of the Z variate sample=$\sqrt{\sum(z - z')^2 /(N-1)}$ and Cs=coefficient of skew of variate Z=$[N\sum(z - z')^3]/(N-1)(N-2)(\sigma_z)^3$

The variations of K_z=f(Cs,T) can be obtained from table.After finding z_T, the corresponding x_T is obtained by equation z=log x as x_T=antilog(z_T).

When the skew is zero i.e. Cs=0, the log-pearson TypeIII distribution reduces to log normal distribution which plots as a straight line on logarithmic probability paper.

REGIONAL FLOOD FREQUENCY ANALYSIS

When the available data at a catchment is too short to conduct frequency analysis, a regional analysis is adopted. In this a hydrologically homogenous region from the statistical point of view is considered. Available long time data from neighbouring catchments are tested for homogeneity and a group of stations satisfying the test are identified. This group of stations constitutes a region and all the station data of this region are pooled and analysed as a group to find the frequency characteristics of the region.The mean annual flood Q_{ma}, which corresponds to a recurrence interval of 2.33 years is used for non-dimensionalising the results. The variation of Q_{ma} with drainage area and the variation of Q_T/Q_{ma} with T where Q_T is the discharge for any T, are the basic plots prepared in this analysis.An independent relation defining the mean annual flood in terms of basin characteristics is the required.Often this is a simple relation brtween mean annual flood and drainage area, although other parameters such as basin slope, mean elevation, percentage of area of area in lakes, and mean precipitation over the catchment may be employed.

A probability curve for an ungauged basin is then constructed by first determining the mean annual flood and then converting the ratio scale of the regional –frequency curve to flow by multiplying the ratios by the estimated meam annual flood. The weakness of this approach lies in the assumption that all the streams will show the same variance of Q_T/Q_{ma} (slope of the cummulative frequency function).Statistical tests may indicate that several streams could have the same function,but they can never prove the functions are identical.Differences between frequency curves of small streams may be real differences arising from different regimes.They may, on the other hand be apparent differences arising from the variability of short samples.

If the regional approach is used, considerable care should be taken to select streams as nearly similar in hydrologic characteriticsas possible.They should have similar vegetal cover, land use, topographic conditions,and geologic characteristics, and the range of the drainage area should not be excessive.They should also have similar rainfall and evapotranspiration characteritics.

DESIGN STORM

To estimate the design flood for a project by the use of a unit hydrograph, one needs the design storm. This can be the storm-producing PMP for deriving PMF or a standard project storm (SPS) for calculation of standard project flood (SPF).The following procedure is adopted in India:

(i) The duration of the critical rainfall is first selected. This will be the basin lag if the flood peak is of interest.If the flood volume is of interest, the duration of the longest storm experienced in the basin is selected.

(ii)Past major storms in the region which conceivably could have ocurredin the basin under study are selected. DAD analysis is performed and the enveloping curve representing the maximum depth-duration relation for the study basin is obtained.

(iii) Rainfall depth for convenient time intervals are scaled from the enveloping curve.These increments are to be arranged to get a critical sequence which produces the maximum flood peak when applied to the relevant unit hydrograph of the basin.The critical sequence of rainfall increments can be obtained by trial and error.Alternatively, increments of precipitation are first arranged in a table of unit hydrogrph ordinates, such that (a) the maximum rainfall increment is against the maximum unit hydrograph ordinate,(b) the second highest rainfall increment is against the second highest unit hydrograph ordinate and so on, and (c) the sequence of rainfall increments arranged above is now reversed, with the last item first and first item last.The new sequence gives the design storm.

(iv) The design storm is then combined with hydrologic abstractions most conducive to high runoff, viz. low initial loss and lowest infiltration rate to get the hyetograph of rainfall excess to operate uponthe unit hydrograph.

