Hydrographs and Irrigation

Index

Precipitation, Evaporation and Evapotranspiration

Index of Wetness

- Index of wetness =

$$\frac{\text{rainfall in a given year at a given place}}{\text{average annual rainfall of that place}} \times 100$$

- % Rain deficiency = 100 - % index of wetness

Aridity index

$$A.I = \frac{PET - AET}{PET} \times 100$$

Where

A.I = Aridity index

PET = Potential Evapo-Transpiration

AET = Actual Evapo-transpiration

- AI ≤ 0 → Non arid
- 1 ≤ A.I ≤ 25 → Mild Arid
- 26 ≤ A.I ≤ 50 → Moderate arid
- A.I > 50 → Severe Arid

Optimum Number of rain Gauge: (N)

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

$$C_V = \frac{\sigma_{n-1}}{\bar{X}} \times 100$$

$$\sigma_{n-1} = \sqrt{\frac{\Sigma(X - \bar{X})}{(n-1)}}$$

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

where

C_V = Coefficient of variation,

$\in$ = Allowable % Error,

σ = Standard deviation of the data, n = Number of stations,

$\bar{x}$ = mean of rainfall value

Estimation of missing rainfall data

$$(a)\ P_x \cdot = \frac{P_1 + P_2 + \ldots + P_n}{(n)}$$

$$If\ N_1, N_2 \ldots N_n < 10\%\ of\ N_x$$

where,

$N_1, N_2, \ldots N_x \ldots N_n$ are normal annual precipitation of 1,2,…x…n respectively.

$P_1, P_2 \ldots P_n$ are rainfall at station 1,2,…. N respectively.

And P_x is the rainfall of station x.

Case: A minimum number of three stations closed to station 'x'

$$P_x = \frac{P_1 + P_2 + P_3}{3}$$

$$(b)\ \ P_x = \frac{N_x}{n}\left[\frac{P_1}{N_1} + \frac{P_2}{N_2} + \ldots + \frac{P_n}{N_n}\right]$$

If any of N_1, N_2, $N_3 \ldots N_n$ > 10% of N_x

Mean rainfall Data

To convert the point rainfall values at various into an average value over a catchment the following three methods are in use

(i) Arithmetic Avg Method:

When the rainfall measured at various stations in a catchment area is taken as the arithmetic mean of the station values.

$$P_{avg} = \frac{P_1 + P_2 + \ldots + P_n}{n}$$

Where,

P_1, $P_2 \ldots P_n$ are rainfall values of stations 1,2…n respectively.

In practice, this method is used very rarely.

(ii) Thiessen Polygon Method:

In this method, the rainfall recorded at each station is given a weightage on the basis of an area closest to the station.

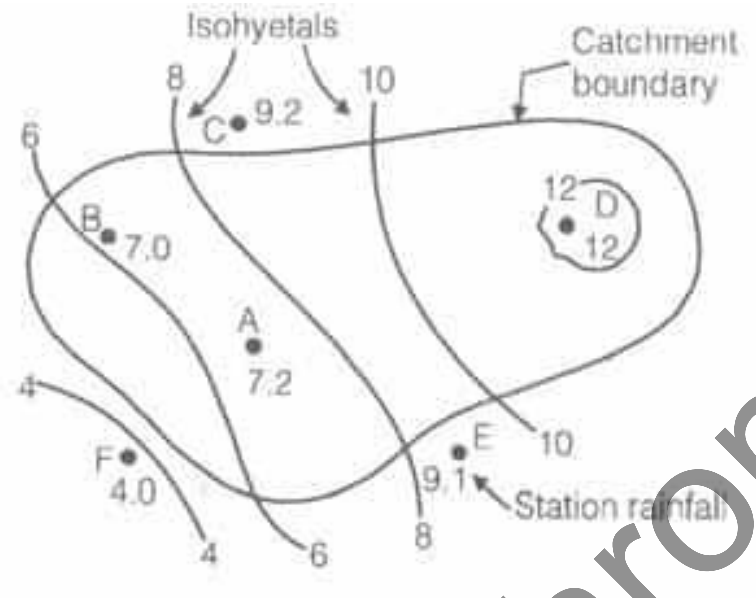
$$P_{avg} = \frac{P_1A_1 + P_2A_2 + \ldots + P_nA_n}{A_1 + A_2 + \ldots + A_n}$$

Where, P_1, $P_2 \ldots P_n$ are the rainfall data of areas A_1, $A_2 \ldots A_n$.

The Thiessen-polygon method of calculating the average precipitation over an area is superior to the arithmetic average method.

(iii) Isohyetal Method:

An isohyet is a line joining points of equal rainfall magnitude. The recorded values for which a real average P is to be determined are then marked on the plot at appropriate stations. Neighboring stations outside the catchment are also considered.



$$P_{avg} = \frac{A_1 \frac{(P_1 + P_2)}{2} + A_2 \frac{(P_2 + P_3)}{2} + A_{n-1} \frac{(P_{n-1} + P_n)}{2}}{A_1 + A_2 + \ldots + A_{n-1}}$$

Evaporation & Evapo-Transpiration

Evaporation and its Measurement

Evaporation is a cooling process in which the latent heat of evaporation of about 585 cal/gm is provided by the water body. in this process, liquid changes into gaseous phase at the free surface, below the boiling point through the transfer of heat energy.

Dalton's Law

The rate of evaporation is proportional to the difference between the saturation vapour pressure at the water temperature, e_s and the actual vapour pressure in the air e_a .Thus

Thus,

***E* = *K*(e_s-e_a)**

Where,

E = Rate if evaporation (mm/day)

e_s = Saturation vapour pressure of air (mm)

e_a = Actual vapour pressure of air (mm)

e_s-e_a Saturation deficiency

Measurement of Evaporation

1. **ISI standard pan** Lake evaporation = C_p × pan Evaporation
   Where, C_p pan coefficient
   = 0.8 for ISI pan
   = 0.7 for class A-Pan
2. **Empirical Evaporation Equations** (Meyer's Formula)
   $$E = k_m(e_s - e_a)\left[1 + \frac{V_9}{16}\right]$$
   Where, k_m = Coefficients which accounts for size of water body.
   = 0.36 (for large deep water)
   $\simeq$ 0.50 (for small and shallow waters)
   e_s = Saturation vapour pressure of air in mm of Hg.
   e_a = Actual vapour pressure of overlying air in mm at Hg at the specified height of 8 m.
   V_9 = monthly mean wind velocity in km/hr at about 9 m above the ground level.
3. **1/7th power La**

$$\frac{V_1}{V_2} = \left(\frac{H_1}{H_2}\right)^{1/7}$$

Where, V_1 is the wind velocity at height H_1 and V_2 is the wind velocity at height H_2.

Water Budget Method

This is the simplest method but it is least reliable it is used for rough calculation, it is based on mass conversation principle.

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

Where,

P=Daily precipitation on the water surface.

V_{is} = Daily surface inflow into lake.

V_{os} = Daily surface outflow from lake.

V_{ig} = Daily underground inflow into the lake.

V_{og} = Daily underground outflow from the lake.

E = Daily Evaporation

ΔS = change in storage of lake

= +ve if increase in storage

= -ve if decrease in storage

T_L = Daily transpiration loss from the plants on the lake.

Energy Budget Method

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

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

Where,

H_n = Net heat energy received by the water surface

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

$H_c(1-r)$ = incoming solar radiation into a surface of reflection coefficient, r

H_b = Back radiation from water body

H_g = Heat flux into the ground

H_S = Heat stored in water body

H_i = Net heat conducted out the system by water flow (advected energy)

β = Bowen's ratio

δ = Density of water

L = Latent heat of evaporation.

Evapo-Transpiration

While transpiration takes place, the land area in which plants stands, also loses moisture by the evaporation of water from soil and water bodies. In hydrology and irrigation practice, it is found that evaporation and transpiration processes can be considered advantageously under one head as evapo-transpiration.

The real evapo-transpiration occurring in a specific situation is called actual evapo-transpiration (AET).

- **Penman's Method**

Penman's equation is based on sound theoretical reasoning and is obtained by a combination of the energy balance and mass transfer approach.

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

Where, PET = daily evaporation in mm/day.

A = slope of the saturation vapour pressure v/s temperature curve at the mean air temperature in mm of Hg per °C.

H_n = Net radiation in mm of evaporable water per day

E_a = Parameter including wind velocity and saturation deficit.

γ = Psychometric constant

= 0.49 mm of Hg/°C

It is based on mass transfer and energy balance.

Transpiration Loss (T)

$T = (w_1+w_2)-w_2$

Where,

w_1 = Initial weight of the instrument

W = Total weight of water added for full growth of plant.

w_2 = Final weight of instruction including plant and water

T = Transpiration loss.

Infiltration and Runoff

Infiltration

The process of water entering into the soil is called infiltration. Actually, the infiltration occurs on the ground surface plane. Below the surface, the penetration further is called percolation. Whatever rainfall occurs on surface of earth, some quantities infiltrate.

- **Horton's Equation**: He observed that the infiltration capacity reduced in an exponential fashion from an initial, maximum rate f_{cf} to a final constant rate f_{co}. Horton expressed the decay of infiltration capacity with time as an exponential decay given by

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

Where,

f = infiltration capacity at any time t from the start of the rainfall

f_o = initial infiltration capacity at t = 0

f_c = final steady state value

t_d = Duration of rainfall

k_h = constant depending on soil.

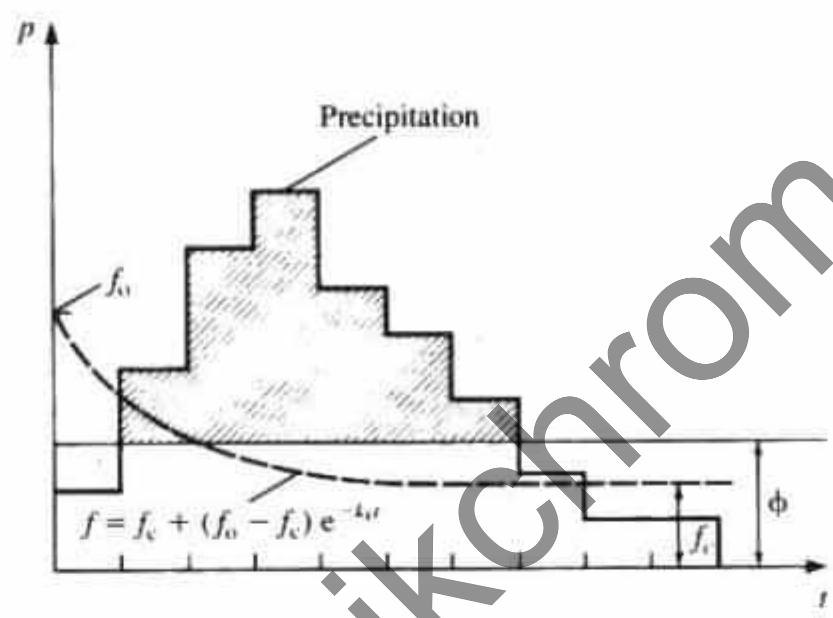


Illustration of ϕ index and Horton's equation for infiltration*

Infiltration Indices

In hydrological calculations involving floods, it is found convenient to use a constant value of filtration rate for the duration of the storm. The defined average infiltration rate is called infiltration index. Also this is the average **infiltration** rate during the time when the rainfall intensity exceeds the **infiltration** rate.

