SUBJECT:HYDRAULIC IRRIGATION ENGINEERING

COURSE:DIPLOMA

BRANCH:CIVIL

SEMESTER:4TH

THEORY:2

LECTURE NOTES PREPARED BY

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D. COURSE CONTENTS:

PART: A (Hydraulics)

1 HYDROSTATICS:

1.1 Properties of fluid: density, specific gravity, surface tension, capillarity,

viscosity and their uses

1.2 Pressure and its measurements: intensity of pressure, atmospheric

pressure, gauge pressure, absolute pressure and vacuum pressure; relationship

between atmospheric pressure, absolute pressure and gauge pressure; pressure

head; pressure gauges.

1.3 Pressure exerted on an immersed surface: Total pressure, resultant

pressure, expression for total pressure exerted on horizontal & vertical surface.

2 KINEMATICS OF FLUID FLOW:

2.1 Basic equation of fluid flow and their application: Rate of discharge,

equation of continuity of liquid flow, total energy of a liquid in motion- potential,

kinetic & pressure, Bernoulli's theorem and its limitations. Practical applications of

Bernoulli's equation.

2.2 Flow over Notches and Weirs: Notches, Weirs, types of notches and weirs,

Discharge through different types of notches and weirs-their application (No

Derivation)

2.3 Types of flow through the pipes: uniform and non uniform; laminar and

turbulent; steady and unsteady; Reynold's number and its application

2.4 Losses of head of a liquid flowing through pipes: Different types of major

and minor losses. Simple numerical problems on losses due to friction using

Darcy's equation, Total energy lines & hydraulic gradient lines (Concept Only).

2.5 Flow through the Open Channels: Types of channel sections-rectangular,

trapezoidal and circular, discharge formulae- Chezy's and Manning's equation,

Best economical section.

3 PUMPS:

3.1 Type of pumps

3.2 Centrifugal pump: basic principles, operation, discharge, horse power &

efficiency.

3.3 Reciprocating pumps: types, operation, discharge, horse power & efficiency

PART: B (Irrigation Engineering)

1 Hydrology

1.1 Hydrology Cycle

1.2 Rainfall: types, intensity, hyetograph

1.3 Estimation of rainfall, rain gauges, Its types(concept only),

1.4 Concept of catchment area, types, run-off, estimation of flood discharge by

Dicken's and Ryve's formulae

2 Water Requirement of Crops

2.1 Definition of irrigation, necessity, benefits of irrigation, types of irrigation

2.2 Crop season

2.3 Duty, Delta and base period their relationship, overlap allowance, kharif

and rabi crops

2.4 Gross command area, culturable command area, Intensity of Irrigation,

irrigable area, time factor, crop ratio

3 FLOW IRRIGATION

3.1 Canal irrigation, types of canals, loss of water in canals

3.2 Perennial irrigation

3.3 Different components of irrigation canals and their functions

3.4 Sketches of different canal cross-sections

3.5 Classification of canals according to their alignment, Various types of canal

lining – Advantages and disadvantages

4 WATER LOGGING AND DRAINAGE :

4.1 Causes and effects of water logging, detection, prevention and remedies

5 DIVERSION HEAD WORKS AND REGULATORY STRUCTURES

5.1 Necessity and objectives of diversion head works, weirs and barrages

5.2 General layout, functions of different parts of barrage

5.3 Silting and scouring

5.4 Functions of regulatory structures

6 CROSS DRAINAGE WORKS :

6.1 Functions and necessity of Cross drainage works - aqueduct, siphon, super passage, level crossing

6.2 Concept of each with help of neat sketch

7 DAMS

7.1 Necessity of storage reservoirs, types of dams

7.2 Earthen dams: types, description, causes of failure and protection measures.

7.3 Gravity dam- types, description, Causes of failure and protection measures.

7.4 Spillways- Types (With Sketch) and necessity.

Chapter-I

HYDROSTATICS

Hydrostatic is that branch of science which relating to fluids at rest or to the pressures they exert or transmit **Hydrostatic Pressure**.

Fluid:-

Fluid is a substance that continuously deforms (flows) under an applied shear stress. Fluids are a subset of the phase of matter and include liquids, gases, plasmas and, to some extent, plastic solids. Fluids can be defined as substances which have zero shear modulus or in simpler terms a fluid is a substance which cannot resist any shear force applied to it.

- Fluid is a substance which is capable of flowing
- Conform the shape of the containing vessel
- Deform continuously under application of small shear force

1.1 PROPERTIES OF FLUID:-

Density:-

The density of a fluid, is generally designated by the Greek symbol $\rho(rho)$ is defined as the mass of the fluid over a unit volume of the fluid at standard temperature and pressure. It is expressed in the SI system as kg/m^3.

$$\rho = \lim \frac{\Delta m}{\Delta V} = \frac{dm}{dV}$$

If the fluid is assumed to be uniformly dense the formula may be simplified as:

$$\rho = \frac{m}{V}$$

Example: - setting of fine particles at the bottom of the container.

Specific Weight:-

The specific weight of a fluid is designated by the Greek symbol γ(gamma), and is generally defined as the weight per unit volume of the fluid at standard temperature and pressure. In SI systems the units is N/m^3.

$$\lambda = \rho * g$$

g = local acceleration of gravity and ρ = density

Note: It is customary to use:

$g = 32.174\ ft/s^2 = 9.81\ m/s^2$

$\rho = 1000\ kg/m^3$

Relative Density (Specific Gravity):-

The relative density of any fluid is defined as the ratio of the density of that fluid to the density of the standard fluid. For liquids we take water as a standard fluid with density $\rho=1000\ kg/m^3$. For gases we take air or O_2 as a standard fluid with density, $\rho=1.293\ kg/m^3$.

Specific volume:-

Specific volume is defined as the volume per unit mass. It is just reciprocal of mass density. It is expressed in m^3/kg.

Viscosity:-

Viscosity (represented by μ, Greek letter mu) is a material property, unique to fluids, that measures the fluid's resistance to flow. Though a property of the fluid, its effect is understood only when the fluid is in motion. When different elements move with different velocities, each element tries to drag its neighboring elements along with it. Thus, shear stress occurs between fluid elements of different velocities.

Viscosity is the property of liquid which destroyed the relative motion between the layers of fluid.

- It is the internal friction which causes resistance to flow.
- Viscosity is the property which control the rate of flow of liquid

Viscosity is due to two factors-

a) Cohesion between the liquid molecules.

b) Transfer of momentum between the molecules.

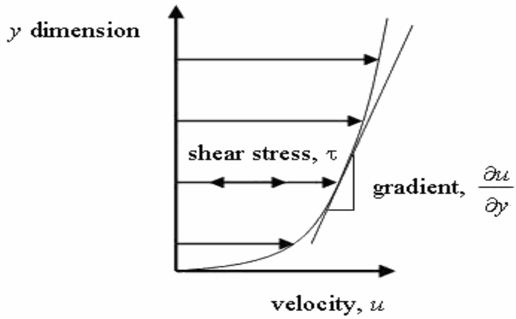


Fig. 1.1

The relationship between the shear stress and the velocity field was that the shear stresses are directly proportional to the velocity gradient. The constant of proportionality is called the coefficient of dynamic viscosity.

$$\tau = \mu \frac{\partial u}{\partial y}$$

UNIT OF VISCOSITY

- In mks system unit of viscosity is kgf-sec/m^2
- In cgs system unit of viscosity is dyne-sec/cm^2
- In S.I system unit of viscosity is Newton-sec/m^2

Kinematic viscosity:-

Another coefficient, known as the kinematic viscosity (ν, Greek nu) is defined as the ratio of dynamic viscosity and density.

I.et, $\upsilon = \mu/\rho$ = viscosity/density

In mks & S.I system unit of kinematic viscosity is meter^2/sec

In cgs system unit of kinematic viscosity is stoke.

SURFACE TENSION:-

Surface tension is defined as the tensile force acting on the surface of a liquid in contact with a gas or on the surface between two immiscible liquids such that the contact surface behaves like a membrane under tension. The magnitude of this force per unit length of the free surface will have the same value as the surface energy per unit area. It is denoted by Greek letter sigma(σ). In MKS units, it is expressed as kgf/m while in SI unit is N/m.

It is also defined as force per unit length, or of energy per unit area. The two are equivalent—but when referring to energy per unit of area, people use the term surface energy—which is a more general term in the sense that it applies also to solids and not just liquids.

Capillarity:-

Capillarity is defined as a phenomenon of rise or fall of a liquid surface in a small tube relative to the adjacent general level of liquid when the tube is held vertically in the liquid. The rise of liquid surface is known as capillary rise while the fall of the liquid surface is known as capillary depression. It is expressed in terms of cm or mm of liquid. Its value depends upon the specific weight of the liquid, diameter of the tube and surface tension of the liquid.

1.2 Pressure and its measurement:-

INTENSITY OF PRESSURE:-

Intensity of pressure is defined as normal force exerted by fluid at any point per unit area. It is also called specific pressure or hydrostatic pressure

P=df/da

- If intensity of pressure is uniform over an area “A” then pressure force exerted by fluid equal to

  Mathematically F=PA

- If intensity of pressure is not uniform or vary point to point then pressure force exerted by fluid equal to integration of P*A

  Mathematically F=∫PA

- Unit of pressure

  - $1N/m^2$ =1 Pascal
  - $1KN/m^2$= 1 kilo Pascal
  - Kilo Pascal= 1kpa = 10^3 Pascal
  - 1 bar = 10^5 Pascal = 10^5 N/m^2

Pascal’s law:-

It states that the pressure or intensity of pressure at a point in a static fluid is equal in all direction.

Atmospheric Pressure:-

The atmospheric air exerts a normal pressure upon all surface with which it is in contact and it is called atmospheric pressure. It is also called parametric pressure.

Atmospheric pressure at the sea level is called standard atmospheric pressure.

S.A.P = 101.3 KN/m^2 = 101.3 kpa = 10.3m of H_2O

= 760 mm of Hg

=10.3 (milli bar)

Gauge pressure:-

It is the pressure which measure with help of pressure measuring device in which atmospheric pressure taken as datum.

The atmospheric pressure on scale is marked as zero.

Absolute pressure:-

Any pressure measure above absolute zero pressure is called absolute pressure.

Vacuum pressure:-

Vacuum pressure is defined as the pressure below the atmospheric pressure.

RELATIONSHIP BETWEEN ABSOLUTE PRESSURE, GAUGE PRESSURE, VACUUM PRESSURE:-

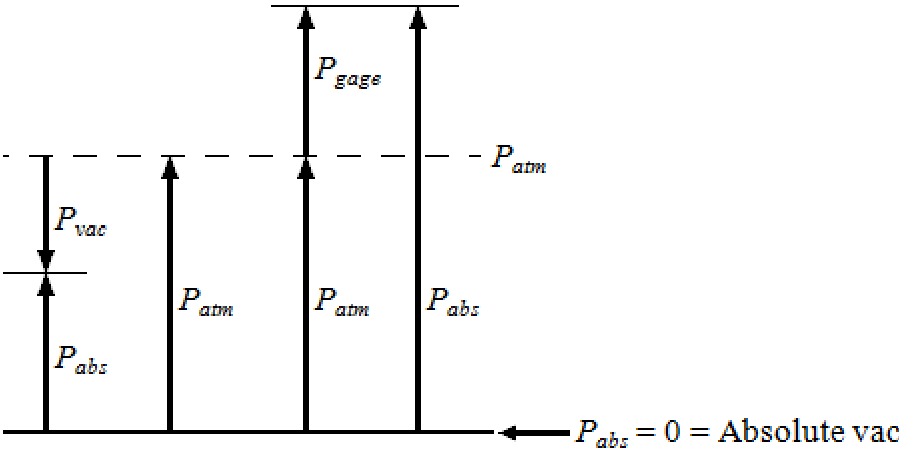


Fig. 1.2

❖ Equations

$P_{gage} = P_{abs} - P_{atm}$	gauge pressure
$P_{vac} = P_{atm} - P_{abs}$	vacuum pressure
$P_{abs} = P_{atm} + P_{gage}$	absolute pressure

❖ Nomenclature

P_{abs}	absolute pressure
P_{gage}	gage pressure
P_{vac}	vacuum pressure
P_{atm}	atmospheric pressure

Pressure Head:-

pressure head is the internal energy of a fluid due to the pressure exerted on its container. It may also be called **static pressure head** or simply **static head** (but not **static head pressure**). It is mathematically expressed as:

$$\psi = \frac{p}{\gamma} = \frac{p}{\rho g}$$

where

- ψ is pressure head (Length, typically in units of m);
- p is fluid pressure (force per unit area, often as Pa units); and
- γ is the specific weight (force per unit volume, typically N/m^3 units)
- ρ is the density of the fluid (mass per unit volume, typically kg/m^3)
- g is acceleration due to gravity (rate of change of velocity, given in m/s^2)

If intensity of pressure express in terms of height of liquid column, which causes pressure is also called pressure head.

Mathematically, $h = P/w$

Pressure Gauges :-

The pressure of a fluid is measured by the following devices:-

1. manometers
2. mechanical gauges

Manometers:-Manometers are defined as the devices used for measuring the pressure at a point in a fluid by balancing the column of fluid by the same or another column of the fluid. They are classified as:

a) Simple manometers

b) Differential manometer

Mechanical gauges:-mechanical gauges are defined as the devices used for measuring the pressure by balancing the fluid column by the spring or dead weight. The commonly used mechanical gauges are:-

a) Diaphragm pressure gauge

b) Bourdon tube pressure gauge

c) Dead weight pressure gauge

d) Bellows pressure gauge

1.3 PRESSURE EXERTED ON IMMERSED SURFACE:-

Hydrostatic forces on surfaces:-

Hydrostatic means the study of pressure exerted by a liquid at rest. The direction of such pressure is always perpendicular to the surface to which it acts.

Forces on Submerged Surfaces in Static Fluids

These are the following features of statics fluids:-

- Hydrostatic vertical pressure distribution
- Pressures at any equal depths in a continuous fluid are equal
- Pressure at a point acts equally in all directions (Pascal's law).
- Forces from a fluid on a boundary acts at right angles to that boundary.

Fluid pressure on a surface:-

Pressure is defined as force per unit area. If a pressure p acts on a small area δA then the force exerted on that area will be

$$F = p\delta A$$

TOTAL PRESSURE:-

Total pressure is defined as the force exerted by a static fluid on a surface when the fluid comes in contact with the surface.

Mathematically **total pressure**,

$P = p_1 a_1 + p_2 a_2 + p_3 a_3 \ldots\ldots\ldots\ldots$

Where,

- p_1, p_2, p_3 = Intensities of pressure on different strips of the surface, and
- a_1, a_2, a_3 = Areas of corresponding strips.

The position of an immersed surface may be,

- Horizontal
- Vertical
- Inclined

Total Pressure On A Horizontal Immersed Surface

Consider a plane horizontal surface immersed in a liquid as shown in figure 1.

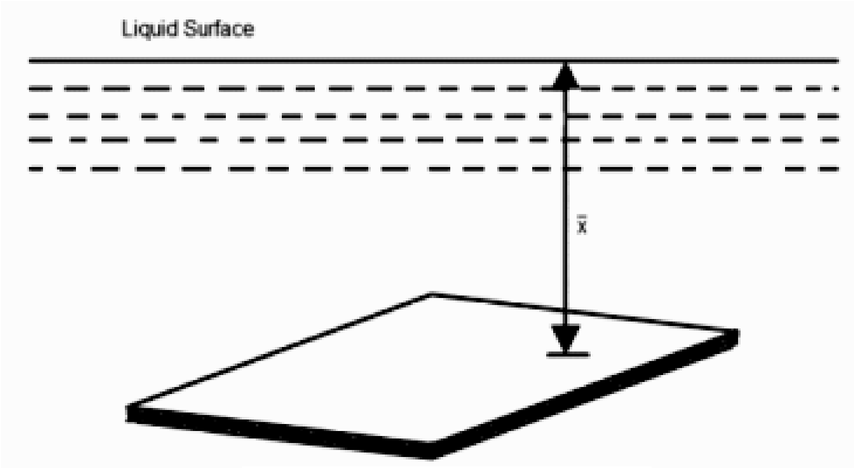


Fig. 1.3

- ω = Specific weight of the liquid
- A = Area of the immersed surface in in^2
- χ = Depth of the horizontal surface from the liquid level in meters

We know that the **Total pressure** on the surface,

P = Weight of the liquid above the immersed surface

= Specific weight of liquid * Volume of liquid

= Specific weight of liquid * Area of surface * Depth of liquid

= $\omega A \chi kN$

Total Pressure On A Vertically Immersed Surface

Consider a plane vertical surface immersed in a liquid shown in figure 2.

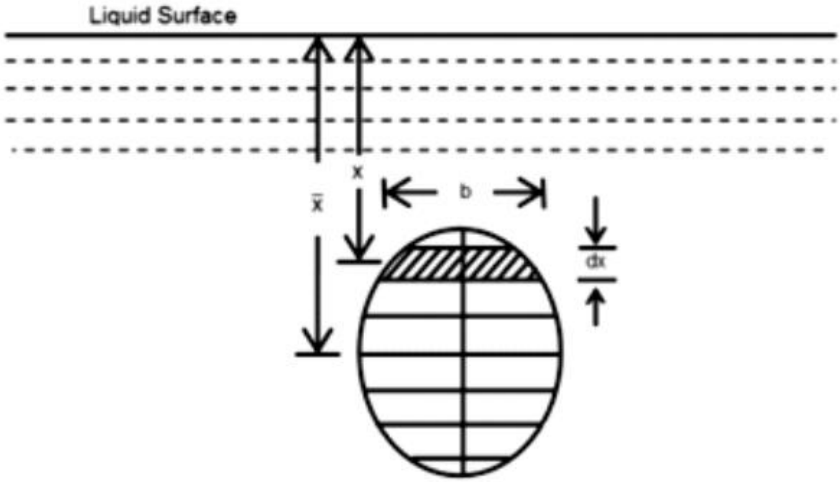


Fig. 1.4

Let the whole immersed surface is divided into a number of small parallel stripes as shown in figure.

Here,

- ω = Specific weight of the liquid
- A = Total area of the immersed surface
- χ = Depth of the center of gravity of the immersed surface from the liquid surface

Now, consider a strip of thickness *dx*, width *b* and at a depth *x* from the free surface of the liquid.

The intensity of pressure on the strip = $\omega\chi$

and the area of strip =b.d*x*

∴Pressure on the strip = Intensity of pressure * Area = $\omega\chi$ .bd*x*

Now, Total pressure on the surface,

$$P = \int wx.bdx\,.$$

$$= w\int x.bdx$$

But, $w\int x.bdx$ = Moment of the surface area about the liquid level = $A\bar{x}$

$$\therefore P = wA\bar{x}$$

1.4 FLOTATION AND BUOYANCY:-

Archimedes Principle:-

Archimedes' principle indicates that the upward buoyant force that is exerted on a body immersed in a fluid, whether fully or partially submerged, is equal to the weight of the fluid that the body displaces. Archimedes' principle is a law of physics fundamental to fluid mechanics. Archimedes of Syracuse formulated this principle, which bears his name.

Buoyancy:-

When a body is immersed in a fluid an upward force is exerted by the fluid on the body. This is upward force is equal to weight of the fluid displaced by the body and is called the force of buoyancy or simple buoyancy.

Centre of pressure:-

The center of pressure is the point where the total sum of a pressure field acts on a body, causing a force to act through that point. The total force vector acting at the center of pressure is the value of the integrated pressure field. The resultant force and center of pressure location produce equivalent force and moment on the body as the original pressure field.

Pressure fields occur in both static and dynamic fluid mechanics. Specification of the center of pressure, the reference point from which the center of pressure is referenced, and the associated force vector allows the moment generated about any point to be computed by a translation from the reference point to the desired new point. It is common for the center of pressure to be located on the body, but in fluid flows it is possible for the pressure field to exert a moment on the body of such magnitude that the center of pressure is located outside the body.

Center of buoyancy:-

It is define as the point through which the force of buoyancy is supposed to act. As the force of buoyancy is a vertical force and is equal to the weight of the fluid displaced by the body, the center of buoyancy will be the center of gravity of the fluid displaced.