DESIGN FLOOD

The PMF is used in situations where the failure of 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 structure would cause less severe damage.Typically the SPF is about 40% to60% of the PMF for the same drainage area.The CWC has recommended as follows:

Sr. No.	Structure	Recommended Design Flood
1.	Spillways for major and medium projects with storages more than 60MCM	(a) PMF determined by unit hydrograph and PMP (b) If (a) is not applicable or possible ,adopt Flood-Frequency method with T=1000years
2.	Permanent Barrages and minor dams with capacity less than 60MCM	(a) SPF determined by unit hydrograph and SPS which is usually the largest recorded storm in the region (b) Flood with a return period of 100 years Adopt (a) or (b) whichever gives higher value.
3.	Pick-up Weirs	Flood with a return period of 100 or 50 years depending on the importance of the project.
4.	Aqueducts	Flood with T=50 years for waterways and Flood with T=100years for foundations and freeboard

5.	Project with very scanty or inadequate data	Empirical Formulae

IS:11223-1985: Guidelines for fixing spillway capacity have made following recommendations:

Class	Gross Storage(MCM)	Hydraulic Head (m)	Inflow Design Flood
Small	0.5 to 10.0	7.5 to 12.0	100-year flood
Intermediate	10.0 to 60.0	12.0 to 30.0	SPF
Large	>60.0	>30.0	PMF

RISK, RELIABILITY & SAFETY FACTOR

The designer of a hydraulic structure always faces a nagging doubt about the risk of failure of his structure.This is because the estimation of the hydraulic design values, (such as the design flood discharge and the river stage during the design flood) involve a natural or inbuilt uncertainty and as such a hydrological risk of failure. As an example, consider a weir with an expected life of 50 years and designed for a flood magnitude of return period T=100 years.This weir may fail if a flood magnitude greater than the design flood occurs within the life period (50years) of the weir.

The probabilty of occurence 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 $R.'$ =1-(probability of non-occurrence of the event $x \geq x_T$ in years) =1-$(1-P)^n$ =1-$[1-1/T]^n$ whereP=probability P($x \geq x_T$)=1/T and T=return period

The reliability Re is defined as Re= 1-$R.'$ = $[1-1/T]^n$

It can be seen that the return period for which a structure should be designed depends upon the acceptable level of risk. In practice, the acceptable risk is governed by economic and policy considerations.

SAFETY FACTOR

In addition to the hydrological uncertainty, as mentioned above, a water resource development project will have many other uncertainties.These may arise out of structural, constructional, operational and environmental causes as well as from non-technological considerations such as economic, sociological and political causes.As such, any water resource development project will have safety factor for a given hydrological parameter M as defined below:

$(SF)_m$=(actual value of parameter adopted in the design of the project)/(Value of the parameter M obtained from hydrological considerations only)=C_{am}/C_{hm}

The parameter M includes such items as flood discharge magnitude, maximum river stage, reservoir capacity and free board.The difference (C_{am}-C_{hm}) is known as safety margin.The concept of risk, reliability and safety factor form the building blocks of the emerging field of reliabilty based design.

FLOOD ROUTING

Flood routing is a process of predicting the temporal and spatial variations of a hydrograph as the flood wave traverses a river reach or reservoir.Giventhe flow at an upstream point,routing can be used to compute the flow at a downstream point.The principle of routing apply also to computation of the effect of a reservoir on the shape of a flood wave.

TYPES OF FLOOD ROUTING

(i) Reservoir Routing: In reservoir routing the effect of a flood wave entering a reservoir is studied.Knowing the volume-elevation characteristics of the reservoir and the outflow-elavation relationship for the spillways and other outlet structures in the reservoir, the effect of a flood wave entering a reservoir is studied to predict the variations of reservoir elevation and outflow discharge with time.This type of reservoir routing is essential 9i) in the design of capacity of spillways and other reservoir outlet structures, and (ii) in the location and sizing of capacity of reservoirs to meet specific requirements.

(ii) Channel Routing: In this type the change in the shape of a hydrograph as it travels down a channel is studied. By cosidering a channel reach and an input hydrograph at the upstream end,this form of routing aims to predict the flood hydrograph at various sections of the reach.Information on the flood-peak attenuation and the duration of the highwater levels obtained by channel routing is of utmost importance in flood forecasting operations and flood protection works.

METHODS OF FLOOD ROUTING

(i)Hydological Routing: Flood wave movement in natural channels traditionally has been treated in design and prediction by applying hydrologic routing procedures.Such procedures solve the continuity equation (or storage equation) for an extended reach of the river, usually bounded by selected gauged points.