The W – **index** can be derived from the observed rainfall and runoff data. It differs from the φ - **index** in that it excludes surface storage and retention.

(i) **W-index**:

In an attempt to refine the φ-index the initial losses are separated from the total abstractions and an average value of infiltration rate, called W-index, is defined as

$$W\text{-}index = \frac{P - R - I_a}{t_e}$$

Where, P = Total storm precipitation (cm)

R = Total storm runoff (cm)

I_a = initial losses (cm)

t_e = Duration of rainfall excess

W-index = Avg. rate of infiltration (cm/hr)

(ii) **φ-index**:

The φ index is the average rainfall above which the rainfall volumes is equal to the runoff volume. The φ index is derived from the rainfall hyetograph with the edge of the resulting run- off volume.

φ-index = (I-R)/24

Where,

R = Runoff in cm from a 24- h rainfall of intensity I cm/day

Runoff

Runoff can be described as the part of the water cycle that flows over land as surface water instead of being absorbed into groundwater or evaporating. It thus represents the output from the catchment in a given unit of time.

There are a variety of **factors that affect runoff**. Some of those include:

1. Amount of Rainfall

2. Permeability

3. Vegetation

4. Slope

Direct Runoff

The part of runoff which enters the stream quickly after the rainfall or snow melting. To design soil conservation structure with proper capacity it is necessary to estimate peak runoff rate.

It includes surface runoff, prompt interflow and rainfall on the surface of the stream. In the case of snow-melt, the resulting flow entering the stream is also a direct runoff, sometimes terms such as direct storm runoff are used to designate direct runoff.

Base Flow

Baseflow (also called drought **flow**, groundwater recession **flow**, low **flow**, low-water **flow**, low-water discharge and sustained or fair-weather runoff) is the portion of streamflow that comes from "the sum of deep subsurface **flow** and delayed shallow subsurface **flow.** Also, it is the delayed flow that reaches a stream essentially as groundwater flow is called base flow.

(i) Direct runoff = surface runoff + Prompt interflow

(ii) Direct runoff = Total runoff- Base flow

(iii) Form Factor

$$\frac{A}{l^2}$$

where, A = Area of the catchment *l* Axial length of basin.

(iv) Compactness coefficient

$$= \frac{P}{2\pi r_e} r_e = \sqrt{\frac{A}{\pi}}$$

r_e = Radius of an equivalent circle whose Area is equal to area of the catchment (A)

(v) Elevation of the water shed, (z)

$$z = \frac{A_1 z_1 + A_2 z_2 + ... + A_n z_n}{A_1 + A_2 + ...A_n}$$

Where, A_1, A_2 … Area between successive contours.

Z_1, z_2 … mean elevation between two successive contours.

Methods to Compute Runoff

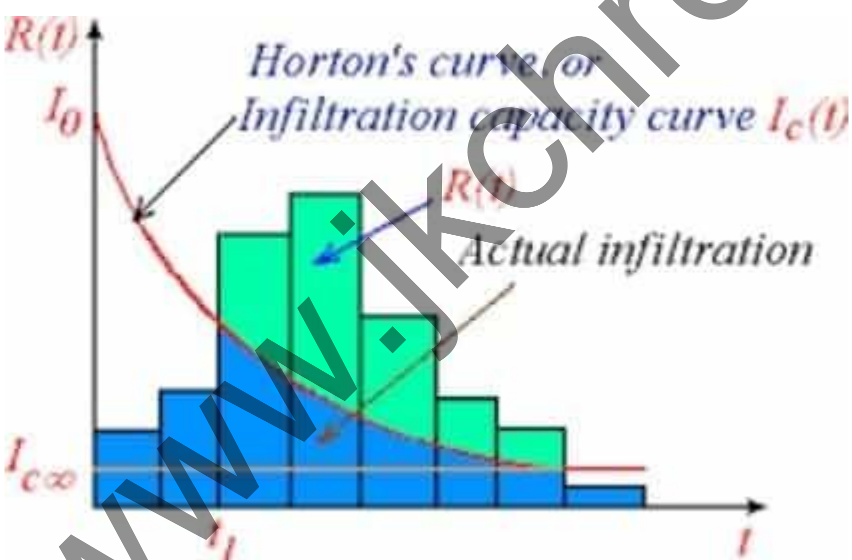
(i) By Runoff coefficient

Q = KP where, p = precipitation

K = Runoff coefficient

Q = Runoff

(ii) By infiltration Capacity Curve



(iii) By Rational Formula

$$Q_p = \frac{1}{36} \cdot kP_C A$$

Where, k = Runoff coefficient

P_C = Critical design rainfall intensity in cm/hr

A = Area of catchment in the hectare

Q_p = Peak discharge in m^3/sec.

Hydrograph and its Analysis

Hydrograph

- A plot of the discharge in a stream plotted against time chronologically is called a hydrograph. OR
- Special graphs that show changes in a river's discharge over a period of time, usually in relation to a rainfall event. OR
- A hydrograph is a graph showing the rate of flow (discharge) versus time past a specific point in a river, or other channel or conduit carrying flow. The rate of flow is typically expressed in cubic meters or cubic feet per second (m^3/s).

Basic Terms

The discharge is measured at a specific point in a river and is typically time variant.

- **Rising limb**: The rising limb of hydrograph, also known as concentration curve, reflects a prolonged increase in discharge from a catchment area, typically in response to a rainfall event
- **Recession (or falling) limb**: The recession limb extends from the peak flow rate onward. The end of storm flow (quick flow or direct runoff) and the return to groundwater-derived flow (base flow) is often taken as the point of inflection of the recession limb. The recession limb represents the withdrawal of water from the storage built up in the basin during the earlier phases of the hydrograph.
- **Peak discharge**: The highest point on the hydrograph when the rate of discharge is greatest
- **Lag time**: The time interval from the centre of mass of rainfall excess to the peak of the resulting hydrograph
- **Time to peak**: Time interval from the start of the resulting hydrograph
- **Discharge**: The rate of flow (volume per unit time) passing a specific location in a river or other channel

Factors affecting hydrograph shape

1. **Drainage characteristics**: Basin area, basin shape, basin slope, soil type and land use, drainage density, and drainage network topology. Most changes in land use tend to increase the amount of runoff for a given storm.
2. **Rainfall characteristics**: Rainfall intensity, duration, and their spatial and temporal distribution; and storm motion, as storm moving in the general downstream direction tend to produce larger peak flows than storms moving upstream.

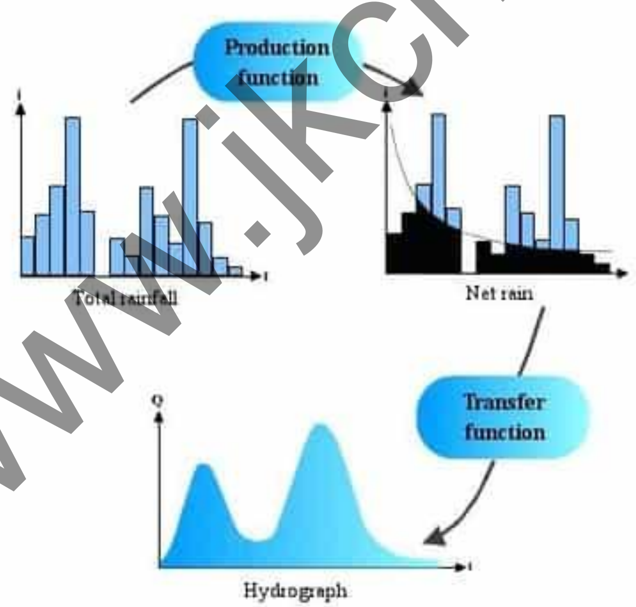
Why Construct & Analyse Hydrographs?

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

Relation rainfall - runoff

A rainfall defined in time and space that falls on a catchment produces a hydrograph.

Given figure defines certain essential elements of the hydrograph resulting from a **hyetograph**(Rainfall intensity **vs**. time).



To describe the processes that occur when the rain is transformed into a flow hydrograph (by Horton's postulate), we apply two functions called production function and transfer function. The production function allows determination of the net rain hyetograph starting from the total rain. The transfer function allows determination of the hydrograph resulting from the net rain. The net rain represents the part of total rain that contributes to the flow process.

$$D = \frac{d}{2}\sqrt{\frac{V_2 - V_1}{V_2 + V_1}}$$
$$= \frac{30}{2}\sqrt{\frac{4000-600}{4000+600}}$$
$$= 12.9\text{m}$$

The hydrograph is represented by an asymmetric curve. Peak flow is represented by the following formula:

Q_p = C.i.A

where:		
	C	runoff coefficient (depends on the catchment characteristics)
	i	rainfall intensity in time t_c
	A	area of catchment

It can be defined:

- Response time of the catchment t_r - represents the time interval that separates the net rain gravity centre from the peak flow or sometimes the gravity centre of the flow hydrograph.
- Time of concentration t_c - is the time required by rain fallen on the catchment to flow from the farthest point to the measuring point of the river. Thus, after time t_c from the beginning of the rainfall, the whole catchment is considered to contribute to the flow. The value of *i*, the main intensity, assumes that the rate of rainfall is constant during t_c, and that all measured rainfall over the area contributes to the flow. The peak flow Q_p occurs after the period t_c.
- Rising limb t_m - is the time from the beginning of the rain to the peak of the hydrograph
- Base time t_b - represents the direct flow duration.
- Time to Peak t_p - Time from the beginning of the rising limb to the occurrence of the peak discharge

Total streamflow during a precipitation event includes the baseflow existing in the basin prior to the storm and the runoff due to the given storm precipitation. Total streamflow hydrographs are usually conceptualised as being composed of:

a) **Direct Runoff**, which is composed of contributions from surface runoff and quick interflow. Unit hydrograph analysis refers only to direct runoff.

b) **Baseflow**, which is composed of contributions from delayed interflow and groundwater runoff.

c) **Surface runoff** includes all overland flow as well as all precipitation falling directly onto stream channels. Surface runoff is the main contributor to the peak discharge.

d) **Interflow** is the portion of the streamflow contributed by infiltrated water that moves laterally in the subsurface until it reaches a channel. Interflow is a slower process than surface runoff. Components of interflow are quick interflow, which contributes to direct runoff, and delayed interflow, which contributes to baseflow.

e) **Groundwater runoff** is the flow component contributed to the channel by groundwater. This process is extremely slow as compared to surface runoff.