METACENTER:-

The metacentric height (GM) is a measurement of the initial static stability of a floating body. It is calculated as the distance between the centre of gravity of a ship and its metacentre. A larger metacentric height implies greater initial stability against overturning. Metacentric height also has implication on the natural period of rolling of a hull, with very large metacentric heights being associated with shorter periods of roll which are uncomfortable for passengers. Hence, a sufficiently high but not excessively high metacentric height is considered ideal for passenger ships.

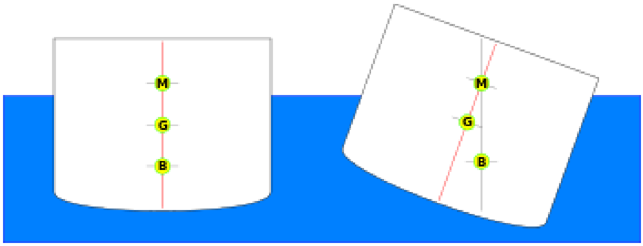


Fig. 1.5

The metacentre can be calculated using the formulae:

$$KM = KB + BM$$

$$BM = \frac{I}{V}$$

Metacentric height:-

The distance between the meta-center of a floating body and a center of gravity of the body is called metacentric height.

MG =BM-BG

MG=I/V-BG

Stability of a submerged body:-

Stable condition:-

- ❖ For stable condition w = f_b and the point "B" above the CG of the body.

Unstable equilibrium;-

- ❖ For unstable equilibrium w = f_b and the point B is below the CG of the body.

Neutral equilibrium:-

- ❖ If the force of buoyancy is act as CG of the body.

Stability of a floating body:-

- ❖ For stable condition w = f_b and the meta centre *"m"* is about the CG of the body.
- ❖ For unstable equilibrium w = f_b and the metacentre *"m"* is below CG of the body.

- In neutral equilibrium $w = f_b$ and metacentre "m" is acting at CG of the body.

Chapter-II

KINEMATICS OF FLUID FLOW

2.1 Basic equation of fluid flow and their application:-

Energy of a Liquid in Motion:-

The energy, in general, may be defined as the capacity to do work. Though the energy exits in many forms, yet the following are important from the subject point of view:

1. Potential energy,
2. Kinetic energy, and
3. Pressure energy.

Potential Energy of a Liquid Particle in Motion:-

It is energy possessed by a liquid particle by virtue of its position. If a liquid particle is *Z m*

above the horizontal datum (arbitrarily chosen), the potential energy of the particle will be *Z* metre-kilogram (briefly written as mkg) per kg of the liquid. The potential head of the liquid, at point, will be *Z* metres of the liquid.

Kinetic Energy of a Liquid Particle in Motion:-

It is the ehergy, possessed by a liquid particle, by virtue of its motion or velocity. If a liquid particle is flowing with a mean velocity of v metres per second; then the kinetic energy of the particle will be $V^2/2g$ mkg per kg of the liquid. Velocity head of the liquid, at that velocity, will be $V^2/2g$ metres of the liquid.

Pressure Energy of a Liquid Particle in Motion:-

It is the energy, possessed by a liquid particle, by virtue of its existing pressure. If a liquid particle is under a pressure of p kN/m2 (i.e., kPa), then the pressure energy of the particle-will be $\frac{p}{w.}$ mkg per kg of the liquid, where w is the specific weight of the liquid. Pressure head of the liquid

under that pressure will be $\frac{p}{w.}$ metres of the liquid.

Total Energy of a Liquid Particle in Motion:-

The total energy of a liquid, in motion, is the sum of its potential energy, kinetic energy and pressure energy, Mathematically total energy,

$E = Z + V^2/2g + \frac{p}{w.}$ m of Liquid.

Total Head of a Liquid Particle in Motion:-

The total head of a liquid particle, in motion, is the sum of its potential head, kinetic head and pressure head. Mathematically, total head,

$H = Z + V^2/2g + \frac{p}{w.}$ m of liquid.

Example

Water is flowing through a tapered pipe having end diameters of 150 mm and 50 mm respectively. Find the discharge at the larger end and velocity head at the smaller end, if the velocity of water at the larger end is 2 m/s. Solution. Given: d_1 = 150mm = 0•15 m ; d_2= 50 mm= 0•05 m and V_1 = 2•5 m/s. Discharge at the larger end We know that the cross-sectional area of the pipe at the larger end,

$a_1 = \frac{\pi}{4} \times (0.15)2 = 17.67 \times 10^{-3} m^2$

and discharge at the larger end,

$Q_1 = a_1.v_1 \quad = (17.67xl0^{-3})x2.5 \quad = 44.2 \text{ x } 10^{-3}\ m^3/s$

= 44.2Jitres/s Ans.

Velocity head at the smaller end

We also know that the cross-sectional area of the pipe at the smaller end,

$A_2 = \frac{\pi}{4} \times (0.15)2 = 1.964 \times 10^{-3} m^2$

Since the discharge through the pipe is continuous, therefore

$a_1.v_1 = a_2.v_2$

or $\quad v2 = \dfrac{a1.v1}{a2} = [(17.67 \times 10^{-3}) \times 2.5]/1.964 \times 10^{-3} = 22.5\ m/s$

:. Velocity head at the smaller end

$V_2^2/2g=(22.5)^2/2 \times 9.81=25.8\ m$ *Ans*

Bernoulli's Equation:-

It states, "For a perfect incompressible liquid, flowing in a continuous stream, the total nergy; of a particle remains the same, while the particle moves from one point to another." This statement is based on the assumption that there are no "losses due to friction in the pipe. Mathematically,

$$Z + V2/2g + \frac{p}{w.} = \text{Constant}$$

where

Z = Potential energy,

$V^2/2g$ =Kinetic energy, and

$\frac{p}{w.}$ = Pressure energy.

Proof

Consider a perfect incompressible liquid, flowing through a non-uniform pipe as shown in Fig-

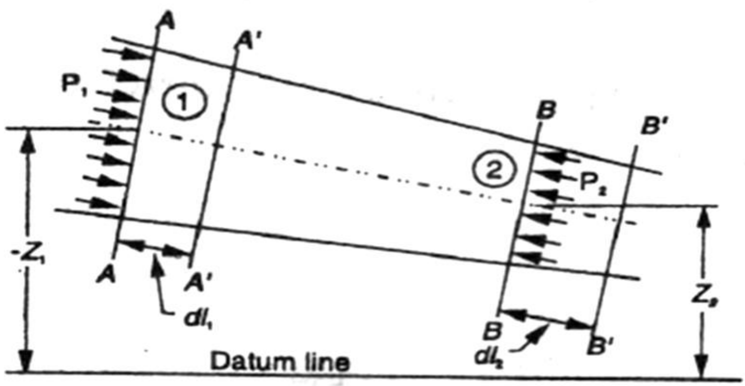


Fig. 2.1

Let us consider two sections AA and BB of the pipe. Now let us assume that the pipe is running full and there is a continuity of flow between the two sections.

Let
Z_1 = Height of AA above the datum,
P_1= Pressure at AA,
V_1 = Velocity of liquid at AA,
A_1 = Cross-sectional area of the pipe at AA, and
Z_2,P_2,V_2,A_2= Corresponding values at BB.
Let the liquid between the two sections AA and BB move to A' A' and B' B' through very small lengths dl_1 and dl_2 as shown in Fig. This movement of the liquid between AA and BB is equivalent to the movement 'of the liquid between AA and A' A' to BB and B' B', the remaining liquid between A' A' and BB being uneffected.
Let W be the weight of the liquid between AA and A' A'. Since the flow is continuous, therefore
$W = wa_1dI_1 = wa_2dL_2$

or $a_1 \times dl_1 = \frac{W}{w}$ ...(i)

Similarly $a_2dl_2 = \frac{W}{w}$

$\therefore a_1 . dL_1 = a_2 dL_2$ ...(ii)

We know that work done by pressure at AA, in moving the liquid to A' A'
= Force x Distance = $P_1. a_1 . dL_1$
Similarly, work done by pressure at BB, in moving the liquid to B' B'
$=-P_2a_2dl_2$
...(Minus sign is taken as the direction of P_2 is opposite to that of P_1)
:. Total work done by the pressure
$= P_1a1dl_1- P_2a_2dl_2$

$=P_1 a1dl_1 - p_{2a}1dl_1$
$\ldots(a_1dl_1=a_2dl_2)$
$= a_1.dl_1 (P_1- P_2) = \frac{W}{w} (P_1 - P_2) \ldots(a_1.dl_1 = \frac{W}{w})$
Loss of potential energy $= W (Z_1-Z_2)$
and again in kinetic energy $=W[(V_2^2/2g)-(V_1^2/2g)]= \frac{W}{2g} (v_2^2-v_1^2)$
We know that loss of potential energy + Work done by pressure
= Gain in kinetic energy
$\therefore W (Z_1-Z_2)+ \frac{W}{w} (P_1 - P_2) = \frac{W}{2g} (v_2^2-v_1^2)$
$(Z_1-Z_2)+(p_1/w)-(p_2/w)=v_2^2/2g-v_1^2/2g$
Or $Z_1 + v_1^2/2g+ (p_1/w)=Z_2+ v_2^2/2g+(p_2/w)$
which proves the Bernoulli's equation.

Euler's Equation For Motion

The "Euler's equation for steady flow of an ideal fluid along a streamline is based on the
Newton's Second Law of Motion. The integration of the equation gives Bernoulli's equation in the form of energy per unit weight of the flowing fluid. It is based on the 'following assumptions:
1. The fluid is non-viscous (i.e., the frictional losses are zero).
2. The fluid is homogeneous and incompressible (i.e., mass density of the fluid is constant).
3. The flow is continuous, steady and along the streamline.
4. The velocity of flow is uniform over the section.
5. No energy or force (except gravity and pressure forces) is involved in the flow.

Consider a steady' flow of an ideal fluid along a streamline. Now consider a small element
AB of the flowing fluid as shown in Fig.

Let

dA = Cross-sectional area of the fluid element,
ds = Length of the fluid element,
dW = Weight of the fluid 5!1ement,
p = Pressure on the element at A,
p + dp = Pressure on the element at B, and
v = Velocity of the fluid element.
We know that the external forces tending to accelerate the fluid element in the direction of the streamline
= p. dA - (p + dp) dA

Fig. 2.2

= -dp.dA
We also know that the weight of the fluid element,
dW = ρ g. dA . ds
From the geometry of the figure, we find that the component of the weight of the fluid element
,in the direction of flow
= - ρ g . dA . ds cosθ

= - ρ g . dA . ds($\frac{dz}{ds}$)

…cosθ =$\frac{dz}{ds}$

= - ρ g. dA. dz
$\therefore mass\ of\ the\ fluid\ element$ = ρ .dA.ds
,We see that the acceleration of the fluid element

$$\frac{dv}{dt} = \frac{dv}{ds} \times \frac{ds}{dt} = v.\frac{dv}{ds}$$

Now, as per Newton's Second Law of Motion, we know that
Force = Mass x Acceleration

(- dp . dA) - (ρ g . dA . dz-) = ρ . dA . ds $\times \frac{dv}{ds}$

$$\frac{dp}{\rho} + g.dz = v.dv$$

…(dividing both side by -ρdA)

Or $\frac{dp}{\rho} + g.dz + v.dv = 0$

This is the required Euler's equation for motion and is in the form of a differential equation. Integrating the above equation,
'

$$\frac{1}{\rho}\int dp + \int g.dz + \int v.dv = constant$$

$$\frac{p}{\rho} + gz + v^2/2 = constant$$

$$P + wZ + Wv^2/2g = constant$$

$$\frac{p}{w} + Z + v^2/2g = constant \text{ (Dividing by w)}$$

or in other words, $\frac{p1}{w} + Z_1 + (v_1^2/2g) = \frac{p2}{w} + Z_2 + (v_2^2/2g)$
which proves the Bernoulli's equation.

Limitations of Bernoulli's Equation:-

The Bernoulli's theorem or Bernoulli's equation has been derived on certain assumptions, which are rarely possible. Thus the Bernoulli's theorem has the following limitations:

1. The Bernoulli's equation has been derived under the assumption that the velocity of every liquid particle, across any cross-section of a pipe, is uniform. But, in actual practice, it is not so. The velocity of liquid particle in the centre of a pipe is maximum and gradually decreases towards the walls of the pipe due to the pipe friction. Thus, while using the Bernoulli's equation, only the mean velocity of the liquid should be taken into account.
2. The Bernoulli's equation has been derived under the assumption that no external force, except the gravity force, is acting on the liquid. But, in actual practice, it is not so. There are always some external forces (such as pipe friction etc.) acting on the liquid, which effect the flow of the liquid. Thus, while using the Bernoulli's equation, all such external forces should be neglected. But, ifsorne energy is supplied to, or, extracted from the flow, the same should also be taken into account.

3. The Bernoulli's equation has been derived, under the assumption that there is. no loss of energy of the liquid particle while flowing. But, in actual practice, -it is rarely so. In a turbulent flow, some kinetic energy is converted into heat energy. And in a viscous flow, some energy is lost due to shear forces. Thus, while using Bernoulli's equation, all such losses should be neglected.

4. If the liquid is flowing in a curved path, the energy due to centrifugal force should also be taken into account.

Example

The diameter of a pipe changes from 200 mm at a section 5 metres-above datum = to 50 mm at a section 3 metres above datum. The pressure of water at first section is 500 kPa. If the velocity of flow at the first section is 1 m/s, determine the intensity of pressure at the second section.

Solution.'Gi~en: d_1= 200 mm = 0.2 m; Z_1= 5 m; d2 = 50 mm = 0.05 m z_2 = 3 m; p = 500/
kPa = 500 kN/m2 and V_1 = 1 mls.

Let

V_2 = Velocity of flow at section 2, and

P_2 = Pressure at section 2. We know that area of the pipe at section 1

$a_1=\frac{\pi}{4}\times 0.2^2=31.42\times 10^{-3}m^2$

and area of pipe at section 2 $a_1=\frac{\pi}{4}\times 0.05^2=1.964\times 10^{-3}m^2$

Since the discharge through the pipe is continuous,therefore $a_1.V_1 = a_2.V_2$

$V_2=\frac{a1.v1}{a2}$ =[(31.42× 10 -3)× 1]/ 1.964× 10 -3=16m/s

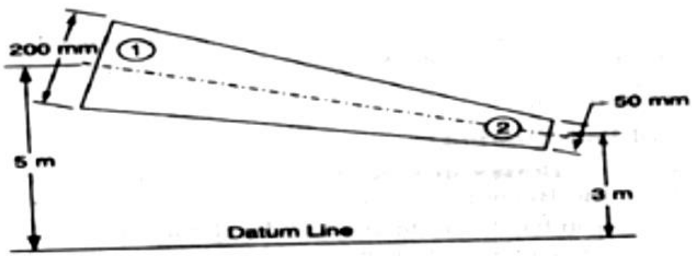


Fig. 2.3

Applying Bernoulli's equation for both the ends of the pipe,

$Z_1 + v_1^2/2g + (p_1/w) = Z_2 + v_2^2/2g + (p_2/w)$

$5+(1)^2/(2\times 9.81) + 500/9.81 = 3+(16)^2/2X9.81+\frac{p2}{9.81}$

P2 = 40 x 9.81 = 392.4 kN/m^2 = 392.4 kPa Ans

practical Applications of Bernoulli's Equation

The Bernoulli's theorem or Bernoulli's equation is the basic equation which has the widest applications in Hydraulics and Applied Hydraulics. Since this equation is applied for the derivation .of many formulae, therefore its clear understanding is very essential. Though the Bernoulli's equation has a number of practical applications. yet in this chapter we shall discuss its applications on the following ‘hydraulic devices :

1. Venturi meter.
2. Orifice meter.
3. Pitot tube.

Venturimeter

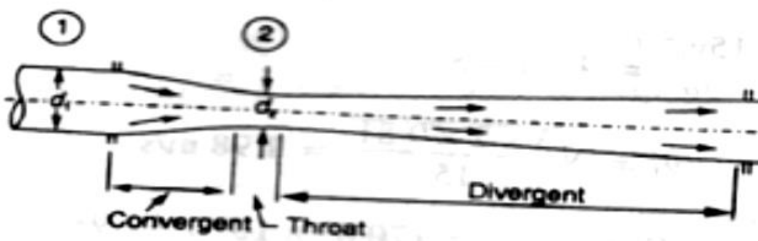


Fig. 2.4

A venturi meter is an apparatus for finding out the discharge of a liquid flowing in a pipe. A- venture meter, in its simplest form, consists of the following three parts:

(a) Convergent cone.
(b) Throat.
(c) Divergent cone.

(a) Convergent cone

It is a short pipe which converges from a diameter d_1 (diameter of the pipe. in which the venture meter is fitted) to a smaller diameter d_2: The convergent cone is also known as inlet of the venturi meter. The slope of the converging sides is between 1 in 4 or 1 in 5 as shown in Fig.

(b) Throat

It is a small portion of circular pipe in which the diameter d_2 is kept constant as shown in Fig.

(c) Divergent cone

It is a pipe, which diverges from a diameter d_2 to a large diameter d_1. The divergent cone is also known as outlet of the venture meter. The length of the divergent cone is about 3 to 4 times than that of the convergent cone as shown in Fig.

A little consideration will show that the liquid, while flowing through the venture meter, is accelerated between the sections 1 and 2 (i.e., while flowing through the convergent cone). As a result of the acceleration, the velocity of liquid at section 2 (i.e., at the throat) becomes higher than that at section 1. This increase in velocity results in considerably decreasing the pressure at section 2.1fthe pressure head at the throat falls below the separation head (which is 2.5 metres of water), then there will be a tendency of separation of the liquid flow, In order to avoid the tendency of separation at throat, there is always a fixed ratio of the diameter of throat and the pipe (i.e., dz/dt). This ratio varies from 1/4 to 3/4, but the most suitable value is 1/3 to 1/2.

The liquid, while flowing through the venture meter, is decelerated (i.e., retarded) between the sections 2 and 3 (i.e., while flowing through the divergent cone). As a result of this retardation, the velocity of liquid decreases which, consequently, increases the pressure. If the pressure is rapidly recovered, then there is every possibility for the stream of liquid to break away from the walls of the metre due to boundary layer effects. In order to avoid the tendency of breaking away the stream of liquid, the divergent cone is made sufficiently longer. Another reason for making the divergent cone longer is to minimise the frictional losses. Due to these reasons, the divergent cone is 3 to 4 times longer than convergent cone as shown in Fig.

Discharge through a Venturi meter

Consider a venture meter through which some liquid is flowing as shown in Fig.

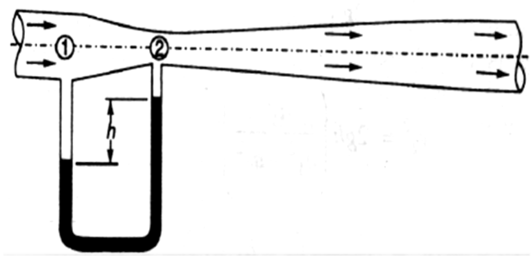


Fig. 2.5

Let

P_1 = Pressure at section 1,

V_1 = Velocity of water at section 1,

Z_1 = Datum head at section 1,

a_1 = Area of the venturi meter at section 1, and

p_2,v_2,z_2,a_2 = Corresponding values at section 2.

Applying Bernoulli's equation at sections 1 and 2. i.e

$Z_1 + v_1^2/2g+ (p_1/w)=Z_2+ v_2^2/2g+(p_2/w)$

……..(1)

Let us pass our datum line through the axis of the venture meter as shown in Fig.

Now $Z_1=0$ and $Z_2=0$

$\therefore$ $v_1^2/2g+(p_1/w)=v_2^2/2g+(p_2/w)$

Or $(p_1/w)-(p_2/w)=v_2^2/2g-v_1^2/2g$

……..(2)

Since the discharge at sections 1 and 2 is continuous, therefore

$V_1=a_2v_2/a_1$ $(a_1v_1=a_2v_2)$

$V_1^2=a_2^2v_2^2/a_1^2$

……..(3)

Substituting the above value of v_1^2 in equation (2),

$\frac{p1}{w}-\frac{p2}{w}=v_2^2/2g-(a_2^2/a_1^2 X\ v_2^2/2g)$

$= v_2^2/2g(1-a_2^2/a_1^2)=v_2^2/2g[(a_1^2-a_2^2)/a_1^2]$

We know that $\frac{p1}{w}-\frac{p2}{w}$ is the difference between the pressure heads at sections 1 and 2 when the pipe is horizontal, this difference represents the venturi head and is denoted by h.