(ii)Hydraulic Routing: Routing of unusual events such as a reversal of flow, complex channel systems as in a delta,flood waves due to failure of dam,or by the probable maximumflood and situations with variable backwater are the special cases where hydrologic routing methods may not suffice.Hydraulic routing based on a solution of the energy or momentum equations is an alternate to the hydrologic methods.This method deal directly with the hydraulic characteristics of the channel and, in some cases, also take dynamic effects into account.

BASIC EQUATIONS

The passage of a flood hydrograph through a reservoir or a channel reach can be classified as gradually varied unsteady flow.The equation of continuety used in all hydrologic routing as the primary equation states that the difference between the inflow and outflow rates is equal to the rate of change of storage, i.e. $I-O=dS/dt$ Or $\Delta S=S_2-S_1=\int_{t1}^{t2} Idt - \int_{t1}^{t2} Odt$

Alternatively, to provide a form more convenient for hydrologic routing, it is commonly assumed that in a small time interval of routing period Δt the difference between the total inflow volume and total outflow volume in a reach is equal to the change in storage in that reach$I'\Delta t-O'\Delta t=\Delta S$, where I'= average inflow in time Δt, O'=average outflow in time Δt and ΔS=change in storage.By taking $I'=(I_1+I_2)/2$,$O'=(O_1+O_2)/2$ and $\Delta S=S_2-S_1$ with suffixes 1 and 2 to denote the beginning and end of time interval Δt,an equation can be expressed as $[(I_1+I_2)/2]\Delta t-[(O_1+O_2)/2]\ \Delta t=S_2-S_1.$

The time interval Δt should be sufficiently short so that inflow and outflow hydrographs can be assumed to be straight lines in that time interval.Further Δt must be shorter than the time of transit of the flood wave through the reach.The routing period should never be greater than the time of travel through the reach, for if it were, it would be possible for the wave crest to pass completely through the reach during a routing period.Generally a routing period between ½ to1/3 of the time of travel will work quite well.

In the differential form, the equation of continuity for unsteady flow in a reach with no lateral flow is given by $\frac{\partial Q}{\partial x}$ +T $\frac{\partial y}{\partial t}$ =0 where T=top width of the section and y=depth of flow

The equation of motion for a flood wave is derived from the application of the momentum equation as

$\frac{\partial y}{\partial x} + \frac{V}{g}\frac{\partial V}{\partial x} + \frac{1}{g}\frac{\partial V}{\partial t}=S_0-S_f$ where V=velocity of flow at any section, S_0=channel bed slope and S_f= slope of energy line.These above two equations were first developed by A.J.C.Barre de Saint Venant and therefore are commonly known as St.Venant equations.Hydraulic-flood routing involves the numerical solution of St.Venant equations.

HYDROLOGIC STORAGE ROUTING

A flood wave I(t) enters a reservoir provided with an outlet such as a spillway.The outflow is a function of the reservoir elevation only,i.e.Q=Q(h).The storage in the reservoir is a function of reservoir elevation ,S=S(h).Further,due to the passage of the flood wave through the reservoir,the water level in the reservoir changes with time, h=h(t) and hence the storage and discharge change with time.It is required to find the variations of S,h and Q with time,i.e. find S=S(t),Q=Q(t) and h=h(t) given I=I(t).

If an uncontrolledspillway is providedin a reservoir, typically $Q=\frac{2}{3}C_{d\sqrt{2g}}\ L_e\ H^{3/2}$=Q (h) where H=head over spillway, L_e= effective length of spillway crest and C_d=coefficient of discharge.

For reservoir routing, the following data have to be known:

(i) Storage volume vs elevation for the reservoir;

(ii) Water-surface elevation vs outflow and hence storage vs outflow discharge;

(iii) Inflow hydrograph, $I=I(t)$; and

(iv) Initial values of S, I and Q at time $t=0$.