Types of hydrograph can include:

- Storm/Flood hydrographs - A **storm hydrograph** is a way of displaying how the discharge of a river can change over time in response to a rainfall event. The discharge of a river is just the amount of water passing a certain point every second and is calculated by multiplying the cross-sectional area of the river by its velocity
- Annual hydrographs
- Direct Runoff Hydrograph - **Direct runoff hydrograph(DRH)** resulting from one unit (e.g., 1 cm) of effective rainfall occurring uniformly over that watershed at a uniform rate over a unit period of time

Unit Hydrograph

This method was first suggested by Sherman in 1932 and has undergone many refinements since then.

- A *unit hydrograph* is defined as they hydrograph of direct runoff resulting from one unit depth (1 cm) of rainfall excess occurring uniformly over the basin and at a uniform rate for a specified duration (D hours). OR

- A *unit hydrograph* (UH) is the hypothetical unit response of a watershed (in terms of runoff volume and timing) to a unit input of rainfall. It can be defined as the *direct runoff hydrograph* (DRH) resulting from one unit (e.g.,1 cm) of *effective rainfall o*ccurring uniformly over that watershed at a uniform rate over a unit period of time. As a UH is applicable only to the direct runoff component of a hydrograph (i.e., surface runoff), a separate determination of the baseflow component is required. OR
- It can be defined as the direct **runoff hydrograph** (DRH) resulting from one unit (e.g., 1 cm) of **effective** rainfall occurring uniformly over that watershed at a uniform rate over a unit period of time.

A UH is specific to a particular watershed and specific to a particular length of time corresponding to the duration of the effective rainfall. That is, the UH is specified as being the 1-hour, 6-hour, or 24-hour UH, or any other length of time up to the *time of concentration* of direct runoff at the watershed outlet. Thus, for a given watershed, there can be many unit hydrographs, each one corresponding to a different duration of effective rainfall

The following are essential steps in deriving a unit hydrograph from a single storm:

1. Separate the baseflow and obtain the direct runoff hydrograph (DRH).
2. Compute the total volume of direct runoff and convert this volume into the equivalent depth of effective rainfall (in 1 cm) over the entire basin.
3. Normalize the direct runoff hydrograph by dividing each ordinate by the equivalent volume (cm) of direct runoff (or effective rainfall)
4. Determine the effective duration of excess rainfall. To do this, obtain the effective rainfall hyetograph (e.g., use the φ-index, the Horton) and its associated duration. This duration is the duration associated with the unit hydrograph

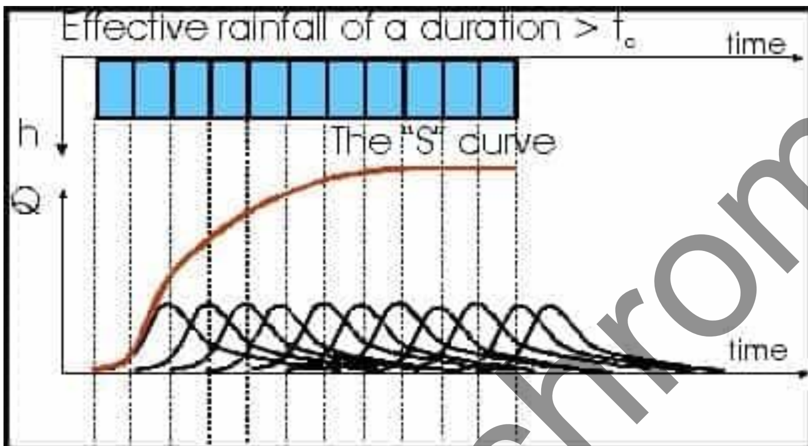
Unit hydrographs are fundamentally linked to the duration of the effective rainfall event producing them. They can only be used to predict direct runoff from storms of the same duration as that associated with the UH, or from storms which can be described as a sequence of pulses, each of the same duration as that associated with the UH.

Unit Hydrographs for Different Effective Duration

A unit hydrograph for a particular watershed is developed for a specific duration of effective rainfall. When dealing with a rainfall of different duration a new unit hydrograph must be derived for the new duration. The linearity property implicit in the UH analysis can be used to generate UH's associated with larger or smaller effective rainfall pulse duration. This procedure is sometimes referred to as the S-curve Hydrograph method.

1. **S-Curve Hydrograph Method**

An S-hydrograph represents the response of the basin to an effective rainfall event of infinite duration. Assume that a UH of duration D is known and that a UH for the same basin but of duration D' is desired. The first step is to determine the S-curve hydrograph by adding a series of (known) UH's of duration D, each lagged by a time interval D. The resulting superposition represents the runoff resulting from a continuous rainfall excess of intensity 1/D.



Lagging the S-curve in time by an amount D' and subtracting its ordinates from the original unmodified S-curve yields a hydrograph corresponding to a rainfall event of intensity 1/D and of duration D'. Consequently, to convert this hydrograph whose volume is D'/D into a unit hydrograph of duration D', its ordinates must be normalized by multiplying them by D/D'. The resulting ordinates represent a unit hydrograph associated with an effective rainfall of duration D'.

A hydrologic system (a basin) is said to be a linear system if the relationship between storage, inflow, and outflow is such that it leads to a linear differential equation.

Assumptions

Time invariance:

The first basic assumption is that the direct-runoff response to a given effective rainfall in a catchment is time invariant. This implies that the DRH for a given ER in a catchment is always the same irrespective of when it occurs.

Linear Response:

The direct-runoff response to the rainfall excess is assumed to be linear. This is the most important assumptions of the unit-hydrograph theory. Linear response means that if an input X_1(t) cause an output y_1(t) an output x_2(t) causes an output y_2(t) then an input x_1(t) + x_2(t) gives an output $y_1(t) + y_2(t)$. Consequently if x_2(t) = rx_1(t), then y_2(t) = r_1y_1(t). Thus, if the rainfall excess in a duration D is r times the unit depth, the resulting DRH will have ordinates bearing ratio r to those of the corresponding D-h unit hydrograph.

$$t'_B = t_B + (n-1)D$$

Where,

$t'_B =$ The base period of T hr U.H

t_B = Base period of D hr U.H

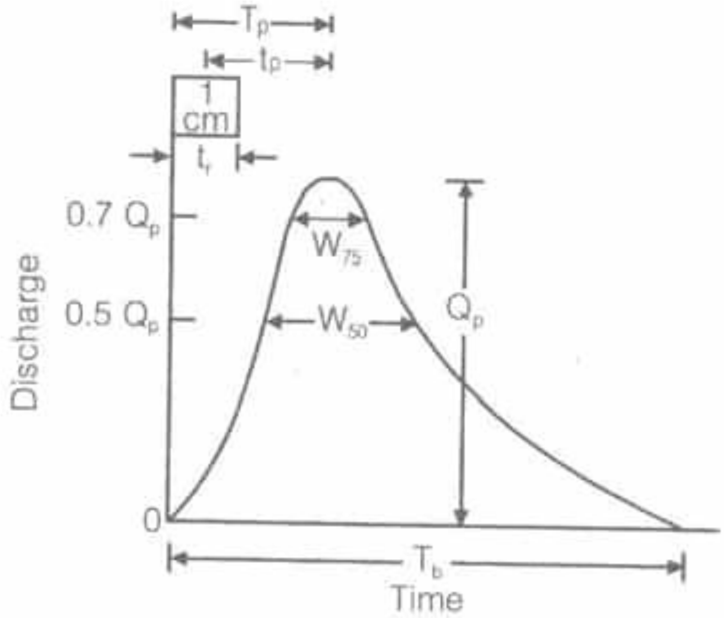
Also, T>D

T = n.D where 'n' is an integer.

Synthetic Hydrograph

A **synthetic unit hydrograph** retains all the features of the **unit hydrograph** but does not require rainfall-runoff data. A **synthetic unit hydrograph** is derived from theory and experience, and its purpose is to simulate basin diffusion by estimating the basin lag based on a certain formula or procedure.

Synder's Method: Synder (1938), based on a study of a large number of catchment in the Appalachian Highlands of eastern United States developed a set of empirical equations for synthetic unit hydrograph in those areas. These equations are in use in the USA. And with some modifications in many other countries, and constitute what is known as Synder's synthetic unit hydrograph.



(i) $t_p = C_t\left[L \cdot L_{Ca}\right]^{0.3}$

Where,

t_p = Time interval between mid-point of unit rainfall excess and peak of unity hydrograph in hour

L = Length of main stream

L_{Ca} = The distance along the main stream from the basin outlet to a point on the stream which is nearest to the centrod of basis (in KM)

C_t = Regional constant 0.3 < C_t 0.6

(ii) $t_p = C_t\left[\frac{L \cdot L_{Ca}}{\sqrt{S}}\right]^n$

S = Basin slope.

N = Constant = 0.38.

(iii) $t_r = \frac{t_p}{5.5}$

t_r = Standard duration of U.H in hour

$$(iv)\ Q_{PS} = \frac{2.78 C_P A}{t_p}$$

Where, C_p = Regional constant = 0.3 to 0.92.

A = Area of catchment in km^2.

Q_{PS} = Peak discharge in m^3/s.

(v) $t'_p = 0.955\ t_p + 0.25\ t_r$

where, t_R = standard rainfall duration.

t'_p =

Basin lag for non-standard U.H.

$$(vi)\ Q_p = \frac{2.78 C_P A}{t'_P}$$

$$(vii)\ t_B = \left(72 + 3t'_p\right) hour,$$

for a large catchment.

Where,

t_B = Base time of synthetic U.H

$$t_B = 5\left[t'_p + \frac{t_R}{2}\right] hour,$$

for the small catchment.

$$(viii)\ W_{50} = \frac{5.87}{(q)^{1.08}}$$

W_{50} = width of U.H in hour at 50% peak discharge.

(ix) $W_{75} = (W_{50})/1.75$

W_{75} = Width of U.H in hours at 75% peak discharge.

$$(x)\ q = \frac{Q_P}{A}$$

where, Q_P = Peak discharge in m^3/sec.

A = Area in km^2.