Or $h=v_2^2/2g[(a_1^2-a_2^2)/a_1^2]$

Or $v_2^2=2gh[a_1^2/(a_1^2-a_2^2)]$

$\therefore$ $v_2=\sqrt{2gh}[a_1/\sqrt{\square}(a_1^2-a_2^2)]$

We know that the discharge through a venture meter,

$Q =$ Coefficient of venturi meter x $a_2\,v_2$

$=C.a_2v_2=[Ca_1a_2/\sqrt{\square}(a_1^2-a_2^2)]\times\sqrt{2gh}$

Example

A venture meter with a 150 mm diameter at inlet and 100 mm at throat is,laid with its axis horizontal and is used for measuring the flow of oil specific gravity O.9. The oil-mercury differential manometer shows a gauge difference of200 mm. Assume coefficient of the metre as O.9 Calculate the discharge in litres per minute.

Solution. Given: d_1 = 150 mm = 0.15 m; d_2 = 100 mm = 0.1 rn; Specific gravity of oil = 0.9
h = 200 mm = O.2 m of mercury and C = 0.98.

We know that the area at inlet,

$a_1=\frac{\pi}{4}\times 0.15^2=17.67\times 10^{-3}m^2$

and the area at throat,

$a_2=\frac{\pi}{4}\times 0.1^2=7.854\times 10^{-3}m^2$

We also know that the difference of pressure head,

H=0.2(13.6-0.9/0.9)=2.82 m of oil

and the discharge through the venturi meter,

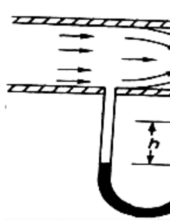
$Q=[Ca_1a_2/\sqrt{}(a_1^2-a_2^2)]\times\sqrt{2gh}$

$=63.9\ X\ 10^{-3}m^3/s=63.9lit/s$ Ans.

0rifice Metre

An orifice metre is used to measure the discharge in a pipe. An orifice metre, in its simplest
form, consists of a plate having a sharp edged circular hole known as an orifice. This plate is fixed inside a pipe as shown in Fig. c A mercury manometer is inserted to know the difference

of pressures between the pipe an? the throat (i.e., orifice).



Let

h = Reading of the mercury manometer,

P_1 = Pressure at inlet,

V_1 = Velocity of liquid at inlet,

a_1 = Area of pipe at inlet, and

P_2, v_2, a_2= Corresponding values at the throat.

Fig. 2.6

Now applying Bernoulli's equation for inlet of the pipe and the throat,

$Z_1 + v_1^2/2g + (p_1/w) = Z_2 + v_2^2/2g + (p_2/w)$ ....(i)

$(p_1/w) - (p_2/w) = v_2^2/2g - v_1^2/2g$

Or $h = v_2^2/2g - v_1^2/2g = 1/2g(v_2^2 - v_1^2)$ ....(ii)

Since the discharge is continuous, therefore $a_1.v_1 = a_2 v_2$

$V_1 = a_2/a_1 \times v_2$ or $v_1^2 = a_2^2/a_1^2 \times v_2^2$

Substituting the above value of v_1^2 in equation (ii)

$h = 1/2g(v_2^2 - a_2^2/a_1^2 \times v_2^2) = v_2^2/2g \times (1 - a_2^2/a_1^2) = v_2^2/2g[(a_1^2 - a_2^2)/a_1^2]$

$\therefore$ $v_2^2 = 2gh[a_1^2/(a_1^2 - a_2^2)]$ or $v_2 = \sqrt{\ } 2gh[a_1/\sqrt{\ }(a_1^2 - a_2^2)]$

We know that the discharge,

Q = Coefficient of orifice metre x $a_2 . v_2$

$= [Ca_1 a_2/\sqrt{\ }(a_1^2 - a_2^2)] \times \sqrt{2gh}$

Example. An orifice metre consisting of 100 mm diameter orifice in a 250 mm diameter pipe has coefficient equal to 0•65. The pipe delivers oil (sp. gr. 0•8). The pressure difference on the two sides of the orifice plate is measured by a mercury oil differential inano meter.lfthe differential gauge reads 80 mm of mercury, calculate the rate of flow in litresls.

Solution. Given: d_2 = 100 mm = 0.1 m; d_1 = 250 mm = o.25 m; C = 0.65 ; Specific gravity of oil = 0.8 and h = 0.8 m of mercury.

We know that the area of pipe,

$a_1 = \frac{\pi}{4} \times 0.25^2 = 49.09 \times 10^{-3} m^2$

and area of throat

$a_2 = \frac{\pi}{4} \times 0.1^2 = 7.854 \times 10^{-3} m^2$

We also know that the pressure difference,

h = 0.8[(13.6-0.8)/0.8]=12.8 m of oil

and rate of flow,

$Q=[Ca_1a_2/\sqrt{}(a_1^2-a_2^2)] \times \sqrt{2gh}$

$=82 \times 10^{-3}$ m^3/s=82 lit/s Ans

Pitot Tube.

A Pitot tube is an instrument to determine the velocity of flow at the required point in a pipe or a stream. In its simplest form, a pitot tube consists of a glass tube bent a through 90° as shown in Fig.
The lower end of the tube faces the direction of the flow as shown in Fig. The liquid rises up in the tube due to the pressure exerted by the flowing liquid. By measuring the rise of liquid in the tube, we can find out the velocity of the liquid flow.
Let h = Height of the liquid in the pitot tube above the surface,

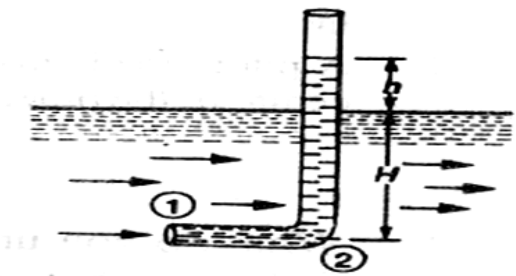


Fig. 2.7

H = Depth of tube in the liquid, and

v = Velocity of the liquid.

Applying Bernoulli's equation for the sections 1 and 2,

$H+v^2/2g=H+h$

…..($z_1=z_2$)

$h=v^2/2g$

∴ $v = \sqrt{2gh}$

Example .

A pltot tube was inserted in a pipe to measu!e the velocity of water in it. If (

water rises the tube is 200 mm, find the velocity of water.

Solution. Given: h = 200 mm = 0.2 m.

We know that the velocity of water in the pipe,

$v = \sqrt{2gh} = \sqrt{(2 \times 9.81 \times 0.2)} = 1.98$ m/s Ans.

Rate of Discharge

The quantity of a liquid, flowing per second through a section of a pipe or a channel, is known as the rate of discharge or simply discharge. It is generally denoted by Q. Now consider a liquid flowing through a pipe.

Let, a = Cross-sectional area of the pipe, and

v = Average velocity of the liquid,

:. Discharge, Q = Area × Average velocity = a.v

Notes: 1. If the area is in m^2 and velocity in m/s, then the discharge,

$$Q = m^2 \text{ x m/s} = m^3/s = \text{cumecs}$$

2. Remember that $1m^3 = 1000$ litres.

Equation of Continuity of a Liquid Flow

If an incompressible liquid is continuously flowing through a pipe or a channel (whose cross- sectional area may or may not be constant) the quantity of liquid passing per second is the same at all sections. This is known as the equation of continuity of a liquid flow. It is the first and fundamental equation of flow.

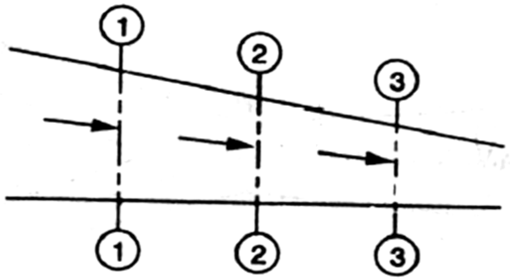


Fig. 2.8

CONTINUITY OF A LIQUID FLOW

Consider a tapering pipe through which some liquid is flowing as shown in Fig

Let , a_1 = Cross-sectional area of the pipe at section 1-1, and
v_1= Velocity of the liquid at section 1-1,
Similarly , a_2 , v_2= Corresponding values at section 2-2,
and a_3 , v_3 = Corresponding values at section 3-3.
We know that the total quantity of liquid passing through section 1-1,

$$Q_1 = a_1.v_1 \quad \text{……………………………(i)}$$

Similarly, total quantity of liquid passing through section 2-2,

$Q_2 = a_1.v_1$ ………………….(ii)

and total quantity of the liquid passing through section 3-3,

$Q_3 = a_3.v_3$ ………………….(iii)

From the law of conservation of matter, we know that the total quantity of liquid passing through the sections 1-1, 2-2 and 3-3 is the same. Therefore

$Q_1 = Q_2 = Q_3 =$ .... or $a_1. v_1 = a_2.v_2 = a_3.v_3$ .... and so on.

Example : Water is flowing through a pipe of 100 mm diameter with an average velocity
10 m/s. Determine the rate of discharge of the water in litres/s. Also determine the velocity of water
At the other end of the pipe, if the diameter of the pipe is gradually changed to 200 mm.

Solution. Given: $d_1 =$ 100 mm = 0.1 m; V_1 = 10 m/s and $d_2 =$ 200 mm = 0.2 m.

Rate of discharge

We know that the cross-sectional area of the pipe at point 1,

$$a_1 = \left(\frac{\pi}{4}\right) x (0.1)^2 = 7.854x10^{-3}\ m^2$$

and rate of discharge, Q = $a_1.v_1$ = $(7.854 \times 10^{-3})x\ 10 = 78.54 \times 10^{-3}\ m^3/s$

= 78.54 litres/s **Ans.**

Velocity of water at the other end of the pipe

We also know that cross-sectional area of the pipe at point 2,

$$a_2 = \left(\frac{\pi}{4}\right) x (0.2)^2 = 31.42x10^{-3}\ m^2$$

and velocity of water at point 2 ,$v_2 = \frac{Q}{a2}$ = $((78.54 \times 10^{-3})/(31.42x10^{-3}))$=2.5m/s **Ans.**

2.2 Flow over Notches:-

A notch is a device used for measuring the rate of flow of a liquid through a small channel or a tank. It may be defined as an opening in the side of a tank or a small channel in such a way that the liquid surface in the tank or channel is below the top edge of the opening.

A weir is a concrete or masonry structure, placed in an open channel over which the flow occurs. It is generally in the form of vertical wall, with a sharp edge at the top, running all the way across the open channel. The notch is of small size while the weir is of a bigger size. The notch is generally made of metallic plate while weir is made of concrete or masonry structure.

1. Nappe or Vein. The sheet of water flowing through a notch or over a weir is called Nappe or Vein.

2. Crest or Sill. The bottom edge of a notch or a top of a weir over which the water flows, is known as the sill or crest.

Classification Of Notches And Weirs:-

The notches are classified as :

I. According to the shape of the opening:

(a) Rectangular notch,

(b) Triangular notch,

(c) Trapezoidal notch, and

(d) Stepped notch.

2. According to the effect of the sides on the nappe:

(a) Notch with end contraction.

lb) Notch without end contraction or suppressed notch e,

Weirs are classified according to the shape of the opening the' shape of the crest, the effect of the sides on the nappe and nature of discharge. The following are important classifications.

Discharge Over A Rectangular Notch Or Weir

The expression for discharge over a rectangular notch or weir is the same.

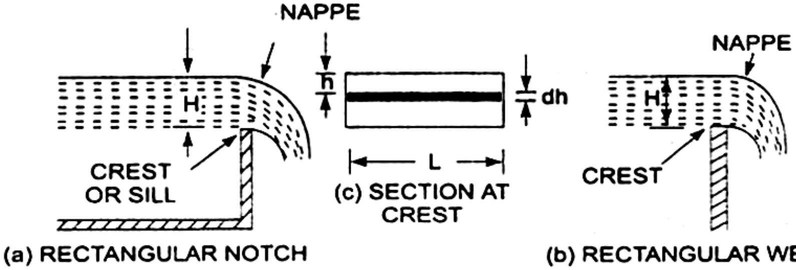


Fig. 2.9

Rectangular notch and 'weir:-

Consider a rectangular notch or weir provided in a channel carrying water as shown in Fig Let H = Head of water over the crest L = Length of the notch or weir

The total discharge, $Q=\frac{2}{3}\times c_d \times L \times \sqrt{2g}[H]^{3/2}$

Problem - 1

Find the discharge of water flowing over a rectangular notch 0/2 In length when the constant head over the notch is 300 mm. Take *cd* = 0.60.

Solution. Given:

Length of the notch, L=2.0m

Head over notch, H = 300 m = 0.30 m

C_d=0.06

Discharge $Q=\frac{2}{3}\times c_d \times L \times \sqrt{2g}[H]^{3/2}$

$=\frac{2}{3} \times 0.6 \times 2.0 \times \sqrt{2} \times 9.81 \times [0.30]_{1.5}$ m3/s

= 3.5435 x 0.1643 = 0.582 m3/s. Ans,

Problem 2

Determine the height of a rectangular weir of length 6 m to be built across a Rectangular channel. The maximum depth of water on the upstream side of the weir is 1.8m and discharge is 2000 litres/s. Take Cd = 0.6 and neglect end contractions.

Solution. Given:

Length of weir, L=6m

Depth of water, H1=1.8m

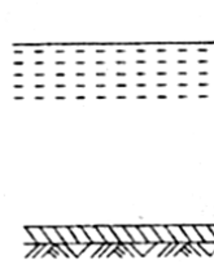
Discharge, Q = 2000 litIs = 2 m3/s

Cd/=0.6

Let H is the height of water above the crest of weir and H2 =height of weir

The discharge over the weir is given by the equation .

$$Q=\frac{2}{3} \times c_d \times L \times \sqrt{2g[H]}_{3/2}$$



$$2=\frac{2}{3} \times 0.6 \times 6 \times \sqrt{2} \times 9.81 \times [H]_{3/2}$$

$=10.623\ H^{3/2}$

$= H^{3/2}=\frac{2.0}{10.623}$

Fig. 2.10

$H=\left(\frac{2.0}{10.623}\right)^{2/3}=0.328$ m

Height of weir, H2 = H1- H

= Depth of water on upstream side - H

= 1.8 - .328 = 1.472 m. Ans.

Discharge Over A Triangular Notch Or Weir:-

The expression for the discharge over a triangular notch or weir is the same. It is derived as : Let H = head of water above the V notch

θ = angle of notch

Total discharge, $Q=\frac{8}{15}\times C_d \times \frac{\tan\theta}{2}\times\sqrt{2g}\times H^{5/2}$

For a right angle V Notch ,if C_d=0.6

$\theta = 90^0$, $\tan\frac{\theta}{2}=1$

Discharge, $Q = \frac{8}{15}\times 0.6\times 1\times\sqrt{2\times 9.81}\times H^{5/2}$

$=1.417\times H^{5/2}$

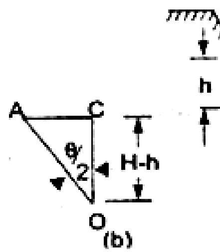


Fig. 2.11

Problem -1

Find the discharge over a triangular notch of angle 60° when the head over the V-notch is 0.3 m. Assume C_d = 0.6.

Solution. Given an Angle of V-notch, e = 60°

Head over notch, H=0.3 m

C_d = 0.6

Discharge, Q over a V-notch is given by equation

$Q=\frac{8}{15}\times C_d\times\frac{\tan\theta}{2}\times\sqrt{2g}\times H^{5/2}$

$\frac{8}{15}\times C_d\times\frac{0.6\tan 60}{2}\times\sqrt{2\times 9.81}\times(0.3)^{5/2}$

= 0.8182 x 0.0493 = 0.040 m3/s. Ans,

Problem -2

Water flows over a rectangular weir 1 m wide at a depth of 150 mm and afterwards passes through a triangular right-angled weir. Taking C_d for the rectangular and triangular weir as 0.62 and 0.59 respectively, find the depth over the triangular weir.

Solution. Given:

For rectangular weir. Length= L = 1 m

Depth of water, H = 150mm=0.15m

C_d = 0.62

For triangular weir.

θ = 90°

C_d = 0.59

Let depth over triangular weir = H_1

The discharge over the rectangular weir IS given by equation

$Q=\frac{2}{3}\times C_d\times L\times\sqrt{2g}[H]^{3/2}$

$=\frac{2}{3}\times 0.62\times 1.0\times\sqrt{2\times 9.81}\times(0.15)^{3/2}$

=0.10635 m^3/s

The same discharge passes through the triangular right-angled weir. But discharge. Q. is given by the equation

$$Q=\frac{8}{15}\times C_d \times \frac{\tan\theta}{2}\times\sqrt{2g}\times H^{5/2}$$

$$0.10635=\frac{8}{15}\times 0.59\times\frac{\tan 90}{2}\times\sqrt{2g}\times H_1^{5/2}$$

{ $\theta = 90^0$ and $H=H_1$ }

$$=\frac{8}{15}\times 0.59\times 1\times 4.429\times H_1^{5/2}$$

$$=1.3936\ H_1^{5/2}$$

$$H_1^{5/2}=\frac{0.10635}{1.3936}=0.07631$$

$H_1=(0.07631)^{0.4}=0.3572$ m , Ans

Discharge Over A Trapezoidal Notch Or Weir:-

A trapezoidal notch or weir is a combination of a rectangular and triangular notch or weir. Thus the total discharge will be equal to the sum of discharge through a rectangular weir or notch and discharge through a triangular notch or weir.

Let H = Height of water over the notch

L = Length of the crest of the notch

C_{d1} = Co-efficient or discharge. for rectangular portioo ABCD of Fig.

C_{d2} = Co-efficient of discharge for triangular portion [FAD and BCE]

The-discharge through rectangular portion ABCD is given by or $\qquad Q_1 = \frac{2}{3} \times C_{d1} \times L \times \sqrt{2g} \times H^{3/2}$

The discharge through two triangular notches FDA and BCE is equal to the discharge through a single triangular notch of angle e and it is given by equation

$$Q_2 = \frac{2}{3} \times C_{d2} \times \frac{\tan\theta}{2} \times \sqrt{2g} \times H^{5/2}$$

Discharge through trapezoldal notch or weir FDCEF = $Q_1 + Q_2$

$$= \frac{2}{3} \times C_{d1} L \sqrt{2g} \times H^{3/2} + \frac{8}{15} C_{d2} \times \frac{\tan\theta}{2} \times \sqrt{2g} \times H^{5/2}$$

Problem -1 Find the discharge through a trapezoidal notch which is 1 m wide at the tap and 0.40 m at the bottom and is 30 cm in height. The head of water On the notch is 20 cm. Assume C_d for rectangular portion = 0.62 while for triangular portion = 0.60.

Solution. Given:

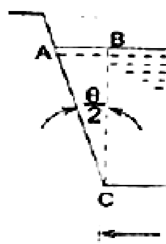
Top width $\qquad AE = 1$ m

Base width, $\qquad CD = L = 0.4$ m

Head of water, $\qquad H = 0.20$ m

For rectangular portion, $\qquad C_{d1} = 0.62$

From ΔABC ,we have



$$\frac{\tan\theta}{2} = \frac{AB}{BC} = \frac{\frac{AE - CD}{2}}{H}$$

$$= \frac{\frac{1.0 - 0.4}{2}}{0.3} = \frac{\frac{0.6}{2}}{0.3} = \frac{0.3}{0.3} = 1$$

Fig. 2.12

Discharge through trapezoidal notch is given by equation

$$Q = \frac{2}{3} C_{d1} \times L \times \sqrt{2g} \times H^{3/2} + \frac{8}{15} C_{d2} \times \frac{\tan\theta}{2\sqrt{2g}} \times H^{5/2}$$

$= \frac{2}{3} \times 0.62 \times 0.4 \times \sqrt{2 \times 9.81} \times (0.2)^{3/2} + \frac{8}{15} \times 60 \times 1 \times \sqrt{2 \times 9.81} \times (0.2)^{5/2}$

$= 0.06549 + 0.02535 = 0.09084$ m^3/s = 90.84 litres/s. Ans

Discharge Over A Stepped Notch:-

A stepped notch is a combination of rectangular notches. The discharge through 'stepped notch is equal to the sum of the discharges' through the different rectangular notches.

Consider a stepped notch as shown in Fig.

Let H_1 = Height of water above the crest of notch (1).