As the horizontal water surface is used in the reservoir, the storage routing is also known as Level Pool Routing. Two commonly used semi-graphical methods and a numerical method are described below:

MODIFIED PUL'S METHOD

The equation $[(I_1+I_2)/2]\ \Delta t-[(O_1+O_2)/2]\ \Delta t=S_2-S_1$ can be re-arranged as

$[[(I_1+I_2)/2]\ \Delta t\ +[S_1-O_1\Delta t/2] = [S_2+O_2\Delta t/2]$. At the start of flood routing, the initial storage and outflow discharges are known. In the above equation all the terms in the left hand side are known at the beginning of a time step Δt. Hence the value of the function $[S_2+O_2\Delta t/2]$ at the end of the time step can be calculated by above equation. Since the relation $S=S(h)$ and $Q=Q(h)$ are known, $[S+O\Delta t/2]_2$ will enable one to determine the reservoir elevation and hence the discharge at the end of the time step. The procedure is repeated to cover the full inflow hydrograph.

GOODRICH METHOD

The equation $[(I_1+I_2)/2]\ \Delta t-[(O_1+O_2)/2]\ \Delta t=S_2-S_1$ can be re-arranged as

$I_1+I_2+ (2S_1/t-O_1) =2S_2/t+O_2$

The solution of above equation requires a routing curve showing (2S/t+O) versus O. All the terms on left hand side of the equation are known, and a value of $(2S_2/t+O_2)$ can be computedThe corresponding values of O_2 can be determined from the routing curve.The computation is then repeated for succeding routing periods.It may be noted that (2S/t-O) is easily computed as (2S/t+O)-2O.

STANDARD FOURTH-ORDER RUNGE-KUTTA (SRK) METHOD

dS=A (H).dH where S=storage at a water surface elevation H in the reservoir=S(H)

A=area of the reservoir at elevationH=function of H=A(H)

O=outflow from the reservoir=function of H=O(H)

By continuity equation

$\frac{dS}{dt}$ =I(t)-O(H)=A(H)$\frac{dH}{dt}$

$\frac{dH}{dt}$=$\frac{I(t)-O(H)}{A(H)}$= Function of (t,H)

If the routing is conducted from the initial condition,(at $t=t_o$ and $I=I_o;O=O_o,H=H_o,S=S_o$) in time steps Δt, the water surface elevation H at (i+1)th step is given by SRK method as

$H_i +1=H_i+\frac{1}{6}(K_1+2K_2+2K_3+K_4)\Delta t$

where $K_1=F(t_i,H_i)$

$K_2=F[(t_i)+\frac{\Delta t}{2},H_i+\frac{1}{2}K_1\Delta t]$

$K_3= F[(t_i)+\frac{\Delta t}{2},H_i+\frac{1}{2}K_2\Delta t]$

$K_4= F[(t_i)+\Delta t,H_i+K_3\Delta t]$

Starting from the known initial conditions and knowing O vs H and A vs H relationships, a given hydrograph $I=I_t$ is routed by selecting a time step Δt.At any time $t=(t_o+i\Delta t)$, the value of Hi is known and the coefficients K_1, K_2, K_3, K_4 are determined by repeated appropriate evaluation of the function F(t,H).It is seen that SRK method directly determines H_{i+1} by four evaluations of the function F(t,H).Knowing the values of H at various time intervals,i.e.H=H(t), the other variables O(H) and S(H) can be calculated to complete the routing operation.

ATTENUATION

Owing to the storage effect, the peak of outflow hydrograph will be smaller than that of the inflow hydrograph.This reduction in the peak value is called Attenuation.Further, the peak of the outflow occurs after the peak of the inflow;the time difference between the two peaks is known as lag.

HYDROLOGIC CHANNEL ROUTING

Routing in natural river channels is complicated by the fact that storage is not a function of outflow alone.The resulting curve in storage vs simultaneous outflow plot is usually a wide loop indicating greater storage for agiven outflow during rising stages than during falling. The storage beneath a line parallel to the streambed is called prism storage; between this line and the actual profile,wedge storage.During rising stages a considerable volume of wedge storage may exist before any large increase in outflow occurs.During falling stages, inflow drops more rapidly than outflow, and the wedge-storage volume becomes negative.Routing in streams requires a storage relationship which adequately represents the wedge storage.This is usually done by including inflow as a parameter in the storage equation.