Types of irrigation systems & Water Requirement of Crops

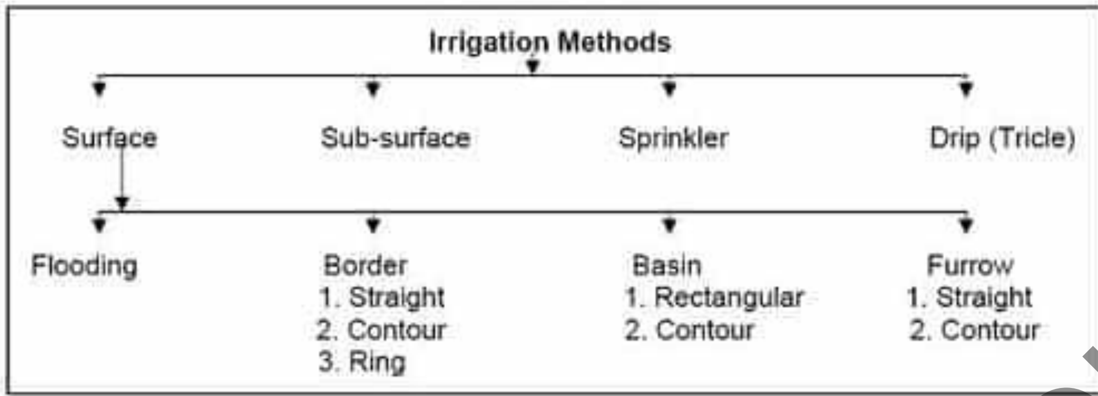
Types of Irrigation Systems

Major aim of irrigation systems is to help out in the growing of agricultural crops and vegetation by maintaining with the minimum amount of water required, maintenance of landscapes and re-vegetation of disturbed soils. Irrigation systems are also used for dust repression, removal of sewage, and in mining.

On the contrary, agriculture that relies only on direct rainfall is referred to as rain-fed or dry-land farming.

Techniques of Irrigation

In India, the irrigated area consists of about 36 percent of the net sown area. There are various techniques of irrigation practices in different parts of India. These methods of irrigation differ in how the water obtained from the source is distributed within the field. In general, the goal of irrigation is to supply the entire field homogeneously with water, so that each plant has the amount of water it needs, neither too much nor too little. Irrigation in India is done through wells, tanks, canals, perennial canal, and multi-purpose river valley projects.

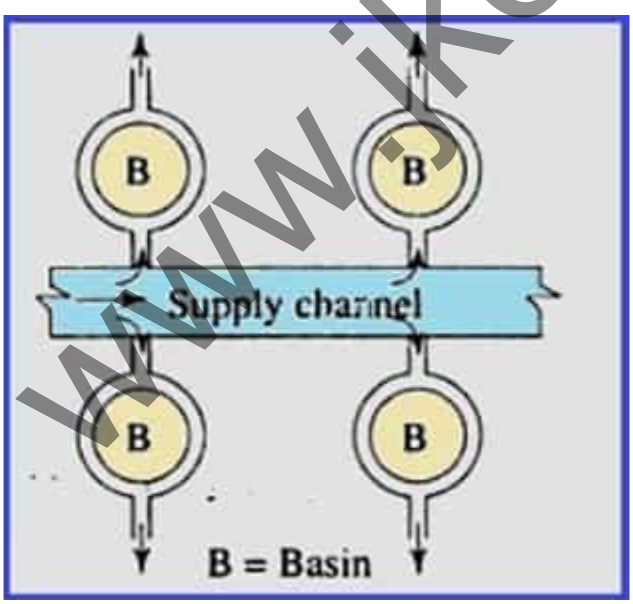


A) Surface Irrigation

In this technique, water flows and spreads over the surface of the land. Varied quantities of water are allowed on the fields at different times. Therefore, it is very difficult to understand the hydraulics of surface irrigation.

Surface irrigation technique is broadly classified as

1. Basin irrigation - Basin irrigation is common practice of surface irrigation. If a field is level in all directions, is encompassed by a dyke to prevent runoff, and provides an undirected flow of water onto the field, it is herein called a basin. It may be furrowed or ridged, have raised beds for the benefit of certain crops, but as long as the inflow is undirected and uncontrolled into these field modifications, it remains a basin.



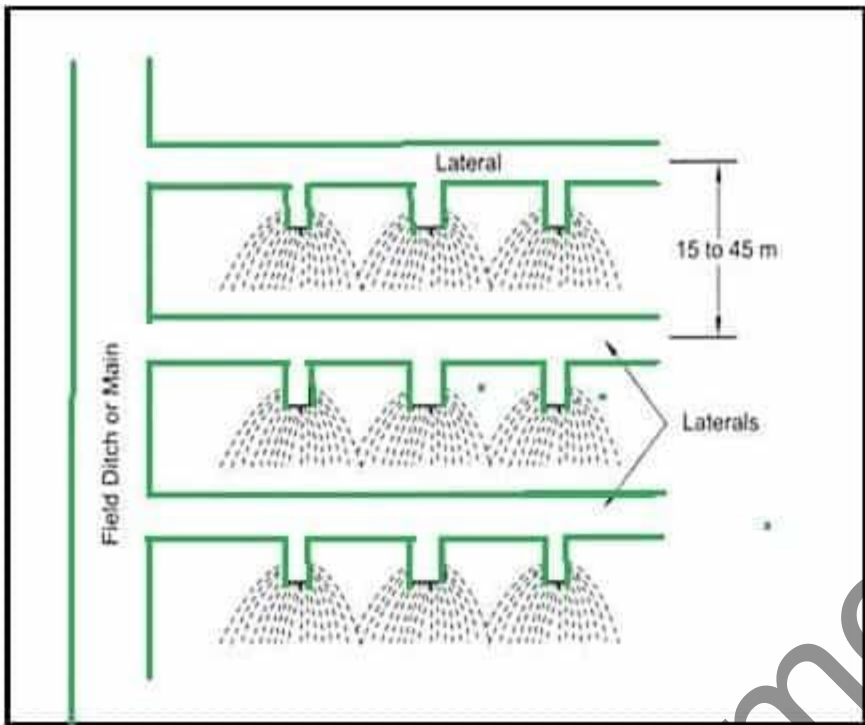
2. Furrow irrigation - In furrow irrigation technique, trenches or “furrows” are dug between crop rows in a field. Farmers flow water down the furrows (often using only gravity) and it seeps vertically and horizontally to refill the soil reservoir. Flow to each furrow is individually controlled. Furrow irrigation is suitable for row crops, tree crops and because water does not directly contact the plants, crops that would be damaged by direct inundation by water such as tomatoes, vegetables, potatoes and beans. It is one of the oldest system of irrigation. It is economical and low-tech making it particularly attractive in the developing world or places where mechanized spray irrigation is unavailable or impractical.

There are numerous **advantages** of Furrow technique of irrigation:

1. Large areas can be irrigated at a time.
2. It saves labour since once the furrow is filled, it is not necessary to give water a second time.
3. It is a reasonably cheaper method.
4. Plants get proper quantity of water by this system.

Major **drawback** of furrow system of irrigation is ensuring uniform dispersal of water over a given field. . Other problem with furrow irrigation is the increased potential for water loss due to runoff.

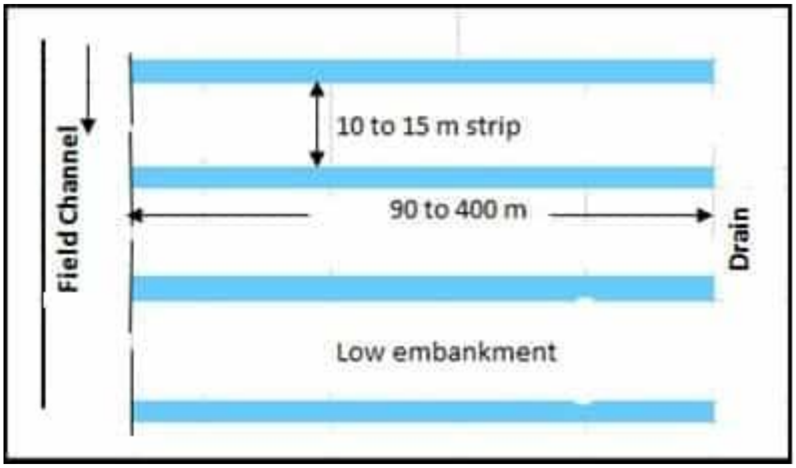
- **Uncontrolled flooding**: There are many cases where croplands are irrigated without regard to efficiency or consistency. These are usually situations where the value of the crop is very small or the field is used for grazing or recreation purposes. Small land holdings are generally not subject to the range of surface irrigation practices of the large industrial farming systems. The assessment methods can be applied if desired, but the design techniques are not generally applicable nor need they be since the irrigation practices tend to be minimally managed.
- **Free flooding** - In free flooding method, water is applied to the land from field ditches without any check or guidance to the flow. The land is divided into plots or kiaries of suitable size depending on porosity of soil. Water is spread over the field from watercourse. The irrigation operation begins at the higher area and proceeds towards the lower levels. The flow is stopped when the lower end of the field has received the desired depth of water.



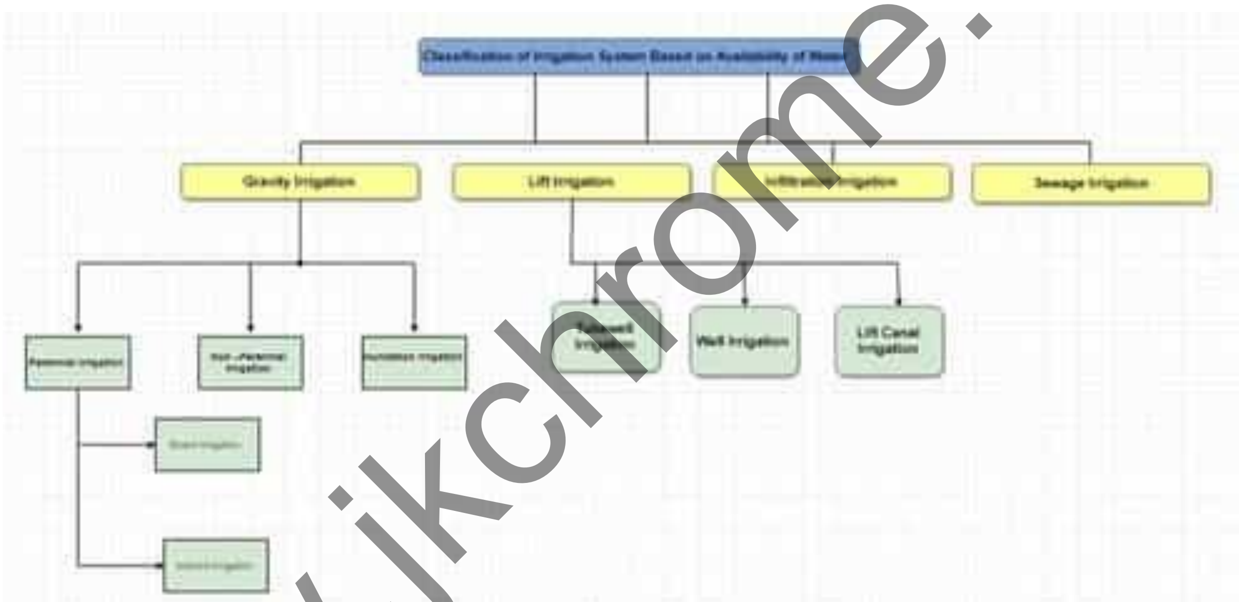
This technique is beneficial for newly established farms where making furrows is very expensive. This method is economical and can be effectively used where water supply is in plenty. This method is suitable for the fields with an irregular surface in which other techniques are difficult to apply.