L_1 = Length of notch 1,

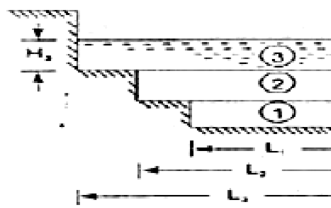
H_2, L_2 and H_3, L_3 are corresponding values for notches 2 and respectively.

C_d=Co-efficient of discharge for all notches

Total discharge $Q=Q_1+Q_2+Q_3$

Fig. 2.12

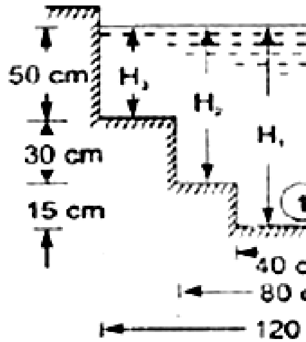
$Q=\frac{2}{3} \times C_d \times L_1 \times \sqrt{2g}[H_1^{3/2}-H_2^{3/2}]+\frac{2}{3} \times C_d \times L_2 \times \sqrt{2g}[H_2^{3/2}-H_3^{3/2}]+\frac{2}{3} C_d \times L_3 \times \sqrt{2g} \times H_3^{3/2}$



Problem Fig. 2.13

Fig. 1 shows a stepped notch. Find the discharge through the notch if Cd for all section = 0.62.

Solution. Given:



L_1 = 40 cm, L_2 = 80 cm,

L_3 = 120cm

H_1 = 50 + 30 + 15 = 95 crn,

H_2=80 cm,H_3=50 cm,

C_d=0.62

Total Discharge ,$Q=Q_1+Q_2+Q_3$

where

$Q_1=\frac{2}{3}\times C_d\times L_1\times\sqrt{2g}[H_1^{3/2}-H_2^{3/2}]$

$=\frac{2}{3}\times 0.62\times 40\times\sqrt{2\times 981}\times[95^{3/2}-80^{3/2}]$

$=154067 cm^3/s$ =154.067 lit/s

Fig. 2.14

$Q_2=\frac{2}{3}\times C_d\times L_2\times\sqrt{2g}[H_2^{3/2}-H_3^{3/2}]$

$=\frac{2}{3}\times 0.62\times 80\times\sqrt{2\times 981}\times[80^{3/2}-50^{3/2}]$

$=530141\ cm^3/s$

=530.144 lit/s

$Q_3=\frac{2}{3}C_d\times L_3\times\sqrt{2g}\times H_3^{3/2}$

$=\frac{2}{3}\times 0.62\times 120\times\sqrt{2\times 981}\times 50^{3/2}$

$=776771\ cm^3/s$

=776.771 lit/s

$\therefore\ Q=Q_1+Q_2+Q_3$

$=154.067+530.144+776.771$

$=1460.98$ lit/s   Ans.

Velocity Of Approach

Velocity of approach is defined as the velocity with which the water approaches or reaches the weir or notch before it flows over it. Thus if V_a is the velocity of approach, then an additional head h_a equal to $V_a^2/2g$ due to velocity of approach, is acting on the water. flowing over the notch. Then initial height of water over the notch becomes $(H+h_a)$ and final height becomes equal to h_a,' Then all the formulae are changed taking into consideration of velocity of approach.

The velocity of approach, V_a is determined by finding the discharge over the notch or weir neglecting velocity of approach. Then dividing the -discharge-by the cross-sectional area of the channel .on the' upstream side of the weir or notch, the velocity of approach is obtained . Mathematically,

$$V_a=\frac{Q}{Area\ of\ Channel}$$

This velocity of approach is used to find an additional head ($h_a= V_a^2/2g$).Again the discharge is calculated and above process is repeated for more accurate discharge.

Discharge over a rectangular weir, with velocity of approach

$$=\frac{2}{3}\times C_d\times L\times\sqrt{2g}\,[(H_1+h_a)^{3/2}-h_a^{3/2}]$$

Problem:-

Water is flowing in a rectangular channel of 1 m wide and 0.75 m deep. Find the discharge over a rectangular weir of crest length 60 cm if the head of water over the crest of weir is

20 cm and water from channel flows over the weir. Take Cd = 0.62. Neglect end contractions. Take

velocity of approach into consideration.

Solution. Given:

Area of channel, A = Width x depth = 1.0 x 0.75 = 0.75 m^2

Length of weir, L = 60 cm = 0.6 m

Head of water, H_1 = 20 cm = 0.2 m

C_d = 0.62

Discharge over a rectangular weir without velocity of approach is given by

$$Q=\frac{2}{3}C_d \times L \times \sqrt{2g} \times H_1^{3/2}$$

$$=\frac{2}{3} \times 0.62 \times 0.6 \times \sqrt{2 \times 9.81} \times (0.2)^{3/2}$$

$=0.0982\ m^3/s$

velocity of approach $V_a=\frac{Q}{A}=\frac{0.0982}{0.75}=0.1309\frac{m}{s}$

Additional head $h_a=V_a^2/2g$

$=(0.1309)^2/2 \times 9.81 = 0.0008733\ m$

Then discharge with velocity of approach is given by equation

$$Q=\frac{2}{3} \times C_d \times L \times \sqrt{2g}\,[(H_1+h_a)^{3/2}-h_a^{3/2}]$$

$=2/3 \times 0.62 \times 0.6 \times \sqrt{(2 \times 9.81}[(0.2 + 0.00087)^{3/2} - (0.00087)^{3/2}]$

= 1.098 [0.09002- .00002566]
= 1.098 x 0.09017
=0.09881 m^3/s. Ans

Types of Weirs :-

Though there are numerous types of weirs, yet the following are important from the subject point of view :

1. Narrow-crested weirs,
2. Broad-crested weirs,
3. Sharp-crested weirs,
4: Ogee weirs, and
5. Submerged or drowned weirs.

Discharge over a Narrow-crested Weir :-

The weirs are generally classified according to the width of their crests into two types. i.e.
narrow-crested weirs and broad crested weirs.

Let b = Width of the crest of the weir, and
H = Height of water above the weir crest.

If 2b is less than H,the weir is called a narrow-crested weir. But if 2b is more than H . it is called a broad-crested weir.
A narrow-crested weir is hydraulically similar to an ordinary weir or to a rectangular weir .Thus, the same formula for discharge over a narrow-crested weir holds good, which we derived from an ordinary weir .

$$Q = \frac{2}{3} \times C_d . L\sqrt{2g} \times H^{3/2}$$

Where, Q = Discharge over the weir,

Cd = Coefficient of discharge,

L = Length of the weir, and

H = Height of water level above the crest of the weir.

Example A narrow-crested weir of 10metres long is discharging water under a constant head of 400 mm. Find discharge over the weir in litresls. Assume coefficient of discharge as 0.623.

Solution. Given: L = 10 m; H= 400 mm = 0.4 m and C_d = 0.623.
We know that the discharge over the weir,

$$Q = \frac{2}{3} \text{ X } C_d .L\sqrt{2g} \text{ x } H^{3/2}$$

$$= \frac{2}{3} \text{ x}0.623\text{x}10\sqrt{(2 \times 9.81)} \times (0.4)^{3/2}$$

$$=46.55 \text{ m}^2/\text{s} = 4655 \text{ lit/s}$$

Discharge over a Broad-crested Weir :-

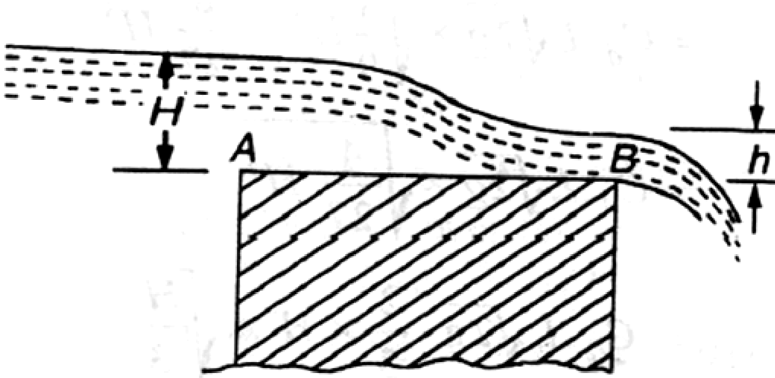


Fig. 2.15

Broad-crested weir

Consider a broad-crested weir as shown in Fig. Let A and B be the upstream and downstream ends of the weir.

Let H = Head of water on the upstream side of the weir (i.e., at A),

h = Head of water on the downstream side of the weir (i.e., at B),

v = Velocity of the water on the downstream side of the weir

(i.e., at B),

C_d = Coefficient of discharge, and

L = Length of the weir.

$$Q=1.71C_d .L \times H^{3/2}$$

Example A broad-crested weir 20 m long is discharging water from a reservoir in to channel. What will be the discharge over the weir, if the head of water on the upstream and downstream sides is 1m and 0.5 m respectively? Take coefficient of discharge for the flow as 0.6 .

Solution. Given: L = 20 m; H = 1 m; h = 0.5 m and C_d = 0.6.

We know that the discharge over the weir,

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

$$= 0.6 \times 2.0 \times 0.5 \times \sqrt{2 \times 9.81(1-0.5)} \ m^3/s$$

$$= 6 \times 3.13 = 18.8 \ m^3/s$$ **Ans.**

Discharge over a Sharp-crested Weir :-

It is a special type of weir, having a sharp-crest as shown in Fig. The water flowing over the crest comes in contact with the crest -line and then springs up from the crest and falls as a trajectory as shown in Fig.

In a sharp-crested weir, the thickness of the weir is kept less than half of the height of water on the weir. i.e.,

$$b<(H/2)$$

where , b = Thickness of the weir,

and H = Height of water, above the crest of the weir.

The discharge equation, for a sharp crested weir, remains the same as that of a rectangular weir. i.e.,

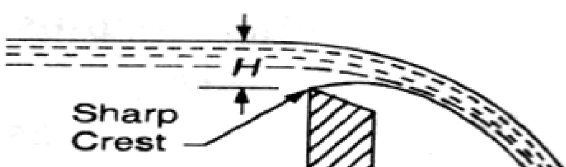


Fig. 2.16

Sharp-crested weir :-

$$Q = \frac{2}{3} \times C_d . L\sqrt{2g} \times H^{3/2}$$

Where, C_d = Coefficient of discharge, and
L = Length of sharp-crested weir

Example In a laboratory experiment, water flows over a sharp-crested weir 200 mm long under a constant head of 75mm. Find the discharge over the weir in litres/s, if $C_d = 0.6$.

Solution. Given: L = 200 mm = 0.2 m; H = 75 mm = 0.075 m and $C_d = 0.6$.

We know that the discharge over the weir,

$$Q = \frac{2}{3} \times C_d . L\sqrt{2g} \times H^{3/2}$$

$$= \frac{2}{3} \times 0.6 \times 0.2 \times \sqrt{2 \times 9.81} \times (0.075)^{3/2}$$

$= 0.0073\ m^3/s = 7.3$ litres/s. **Ans.**

Discharge over an Ogee Weir :-

It is a special type of weir, generally, used as a spillway of a dam as shown in Fig.

, The crest of an agee weir slightly rises up from the

point A ,(i.e., crest of the sharp-crested weir) and after reaching the maximum rise of 0.115 H (where H is the height of a water above the point A) falls in a parabolic form as shown in Fig.

The discharge equation for an agee weir remains the same as that of a rectangular weir. i.e.,

$$Q = \frac{2}{3} \text{ X } C_d .L\sqrt{2g} \text{ x } H^{3/2}$$

Where C_d = Co-efficient of discharge and

L= Length of an ogee weir

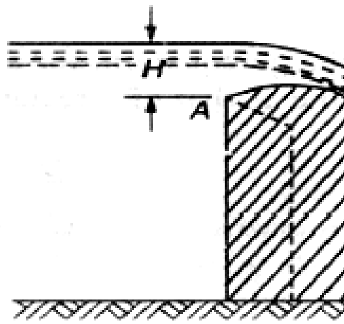


Fig. 2.17

Example

An ogee weir 4 metres long has 500 mm head of water. Find the discharge over the weir, if C_d = 0.62.

.

Solution. Given: L = 4 m; H = 500 mm = 0.5 m and C_d= 0.62.

We know that the discharge over the weir,

$$Q = \frac{2}{3} \text{ X } C_d .L\sqrt{2g} \text{ x } H^{3/2}$$

$= \frac{2}{3} \times 0.62 \times 4\sqrt{2 \times 9.81} \times (0.5)^{3/2}$ m^3/s

$= 7.323 \times 0.354 = 2.59$ m^3/s= 2590 litres/s   Ans

Discharge over a Submerged or Drowned Weir :-

When the water level on the downstream side of a weir is above the top surface of weir, it is known a submerged or drowned weir as shown in Fig

The total discharge, over such a weir, is found out by splitting up the height of water, above the sill of the weir, into two portions as discussed below:

Let H_1 = Height of water on the upstream side of the weir, and

H_2 =height of water on the downstream side of the weir.

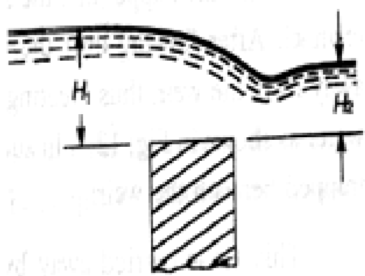


Fig. 2.18

The discharge over the upper portion may be considered as a free discharge under a head of water equal to $(H_1 – H_2)$. And the discharge over the lower portion may be considered as a submerged discharge under a head of H_2 . Thus discharge over the free portion (i.e., upper portion),

$$Q1 = \frac{2}{3}\ X\ Cd\ .L\sqrt{2g}\ x\ (H1\text{-}H2)^{3/2}$$

Submerged weir :-

and the discharge over the submerged (i.e., lower portion),

$Q_2 = C_d . L. H_2. \sqrt{2g(H_1 - H_2)}$

:. Total discharge, $Q = Q_1 + Q_2$

Example A submerged sharp crested weir 0.8 metre high stands clear across a channel having vertical sides and a width of 3 meters. The depth of water in the channel of approach is 1.2 meter. And 10 meters downstream from the weir, the depth of water is 1 meter. Determine the discharge over the weir in liters per second. Take C_d as 0.6.

Solution. Given: L = 3 m and Cd = 0.6.

From the geometry of the weir, we find that the depth

of water on the upstream side,

$H_1 = 1.25 - 0.8 = 0.45$ m and depth of water on the downstream side,

$H_2 = 1 - 0.8 = 0.2$ m

We know that the discharge over the free portion of the weir

$$Q_1 = \frac{2}{3}\ X\ Cd\ .L\sqrt{2g}\ x\ (H1\text{-}H2)^{3/2}$$

$$= \frac{2}{3} \times 0.6 \times 3 \times \left(\sqrt{2 \times 9.81}\right)(0.45 - 0.20)^{3/2}$$

$= 5.315 \times 0.125 = 0.664\ m^3/s = 664$ liters/s ... (i)

and discharge over the submerged portion of the weir,

$Q_2 = C_d . L. H_2. \sqrt{2g(H_1 - H_2)}$

$= 0.6 \times 3 \times 0.2 \sqrt{2 \times 9.81(0.45 - 0.2)}$ m^3/s

$= 0.36 \times 2.215 = 0.797$ m^3/s = 797 liters/s ... (ii)

:. Total discharge: $Q = Q_1 + Q_2 = 664 + 797 = 1461$ liters/ s **Ans.**

2.3 Flow over Weirs:-

An open channel is a passage through which the water flows under the force of gravity - atmospheric pressure. Or in other words, when the free surface of the flowing water is in contact, with the atmosphere as in the case of a canal, a sewer or an aquaduct, the flow is said to be through an open channel. A channel may be covered or open at the top. As a matter of fact, the flow of water in an open channel, is not due to any pressure as in the case of pipe flow. But it is due to the slope the bed of the channel. Thus during the construction of a channel, a uniform slope in its bed is provided to maintain the flow of water.

Chezy's Formula for Discharge through an Open Channel :-

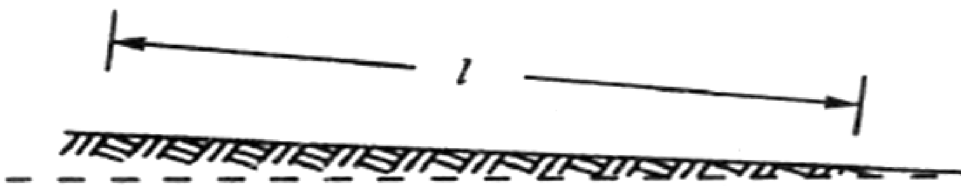


Fig. 2.19

Sloping bed of a channel :-

Consider an open channel of uniform cross-section and bed slope as shown in Fig.

Let

I = Length of the channel,

A = Area of flow,

v = Velocity of water,

p = Wetted perimeter of the cross-section, m=

f = Frictional resistance per unit area at unit velocity, and

i = Uniform slope in the bed.

$m = \frac{A}{P}$ ……..(known as hydraulic mean depth or hydraulic radious)

∴ Discharge Q= A X v=AC$\sqrt{mi}$

Example.

A rectangular channel is 1. 5 metres deep and 6 metres wide. Find the discharge through channel, when it runs full. Take slope of the bed as 1 in 900 and Chezy's constant as 50.

Solution. Given: d = 1.5 m; b = 6 m; i = 1/900 and C = 50.

We know that the area of the channel,

A = b.d = 6 x 1.5 = 9 m^2

and wetted perimeter, D = b + 2d = 6 + (2 x 1.5) = 9 m

:. Hydraulic mean depth, $m = \frac{A}{P} = 1$ m

and the discharge through the channel,

Q = AC$\sqrt{mi}$ = 9x50$\sqrt{(1 \times 1/900)}$= 15m^3/s Ans.

Manning Formula for Discharge :-

Manning, after carrying out a series of experiments, deduced the following relation for the value of C in Chezy's formula for discharge:

$C = \frac{1}{N} \times m^{1/6}$

where N is the Kutter's constant

Now we see that the velocity,

$v = C\sqrt{mi} = M \times m^{2/3} \times i^{1/2}$

where

M =1/N and is known as Manning's constant.

Now the discharge,

$Q =$ Area x Velocity $= A \times 1/N \times m^{2} \times i^{1/2}$

$= A \times M \times m^{2/3} \times i^{1/2}$

Example

An earthen channel with a 3 m wide base and side slopes 1 : 1 carries water with a depth of 1 m. The bed slope is 1in 1600. Estimate the discharge. Take value of N in Manning's formula as 0.04.

Solution.

Given: b = 3 m; Side slopes = 1 : 1; d = 1 m; i = 1/1600 and N = 0.04.

We know that the area of flow,

$$A = \frac{1}{2} \times (3 + 5) \times 1 = 4 \ m^2$$

and wetted perimeter,

$$P = 3 + 2 \times \sqrt{(1)^2+(1)^2} = 5.83 \ m$$

$\therefore$ hydraulic mean depth m = A/P=4/5.83=0.686 m

We know that the discharge through the channel

$$Q = \text{Area} \times \text{Velocity} = A \times 1/N \times m^{2/3} \times i^{1/2}$$

$$= 4 \times 1/0.04 \times 0.686^{2/3} \times (1/1600)^{1/2}$$

$$= 1.945 \ m^3/s \text{ Ans}$$

Channels of Most Economical Cross-sections :-

A channel, which gives maximum discharge for a given cross-sectional area and bed slope is called a channel of most economical cross-section. Or in other words, it is a channel which involves least excavation for a designed amount of discharge. A channel of most economical cross-section is, sometimes: also defined as a channel which has a minimum wetted perimeter; so that there is a minimum resistance to flow and thus resulting in a maximum discharge. From the above definitions, it is obvious that while deriving the condition for a channel of most economical cross-section, the cross-sectional area is assumed tobe constant. The relation between depth and breadth of the section is found out to give the maximum discharge.

The conditions for maximum discharge for the following sections will be dealt with in the succeeding pages :

1. Rectangular section,
2. Trapezoidal section, and
3. Circular section.

Condition for Maximum Discharge through a Channel of Rectangular Section :-

A rectangular section is, usually, not provided in channels except in rocky soils where the faces of rocks can stand vertically. Though a rectangular section is not of

much practical importance, yet we shall discuss it for its theoretical importance only.

Consider a channel of rectangular section as shown in Fig.