ANALYTICAL METHOD OF CHANNEL ROUTING

One expression for storage in a reach of a stream is $S=\frac{b}{a}\left[x\, I^{m/n} + (1-x)\, O^{m/n}\right]$ where a and n are constants from the mean stage–discharge relation for the reach, $q=ag^n$, and b and m are constants in the mean stage-storage relation for the reach,$S=bg^m$.In a uniform rectangular channel,storage would vary with the first power of stage (m=1) and discharge would vary as the $\frac{5}{3}$ power(Manning formula).In natural channel with overbank floodplains the exponent n may approach or become less than unity.The constant x expresses the relative importance of inflow and outflow in determining storage.For a simple reservoir,x=0(inflow has no effect), while if inflow and outflow were equally effective,x would be 0.5. For most streams,x is between 0 and 0.3 with a mean value near 0.2.

MUSKINGUM METHOD

Here '**Muskingum**' is the name of a river in North America on which a related flood routing method was first developed by McCarthy,G.T. vide his paper “The Unit Hydrograph and Flood Routing” presented at conference of North Atlantic Division, U.S.Corps of Engineers in June 1938.Muskingum method assumes that m/n=1 and lets b/a=K. The above general equation then becomes $S=K[x\,I+ (1-x)O]$ where constant K, known as the storage constant, is the ratio of storage to discharge and has the dimension of time.It is approximately equal to the travel time through the reach and in the absence of better data is sometimes estimated in this way.If the flow data on previous floods are available, K andx are determined by plotting S vs [xI+(1-x)O] for various values of x.The best value of x is that which causes the data to plot most nearly as a single valued curve.The Muskingum method assumes that this curve is a straight line with slope K.The units of K depend upon the units of flow and storage.

If equation S=K[x I+ (1-x)O] is substituted for S in equation$[(I_1+I_2)/2]\ \Delta t-[(O_1+O_2)/2]\ \Delta t=S_2-S_1$ and the like terms are collected, the resulting equation reduces to

$O_2=c_oI_2+c_1I_1+c_2O_1$

Where $c_o=-\frac{Kx-0.5t}{K-Kx+0.5t}$, $c_1=\frac{Kx+0.5t}{K-Kx+0.5t}$, $c_2=\frac{K-Kx-0.5t}{K-Kx+0.5t}$

Combining above, $c_o+ c_1+ c_2=1$. In these equations t is the routing period in the same time units as K.With K,x,and t established, values of c_o, c_1,and c_2 can be computed.The routing operation is simply a solution of equation $O_2=c_oI_2+c_1I_1+c_2O_1$with O_2 of one routing period becoming the O_1 of the succeeding period.

FLOOD FORECASTING & CONTROL

Flood control is commonly used to denote all the measures adopted to reduce damages to life and property by floods.Presently many people prefer to use the term flood management instead of flood control as it reflects the activity more realistically.Following measures are adopted for flood mitigation:

1. Structural measures:

Storage and detention reservoirs

Levees(flood embankments)

Flood ways (new channels)

Channel Improvement

Water management

2. Non-structural measures:

Flood plain zoning

Flood forecsting and warning

Evacuation and relocation

Flood insurance

HYDRAULIC METHOD OF FLOOD ROUTING

The hydraulic method of flood routing is essentially a solution of the basic St. Venants equations which are simultaneous, quasi-linear,first order partial differential equations of the hyperbolic type and are not amenable to general analytical solutions.Only for highly simlified cases can one obtain the analytical solution of these equations.The various numerical methods for solving St. Venant equations can be classified broadly into two categories(i)Approximate method, (ii) Comlete numerical methods.

APPROXIMATE METHODS

These are based on the equation of continuity only or on a drastically curtailed equation of motion.The hydrological method of storage routing and Muskingum channel routing belong to this category.Other methods in this category are diffision analogy and kinematic wave models.

COMPLETE NUMERICAL METHODS

These are the essence of the hydraulic method of routing and are classified into many categories as mentioned below each further classified into (i) Implicit method and(ii) Explicit method:

1.Direct method

2.Method of characteristics: (i)Characteristic Nodes,(ii)Rectangular Grid

3.Finite element method

In the direct method,the partial derevatives are replaced by finite differences and the resulting algebric equations are then solved.In the method of characteristics St. Venant equations are converted into a pair of ordinary differential equations and then solved by finite difference techniques.In the finite element method the system is divided into a number of elements and partial differential equations are integrated at the nodal points of the elements.