The major **drawback** of this method is that there is no perfect control over the flow of water to attain high efficiency. Sometimes the flow of water over the soil is too rapid to fulfil soil moisture deficiency. On the other hand, sometimes water is retained on the field for a very long time and consequently, the water is lost in infiltration or deep percolation.

3. Border Strip Method - In this technique of irrigation, a field is divided into number of strips. The width of strip varies from 10 to 15 metres and length varies from 90 m to 400 m. Strips are separated by low embankments or levees. The water is diverted from the field channel into the strips. The water flows gradually towards lower end, wetting the soil as it advances. The surface between two embankments should essentially be level. It assists in covering the entire width of the strip. There is a general surface slope from opening to the lower end. The surface slope from 2 to 4 m/1000 m is best suited. When the slope is steeper, special arrangement is made to prevent erosion of soil.



Classification Based on Availability of Water



1. Gravity Irrigation:

Gravity or flow irrigation is the type of irrigation in which water is available at a higher level as to enable supply to the land by gravity flow. In flow irrigation water is supplied to the fields though the canals off taking from head works. Gravity flow irrigation is cheaper compared to lift irrigation. The gravity irrigation is further classified as under.

1.1 Perennial Irrigation

In this system assured the supply of water throughout the crop period to irrigation requirement of the crops is made available to the command area through storage of water done at the dam or diversion of supply made by means of head works at the off take point of the canal. Perennial irrigation may be either direct or indirect, as follows:

1.1.1 Direct irrigation:

In direct irrigation system, water is directly diverted from the river into the canal by the construction of diversion weir or barrage across the river without attempting to store water.

1.1.2 Indirect irrigation:

It is also termed as storage irrigation. Here water is stored in reserved during monsoon period by the construction of a dam across the river for supply into the off taking canals.

1.2 Non –Perennial Irrigation:

Also called restricted irrigation. Canal supply is generally made available in non-monsoon period from the storage in small dams as in Kandi areas which inadequate to feed all the year round, and/or canal water is not required during monsoon due adequate rainfall in the command area.

1.3 Inundation Irrigation:

Inundation irrigation is done by a canal taking off from a river in flood without any diversion work. It depends on the periodical rise in water level of the river and the supply is drawn through open cuts in the river bank or creeks which are called heads.

2. Tank Irrigation: Tanks on local streams form a significant source of irrigation especially in the peninsula area in the States of Karnataka, Maharashtra and Tamil Nadu. Tank irrigation belongs to category of storage irrigation. Tanks are small sized reservoirs formed by small earthen embankments to store runoff for irrigation. The site is selected within a watershed protected by vegetation and containing minimum of cultivated land so as to ensure minimum rate of sedimentation which lowers its storage capacity. Adequate soil conservation measures are essentially adopted to ensured quantity and quality of water inflow into the tank.

3. Lift Irrigation:

In lift irrigation water is lifted from a river or a canal to the bank to irrigate the land which are not commanded by gravity flow. In lift Irrigation mechanical devices like pumps, or electric motors and pumps are required to be installed for lifting water. Electrical pumps are generally provided for lifting water. Diesel pumping sets are also installed as standby.

Lift Irrigation vs. Gravity Irrigation:

Lift irrigation

1.Costly means of irrigation

2.Less manorial silt in water

3.Working dependent on the operation of machinery

4.Higher water rates.

5. Lift irrigation is a complex system and by and large costly.

Gravity flow irrigation

1.Cheapest means of irrigation

2. Silt in water has manorial value

3. Lifting equipment is not involved

4. Lowest water rates

5. Simple and economical system of irrigation.

4. Well Irrigation

Groundwater:

Groundwater is generally a more dependable source of irrigation than surface water and is free from seeds and plant organisms. The first cost of installation is, however, high. The best water bearing stratum or aquifer is coarse gravel free from sand but such formation are rare to find..

Subsurface Irrigation:

It is termed as subsurface irrigation because in this type of irrigation, water does not wet the soil surface. The underground water nourishes the plant roots by capillarity.

5. Sprinkler Systems

In the sprinkler irrigation network, we have the mains and the subdomains, through which water under pressure is made to flow. Revolving sprinkler heads are then usually mounted on rising pipes attached to the laterals. The water jet comes out through the revolving sprinkler heads, with force. When sprinkler heads are not provided, perforations are made in the pipes, and they are provided with nozzles, through which water jets out and falls on the ground. Generally, such a perforated pipe system operates at low heads; whereas, the revolving heads sprinklers operate on high as well as low heads, depending upon the type of rotary head used.

The advantage of sprinkler irrigation are enumerated below:

1. Seepage losses, which occur in earthen channels of surface irrigation methods, are completely eliminated.
2. Moreover, only the optimum quantity of water is used in this method.
3. Land levelling is not required, and thus avoiding removal top fertile soil, as happens in other surface irrigation methods.
4. No cultivation area is lost for making ditches, as happens in surface irrigation methods. It, thus, results in increasing about 16% of the cropped area.
5. In the sprinkler system, the water is to be applied at a rate lesser than the infiltration capacity of the soil, and thus avoiding surface run, and its bad effects, such as loss of water, washing of topsoil, etc.
6. Fertilizers can be uniformly applied because they are mixed with irrigation water itself.
7. This method leaches down salts and prevents waterlogging or salinity.
8. It is less labour oriented, and hence useful where labour is costly and scarce.
9. Upto 80% efficiency can be achieved, i.e. upto 80% of applied water can be stored in the root zone of plants/.

Limitations of sprinkler irrigation nare also enumerated below:

1. High winds may distort sprinkler pattern, causing non-uniform spreading of water on the crops.

2. In areas of high temperature and high wind velocity, considerable evaporation losses of water may take place.
3. They are not suited to crops requiring frequent and larger depths of irrigation, such as paddy.
4. The initial cost of the system is very high, and the system requires a high technical skill.
5. Only sand and silt free water can be used, as otherwise pump impellers lifting such waters will get damaged.
6. It requires a larger electrical power.
7. Heavy soil with poor intake cannot be irrigated efficiently.
8. Constant water supply is needed for commercial use of equipment.

6. Drip irrigation Method

Drip irrigation, also called trickle irrigation, is the latest field irrigation technique and is meant for adoption at places where there exists acute scarcity of irrigation water and other salt problems. In this method, water is slowly and directly applied to the root zone of the plants, thereby minimizing the losses by evaporation and percolation.

This system involves laying a system of the head, mains, sub mains, laterals, and drop nozzles. Water oozes out of these small drip nozzles uniformly and at a very small rate, directly into the plant roots area.

The head consists of a pump to lift water, so as to produce the desired pressure of about 2.5 atmospheres, for ensuring proper flow of water through the system. The lifted irrigation water is passed through a fertilizer tank, so as to mix the fertilizer directly in the irrigation water, and then through a filter, so as to remove the suspended particles from the water, to avoid clogging of drip nozzles.

Water Requirements of Crops

Every crop requires a certain quantity of water after a certain fixed interval, throughout its period of growth. If natural rain is sufficient and timely so as to satisfy both these requirements, no irrigation water is required for raising that crop.

In a tropical country like India, the natural rainfall is either insufficient, or the water does not fall frequency of the rainfall varies throughout a tropical country, the certain crop may require irrigation in a certain part of the country. The area where irrigation is a must for agriculture is called the **arid region**, while the area in which inferior crops can be grown without irrigation is called a **semiarid region**.

- **Crop Period or Base Period**

The time period that elapses from the instant of its sowing to the instant of its harvesting is called the **crop period**.

The time between the first watering of a crop at the time of its sowing to its last watering before harvesting is called the **base period** or the base of the crop.

Crop period is slightly **more** than the base period, but for all practical purposes, they are taken as one and the same thing, and generally expressed in days.

- **Delta of a Crop (Δ)**

Each crop requires a certain amount of water after a certain fixed interval of time, throughout its period of growth.

The total quantity of water required by the crop for its full growth may be expressed in hectare metre (ha.m) or simply as depth to which water would stand on the irrigated area if the total quantity supplied were to stand above the surface without percolation or evaporation. This total depth of water (in **cm**) required by a crop to come to maturity is called its **delta** (**Δ**).

Example 1: If rice requires about 10cm depth of water at an average interval of about 10 days, and the crop period for rice is 120 days, find out the delta for rice.

Solution: Water is required at an interval of 10 days for a period of 120 days. It evidently means that 12 no. of waterings are required, and each time, 10 cm depth of water is required. Therefore, the total depth of water required.

= 12 x10 cm = 120 cm.

Hence Δ for rice = 120 cm. **Ans.**

Example 2: If wheat requires about 7.5 cm of water after every 28 days, and the base period for wheat is 140 days, find out the value of delta for wheat.

Solution: Assuming the base period to be representing the crop period, as per usual practise, we can easily infer that the water is required at an average interval of 28 days up to a total period of 140days.

This means that 5(140/28) no. of waterings are required 28days

The depth of water required each time = 7.5 cm.

Total depth of water required. In 140 days = 5 x7.5 = 37.5 cm

Hence, Δfor wheat = 37.5 cm. **Ans.**

- **Delta for certain crops**

The average values of deltas for certain crops are shown in the table. These values represent the total water requirement of the crops. The actual requirement of irrigation water may be less, depending upon the useful rainfall. Moreover, these values represent the values on the field, i.e. 'delta on field' which includes losses.

Table: Average Approximate Values of Δ for Certain Important Crops in India	
Crop	**Delta on field(cm)**
Sugarcane	120
Rice	120
Tobacco	75
Garden fruits	60
Cotton	50
Vegetables	45
Wheat	40
Barley	30
Maize	25
Fodder	22.5

- **Duty of Water** (D)

The term **duty** means the "**area of land**" that can be irrigated with the unit volume of irrigation **water**. Quantitatively, **duty** is defined as the area of land expressed in hectares that can be irrigated with unit discharge, that is, 1 cumec flowing throughout the base period, expressed in days.