Let

b = Breadth of the channel, and
d = Depth of the channel.
A= b X d
Discharge Q = A x v = AC $\sqrt{\ }$mi
m=A/P
=d/2

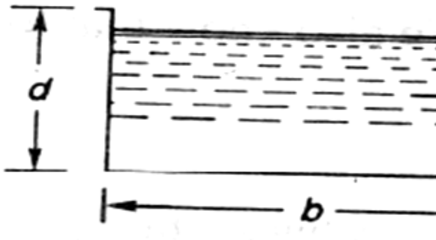


Fig. 2.20

Hence, for maximum discharge or maximum velocity, these two conditions (i.e., b = 2d and m = d/2) should be used for solving the problems of channels of rectangular cross-sections.

Example

A rectangular channel has a cross-section of 8 square metres. Find its size and discharge through the most economical section, if bed slope is 1 in 1000. Take C = 55.

Solution. Given: A = 8 m2

i= 1/1000 = 0.001 and C = 55.

Size of the channel
Let
b = Breadth of the channel, and
d = Depth of the channel.
We know that for the most economical rectangular section,
b = 2d
:. Area (A) 8=b X d= 2d X d =$2d^2$
= b=2 m

And b=2d=2 X 2= 4 m

Discharge through the channel

We also know that for the most economical rectangular section, hydraulic mean depth,

m=d/2=2/2=1 m

and the discharge through the channel,

Q = AC $\sqrt{\square}$mi= 8 x 55$\sqrt{\square}$ 1 X 0.001 m^3/s

= 440 x 0.0316 = 13.9 m^3/s , Ans.

Condition for Maximum Discharge through a Channel of Trapezoidal Section :-

A trapezoidal section is always provided in the earthen channels. The side slopes, in a channel of trapezoidal cross-section are provided, so that the soil can stand safely. Generally, the side slope in a particular soil is decided after conducting experiments on that soil. In a soft soil, flatter side slopes

should be provided whereas in a harder one, steeper side slopes may be provided.

consider a channel of trapezoidal cross- section ABCD as shown in FIg.

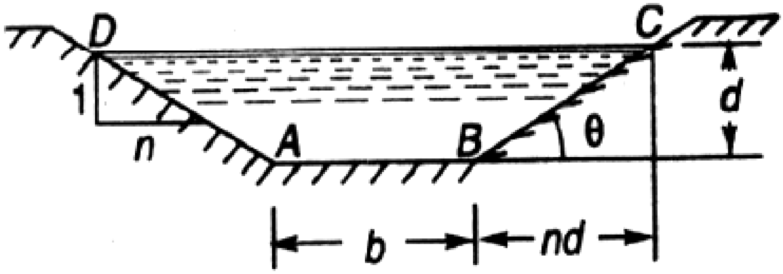


Fig. 2.21

Let
b = Breadth of the channel at the bottom,
d = Depth of the channel and
$\frac{1}{n}$ =side slope (i.e., 1 vertical to n horizontal)

Hence, for maximum discharge or maximum velocity these two
(i.e., b +2nd/2 = d $\sqrt{n2 + 1}$ and m = d/2) should be used for solving problems on channels of
trapezoidal cross-sections.

Example .
A most economical trapezoidal channel has an area of flow 3.5 m^2 discharge in the channel, when running 1 metre deep. Take C = 60 and bed slope 1 in 800.

Solution. Given: A = 3.5 m2 ; d = 1 m; C = 60 and i = 1/800.

We know that for the most economical trapezoidal channel the hydraulic mean depth

m = d/2 = 0.5 m

and discharge in the channel,

Q= A.C.$\sqrt{mi}$ = 5.25 m^3/s Ans.

Example .

A trapezoidal channel having side slopes of 1 : 1 and bed slope of 1 in 1200 is required to carry a discharge of 1800 m^3/min. Find the dimensions of the channel for cross-section. Take Chezy's constant as 50.

Solution.

Given side slope (n)=1

(i.e. 1 vertical to n horizontal),

i= 1/1200 ,Q= 180m^3/min = 3m^3/sec

and C = 50

Let b=breadth of the channel of its bottom and d= depth of the water flow.

We know that for minimum cross section the channel should be most economical and for economical trapezoidal section half of the top width is equal to the slopping side. i.e.

b +2nd/2 = d $\sqrt{n2 + 1}$

or b = 0.828d

∴ Area A = d X (b + nd) = 1.828d^2

We know that in the case of a most economical trapizodial section the hydraulic mean depth m=d/2

And discharge through the channel (Q)= A.C.$\sqrt{mi}$ =1.866$d^{5/2}$

$\therefore$ $d^{5/2}$ =3/1.866 = 1.608

Or d = 1.21 m

$\therefore$ b = 0.828 d = 0.828 X 1.21 = 1 m ANS

Condition for Maximum Velocity through a Channel of Circular Section :-

Consider a channel 'of circular section, discharging water under the atmospheric pressure shown in Fig.

Let r = Radius of the channel,

h = Depth of water in the channel, and

2θ = Total angle (in radians) subtended at the centre by the water

From the geometry of the figure, we find that the wetted perimeter of the channels,

$$P = 2r\theta \qquad ...(i)$$

and area of the section, through which the water is flowing,

$$A = r^2\theta - \frac{r2 \sin 2\theta}{2} = r^2\left(\theta - \frac{\sin 2\theta}{2}\right) \qquad ...(ii)$$

We know that the velocity of flow in an open channel,

Q= A.C.$\sqrt{mi}$

We know that the velocity of flow in an open channel , v= C$\sqrt{mi}$

Problem: Find the maximum velocity of water in a circular channel of 500 mm radius, if the bed slope is 1 in 400. Take manning's constant as 50.

Solution:-

Given d= 500mm = 0.5m or r = 0.5/2 = 0.25m, i=1/400 and M= 50

Let **2** θ = total angle (in radian) subtended by the water surface at the centre of the channel.

Now we know that for maximum velocity , the angle subtended by the water surface at the centre of the channel.

2 θ = $257^{0}30'$ or θ = 128.75^{0} = $128.75 X \frac{\pi}{180}$ = 2.247rad

∴ Area of flow, **A** = $r^2 \left(\theta - \frac{\sin 2\theta}{2}\right)$ = $171m^2$

And perimeter P = $2r\theta$ = 1.124m

∴ hydraulic mean depth m= A/P = 0.171/1.124 =0.152m

And velocity of water v= $MXm^{2/3}Xi^{1/2}$ = 0.71m/s ANS

Chapter-III

PUMPS

3.1 Centrifugal Pumps:-

The hydraulic machines which convert the mechanical energy to hydraulic energy are called pumps. The hydraulic energy is in the form of pressure energy. If the mechanical energy is converted, into pressure energy by means of centrifugal force acting on the fluid, the hydraulic machine is called centrifugal pump.

The centrifugal pump works on the principle of forced vortex flow which means that when a certain mass of liquid is rotated by an external torque, the rise in pressure head of the rotating liquid takes place. The rise in pressure head at any point of the rotating liquid is proportional to the square of tangential velocity of the liquid at that point (i.e. , rise in pressure head $= \frac{v^2}{2g} or \frac{\omega^2 r^2}{2g}$) . Thus at the outlet of the impeller, where radius is more , the rise in pressure head will be more & the liquid will be more & the liquid will be discharged at the outlet with a high pressure head. Due to this high pressure head, the liquid can be lifted to a high level.

Main Parts Of A Centrifugal Pump:-

The followings are the main parts of a centrifugal pump:

1. Impeller
2. Casing
3. Suction pipe with a foot valve & a strainer
4. Delivery Pipe

All the main parts of the centrifugal pump are shown in Fig 19.1

1. **Impeller**: The rotating part of a centrifugal pump is called 'impeller'. It consists of a series of backward curved vanes. The impeller is mounted on a shaft which is connected to the shaft of an electric motor.

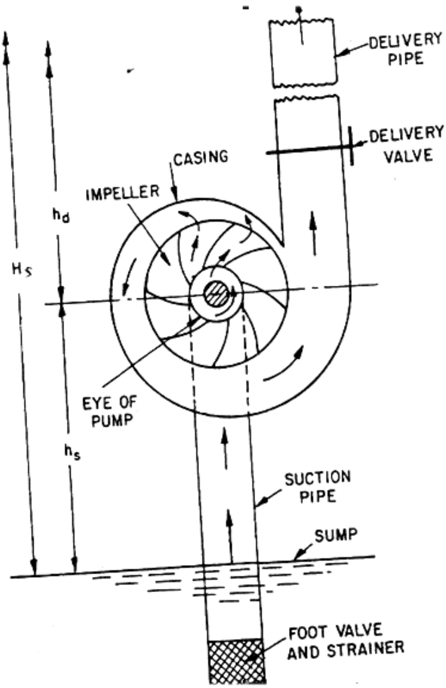
2. **Casing:** The casing of a centrifugal pump is similar to the casing of a reaction turbine. It is an air-tight passage surrounding the impeller & is designed in such a way that the kinetic energy of the water discharged at the outlet of the impeller is converted into pressure energy before the water leaves the casing & enters the delivery pipe. The following three types of the casings are commonly adopted:

   a. Volute **casing** as shown in Fig.19.1

   b. Vortex casing as shown in Fig.19.2(a)

   c. Casing with guide blades as shown in Fig.19.2(b)

a) Volute casing as shown in Fig.3.1the Volute casing, which is surrounding the impeller. It is of spiral type in which area of flow increases gradually. The increase in area of flow decrease velocity of flow. Decrease in velocity increases the pressure of water flowing through casing. it has been observed that in case of volute casing, the efficiency of pump increases.



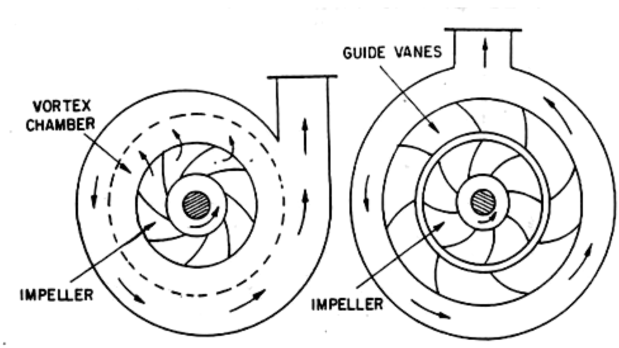
Main parts of a centrifugal pump

Fig. 3.1

b) Vortex casing. if a circular chamber is introduced between the casing and impeller as shown in fig.3.1,the casing is known as vortex casing .by introducing the circular chamber, loss of energy due to formation of eddies is reduced to a considerable extent. thus efficiency of pump is more than the efficiency when only volute casing is provided.

c) Casing with guide blades. This casing is shown in fig.3.1 in which the impeller is surrounded by a series of guide blades mounted on a ring which is known as diffuser. the guide vanes are designed in which a way that the water from the impeller enters the guide vanes without stock. Also the area of guide vanes increases, thus reducing the velocity of flow through guide vanes and consequently increasing the pressure of water. the water from guide vanes then passes through the surrounding casing which is in most of cases concentric with the impeller as shown in fig.3.1.

3. suction pipe with foot-valve and a strainer: A pipe whose one end is connected to the inlet of pump and other end dips into water in a sump is known as suction pipe. A foot valve which is a non-return valve or one –way type valve is fitted at lower end of suction pipe. Foot valve opens only in upward direction. A strainer is also fitted at lower end of suction pipe.



fferent

(a) VORTEX CASING (b) CASING WITH GUIDE BLADES

Fig: 3.2

4. Delivery pipe: a pipe whose one end is connected to outlet of pump and other end delivers water at a required height is known as delivery pipe.

Efficiencies of a centrifugal pump: Efficiencies of a centrifugal pump: In case of a centrifugal pump , the power is transmitted from the shaft of the electric motor to the shaft of the pump & then to the impeller. From the impeller, the power is given to the water. Thus power is decreasing from the shaft of the pump to the impeller & then to the water. The following are the important efficiencies of a centrifugal pump:

a. Manometric efficiencies η_{man}

b. Mechanical efficiencies η_m

c. Overall efficiencies η_o

a) **Manometric Efficiencies η_{man}:** The ratio of the manometric head to the head imparted by the impeller to the water is known as manometric efficiency. It is written as

$$\eta_{max} = \text{Manometric head} / \text{Head imparted by impeller to water}$$

$$= \frac{H_m}{\frac{V_{w2} u_2}{g}} = \frac{g H_m}{V_{w2} u_2} \ldots\ldots\ldots\ldots\ldots$$

The impeller at the impeller of the pump is more than the power given to the water at outlet of the pump. The ratio of the power given to water at outlet of the pump to the power available at the impeller, is known as manometric efficiency.

The power given to water at outlet of the pump= $\frac{WH_m}{1000} kW$

The power at the impeller = $\frac{Work\ done\ by\ impeller\ per\ second}{1000} kW$

$$= \frac{W}{g} \times \frac{V_{w2} u_2}{1000} kW$$

$$\eta_{max} = \frac{\frac{WH_m}{1000}}{\frac{W}{g} \times \frac{V_{w2} u_2}{1000}} = \frac{gH_m}{V_{w2} \times u_2}$$

b) Mechanical efficiencies:-

The power at the shaft of the centrifugal pump is more than the power available at the impeller of the pump . The ratio of the power available at the impeller to the power at the shaft of the centrifugal pump is known as mechanical efficiency. It is written as

$\eta_m =$ Power at the impeller/Power at the shaft

The power at the impeller in kW=Work done by impeller per second/10000

$$= \frac{W}{g} \times \frac{V_{w2} u_2}{1000}$$

$$\eta_m = \frac{\frac{W}{g}\left(\frac{V_{w2} u_2}{1000}\right)}{S.P.} \ldots\ldots\ldots\ldots\ldots$$

Where S.P.= Shaft Power

c) Overall efficiencies η_o

It is defined as the ratio of power output of the pump to the power input to the pump . The power output of the pump in kW

$$= \frac{Weight\ of\ water\ lifted\ * H_m}{1000} = \frac{WH_m}{1000}$$

Power input to the pump =Power supplied by the electric motor

= S.P. of the pump

$$= \eta_o = \frac{\left(\frac{WH_m}{1000}\right)}{S.P.} \ldots\ldots\ldots\ldots\ldots\ldots$$

$$= \eta_{man} \times \eta_m \ldots\ldots\ldots\ldots\ldots\ldots\ldots$$

Problem 3.1: The internal & external diameters of the impeller of a centrifugal pump are 200mm & 400mm respectively. The pump is running at 1200 r.p.m. The vane angles of the impeller at inlet & outlet are 20^0 & 30^0 respectively. The water enters the impeller radially & velocity of flow is constant. Determine the velocity of flow per metre sec.

Solution: **I**nternal Dia. Of impeller,=D_1=200mm=0.20m

External Dia. Of impeller ,=D_2=400mm=0.40m

Speed N=1200r.p.m

Vane angle at inlet , $\theta = 20^0$

Vane angle at outlet, $\phi = 30^0$

Water enter s radially means, $\alpha = 90^0$ and $V_{w1} = 0$

Velocity of flow , $= V_{f1} = V_{f2}$

Tangential velocity of impeller at inlet & outlet are,

$$u_1 = \frac{\Pi D_1 N}{60} = \frac{\Pi \times .20 \times 1200}{60} = 12.56 m/s$$
$$u_2 = \frac{\Pi D_2 N}{60} = \frac{\Pi \times .40 \times 1200}{60} = 25.13 m/s$$

From inlet velocity triangle,

$$\tan\phi = \frac{V_{f1}}{u_1} = \frac{V_{f2}}{12.56}$$
$$V_{f1} = 12.56 \tan\theta = 12.56 \times \tan 20 = 4.57 m/s$$
$$V_{f2} = V_{f1} = 4.57 m/s$$

Problem 3.2: A centrifugal pump delivers water against a net head of 14.5 metres & a design speed of 1000r.p.m .The values are back to an angle of 30^0 with the periphery. The impeller diameter is 300mm & outlet width 50mm. Determine the discharge of the pump if manometric efficiency is 95%.

Solution: Net head, $H_{m=}$ 14.5m

Speed, N =1000r.p.m

Vane angle at outlet, $\phi = 30^0$

Impeller diameter means the diameter of the impeller at outlet

Diameter, $D_2 = 300mm = 0.30m$

Outlet width, $B_2 = 50mm = 0.05m$

Manometric efficiency, $\eta_{man} = 95\%=0.95$

Tangential velocity of impeller at outlet,

$$u_2 = \frac{\pi D_2 N}{60} = \frac{\pi \times .30 \times 1000}{60} = 15.70 m/s$$

Now using equation

$$\eta_{\max} = \frac{gH_m}{V_{w2}u_2}$$

$$0.95 = \frac{9.81 \times 14.5}{V_{w2} \times 15.70}$$

$$V_{w2} = \frac{0.95 \times 14.5}{0.95 \times 15.70} = 9.54 m/s$$

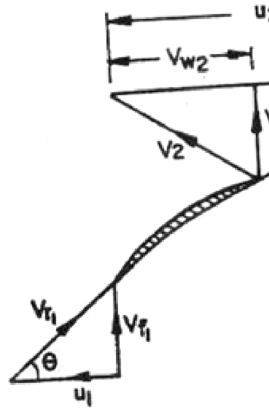


Fig. 3.3

Refer to fig(3.3). From outlet velocity triangle, we have

$$\tan\phi = \frac{V_{f2}}{(u_2 - V_{w2})}$$

$$\tan 30^0 = \frac{V_{f2}}{(15.70 - 9.54)} = \frac{V_{f2}}{6.16}$$

$$V_{f2} = 6.16 \times \tan 30^0 = 3.556 m/s$$

$$Disch\arg e = Q = \pi \times D_2 \times B_2 \times V_{f2}$$

$$= \pi \times 0.30 \times 0.05 \times 3.556 m^3/s = 0.1675 m^3/s$$

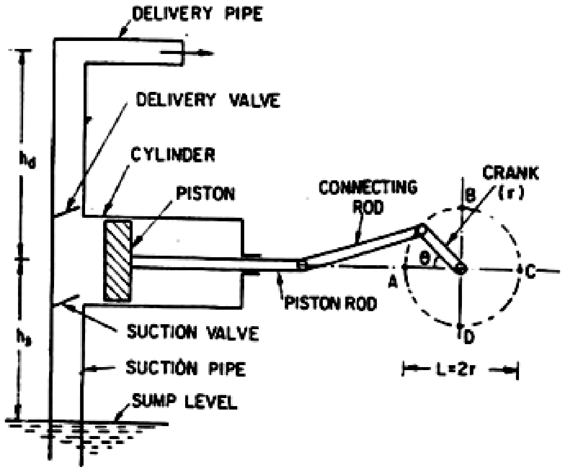
3.2 Reciprocating Pump:-

Introduction:-

We have defined the pumps as the hydraulic machines which convert the mechanical energy to hydraulic energy which is mainly in the form of pressure energy. If the mechanical energy is converted into hydraulic energy (or pressure energy) by sucking the liquid into a cylinder in which a piston is reciprocating (moving backwards and forwards), which exerts the thrust on the liquid & increases its hydraulic energy (pressure energy), the pump is known as reciprocating pump.

Main parts of a reciprocating pump:-

The following are the main parts of a reciprocating pump as shown in fig (3.4)



Main parts of a reciprocating pump.

1. A cylinder with a piston, piston rod, connecting rod and a crank,
2. Suction pipe,
3. Delivery pipe,
4. Suction valve, and
5. Delivery valve.

Fig. 3.4

Discharge through a Reciprocating Pump: Consider a single acting reciprocating pump as shown in fig ().

Let D= dia. Of the cylinder

A= C/s area of the piston or cylinder

$= \frac{\pi}{4} D^2$

r= Radius of crank

N=r.p.m of the crank

L=Length of the stroke=2*r

h_s = height of the axis of the cylinder from water surface in sump

h_d = Height of the delivery outlet above the cylinder axis (also called delivery head)

Volume of water delivered in one revolution or discharge of water in one revolution

= Area * Length of stroke = A*L

Number of revolution per second, = $\frac{N}{60}$

Discharge of the pump per second , Q= Discharge in one direction × No. of revolution per second

$$= A \times L \times \frac{N}{60} = \frac{ALN}{60}$$

………………………………

Wt. of water delivered per second, W = $\rho g Q = \frac{\rho g ALN}{60}$

……………………………

Work done by Reciprocating Pump : Work done by the reciprocating pump per sec. is given by the reaction as

Work done per second = Weight of water lifted per second × Total height through which water is lifted

$$= W \times (h_s + h_d)$$

Where $\left(h_s + h_d\right)$= Total height through which water is lifted

From equation () Weight, W is given by $W = \frac{\rho g A L N}{60}$

Substituting the value of W in equation () we get

Work done per second =

$$\frac{\rho g A L N}{60}\left(h_s + h_d\right)$$ ....