The numerical schemes are further classified into implicit method and explicit method. In the implicit method the dependent variables occur implicitly and the equtions are non-linear.In the explicit method the algebric equations are linear and the dependent variables are extracted explicitly at the end of each time step.Each of these two methods have a host of finite differenc schemes to choose.

UNIT-5

Groundwater: Introduction, forms of subsurface water, aquifers & its properties, Compressibility of aquifers, flow equations for confined and unconfined aquifers, well hydraulics- steady and unsteady flow to a well in confined aquifer, well losses, specific capacity, ground water irrigation, rain water harvesting.

GROUNDWATER

About 30% of the world's fresh water resources exist in the form of groundwater. It is relatively free of pollution and is especially useful for domestic use in villages and small towns.In arid regions groundwater is often the only reliable source of water for irrigation.Aside from its direct use, groundwater is also an important phase of the hydrologic cycle.Most of the flow of perennial streams originates from subsurface water as base flow, while a large part of the flow of ephemeral streams may percolate beneath the surface.

FORMS OF SUBSURFACE WATER

The two major subsurface zones are divided by an irregular surface called the **water table**.The water table is the locus of points (in unconfined material) where hydrostatic pressure equals atmospheric pressure.Above the water table, in the **vadose zone**, soil pores may contain either air or water; hence it is sometimes called **zone of aeration**.In the **phreatic zone**, below the water table, interstices are filled with water; sometimes this is called the **zone of saturation.**

ZONE OF AERATION

In this zone the soil pores are only partially saurated with water.The space between the land surface and the water table marks the extent of this zone.It has three subzones:

SOIL WATER ZONE: This lies close to the ground surface in the major root band of the vegetation from which the water is lost to the atmosphere by evapotranspiration.

CAPILLARY FRINGE: In this zone the water is held by cappilary action. This zone extends from the water table upwards to the limit of capillary rise.

INTERMEDIATE ZONE: This lies between the soil water zone and the capillary frinze.

The thickness of the zone of aeration and its constituent subzones depend upon the soil structure and moisture content and vary from region to region.If the water table is close to the ground surface, the capillary fringe and the soil moisture region may overlap, but where the water table is deep, an intermediate region exists where moisture levels remain constant at the field capacity of the soil and rock of the region.

SATURATED OR PHREATIC ZONE

This zone, also known as groundwater zone**,** is the space in which all the pores are filled with water.The water table forms its upper limit and marks a free surface i.e. a surface having atmospheric pressure.The saturated formations are classified into four categories:

(i) Aquifer, (ii) Aquitard, (iii) Aquiclude, and (iv) Aquifuge.

AQUIFER

A geologic formation of earth material which not only stores water but transmits it from one point to another and yields it in quantities sufficient to permit economic development is called an aquifer.It transmits water relatively easily due to its high permeability.Unconsolidated deposits of sand and gravel form good aquifers.

AQUITARD

It is a formation through which only seepage is possible and thus the yield is insignificant compared to an aqifer.It is partly permeable. A sandy clay unit is an example of aquitard.Through an aquitard appreciable quntities of water may leak into an aquifer below it.

AQUICLUDE

It is a geological formation which is essentially impermeable to the flow of water, though it may contain large amounts of water due to its high porosity. Clay is an example of an aquiclude.

AQUIFUGE

It is a geological formation which is neither porous nor permeable i.e. it has no interconnected openings and can not hold or transmit water.Massive compact rock without any fractures is an aquifuge.

AQUIFERS & ITS PROPERTIES

The availability of groundwater from an aquifer at a place depends upon the rates of withdrawal and replenishment (recharge).Aquifers play the roles of both a transmission conduit and a storage.Aquifers are classified as unconfined aquifers and confined aquifers on the basis of their occurence and field situation.

UNCONFINED AQUIFERS

An unconfined aquifer (also known as water table aquifer) is one in which a free water surface, i.e. a water table exists.Only the saturated zone of this aquifer is of importance in groundwater studies.Recharge of this aquifer takes place through infiltration of precipitation from the ground surface. A well driven into an unconfined aquifer will indicate a static water level corresponding to the water table at that station.