- **Relation between Duty(D) and Delta(Δ)**

Δ = 8.64B/D (metres)

Where,

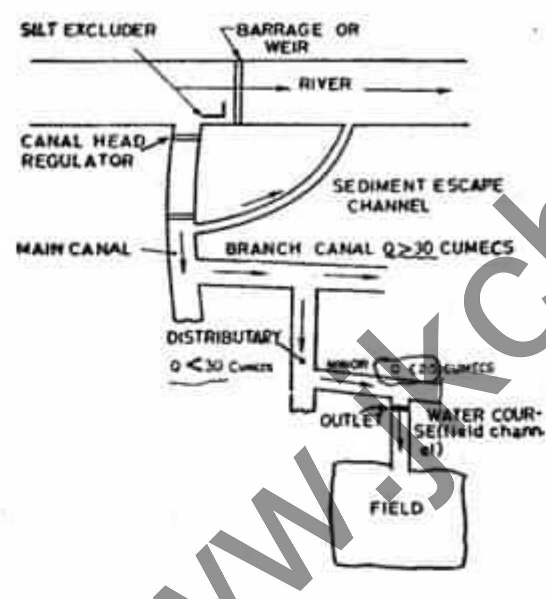
Δ is in **meter**, **B** is in **days**; and

D is duty in **hectares/cumec**.

During the passage of water from these irrigation channels, water is lost due to evaporation and percolation. These losses are called Transit losses or Transmission or Conveyance losses in channels.

Layout of Canal System

Duty of water for a crop is the number of hectares of land which the water can irrigate. Therefore, if the water requirement of the crop is more, less number of hectares of land it will irrigate. Hence, if water consumed is more, duty will be less. It, therefore, becomes clear that the duty of water at the head of the watercourse will be less than the because when water flows from the head of the watercourse and reaches the field, some water is lost as transit losses.



Applying the same reasoning, it can be established that duty of water at the head of a minor will be less than that at the head of the watercourse; duty at the head of a distributary will be less than that at the head of a minor, duty at the head of a branch canal will be less than that at the head of a minor, duty at the head of the main canal will be less that the duty at the head of a branch canal.

Duty of water, therefore, **varies from one place to another** and increases as we move downstream from the head of the main canal towards the head of the branches or watercourses. The duty at the head of the watercourse (i.e. at the outlet point is generally the endpoint of Irrigation Department.

Factors Affecting Duty of Water

1. **Climatic and season**
2. **Useful rainfall**
3. **Type of soil**
4. **The efficiency of cultivation method**

Irrigation Efficiencies

Efficiency is the ratio of the water output to the water input and is usually expressed as the percentage. Input minus output is nothing but losses, and hence, if losses are more, the output is es and, therefore, efficiency is less. Hence, efficiency is inversely proportional to the losses. Water is lost in irrigation during various processes and, therefore, there are different kinds of irrigation efficiencies, as given below.

(i) Efficiency of water-conveyance (η_c): It is a ratio of the water delivered into the fields from the outlet point of the channel to the water pumped into the channel at the starting point. It may be represented by ηc. It takes the conveyance or transit losses into account.

(ii) Efficiency of water application (η_a): It is the ratio of the quantity of water stored into the root zone of the crops to the quantity of water actually delivered into the field. it may be represented by ηa. It may also be termed as farm efficiency, as it takes into account the water is lost on the farm.

(iii) Efficiency of water storage (η_s): It is the ratio of the water stored in the root zone during irrigation to the water needed in the root zone prior to irrigation (i.e., field capacity –existing moisture content). It may be represented by ηs.

(iv) Efficiency of water use (η_u): It is the ratio of the water beneficially used, including leaching water, to the quantity of water delivered. It may be represented by ηu.

Example 3: Once cumec of water is pumped into a farm distribution system. 0.8 cumec is delivered to a turnout, 0.9 kilometres from the well. Compute the conveyance efficiency.

Solution: By definition

η_c = Output/ Input x 100 = 0.8/1.0. 100 = 80%

Example 4: 10 cumecs of water is delivered to a 32-hectare field, for 4 hours. Soil probing after the indicated that 0.3 metres of water has been stored in the root zone. Compute the water application efficiency.

Solution: Volume of water supplied by 10 cumecs of water applied for 4 hours =(10 x 4 x 60x 60)m^3 = 1,44,000 m^3

= 14.4 x10^4 m^3 = 14.4m x $10^4 m^2$ = 14. 4ha.m.

Depth of water applied =

volume/area = 1,44, 000/32,0, 000 = 144/320 = .45

Input = 14.4 ha.m

Output = 32 hectares land is storing water upto 0.3 m depth,

Output = 32 x0.3 ha.m = 9.6 ha.m

Water application efficiency (ηa) = Output/ Input x 100 = (9.6/14.4) x 100= 67%

(v) Uniformity coefficient or Water distribution efficiency:

The effectiveness of irrigation may also be measured by its water distribution efficiency (η_d), which is defined below:

η_d = (1-d/D)x100

Where η_d = Water distribution efficiency

D = Mean depth of water stored during irrigation.

d = Average of the absolute values of deviations from the mean.

The water distribution efficiency represents the extent to which the water has penetrated to a uniform depth, throughout the field. When the water has penetrated uniformly throughout the field, the deviation from the mean depth is zero and water distribution efficiency is 1.0.

Example 5: A stream of 130 litres per second was diverted from a canal and 100 litres per second were delivered to the field. An area of 1.6 hectares was irrigated in 8 hours. The effective depth of the root zone was 1.7 m. The runoff loss in the field was 420 cu. M. The depth of water penetration varied linearly from 1.7 m at the head end of the field to1.1 m at the tail end. Available moisture-holding capacity of the soil is 20 cm per metre depth of soil. It is required to determine the water conveyance efficiency, water application efficiency, water storage efficiency, and water distribution efficiency. Irrigation was started at a moisture extraction level of 50% of the available moisture.

Solution:

(i) Water conveyance efficiency (ηc)

=(Water delivered to the fields/ Water supplied into the canal at the head) x 100

= 100/130 x 100 =77%

(ii) Water application efficiency (ηa)

Water stored in the root zone during irrigation / Water delivered to the field x 100

Water supplied to field during 8 hours @ 100 litres per second

= 100x8 x60 x 60 litres = 2880 cu. m.

Runoff loss in the field = 420 cu. M.

the water stored in the root zone = 2880 –420 = 2460 cu. m.

(iii) Water application efficiency (ηa)

= 2460 /100 = 85.4% Ans. 2880

(iv) Water storage efficiency (ηs) = (Water stored in the root zone during irrigation /

Water needed in the root zone prior to irrigation) x 100

Moisture holding capacity of soil

= 20 cm per m depth x1.7 m depth of root zone = 34 cm

Moisture already available in the root zone at the time of start of irrigation

= 50/100 x 34 =17cm.

Additional water required in the root zone

= 34 –17 = 17 cm.

= 2720 cu. m.

But actual water stored in root zone = 2460 cu. m.

Water storage efficiency (ηs) =2460 /2720 x 100 90% (say)

(v) Water distribution efficiency

Where D = mean depth of water stored in the root zone

D = (1.7+1.1)/2 = 1.4m

d is computed as below:

Deviation from the mean at upper end (absolute value) = |1.7 -1.4| = 0.3

Deviation from the mean at lower end = | 1.1 -1.4 | =0.3

d = Average of the absolute values of deviations from mean = 0.4 +0.3/2 = 0.35

Using equations, we have,

$\eta_{d\,=}$ 75 or 75% Ans.

vi) Consumptive Use or Evapotranspiration (Cu)

Consumptive use for a particular crop may be defined as the total amount of water used by the plant in transpiration (building of plant tissues, etc.) and evaporation from adjacent soils or from plant leaves, in any specified time. The values of consumptive use (Cu) may be different for different crops, and may be different for the same crop at different times and places.

Effective Rainfall (Re)

Precipitation falling during the growing period of a crop that is available to meet the evapotranspiration needs of the crop is called effective rainfall. It does not include precipitation lost through deep percolation below the root zone or the water lost as surface runoff.

Consumptive Irrigation Requirement (CIR)

It is the amount of Irrigation water required in order to meet the evapotranspiration needs of the crop during its full growth. It is, therefore, nothing but the consumptive use itself, but exclusive of effective precipitation, stored soil moisture, or ground water. When the last two are ignored, then we can write

CIR = Cu-Re

Net Irrigation Requirement (NIR)

It is the amount of irrigation water required in order to meet the evapotranspiration need of the crop as well as other needs such as leaching. Therefore, N.I.R. = Cu –Re + Water lost as percolation in satisfying other needs such as leaching.

Estimation of Consumptive Use:

Although various methods have been developed in order to estimate evapotranspiration (consumptive use) value of a crop in an area, but the most simple and commonly used methods are:

(1) Blaney –Criddle Equation, and

(2) Hargreaves class A pan evaporation method

Blaney-Criddle Formula:

It sates that the monthly consumptive use is given by

$$C_u = K.(P/40\ [1.8t + 32])$$

where, C_u = Monthly consumptive use in cm.

k = Crop factor, determined by experiments for each crop, under the environmental conditions of the particular area.

t = Mean monthly temperature in °C

p = Monthly pet cent of annual day light hours that occur during the period.

If p/40 [1.8t +32]is represented by f, we get

Cu = k.f

Example: The monthly consumptive use values for Paddy are tabulated in Table. Determine the total consumptive use. What is the average monthly consumptive use and peak monthly consumptive use?

Table

Dates			Rice (Loam Soil) C_u in cm
June	1-30	Nursery	26.69
July	1-12	Nursery	8.76
July	13-31		14.38
August	1-31		22.73
September	1-30		21.29
October	1-31		25.50
November	1-24		15.06

Solution: The summation of consumptive uses

= 29.69+8.76+14.38+22.73+21.29+25.50+15.06 = 137.41 cm

Hence, total consumptive use for paddy = 137.41 cm.

Average daily consumptive use =

137.4/Period of growth in days =

= 137.41/31+31+31+30+31+24

= 137.41/177 = 0.77 cm. = 0.77x30=23.1 mm.

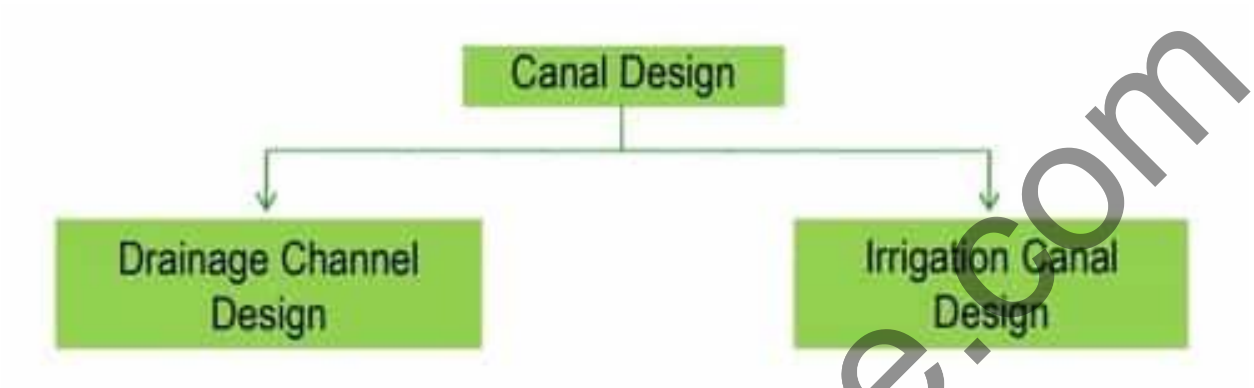
Average monthly consumptive use = 0.77 × 30 = 23.1 mm.