Power required to drive the pump, in kW

$$P = \frac{Work\ done\ per\ second}{1000} = \frac{\rho \times g \times A L N\left(h_s + h_d\right)}{60 \times 1000}$$

$$= \frac{\rho g A L N\left(h_s + h_d\right)}{60{,}000} kW$$ ....

Classification of reciprocating pumps:

The reciprocating pumps may be classified as:

1. According to the water being in contact with one side or both sides of the piston, and
2. According to the number of cylinders provided

If the water is in contact with one side of the piston, the pump is known as single-acting. On the other hand,

If the water is in contact with both sides of the piston, the pump is called double –acting. Hence, classification according to the contact of water is:

I. Single-acting pump

II. Double –acting pump

According to the number of cylinder provided, the pumps are classified as:

I. Single cylinder pump

II. Double cylinder pump

III. Triple cylinder pump

INTRODUCTION

Water on the surface of earth is available in the atmosphere, the oceans, on land and within the soil and fractured rock of the earth’s crust. Water molecules from one location to another are driven due to the solar energy transmitted to the surface of the earth from Sun. Moisture circulates from the earth into the atmosphere through evaporation and then back into the earth as precipitation.

Hydrology:

It is the study of physical geographic which deals with the origin, distribution andproperties of water present in earth surface.

Hydrological Cycle:-

Precipitation

↑

C

o

n

d

e

n

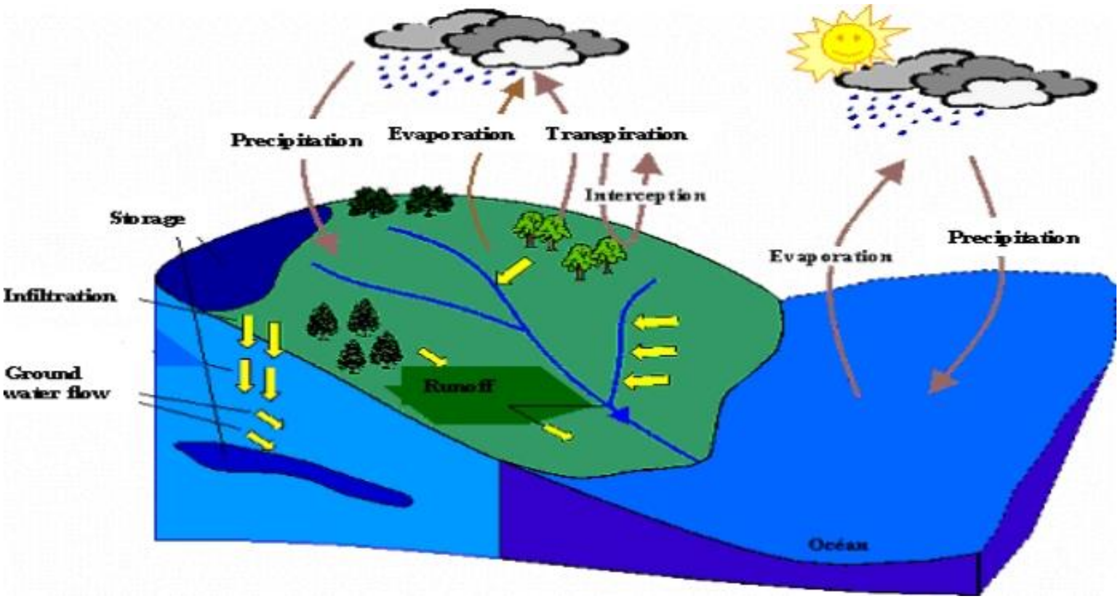
s

ation↑

Formation of Clouds↑

Evaporation
↑

Ocean



Water Budget Equation:-

Mass inflow – Mass outflow = Change in storage$P - R - G - E - T = \Delta S$

Where,

P = Precipitation T = Transpiration

G = Ground water ResourcesR = Runoff

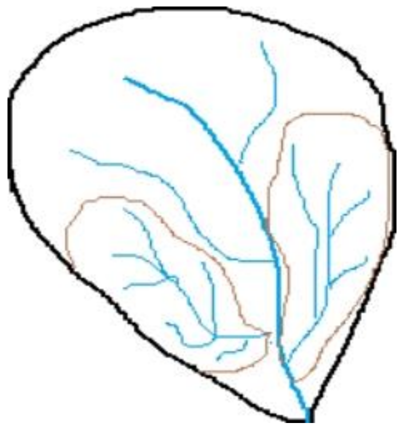
E = Evaporation

ΔS = Chang in Storage

Catchment Area:-

River

Streams



World Water Balance:-

Total quantity of water in the world is estimated to be about 1386 million cubic kilometer (M Km^3). About 96.5% of this water is contained in the oceans as saline water. Some of the water on the land amounting to about 1% of the total water is also saline. Thus, only about 35.0 M Km^3 of fresh water is available. Out of this about 10.6 M Km^3 is both liquid and fresh and the remaining 24.4 M Km^3 is contained in frozen state as ice in the polar regions and on mountain tops and glaciers.

PRECIPITATION

INTRODUCTION:

The term "precipitation" denotes all forms of water that reach the earth from the atmosphere. The usual forms are rainfall, snowfall, hail, frost and dew.

The magnitude of precipitation varies with time and space. For precipitation to form: (i) the atmosphere must have moisture, (ii) there must be sufficient nuclei present to aid condensation , (iii) weather conditions must be good for condensation of water vapour to take place, and (iv) the products of condensation must reach the earth.

FORMS OF PRECIPITATION:

Some of the common forms of precipitation are rain, snow, drizzle, glaze, sleet andhail.

1. ***Rain***

It is the principal form of precipitation in India. The term rainfall is used to describe precipitation in the form of water drops of sizes larger than 0.5mm. The maximum size of a raindrop is 6mm. Any drop larger in size than this tends to break up into drops of smaller sizes during its fall from the clouds. On the basis of its intensity rainfall is classified as follows:

Light rain: trace to 2.5 mm/hr Moderate rain: 2.5mm/hr to 7.5mm/hrHeavy rain: > 7.5mm/hr

2. Snow

Snow is another important form of precipitation. Snow consists of ice crystals which usually combine to form flakes. When fresh, snow has an initial density varying from 0.06 to 0.15g/cm^3 and it is usual to assume an average density of 0.1 ^{g}/cm^3. In India, snow occurs only in the Himalayan regions.

3. Drizzle

A fine sprinkle of numerous water droplets of size less than 0.5mm and intensity less than 1mm/hr is known as drizzle. In this, the drops are so small that they appear to float in the air.

4. Glaze

When rain or drizzle comes in contact with cold ground at 0^0C, the water drops freeze to form an ice coating called glaze or freezing rain.

5. Sleet

It is frozen raindrops of transparent grains which form when rain falls through air at sub freezing temperature. In Britain, sleet denotes precipitation of snow and rain simultaneously.

6. Hail

It is a showery precipitation in the forms of irregular pellets of lump of ice of size more than 8mm. Hails occur in violent thunderstorms in which vertical currents are very strong.

WEATHER SYSTEMS FOR PRECIPITATION:

For the formation of clouds and subsequent precipitation, it is necessary that the moist air masses cool to form condensation. This is normally accomplished by adiabatic cooling of moist air through a process of being lifted to higher altitude. Some of the terms and processes connected with weather systems associated with precipitation are given below.

1. Front

A front is the interface between two distinct air masses. Under certain favorable conditions when a warm air mass and cold air mass meet, the warmer air mass is lifted over the colder one with the formation of front. The ascending warmer air cools adiabatically with the consequent formation of clouds and precipitation.

2. cyclone

A cyclone is a large low pressure region with circular wind motion. Two types of cyclones are recognized: tropical cyclones and extra tropical cyclones.

(a) Tropical cyclone:

A tropical cyclone, also called cyclone in India, hurricane in USA and typhoon in south East Asia, is a wind system with an intensely strong depression with MSL pressures sometimes below 915 m bars. The normal areal extend of cyclone is about 100-200 km in diameter. The isobars are closely spaced and the winds are anticlockwise in the northern hemisphere. The center of the storm called the eye, which may extend to about 10-50 km in diameter, will be relatively quiet. However, right outside the eye, very strong winds/reaching to as much as 200 km per hr exist. The wind speed gradually decreases towards the outer edge. The pressure also increasesoutwards. The rainfall will normally be heavy in the entire area occupied by the cyclone.

(b) Extra tropical cyclone:

These are cyclones formed in locations outside the tropical zone. Associated with a frontal system, they possess a strong counter clockwise wind circulation in the northern hemisphere. The magnitude of precipitation and wind velocities are relatively lower than those of a tropical cyclone. However, the duration of precipitation is usually longer and the areal extend is also larger.

(3) Anticyclones:

These are regions of high pressure, usually of large areal extent. The weatheris usually calm act the center. Anticyclones cause clockwise wind circulations in the northern hemisphere. Winds are of moderate speed, and at the outer, cloudy and precipitation conditions exist.

(4) Convective precipitation:

In this type of precipitation, a packet of air which is warmer than thesurrounding air due to localized heating rises because of its lesser density. Air from cooler surroundings flows to take up its place, thus setting up a convective cell. The warm air continues to rise, undergoes cooling and results in precipitation. Depending upon the moisture, thermal and other conditions, light showers to thunderstorms can be expected in convective precipitation. Usually, the aerial extent of such rains is small, being limited to a diameter of about 10km.

(5) Orographic precipitation:

The moist air masses may get lifted up to higher altitudes due to the presence of mountain barriers and consequently undergo cooling, condensation and precipitation. Such a precipitation is known as orographic precipitation. Thus, in mountain ranges, the windward slopes of heavy precipitation and the leeward slopes have light rainfall.

ANNUAL RAINFALL:

Considerable areal variation exists for the annual rainfall of the magnitude of 200cm in Assam and north-eastern parts and Western-Ghats, and scanty rainfall in eastern Rajasthan and parts of Gujarat, Maharashtra and Karnataka. The average annual rainfall for the entire country is estimated as 118.3cm.

It is well known that there is considerable variation of annual rainfall in time at a place. The coefficient of variation,

C_v= (100* standard deviation)/ Mean

Of the annual rainfall varies between 15 and 70, from place to place with an average value of about 30. Variability is least in regions of high rainfall and largestin regions of scanty rainfall. Gujarat, Haryana, Punjab and Rajasthan have large variability of rainfall.

Some of the interesting statistics relating to the variability of the seasonal and annual rainfall of India are as follows:

- A few heavy spells of rain contribute nearly 90% of total rainfall.
- While the average annual rainfall of the country is 118 cm, average annual rainfall varies from 10 cm in the western desert to 1100 cm in the north-east region.
- More than 50% rain occurs within 15 days and less than 100 hours in a year.
- More than 805 of seasonal rainfall is produced in 10-20% rain events, each lasting 1-3 days.

MEASUREMENT OF PRECIPITATION

1) Rainfall

Precipitation is expressed in terms of the depth to which rainfall water would standon an area if all the rain were collected on it. Thus, 1cm of rainfall over acatchment area of 1km^2 represents a volume of water equal to 10^4 m^3. In the case of snowfall, an equivalent depth of water is used as the depth of precipitation. The precipitation is collected and measured in a *rain gauge*. Terms such pluviometer ombrometer and hyetometer area also sometimes used to designate a rain gauge.

A rain gauge essentially consists of a cylindrical vessel assembly kept in the open to collect rain. The rainfall catch of the rain gauge is affected by its exposure conditions. To enable the catch of rain gauge to accurately represent the area in the surrounding the rain gauge standard settings are adopted. For setting up a rain gauge the following considerations are important:

The ground must be level and in the open and the instrument must present a horizontal catch surface.

The gauge must be set as near the ground as possible to reduce wind effects but it must be sufficiently high to prevent splashing flooding etc.

The instrument must be surrounded by an open fenced area of at least 5.5 m * 5.5 m. No object should be nearer to the instrument than 30 m or twice the height of the obstruction.

Rain gauges can be broadly classified into two categories as

(i) Non recording gauges (ii) Recording gauges.

A. Non-recording Gauges

The non recording gauge extensively used in India is the *Symon's gauge*. It essentially consists of a circular collection area of 12.7 cm (5.0 inch) diameter connected to a funnel. The rim of the collector is set in a horizontal plane at a height of 30.5 cm above the ground level. The funnel discharges the rainfall catch into a receiving vessel. The funnel and receiving vessel are housed in a metallic container. Fig below shows the details of the installation. Water contained in the receiving vessel is measured by a suitably graduated measuring glass, with accuracy up to 0.1mm.

Recently, the Indian Meteorological Department (IMD) has changed over to the use of fiber glass reinforced polyester rain-gauges, which is an improvement over the *Symon's gauge.* These come in different combinations of collector is in two sizes having areas of 200 and 100 cm^2 respectively. Indian standard (IS: 5225- 1969) gives details of these new rain-gauges.

For uniformity, the rainfall is measured everyday at 8.30a.m.(*IST)* and is recorded as the rainfall of that day. The receiving bottle normally does not hold more than 10cm of rain and as such, in the case of heavy rainfall, the measurements must be done more frequently and entered. However, the last reading must be taken at 8.30 a.m. and the sum of the previous readings in the past 24 hours entered as the total of that day. Proper care, maintenance and inspection of rain-gauges, especially during dry weather to keep the instrument free from dust and dirt, is very necessary. The details of installation of non-recording rain-gauges and measurement of rain are specified in Indian Standard (IS:4986-1968).

This rain-gauge can also be used to measure snowfall. When snow is expected, the funnel and receiving bottle are removed and the snow is allowed to collect in the outer metal container. The snow is then melted and the depth of resulting water measured. Antifreeze agents are sometimes used to facilitate melting of snow. In areas where considerable snowfall is expected, special snow-gauges with shields (for minimizing the wind effect) and storage pipes (to collect snow over longer durations) are used.

B. Recording Gauges

Recording gauges produce a continuous plot of rainfall against time and provide valuable data of intensity and duration of rainfall for hydrological analysis of storms. The following are some of the commonly used recording rain-gauges.

(a)Tipping-Bucket Type

This is a 30.5 cm size rain-gauge adopted for use by the US Weather Bureau. The catch from the funnel falls onto one of a pair of small buckets. These buckets areso balanced that when 0.25 mm of rainfall collects in one bucket, it tips and bringsthe other one in position. The water from the tipped bucket is collected in a storagecan. The tipping actuates an electrically driven pen to trace a record on theclockwork-driven chart. The water collected in the storage can is measured atregular intervals to provide the total rainfall and also serve as a check. It may benoted that the record from the tipping bucket gives data on the intensity of rainfall.Further, the instrument is ideally suited for digitalizing of the output signal. (b)Weighing-Bucket Type

In this rain-gauge, the catch from the funnel empties into a bucket mounted on a weighing scale. The weight of the bucket and its contents are recorded on a clockwork-driven chart. The clockwork mechanism has the capacity to run for as long as one week. This instrument gives a plot of the accumulated rainfall against the elapsed time, i.e. the mass curve of rainfall. In some instruments of this type, the recording unit is so constructed that the pen reverses its direction at every preset value, say 7.5 cm (3inch)so that a continuous plot of storm is obtained.

(c) Natural-Syphon Type

This type of recording rain-gauge is also known as float-type gauge. Here, the rainfall collected by a funnel-shaped collector is led into a float chamber causing a float to rise. As the float rises, a pen attached to the float through a lever system records the elevation of the float on a rotating drum driven by a clockwork mechanism. A siphon arrangement empties the float chamber when the float has reached a preset maximum level. This type of rain-gauge is adopted as the standard recording-type rain-gauge in India and in details is described in Indian Standard (IS: 5235-1969).

RAINGAUGE NETWORK:

- **NETWORK DENSITY**

Since the catching area of a rain-gauges very small compared to the areal extent of a storm, it is obvious that to get a representative picture of a storm over a catchment, the number of rain-gauges should be as large as possible, i.e. the catchment area per gauge should be small. Economic considerations restrict the number of gauges to be maintained.

- **ADEQUACY OF RAINGAUGE STATIONS**

Optimum number of stations that should exist to have an assigned percentage of error in the estimation of mean rainfall is obtained by:

$N=(C_v/\varepsilon)^2$
N= optimal number of stations
ε = allowable degree of error in the estimate of the mean rainfallC_v=
coefficient of variation

If there are m stations in the catchment each recording rainfall values P_1, P_2,P_3, $P_i \ldots P_m$ in a known time, the coefficient of variation C_v is calculated as:

$C_v=(100\times\sigma_{m-1}/\bar{P})$
$\sigma_{m-1}=[\sum(P_i-\bar{P})^2/m-1]^{1/2}$
σ_{m-1}= standard deviation
P_i=precipitation magnitude in the i^{th} station$\bar{P}= (\sum P$

$_i)/m$=mean precipitation

- **PREPARATION OF DATA**

Before using any rainfall data in application, it's very necessary to check thedata for consistency and continuity.
The continuity of record may be broken with missing data due to manyreasons such as damage or fault in the rain gauge during a period.

Procedure of missing data estimation:

1. **Statement of the missing data problem:**

Given the annual precipitation values P_1,P_2,P_3,…P_m at neighboring M stations 1,2,3,..,M resp, it is required to find the missing annual precipitation P at a station X not included in the above M station.

Further, the normal precipitations $N_1, N_2, N_3, \ldots N_i, \ldots$ at each of theabove (M+1) stations, including station X, are known.

2. **Procedure:**
   i. If the normal precipitations at various stations are within about 10% of the normal annual precipitation at station X then P_x is calculated as:

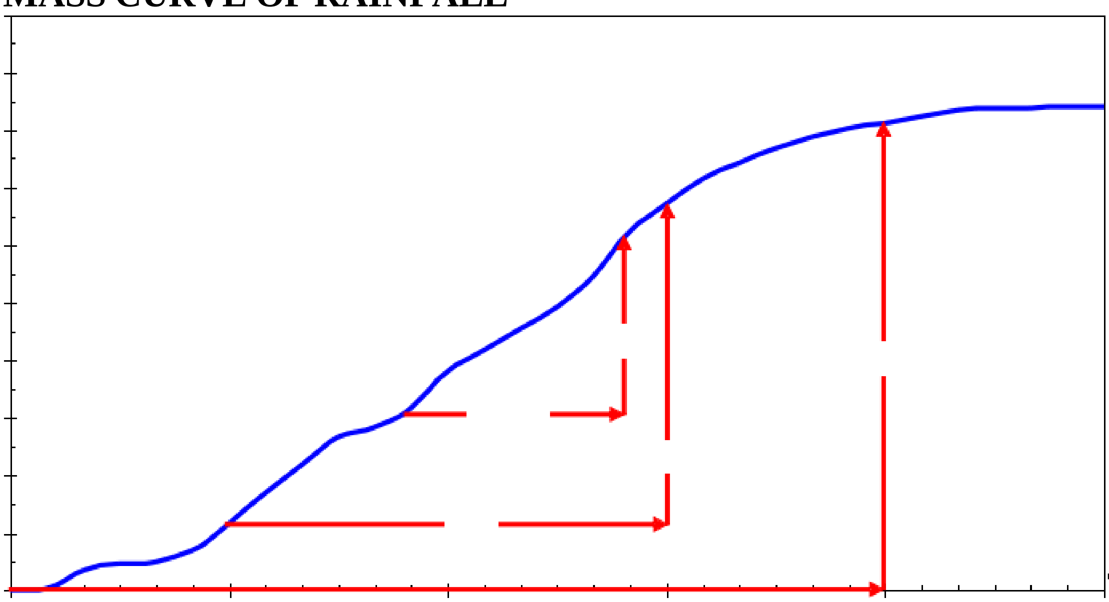
$$P_x=[P_1, P_2, P_3, \ldots P_m]/M$$

   ii. If the normal precipitation varies considerably then P_x is estimated by weighing the precipitation at various stations by the ratios of normal annual precipitations. This method is known as the "normal ratio method":

$$P_x=N_x[P_1/N_1, P_2/N_2, P_3/N_3, \ldots P_m/N_m]/M$$

Presentation of Rainfall Data

MASS CURVE OF RAINFALL-



The mass curve of rainfall is a plot of the accumulated precipitation against time, plotted in chronological order. Records of float type and weighing bucket type gauges are of this form. A typical mass curve of rainfall at a station during a storm is shown in the fig2.9. Mass curves of rainfall are very useful in extracting the information on the duration and the magnitude of a storm. Also, intensities at various time intervals in a storm can be obtained by a slope of the curve. For non- recording rain gauges; mass curves are prepared from knowledge of the approximate beginning and end of a storm and by using the mass curves ofadjacent recording gauge stations as a guide.