CONFINED AQUIFER

A confined aquifer, also known as **artesian** aquifer, is an aquifer which is confined between two impervious beds such as aquiclude or aquifuges.Recharge of this aquifer takes place only in the area where it is exposed at the ground surface.The water in the confined aquifer will be under pressure and hence the piezometric level will be much higher than the top level of the aquifer.At some locations, the piezometric level can attain a level higher than the land surface and a well driven into the aquifer at such a location will flow freely without the aid of any pump.In fact,the term **artesian** is derived from the fact that a large number of such freeflow wells were found in **Artois**,a former province in north France.A confined aquifer is called **leaky aquifer** if either or both of its confining beds are aquitards.

PROPERTIES OF AQUIFERS

The important properties of an aquifer are its capacity to release the water held in its pores and its ability to transmit the flow easily.These properties essentially depend upon the composition of the aquifer.

POROSITY: The amount of pore space per unit volume of the aquifer material is called porosity.In an unconsolidated material the size distribution, packing and shape of particles determine the porosity.In quantitative terms porosity greater than 20% is considered as large, between 5 and 20% as medium and less than 5% as small.

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 specific yield,S_y.The fraction of water held back in the aquifer is known as specific retention,S_r.Thus porosity, $n=S_y+S_r$.The representative values of porosity and specific yield of some common earth materials are given below:

Formation	**Porosity,%**	**Specific Yield,%**
Clay	40-55	1-10
Sand	35-40	10-30
Gravel	30-40	15-30
Sand stone	10-20	5-15
Shale	1-10	0.5-5
Lime stone	1-10	0.5-5

It may be seen from above table that although both clay and sand have high porosity the specific yield of clay is very small compared to that of sand.

DARCEY'S LAW: $Q=KiA=KA\,[-\frac{\Delta H}{\Delta s}]$ where $(-\Delta H)$ is the dropin the hydraulic grade line in a length of Δs of the porpus medium.Darcey's law is a particular case of the general viscous fluid flow.It has been shown valid for laminar flows only.For practical purposes, the limit of the validity of Darcey's law can be taken as Reynold's number of unity i.e. $R_e=\frac{Vda}{\nu}=1$whereR_e=Reynold's number,d_a=reprentative particle size, usually $d_a=d_{10}$ where d_{10} represents a size such that 10% of the aquifer material is of smaller size. ν=kinematic viscosity of water. Except for flow in fissures and caverns, to a large extent ground water flow in nature obeys Darcey's law.

It may be noted that the apparent velocity V used in Darcey's law is not the actual velocity of flow through the pores.Owing to irregular pore geometry the actual velocity of flowvaries from point to point and the bulk pore velocity (v_a) which represents actual speed of travel of water in the porous media is expressed as $v_a=\frac{V}{n}$ where n=porosity.The bulk pore velocityva is the velocity that is obtained by tracking a tracer added to the ground water.

COEFFICIENT OF PERMEABILITY: It reflects combined effect of porous medium and fluid properties.$K=K_o\frac{\gamma}{\mu}=K_o\frac{g}{\nu}$ where $K_o=C\ d_m^2$. The parameter K_o is called specific or intrinsic permeability which is a function of medium only.

STRATIFICATION

Sometimes aquifers may be stratified**,** with different permeabilities in each strata. Two kinds of flow situations are possible in each case.

(i) When the flow is parallel to the stratification, the equivalent permeability Ke of the entire aquifer of thickness $B=\sum_1^n Bi$is $K_e=\frac{\sum_1^n KBi}{\sum_1^n Bi}$**.**

The transmissivity of theformation is T=$K_{e\sum Bi}$=$\sum_{1}^{n} KiBi$

(ii) When the flow is normal to the stratification, the equivalent permeability K_e of the aquifer of length

L=$\sum_{1}^{n} Li$ is Ke=$\frac{\sum_{1}^{n} Li}{\sum_{1}^{n}(\frac{Li}{Ki})}$

The tranmissivity of the aquifer is T=K_e.B

COMPRESSIBILITY OF AQUIFERS

In confined aquifers the total pressure at any point due to overburden is borne by the combined action of the pore pressure and intergrannular pressure. The compressibility of the aquifer and also that of the pore pressure causes a readjustment of these pressures whenever there is a change in storage and thus have an important bearing on the storage characteristics of the aquifer.