Peak monthly consumptive use = 26.69 cm. (Highest value is given)

Lacey, Kennedy and Design of Lined and Unlined Canal

Lacey's and Kennedy's Theory for Canal Design

Canal Design in General



Design Parameters

- The design considerations naturally vary according to the type of soil.
- The velocity of flow in the canal should be critical.
- Design of canals which are known as 'Kennedy's theory' and 'Lacey's theory' are based on the characteristics of sediment load (i.e. silt) in canal water.

Important Terms Related to Canal Design

- **Alluvial soil:** The soil which is formed by the continuous deposition of silt is known as alluvial soil.

- **Non-alluvial soil:** The soil which is formed by the disintegration of rock formations is known as non-alluvial soil.

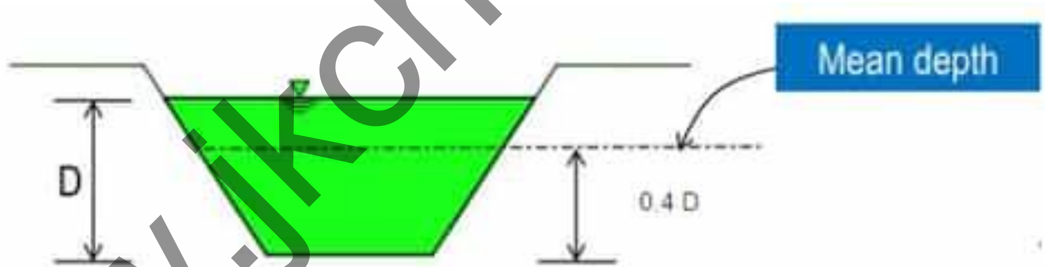
- **Silt factor:** During the investigations works in various canals in alluvial soil, Gerald Lacey established the effect of silt on the determination of discharge and the canal section. So, Lacey introduced a factor which is known as 'silt factor'. It depends on the mean particle size of silt. It is denoted by 'f'. The silt factor is determined by the expression,

$$f = 1.76\sqrt{d_{mm}}$$

where, d_{mm} = mean particle size of silt in mm

Particle	Particle size (mm)	Silt factor (f)
Very fine silt	0.05	0.40
Fine silt	0.12	0.60
Medium silt	0.23	0.85
Coarse silt	0.32	1.00

- **Co-efficient of rugosity:** The roughness of the canal bed affects the velocity of flow. The roughness is caused due to the ripples formed on the bed of the canal. So, a coefficient was introduced by R.G Kennedy for calculating the mean velocity of flow. This coefficient is known as the coefficient of rugosity and it is denoted by 'n'. The value of 'n' depends on the type of bed materials of the canal.
- **Mean velocity:** It is found by observations that the velocity at a depth 0.6D from surface represents the mean velocity (V), where 'D' is the depth of water in the canal or river.
-



(1) Mean velocity by Chezy's formula

$V = C\sqrt{RS}$

Where,

V = mean velocity in m/sec,

C = Chezy's constant,

R = hydraulic mean depth in m

S = bed slope of canal as 1 in *n*.

Again, the Chezy's constant C can be calculated by:

(a) Bazin's Formula:

$$C = \frac{87}{1 + \frac{K}{\sqrt{R}}}$$

Where,

K = Bazin's constant,

R = hydraulic mean depth

(b) Kutter's Formula:

$$C = \left[\frac{\frac{1}{n} + \left(23 + \frac{0.00155}{S}\right)}{1 + \left(23 + \frac{0.00155}{S}\right)\frac{n}{\sqrt{R}}} \right]$$

Where,

n = Co-efficient of rugosity,

S = bed slope,

R = hydraulic mean depth

- **Critical velocity:** When the velocity of flow is such that there is no silting or scouring action in the canal bed, then that velocity is known as critical velocity. It is denoted by 'Vo '. The value of Vo was given by Kennedy according to the following expression,

$V_o = C.D^n$

Where V_o = Critical velocity

D = Depth of channel

C & n = Constants

He found the values of C & n are <u>0.55</u> and <u>0.64</u>

Therefore **$V_o = 0.55 \times D^{0.64}$**

Later he found that critical velocity ratio has huge impact on Critical velocity and he incroporated some changes in above equation.

- **Critical velocity ratio (c.v.r), m:** The ratio of **mean velocity** 'V' to the **critical velocity** 'Vo ' is known as critical velocity ratio (CVR). It is denoted by m i.e.

CVR (m) = V/Vo

When m = 1, there will be no silting or scouring.

When m > 1, **scouring** will occur

When m < 1, **silting** will occur

So, by finding the value of m, the condition of the canal can be predicted whether it will have silting or scouring

- **Regime channel:** When the character of the bed and bank materials of the channel are same as that of the transported materials and when the silt charge and silt grade are constant, then the channel is said to be in its regime and the channel is called regime channel. This ideal condition is not practically possible.

- **Hydraulic mean depth:** The ratio of the cross-sectional area of flow to the wetted perimeter of the channel is known as hydraulic mean depth or radius. It is generally denoted by R.

R = A/P

Where,

A = Cross-sectional area

P = Wetted perimeter

Unlined Canal Design on *Non-alluvial* soil

The non-alluvial soils are stable and nearly impervious. For the design of canal in this type of soil, the coefficient of rugosity plays an important role, but the other factor like silt factor has no role. Here, the velocity of flow is considered very close to critical velocity. So, the mean velocity given by Chezy's expression or Manning's expression is considered for the design of the canal in this soil. The following formulae are adopted for the design.

(1) Mean velocity by Chezy's formula

$$V = C\sqrt{RS}$$

Where,

V = mean velocity in m/sec,
C = Chezy's constant,
R = hydraulic mean depth in m
S = bed slope of canal as 1 in *n*.

Again, the Chezy's constant C can be calculated by:

(a) Bazin's Formula:

$$C = \frac{87}{1+\frac{K}{\sqrt{R}}}$$

Where,

K = Bazin's constant,
R = hydraulic mean depth

(b) Kutter's Formula:

$$C = \left[\frac{\frac{1}{n}+\left(23+\frac{0.00155}{S}\right)}{1+\left(23+\frac{0.00155}{S}\right)\frac{n}{\sqrt{R}}}\right]$$

Where,

n = Co-efficient of rugosity,
S = bed slope,
R = hydraulic mean depth

(2) Mean velocity by Manning's formula

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

(3) Discharge by the following equations:

$$Q = A \times V$$

Where,

Q = discharge in cumec
A = cross-sectional area of water section in m^2
V = mean velocity in m/sec

Note:

- If value of K is not given, then it may be assumed as follows,

For unlined channel, K = 1.30 to 1.75.

For line channel, K = 0.45 to 0.85

- If the value of N is not given, then it may be assumed as follows,

For unlined channel, N = 0.0225

For lined channel, N = 0.333

Example 1:

Design an irrigation channel with the following data:
Discharge of the canal = 24 cumec
Permissible mean velocity = 0.80 m/sec.
Bed slope = 1 in 5000
Side slope = 1:1
Chezy's constant, C = 44

Solution:
We know, $A = 24/0.80 = 30\ m^2$

$30 = (B + D)D$

And $P = B + 2.828\ D$

But, $R = 30/(B + 2.828\ D)$

From Chezy's formula, $V = C\sqrt{RS}$

$$\Rightarrow 0.80 = 44 \times \sqrt{R \times 0.0002}$$

$$\therefore\ R = 1.65\ m$$

Putting the value of R and solving, D = 2.09 m and B = 12.27 m

Unlined Canal Design on Alluvial soil by Kennedy's Theory

R.G Kennedy arrived at a theory which states that the silt carried by flowing water in a channel is kept in suspension by the vertical component of eddy current which is formed over the entire bed width of the channel and the suspended silt rises up gently towards the surface.

The following **assumptions** are made in support of his theory:

1. The eddy current is developed due to the roughness of the bed.
2. The quality of the suspended silt is proportional to bed width.
3. It is applicable to those channels which are flowing through the bed consisting of sandy silt or same grade of silt.

He established the idea of critical velocity ‘Vo ’ which will make a channel free from silting or scouring. From, long observations, he established a relationship between the critical velocity and the full supply depth as follows,

$\mathbf{V_o = C\ D^n}$

The values of C and n where found out as 0.546 and 0.64 respectively, thus

$V_o = 0.546\ D^{0.64}$

Again, the realized that the critical velocity was affected by the grade of silt. So, he introduced another factor (m) which is known as critical velocity ratio (C.V.R).

$V_o = 0.546\ m\ D^{0.64}$

Drawbacks of Kennedy’s Theory

1. The theory is limited to average regime channel only.
2. The design of the channel is based on the trial and error method.
3. The value of m was fixed arbitrarily.
4. Silt charge and silt grade are not considered.
5. There is no equation for determining the bed slope and it depends on Kutter’s equation only.
6. The ratio of ‘B’ to ‘D’ has no significance in his theory.

Design Procedure

1. Critical velocity, $V_o = 0.546\ m\ D^{0.64}$.
2. Mean velocity, $V = C\ (RS)^{1/2}$

Where,

m = critical velocity ratio,

D = full supply depth in m,

R = hydraulic mean depth of radius in m,

S = bed slope as 1 in ‘n’.

The value of ‘C’ is calculated by Kutter’s formula

$$C = \left[\frac{\frac{1}{n} + \left(23 + \frac{0.00155}{S}\right)}{1 + \left(23 + \frac{0.00155}{S}\right)\frac{n}{\sqrt{R}}} \right]$$

Where,

n = rugosity coefficient which is taken as an unlined earthen channel.

3. B/D ratio is assumed between 3.5 to 12.

4. Discharge, Q = A*V

Where,

A = Cross-section area in m^2,

V = mean velocity in m/sec

5. The full supply depth is fixed by trial to satisfy the value of 'm'. Generally, the trial depth is assumed between 1 m to 2 m. If the condition is not satisfied within this limit, then it may be assumed accordingly.