8

Cumulative Rainfall (in.)

7

6

5

4

3 30 min

2

1 1 hr

0

3.07 in

5.56 in

2 hr

8.2 in

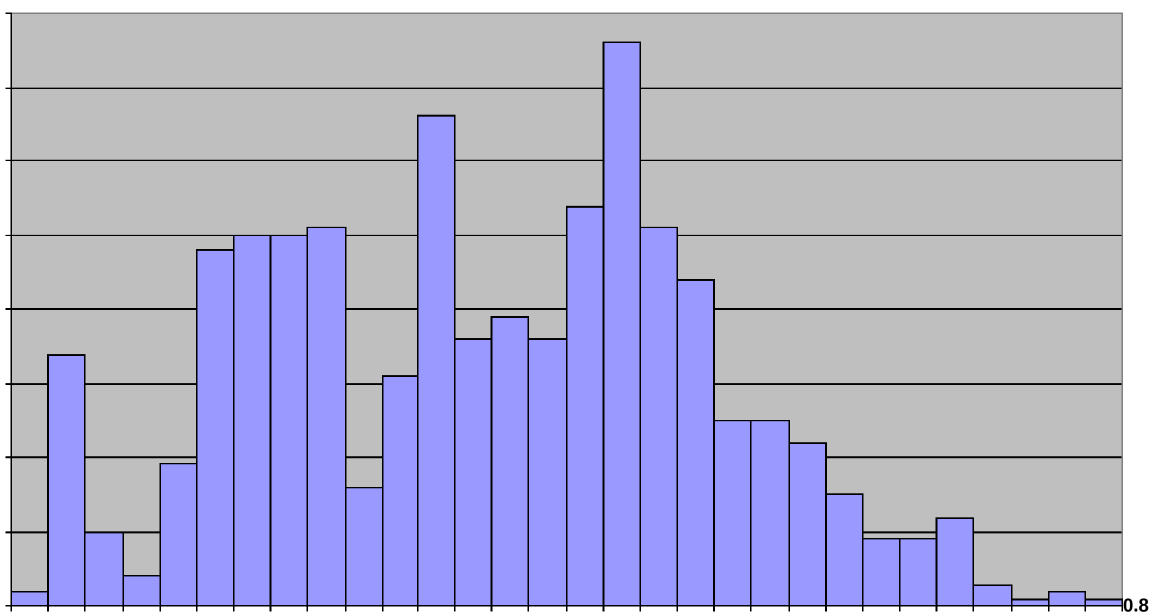
0 30 60 90 120 150

Time (min.)

Mass Curve of Rainfall

HYETOGRAPH

A hyetograph is a plot of the intensity of rainfall against the time interval. The hyetograph is derived from the mass curve and is usually represented as a bar chart (fig.2.10).It is a very convenient way of representing the characteristics of a storm and is particularly important in the development of design storms to predict extreme floods. The area under a hyetograph represents the total precipitation received in the period .The time interval used depends on the purpose ,in urban- drainage problems small durations are used while in flood-flow computations in larger catchments the intervals are of about 6h.



0.5

0.4

0.3

T RAINFALL-

ainfall, also known as station rainfall, refers to the rainfall data of .Depending upon the need, data can be listed as daily, weekly, monthl al or annual values for various periods .Graphically, these data a **POIN** 0.2

0.1

Point r station season represented 0 5 10

15 20 25 30 35 as plots of

ay,re

40 45 50 55 60 65 70 75 80 85 90 95 100 105 110 115 120 125 130 135 140 145 150 magnitude vs chronological time in the form of a bar

Time (min)

diagram. Such a plot however is not convenient for discerning a trend in the rainfall as there will be considerable variations in the rainfall values leading to rapid changes in the plot. The trend is often discerned by the method of moving averages, also known as moving means.

MOVING AVERAGE-

Moving average is a technique for smoothening out the high frequency fluctuations of a time series and to enable the trend, if any, to be noticed. The basic principle is that a window of time range m years is selected. Starting from the first set of m years of data, the average of the data for m years is calculated and placed in the middle year of the range m .The window is next moved sequentially one time unit (year) at a time and the mean of the m terms in the window is determined at each window location .The value of m can be 3 or more years; usually an odd value. Generally the larger size of the range m, the greater is the smoothening .There are many ways of averaging and the method described above is called CENTRAL SIMPLE MOVING AVERAGE.

Mean Precipitation Over An Area:

To convert the point rainfall values at various stations into an average value over acatchment, the following 3 methods are in use:

- Arithmetical-mean method
- Thiessen-mean method
- Isohyetal method

ARITHMETIC-MEAN METHOD:- When the rainfall measured at various stations in a catchment show little variation over catchment area I taken as the arithmetic mean of the station values. Thus, if P_1'P_2……P_i…P_n are the rainfall values in a given period in N stations within catchment then the value of the mean ppt. p over the catchment by the arithmetic mean method is

P==

THIESSEN-MEAN METHOD:

In this method, the rainfall recorded at each station is given a weightage on the basis of an area closest to the station. The procedure of determining the weighing area is as follows: Consider the catchment area as in Fig. below containing six rain gauge stations.

There are three stations outside the catchment but in its neighbor-hood. The catchment area is drawn to scale and the positions of the six stations marked on it. Stations 1 to 6 are joined to for a network of triangles. Perpendicular bisectors for each of the sides of the triangle are drawn. These bisectors form a polygon around each station. The boundary of the catchment, if it cuts the bisectors is taken as the outer limit of the polygon. Thus for station1, the bounding polygon is *abcd*. For station 2, *kade* is taken as the bounding polygon. These bounding polygons are called Thiessen polygons. The areas of these six Thiessen polygons are determined either with a planimeter or by using an overlay grid. If P_1,P_2,..,P_6 are the rainfall magnitudes recorded by the stations 1,2,…,6 respectively, and A_1,A_2,…,A_6 Are the respective areas of the Thiessen polygons then the average rainfall over the catchment P is given by

Thus,in general,for M stations,P==

The ratio A_i/A is called *weightage factor* for each station. The Thiessen-polygon method for calculating the average precipitation over an area is superior to the arithmetic-average method as some weightage is given to the various stations on a rational basis. Further, the rain-gauge stations outside the catchment are also used effectively. Once the weightage factors are determined, the calculation of P is relatively easy for fixed network for stations.

ISOHYETAL METHOD:

An isohyet is a line joining points of equal rainfall magnitude. In the isohyetal method, the catchment area is drawn to scale and the rain-gauge stations are marked. The recorded values for which areal average is to be determined are then marked on the plot at appropriate stations. Neighboring stations outside the catchment are also considered. The isohyets are then drawn by considering point rainfalls as guided and interpolating between them by eye.

The area between two adjacent isohyets are then determined with a planimeter. If the isohyets go out of catchment, the catchment boundary is used as the bounding line. The average value of the rainfall indicated by two isohyets is assumed to be acting over the inter-isohyet area. Thus P1,P2,…..,Pn are the values of isohyets andif a1,a2,….,an-1 are the inter-isohyet areas respectively, then the mean precipitations over the catchment of area A is

P=

It is superior to the two methods.

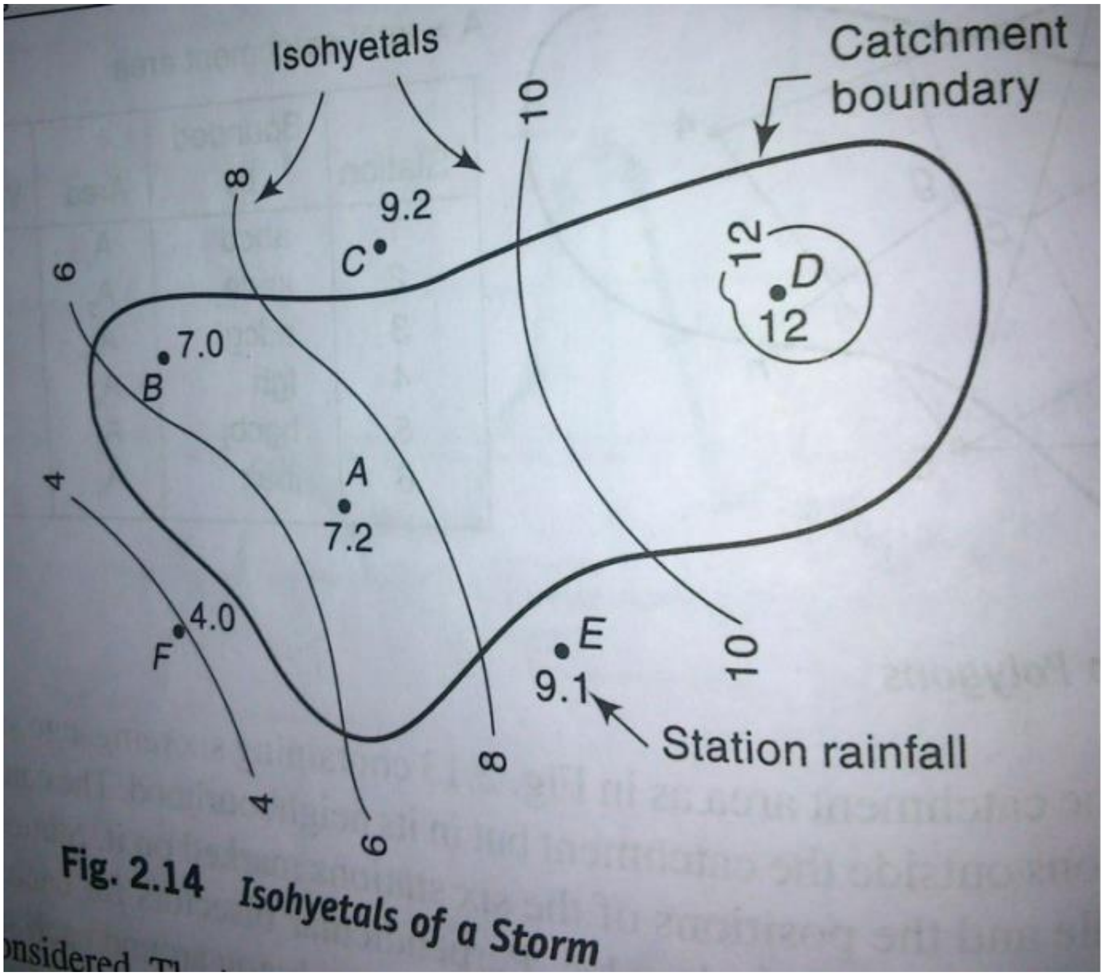


Fig. 2.14 Isohyetals of a Storm

DEPTH-AREA-DURATION (DAD) CURVE:-

DAD analysis is carried out to obtain a curve relating the depth of precipitation D, area of its coverage A and duration of occurrence of the storm D. A DAD curve is agraphical representation of the gradual decrease of depth of precipitation with the progressive increase of the area of storm, away from the storm center, for a given duration taken as the 3rd parameter. This gives a direct relationship between depth, area and duration of ppt. over the region for which the analysis is carried out. The purpose of DAD analysis is to determine the maximum precipitating amounts that have occurred over various sizes of drainage during the passage of storm periods of say 6h, 12h, 24h or other durations. There are two methods available for carrying out DAD analysis-

1. Mass curve method
2. Incremental-isohyetal method

Probable maximum precipitation (PMP):

It is defined as the estimate of the extreme maximum rainfall of a given duration that is physically possible over a basin under critical hydrological and meteorological conditions. This is used to compute flood by using suitable rainfall runoff model.
Two methods of PMP estimation are:

1. Statistical procedure
2. Meteorological approach

The statistical approach of PMP by using Chow's equationPMP
$= P + k\sigma$
Where P is the mean of annual maximum values, σ the standard deviation and k is the frequency factor which varies between 5 to 30 according to rainfall duration.

ABSTRACTIONS FROM PRECIPITATION

LOSSES FROM PRECIPITATION

For a surface water resource engineer, precipitation – runoff =losses
Precipitation – surface runoff = total losses
Total losses = Evaporation + Transpiration + Interception + Depression storage +Infiltration

Evaporation and its estimation :-

- Continuous natural process by which a substance changes from liquid to gaseous state.
- In arid regions, 90% loss is due to evaporation.
- So evaporation may otherwise be defined as loss of water to the atmosphere over the period under consideration.
- The main source of evaporation is solar radiation.
- 1 gm of water requires about 597cal of heat at 0^0c or 1gm of ice at 0 ^{0}c requires about 677cal for vaporization. This heat is latent heat of water and supplied by sun.
- Equivalent molar wt. Of air = 28.95,water vapour is = 18
  So water vapor is 62% lighter than air.
- Due to heat radiation, KE of water surface molecules increases . The surface tension and cohesion force can't hold the water molecules .They project into the atmosphere and due to lighter weight than air ,they can rise up to height where they condense .

- The net evaporation takes place during warm periods.
- Therefore the temperature of water surface is maintained at lower level.
- In a hydrologic cycle, evaporation takes place from all stages , even from thefalling raindrops .
- Average annual rainfall in India is about 1120mm which is equal to 370million hector-m of water.

Total runoff by all the rivers of the country = 170 million hector-mGround water recharge = 37 million hector-m
So loss due to evaporation and transpiration = 163 million hector-m

Factors responsible for evaporation :-

1. **.** Meteorological factors :
   A) Vapour pressure
   B) Solar radiation
   C) Air temperature
   D) Wind velocity
   E) Atmospheric pressure
2. Nature of evaporating surface (Heat storage in water bodies)

3 .Quality of water (Presence Soluble salts)Vapour pressure

- As per Dalton's law of evaporation, 'The rate of evaporation is proportional to the difference between saturated vapour pressure at the water temperature (e_w) and the actual vapour pressure in the air (e_a).

$\mathbf{E_L=C(e_w-e_a)}$

Where E_L=Rate of evaporation (mm/day)
C=Constant e_w, e_a (in mm)

- Evaporation continues till $e_w=e_a$ .If $e_w > e_a$, condensation takes place.Temperature
- Rate of evaporation is directly proportional to increase in water temperature.
- Slight increase due to increase in air temperature also.

Need and classification of irrigation- historical development and merits and demerits of irrigation- types of crops-crop season-duty, delta and base period- consumptive use of crops- estimation of Evapotranspiration using experimental and theoretical methods.

Irrigation- Definition

- Irrigation is an artificial application of water to the soil.
- It is usually used to assist the growing of crops in dry areas and during periods of inadequate rainfall.

Need of the Irrigation

- India is basically an agricultural country, and all its resources depend on the agricultural.
- Water is evidently the most vital element in the plant life.
- Water is normally supplied to the plants by nature through rains.
- However, the total rainfall in a particular area may be either insufficient, or ill-timed.
- Systematic irrigation system – Collecting water during the period of excess rainfall & releasing it to the crop when it is needed.

Less rainfall:

- Artificial supply is necessary
- Irrigation work may be constructed at a place where more water is available & than convey the water where there is less rainfall.

Non uniform rainfall:

- Rainfall may not be uniform over the crop period in the particular area.
- Rains may be available during the starting period of crop but no water may be available at end, with the result yield may be less or crop may be die.
- Collection of water during the excess rainfall & supplied to the crop during the period when there may be no rainfall.

Commercial crops with additional water:

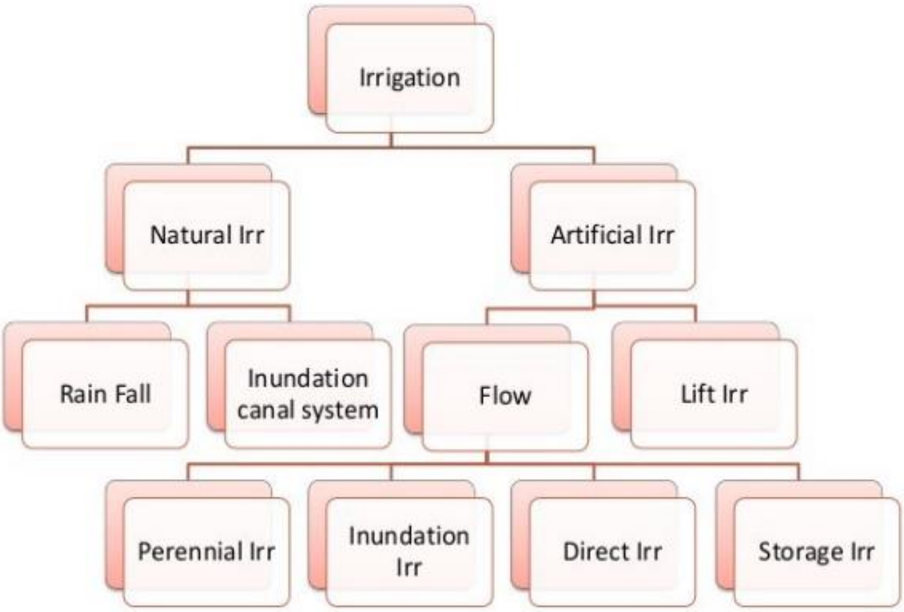
- Rainfall may be sufficient to raise the usual crop but more water may be necessary for raising commercial & cash crop . (Sugarcane, Tea, Tobacco, cotton, cardamom, & indigo)

Controlled water supply:

- Yield of the crop may be increased by the construction of proper distribution system

Benefits of Irrigation:

- Increase in food production
- Protection from famine
- Cultivation of cash crop (Sugarcane, Tobacco, & cotton)
- Addition to the wealth of the country
- Increase the prosperity of people
- Generation of hydro-electric power
- Domestic & industrial water supply
- Inland navigation
- Improvement of communication
- Canal plantations
- Improvement in the ground water storage
- General development of the country.



Types of Irrigation OR Classification of Irrigation:

Natural Irrigation

- No engineering structure is constructed.

1) Rainfall Irrigation

- Rainfall is only used for raising crops.

2) Inundation canal system

- Flood water is utilized for Irrigation purpose by properly direction flow of water.

Artificial Irrigation

- Properly designed engineering structure are constructed.

1) Flow irrigation

- Water flows to the irrigated land by gravity.
- Water sources is to be higher level than the irrigated land.

a) Perennial irrigation :

Water is supplied according to the requirements throughout the crop period throughstorage canal head works & Canal distribution system.

b) Inundation irrigation:

- Lands are submerged & throughly flooded when floods occur in the river.
- Lands are allowed to drain off & the crop are sown.
- Now the soil retains sufficient moisture for the crops to grow.

c) Direct irrigation :

- Water is directly diverted to the canal from the river is called Direct irrigation.
- Discharge in the river shall be higher than the water requirement during the crop period.
- A low diversion weir or a barrage is constructed across the river to rise the water level and divert the same to the canal.
- Direct irrigation can be adopted only where there is enough flow in the river to provide sufficient quantity of water required for irrigation throughout the crop period.

d) Storage Irrigation:

- River flow is not perennial or insufficient during crop period, Storage Irrigation is adopted.
- A dam is construction across the river to store water in the reservoir.
- In some area rain water that run off from a catchment area is stored in tanks and is used for irrigation during the crop period.

2) Lift or well Irrigation:

- Water is lifted up by mechanical such as pump etc or manual to supply for irrigation .
- Lift irrigation is adopted when the water source is lower than the level of lands to be irrigated.

Historical development of Irrigation

- Historically, civilizations have been dependent on development of irrigated agriculture.
- Archaeological investigation has identified evidence of irrigation in **Mesopotamia, Ancient Egypt & Ancient Persia** (at present Iran) as far back as the 6th millennium BCE.