Consider an elemental volume ∆V= (∆x∆y) ∆Z=∆A ∆Z of a compressible aquifer.Adopt following assumptions: (i) the elemental volume is constrained in lateral directions and undergoes change of length in the z-direction only i.e.∆A is constant,(ii) the pore pressure is compressible, and(iii) the solid grains of the aquifer are incompressible but the pore structure is compressible.

By defining the reciprocal of the bulk modulus of elasticity of water as compressibilty of water β, it is written as β=-$\frac{(d\Delta Vw)}{\Delta Vw}$/dp where ∆Vw=volume of water in the chosen element of the aquifer, and p=pressure.By conservation of mass ρ.∆Vw=constant, where ρ=density of water.

Thus ρ d(∆Vw)+∆VWDρ=0 and β=dρ/(ρdp) or dρ=ρβdp

Similarly by considering the reciprocal of the bulk modulus of elasticity of the pore–space skeleton as the compressibility of the pores,α ,is expressed as $\alpha = \frac{d\frac{(\Delta V)}{\Delta}V}{d6z}$ in which бz=intergrannular pressure.

Since ΔV=ΔA.ΔZ with ΔA=constant, $\alpha = \frac{-d\frac{(\Delta Z)}{\Delta}Z}{d6z}$

The total overburden pressure w=p+бz=constant.

Thus dp=-dбz,which when substituted in above equation gives d(ΔZ)=α(ΔZ)dp

As the volume of solids ΔVs in the elemental volume is constant,ΔVs=(1-n)ΔA=constant

d(ΔVs)=(1-n)d(ΔZ)-ΔZ.dn=0 where n=porosity of the aquifer.

dn=α(1-n)dp

Now, the mass of water in the element of volumeΔV,is ΔM=ρn ΔA ΔZ

Or d(ΔM)=ΔV[n dρ+ρ dn+ρn$\frac{d(\Delta Z)}{\Delta Z}$]

$\frac{d(\Delta M)}{\rho\Delta V}$=n$\frac{d\rho}{\rho}$+dn+n $\Delta Z\ d$

$\frac{d(\Delta M)}{\rho\Delta V}$=nβ dp+α(1-n) dp+nαdp=(nβ+α)dp=γ(nβ+α)dh=S_s dh where S_s=γ(nβ+α) and h=piezometric head=z+$\frac{\rho}{\gamma}$ and γ=ρg=weight of unit volume of water.

The term S_s is called specific storage.It has the dimensions of [L^{-1}] and represents the volume of water released from a unit volume of aquifer due to a unit decrease in the piezometric head.The numerical value of Ss is very small being of the order of $1x10^{-4}$ m^{-1}.

By integration of above equation for a confined aquifer of thickness B, a dimensionless storage coefficient S can be expressed as $S=\gamma(n\beta+)B$.The storage coefficient S (also known as Storativity) represents the volume of water released by a column of a confined aquifer of unit cross sectional area under a unit decrease in the piezometric head. The storage coefficient S and the transmissibility coefficient T are known as the formation constants of an aquifer and play very important role in the unsteady flow through the porous media.

For an unconfined aquifer, the coefficient of storage is given by $S=S_y+\gamma(\alpha+n\beta)B_s$ where B_s=saturated thickness of the aquifer.However, the second term on the right hand side is so small relative to S_y that for practicl purposes S is considered equal to S_y i.e.the coefficient of storage is assumed to have the same value as the specific yield for the unconfined aquifers.

FLOW EQUATIONS FOR CONFINED AND UNCONFINED AQUIFERS

WELL HYDRAULICS

STEADY FLOW INTO A WELL IN CONFINED AQUIFER

UNSTEADY FLOW INTO A WELL IN CONFINED AQUIFER

WELL LOSSES

SPECIFIC CAPACITY

GROUND WATER IRRIGATION

RAIN WATER HARVESTING