Example 2:

Design an irrigation channel with the following data:
Full supply discharge = 6 cumec
Rugosity coefficient (n) = 0.0225
C.V.R (m) = 1
Bed slope = 1 in 5000
Assume other reasonable data for the design

Solution:

First Trial:

Assume, Full supply depth, D = 1.5 m

Critical velocity, $V_o = 0.546 \times 1 \times 1.5^{0.64} = 0.707$ [Assume, m = 1]

As m = 1, $V = V_o$

$\therefore A = 6/0.707 = 8.49$ m

$A = (2B + 3)/2 \times 1.5 = 1.5B + 2.25 \Rightarrow B = 4.16$ m

$P = B + 2\sqrt{2} \times 1.5 = B + 4.24 = 8.40$ m

$R = 4.16/8.40 = 1.0$ m

By Kutter's formula, $C = \left[\dfrac{\dfrac{1}{0.0225} + \left(23 + \dfrac{0.00155}{0.0002}\right)}{1 + \left(23 + \dfrac{0.00155}{0.0002}\right)\dfrac{0.0225}{\sqrt{1.0}}}\right] = 44.49$

By Chezy's formula, $V = 44.49 \times \sqrt{(1 \times 0.0002)} = 0.629$ m/sec

C.V.R = 0.629/0.707 = 0.889 < 1

As the C.V.R is much less than 1, the channel will be in silting. So, the design is not satisfactory. Here, the full supply depth is to be assumed by trials to get the satisfactory result.

Second Trial:

Assume, Full supply depth, D = 1.25 m

Critical velocity, $V_o = 0.546 \times 1 \times 1.25^{0.64} = 0.629$ [Assume, m = 1]

As m = 1, $V = V_o$

$\therefore A = 6/0.629 = 9.53$ m

$A = 1.25B + 1.56 \Rightarrow B = 6.38$ m

$P = B + 2\sqrt{2} \times 1.25 = 9.92$ m

R = 0.96 m

By Kutter's formula, $C = \left[\dfrac{\dfrac{1}{0.0225}+\left(23+\dfrac{0.00155}{0.0002}\right)}{1+\left(23+\dfrac{0.00155}{0.0002}\right)\dfrac{0.0225}{\sqrt{0.96}}}\right] = 44.23$

By Chezy's formula, $V = 44.23\times\sqrt{(1\times0.0002)} = 0.613$ m/sec

C.V.R = 0.613/0.629 = 0.97 < 1

In this case, the CVR is very close to 1. So, the design may be accepted. So, finally,

D = 1.25 m and B = 6.28 m

Try Yourself

Problem – 1

Find the maximum discharge through an irrigation channel having the bed width 4 m and fully supply depth is 1.50 m. Given that n = 0.02, S = 0.0002, side slope = 1:1

Assume reasonable data, if necessary. Comment whether the channel will be in scouring or silting.

Problem – 2

Design an irrigation channel with the following data:
Full supply discharge = 10 cumec
Bazin's constant, K = 1.3
C.V.R (m) = 1
B/D ratio = 4
Side slope = 1:1
Assume other reasonable data for the design

Unlined Canal Design on *Alluvial soil* by Lacey's Theory

Lacey's theory is based on the concept of regime condition of the channel. The regime condition will be satisfied if,

- The channel flows uniformly in unlimited incoherent alluvium of the same character which is transported by the channel.
- The silt grade and silt charge remains constant.

- The discharge remains constant.

In his theory, he states that the silt carried by the flowing water is kept in suspension by the vertical component of eddies. The eddies are generated at all the points on the wetted perimeter of the channel section. Again, he assumed the hydraulic mean radius R, as the variable factor and he recognized the importance of silt grade for which in introduced a factor which is known as silt factor 'f'.

Thus, he deduced the velocity as;

$V = (2/5f\ R)^{0.5}$

Where,

V = mean velocity in m/sec,

f = silt factor,

R = hydraulic mean radius in meter

Then he deduced the relationship between A, V, Q, P, S and f are as follows:

- $f = 1.76 \times \sqrt{d_{mm}}$
- $Af^2 = 140 \times V^5$
- $V = \left(\frac{Q \times f^2}{140}\right)^{1/6}$
- $P = 4.75 \times \sqrt{Q}$
- Regime flow equation, $V = 10.8 \times R^{2/3} \times S^{1/3}$

Regime scour depth

$$R = 1.35\left(\frac{q^2}{f}\right)^{1/3}$$

$$Bed\ slope, S = \frac{f^{5/3}}{3340Q^{1/6}}$$

Example 3:

Design an irrigation channel with the following data:

Full supply discharge = 10 cumec

Mean diameter of silt particles = 0.33 mm

Side slope = ½:1

Find also the bed slope of the channel

Solution:

$f = 1.76 \times \sqrt{0.33} = 1.0$ and $V = \left(\dfrac{Q \times 1^2}{140}\right)^{1/6} = 0.64$ m/sec

$A = 10/0.64 = 15.62\ m^2$

$P = 4.75 \times \sqrt{10} = 15.02$ m

$R = 0.47 \times (10/1)^{1/3} = 1.02$ m

$S = \dfrac{1^{5/2}}{3340 \times 10^{1/6}} = 1/4902$

But, $A = BD + 0.5\ D^2$

$\Rightarrow 15.62 = BD + 0.5\ D^2$ ---------------- (i)

$P = B + \sqrt{5}D$

$15.02 = B + 2.24\ D$ ---------------------- (ii)

Solving equation (i) & (ii) D = 1.21 m and B = 12.30 m

Drawbacks of Lacey's Theory

1. The concept of the true regime is theoretical and can not be achieved practically.
2. The various equations are derived by considering the silt factor f which is not at all constant.
3. The concentration of silt is not taken into account.
4. Silt grade and silt charge is not taken into account.
5. The equations are empirical and based on the available data from a particular type of channel. So, it may not be true for a different type of channel.
6. The characteristics of regime channel may not be the same for all cases.

Comparison between Kennedy's and Lacey's theory

Kennedy's Theory	**Lacey's theory**

It states that the silt carried by the flowing water is kept in suspension by the vertical component of eddies which are generated from the bed of the channel.	It states that the silt carried by the flowing water is kept in suspension by the vertical component of eddies which are generated from the entire wetted perimeter of the channel.
It gives relation between 'V' and 'D'.	It gives relation between 'V' and 'R'.
In this theory, a factor known as critical velocity ratio 'm' is introduced to make the equation applicable to different channels with different silt grades	In this theory, a factor known as silt factor 'f' is introduced to make the equation applicable to different channels with different silt grades.
In this theory, Kutter's equation is used for finding the mean velocity.	This theory gives an equation for finding the mean velocity.
This theory gives no equation for bed slope.	This theory gives an equation for bed slope.
In this theory, the design is based on trial and error method.	This theory does not involve trial and error method.

Design of Lined Canal

The lined canals are not designed by the use of Lacey's and Kennedy's theory, because the section of the canal is rigid. Manning's equation is used for designing. The design considerations are,

- The section should be economical (i.e. cross-sectional area should be maximum with minimum wetted perimeter).
- The velocity should be maximum so that the cross-sectional area becomes minimum.
- The capacity of the lined section is not reduced by silting.

Section of Lined Canal:

The following two lined sections are generally adopted:

- **Circular section:** The bed is circular with its center at the full supply level and radius equal to full supply depth 'D'. The sides are tangential to the curve. However, the side slope is generally taken as 1:1.

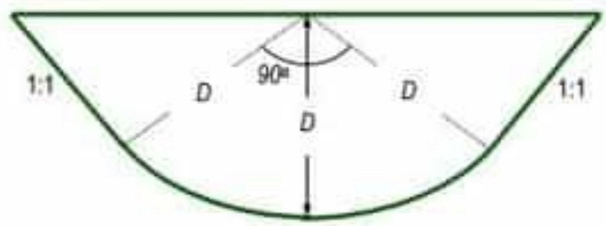


Table – 1:Design parameters for circular section

Design parameter	Side slope		
	1:1	1.5:1	1.25:1
Sectional area (A)	$1.785 \times D^2$	$2.088 \times D^2$	$1.925 \times D^2$
Wetted perimeter (P)	$3.57 \times D$	$4.176 \times D$	$3.85 \times D$
Hydraulic mean depth or radius (R)	$0.5 \times D$	$0.5 \times D$	$0.5 \times D$
Velocity (V)	$V = (1/n) \times R^{2/3} \times S^{1/2}$	–	–
Discharge (Q)	$A \times V$	–	–

- **Trapezoidal section:** The horizontal bed is joined to the side slope by a curve of radius equal to full supply depth D. The side slope is generally kept as 1:1.

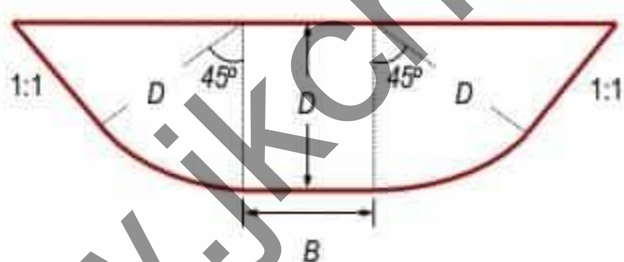


Table – 2:Design parameters for circular section

Design parameter	Side slope		
	1:1	1.5:1	1.25:1
Sectional area (A)	$BD + 1.785 \times D^2$	$BD + 2.088 \times D^2$	$BD + 1.925 \times D^2$
Wetted perimeter (P)	$B + 3.57 \times D$	$B + 4.176 \times D$	$B + 3.85 \times D$
Hydraulic mean depth or radius (R)	A/P	A/P	A/P
Velocity (V)	$V = (1/n) \times R^{2/3} \times S^{1/2}$	–	–
Discharge (Q)	$A \times V$	–	–

Note: For the discharge up to 50 cumec, the circular section is suitable and for the discharge above 50 cumec trapezoidal section is suitable.

Example 4:

Design a lined canal having the following data:

Full supply discharge = 200 cumec
Side slope = 1.25:1
Bed slope = 1 in 5000
Rugosity coefficient = 0.018
Permissible velocity = 1.75 cumec

Solution:

Since the discharge is more than 50 cumec, the trapezoidal section will be acceptable.

From table – 2,

Sectional area, $A = BD + 1.925\ D^2$ ------------- (i)

Wetted perimeter, $P = B + 3.85\ D$ ------------ (ii)

Now, $A = 200/1.75 = 114.28\ m^2$

Again, $V = (1/n)\times R^{2/3}\times S^{1/2}$

$\Rightarrow 1.75 = (1/0.018)\times R^{2/3}\times (0.0002)^{1/2}$

$\therefore\ R = 3.32$ m

$P = 114.28/3.32 = 34.42$ m

From equation (i) $\Rightarrow 114.28 = BD + 1.925\times D^2$

From equation (ii) $\Rightarrow 34.42 = B + 3.85\times D$

Solving equation (i) & (ii) $\Rightarrow D = 4.4$ m (Full supply depth)

$\therefore$ B = 17.5 m (Bed width)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