- In the **“Zana”** valley of the Andes Mountain in **Peru**, archaeologists found remains of three irrigation canals radiocarbon dated from the 4th millennium BCE, the 3rd Millennium BCE & the 9th century CE, These canals are the earliest record of irrigation in the new world.
- The **Indus valley** civilization in Pakistan & North India (from 2600 BCE) also had an early canal irrigation system. Large scale agriculture was used for the purpose of irrigation.
- There is evidence of **ancient Egyptian** Pharaoh Amenemhet-III in the 12th dynasty (about 1800 BCE) using the natural lake of the Faiyum Oasis as a reservoir to store surpluses of water for use during the dry seasons, the lake swelled annually from flooding of the Nile.
- The irrigation works of **ancient Sri Lanka**, the earliest dating from about 300 BCe, in the reign of King Pandukabhaya & under conditions development for the next thousand years, were one of the most complex irrigation systems of the ancient world.
- In the Szechwan region **ancient China** the Dujiangyan Irrigation System was built in 250 BCE to irrigate a large area & it still supplies water today.
- In the **Americas**, extensive irrigation systems were created by numerous groups in prehistoric times. One example is seen in the recent archaeological excavations near the Santa Cruz River in Tucson, Arizona. They have located a village site dating from 4000 years ago.

Present status of Irrigation:

- In the middle of 20th century, the advent of diesel & electric motors led for the first time to system that could pump groundwater out of major aquifers faster than it was recharged.
- This can lead to permanent loss of aquifer capacity, decreased water quality, ground subsidence & other problems.
- The largest contiguous areas of high irrigation density are found in North India & Pakistan along the rivers Ganges & Indus, in the Hai He, Huang He & Yangtze basins in China, along the Nile River in Egypt & Sudan, in the Mississippi-Missouri river basin & in parts of California.

Developmental Aspects of Irrigation:

Irrigation is practiced to maintain the different developmental parameters. Those are:

1. To make up for the soil moisture deficit.
2. To ensure a proper & sustained growth of crops.
3. To make harvest safe.
4. To colonize the cultivable wasteland for horizontal expansion of cultivation.
5. To shift from seasonal cultivation.
6. To promote more intensive cultivation by multiple cropping.
7. To improve the level of agricultural productivity by acting as an agent for adoption of modern technology.
8. To lessen the regional & size-class inequalities in agricultural productivity that will reduce in turn socio-economic imbalances.

Advantages of irrigation

Advantages of irrigation can be direct as well as indirect.

I. Direct Benefits

- The grower has many choices of crops and varieties and can go for multiple cropping for cultivation
- Crop plants respond to fertilizer and other inputs and there by productivity is high.
- Quality of the crop is improved.
- Higher economic return and employment opportunities. It makes economy drought proof.
- Development of pisciculture and afforestation. Plantation is raised along the banks of canals and field boundaries.
- Domestic water supply, hydel power generation at dam site and means of transport where navigation is possible.
- Prevention of damage through flood.

II. Indirect Benefits

- Increase in gross domestic product of the country, revenue, employment, land value, higher wages to farm labour, agro-based industries and groundwater storage.
- General development of other sectors and development of the country
- Increase of food production.
- Modify soil or climate environment – leaching.
- Lessen risk of catastrophic damage caused by drought.
- Increase income & national cash flow.
- Increase labor employment.
- Increase standard of living.
- Increase value of land.
- National security thus self sufficiency.
- Improve communication and navigation facilities.
- Domestic and industrial water supply.
- Improve ground water storage.
- Generation of hydro-electric power.

Disadvantages of Irrigation

The following are the disadvantages of irrigation.

- Water logging.

- Salinity and alkalinity of land.
- Ill aeration of soil.

- Pollution of underground water.
- Results in colder and damper climate causing outbreak of diseases like malaria.

Types of Crops:

1) **Wet crops-** which lands are irrigated and than crop are cultivation
2) **Dry crops-**which do not need irrigation.
3) **Garden crops-** which need irrigation throughout the year
4) **Summer crop (Kharif)-**which are sown during the south west monsoon & harvested in autumn.
5) **Winter crops(rabi)-**which are sown in autumn & harvested in spring.

S.No	Crop	Sown	Harvested
1	Summer season (Kharif crop)		
	Rice	June -July	Oct-Nov
	Maize	June -July	Sep-Oct
	Bajra	June -Aug	Sep-Oct
	Jowar	June -July	Oct-Nov
	Pulses	June -July	Nov-Dec
2	Winter season (Rabi Crops)		
	Wheat, Barley, peas	Oct-Nov	March - April
	Gram	Sep- Oct	March - April
	Tobacco	Feb-Mar	June
	Potato	Oct	Feb
3	Eight Months Crop cotton	May-June	Dec-Jan
4	Annual crop sugercane	Feb-March	Dec-march

6)
Cash crop – which has to be encased in the market. As it cannot be consumed directly by the cultivators.

Crop

Seasons:

- In north India the crop season is divided as Rabi & Kharif.

- Rabi crops are called as winter crops and kharif crops are called as summer crops.
- Kharif crops required more water than rabi crops.
- Rabi starts from 1 st oct and ends on 31 march
- In TamilNadu crops are classified as wet and dry crops.

Crops rotation:

Rotation of crops implies the nature of the crop sown in a particular field is changed year after year.

Necessity for rotation

- The necessity for irrigation when the same crop is grown again and again in the same field, the fertility of land gets reduced as the soil becomes deficient in plant foods favorable to that particular crop.
- If different crops were to be raised there would certainly be more balanced fooding and soil deficient in one particular type of nutrient is allowed to recouped.
- Crop diseases and insect pests will multiply at an alarming rate, if the same crop is to be grown continuously. Rotation will check the diseases.
- A leguminous crop (such as gram) if introduced in rotation will increase nitrogen content of soil thus increasing its fertility.
- The deep rooted and shallow rooted crops in rotation draw their food from different depths of soil. The soil will be better utilized.
- Rotation of crops is beneficial to the farmers as there would be rotation of cash crops, fooder and soil renovating crops.

General rotation of crops can be summarized as:

1. Wheat – great millet – gram.
2. Rice – gram
3. Cotton – wheat – gram.
4. Cotton – wheat – sugarcane
5. Cotton – great millet – gram.

Consumptive Use of Water

- Considerable part of water applied for irrigation is lost by evaporation & transpiration.
- This two processes being difficult to separate are taken as one and called Vapor- transpiration or Consumptive use of water.

Duty :

Delta:

Duty- Area of the crop irrigated/ Volume of water required.

- The depth of water required every time, generally varies depending upon the type of the crop.

- The total depth of water required a crop to nature is called delta.
- Crop period-the time from the instant of its sowing to the instant of harvesting.
- Base Period-time b/w the first supply of water to the land and the last watering before harvesting.

Factor affecting the duty:

1) Soil Moisture

- In clayey soil less water is required since its retentive capacity is more.
- Pervious soil it will be more.

2) Topography

- Uniform distribution depends on topography.
- If the area is sloping the lower portion will get more water than the flat portion, & hence Water requirement is increase.

3) Nature of rainfall

- If rainfall is high over the crop period water requirement becomes less, otherwise it will be more.

4) Nature of crop irrigated

- Dry crop required less water where as wed crop required more water.

5) Method of cultivation:

- If the fields are properly ploughed it will have high retentive capacity & the number of watering are reduced.

6) Season of crop

- Less irrigation water is required for rainy season crop and the duty increased.
- If the crop grown in summer, more irrigation water is required & the duty gets decreased

7) System of Irrigation

- In perennial irrigation, continuous supply of water is given & hence water table is kept high& percolation losses is minimized
- In inundation type wastage is more by deep percolation.

8) Canal Condition

- Well maintained canal will have more duty as the losses is less.

Improving Duty

1. The water losses can be reduced by having the irrigated area nearer to the head of thecanal.

2. Evaporation losses can be minimized by using the water as quickly as possible.

3. Water losses can be minimized by lining the canals.
4. The cultivators should be trained to use water economically without wasting.
5. The soil properties should be studied by establishing research stations in villages.

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**.

Duty and Delta of a Crop Delta:

The total quantity of water required by the crop for its full growth may be expressed in hectare-meter 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 is called delta (Δ).

Problem –1: If rice requires about 10 cm 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*:**

No. of watering required = 120/10 = 12

Total depth of water required in 120 days = $10 \times 12 = 120$ cm

Δ for rice = 120 cm

Problem –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*:**

No. of watering required = 140/28 = 5

Total depth of water required in 140 days = $7.5 \times 5 = 37.5$ cm

Δ for wheat = 37.5 cm

Duty:

- It may be defined as the number of hectares of land irrigated for full growth of a given crop by supply of 1 m^3/s of water continuously during the entire base of that crop.
- Simply we can say that, the area (in hectares) of land can be irrigated for a crop period, B (in days) using one cubic meter of water.

Factors on which duty depends:

1. Type of crop

2. Climate and season
3. Useful rainfall
4. Type of soil
5. Efficiency of cultivation method

Importance of Duty

- It helps us in designing an efficient canal irrigation system.
- Knowing the total available water at the head of a main canal, and the overall duty for all the crops required to be irrigated in different seasons of the year, the area which can be irrigated can be worked out.
- *Inversely, if we know the crops area required to be irrigated and their duties, we can work out the discharge required for designing the channel.*

Measures for improving duty of water:

The duty of canal water can certainly be improved by effecting economy in the use of water by resorting to the following precautions and practices:

(1) Proper Ploughing:

Ploughing should be done properly and deeply so that the moisture retaining capacity of soil is increased.

(2) Methods of supplying water:

The method of supplying water to the agriculture land should be decided according to thefield and soil conditions. For example,

- Furrow method For crops sown ion rows
- Contour method For hilly areas
- Basin For orchards
- Flooding For plain lands

(3) Canal Lining:

It is provided to reduce percolation loss and evaporation loss due to high velocity.

(4) Minimum idle length of irrigation Canals:

The canal should be nearest to the command area so that idle length of the canal is minimumand hence reduced transmission losses.

(5) Quality of water:

Good quality of water should be used for irrigation. Pollution en route the canal should beavoided.

(6) Crop rotation:

The principle of crop rotation should be adopted to increase the moisture retaining capacity and fertility of the soil.

Consumptive use of crops

Definition:

- It is the quantity of water used by the vegetation growth of a given area.
- It is the amount of water required by a crop for its vegetated growth to evapotranspiration and building of plant tissues plus evaporation from soils and intercepted precipitation.
- It is expressed in terms of depth of water. Consumptive use varies with temperature,humidity, wind speed, topography, sunlight hours, method of irrigation, moistureavailability.

Mathematically,

Consumptive Use = Evapotranspiration = Evaporation + transpiration

- It is expressed in terms of depth of water.

Factors Affecting the Consumptive Use of Water

Consumptive use of water varies with:

1. Evaporation which depends on humidity
2. Mean Monthly temperature
3. Growing season of crops and cropping pattern
4. Monthly precipitation in area
5. Wind velocity in locality
6. Soil and topography
7. Irrigation practices and method of irrigation
8. Sunlight hours

Types of Consumptive Water Use

Following are the types of consumptive use,

1. Optimum Consumptive Use
2. Potential Consumptive Use
3. Seasonal Consumptive Use

1. Optimum Consumptive Use:

It is the consumptive use which produces a maximum crop yield.

2. Potential Consumptive Use:

If sufficient moisture is always available to completely meet the needs of vegetation fullycovering the entire area then resulting evapotranspiration is known as Potential Consumptive Use.

3. Seasonal Consumptive Use:

The total amount of water used in the evapo-transpiration by a cropped area during the entire growing season.

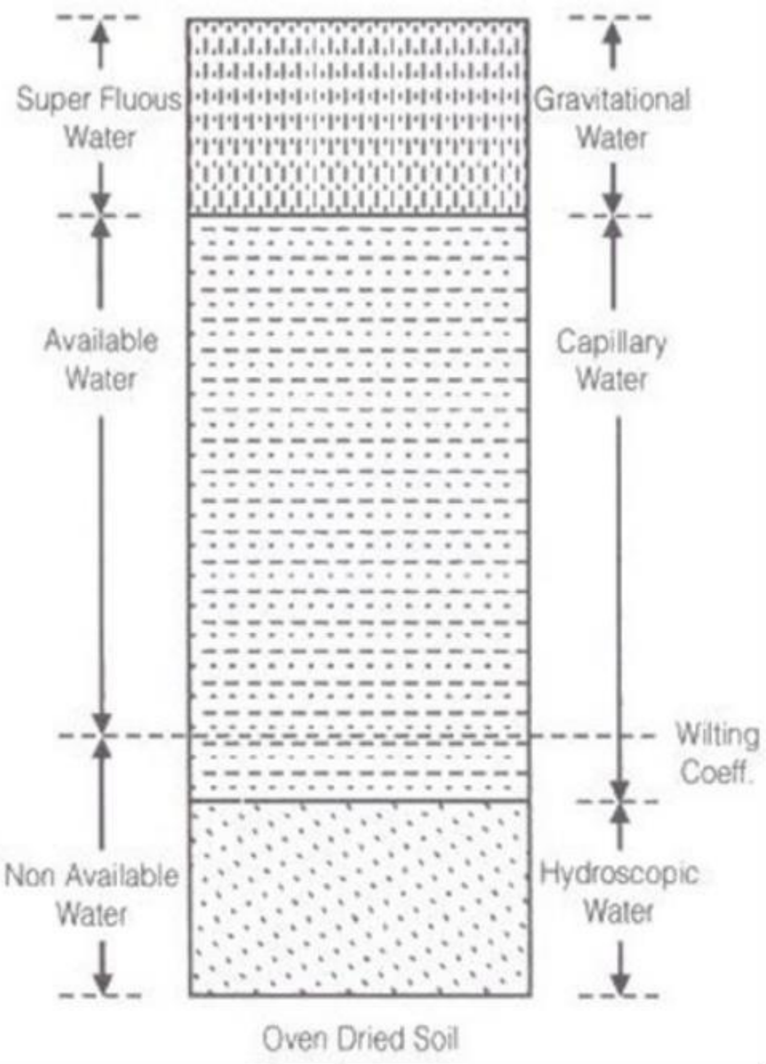
Crop Water Requirements

Soil moisture

Classes and availability of soil water

Water present in the soil may be to classified under three heads

1. Hygroscopic water
2. Capillary water
3. Gravitational water



Hygroscopic water

Water attached to soil particles through loose chemical bonds is termed hygroscopic water. This water can be removed by heat only. But the plant roots can use a very small fraction of this soil moisture under drought conditions.

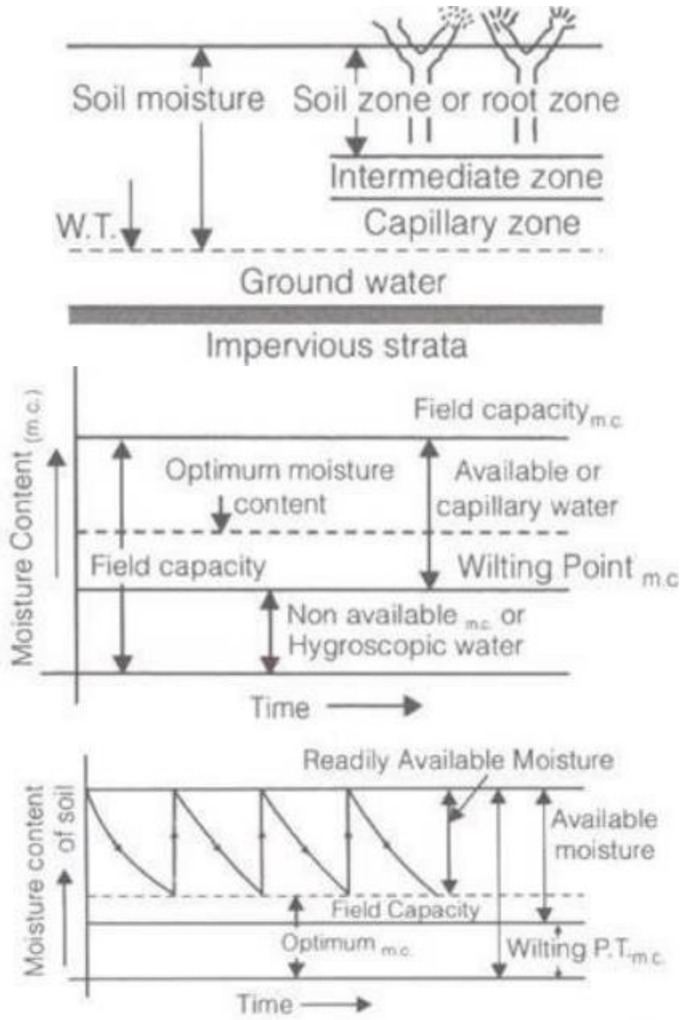
Capillary water

The capillary water is held within soil pores due to the surface tension forces (against gravity) which act at the liquid-vapour (or water-air) interface.

Gravitational water

Gravity water is that water which drains away under the influence of gravity. Soon after irrigation (or rainfall), this water remains in the soil and saturates the soil, thus, preventing circulation of airin the void spaces.

(1)



(4) Available moisture for the plant $= F_C - \phi$

(5) Readily available moisture for the plant = FC -
MoHere FC= field capacity

φ = wilting point or wilting coefficient below plant can’t survive.

$$= \frac{weight\ /\ readily\ available\ moisture\ depth}{consumptive\ use\ rate}$$

Mo= Readily available moisture content

(6) Frequency of Irrigation

(7) $$F_C = \frac{weight\ of\ water\ stored\ in\ soil\ of\ unit\ area}{weight\ of\ same\ soil\ of\ unit\ area}$$

where, weight of water stored in soil of unit area $= \gamma_w \cdot d_w \cdot 1 \cdot$ Weight of some soil of unit area $= \gamma \cdot d \cdot l$

dw= depth of water stored in root zone.

(8) $d_w = \frac{\gamma \cdot d}{\gamma_w} \cdot F_C \quad \gamma \rightarrow$ dry unit wt. of soil

(9) Available moisture depth to plant $d_w^{'} = \frac{\gamma . d}{\gamma_w}(F_C - \phi)$

(10) Readily available moisture depth to plant $d_w^{'} = \frac{\gamma . d}{\gamma_w}(F_C - m_o)$

(11) $F_C = n / G$ where, G = specific gravity and n = porosity

Duty and delta

Duty:

- The duty of water is the relationship between the volume of water and the area of the crop it matures.
- It is defined as the area irrigated per cumec of discharge running for base period B.
- The duty is generally represent by D.

Delta:

- It is the total depth of water required by a crop during the entire base period and is represented by the symbol Δ.

Relation between duty and delta

$$\Delta = \frac{8.64B}{D}$$

Where,

- Δ=Delta in meter

- D = Duty in Ha/cumec
- B = Base period in days

$\Delta = \frac{2B}{D}$ Also

Where,

- Δ=Delta in meter
- B = Base period in days
- D = Duty in acre/cures

Irrigation Requirements of crops

(1) Consumptive Irrigation Requirement (CIR)

CIR = Cu- P_{eff}

Where, Cu= total consumptive use requirementP_{eff}= Effective rainfall.

(2) Net Irrigation Requirement (NIR)

NIR = CIR + Leaching requirement

(3)

$$FIR = \frac{NIR}{\eta_a}$$

(4) Field irrigation requirement (FIR)

(5) Gross irrigation requirement, (GIR)

$$GIR = \frac{FIR}{\eta_c}$$

The equation for this method is,

U = 0.0015 H + 0.9 (Over specified)

U = Consumptive Use

H = Accumulated degree days during the growing season computed from maximum temperature above 32 °F

UNIT -3

DIVERSION AND IMPOUNDING STRUCTURES

Types of Impounding structures - Gravity dam – Forces on a dam -Design of Gravity dams; Earth dams, Arch dams- Diversion Head works - Weirs and Barrages.

Impounding structure

- Impounding structure or dam means a man-made device structure, whether a dam across a watercourse or other structure outside a watercourse, used or to be used to retain or store waters or other materials.
- The term includes: (i) all dams that are 25 feet or greater in height and that create an impoundment capacity of 15 acre-feet or greater, and (ii) all dams that are six feet or greater in height and that create an impoundment capacity of 50 acre-feet or greater.

Diversion headwork.

- Any hydraulic structure, which supplies water to the off-taking canal, is called a headwork.
- A diversion headwork serves to divert the required supply in to the canal from the river.

The purposes of diversion headwork.

1. It raises the water level in the river so that the commanded area can be increased.
2. It regulates the intake of water in to the canal.
3. It controls the silt entry in to the canal.
4. It reduces fluctuations in the level of supply in the river.
5. It stores water for tiding over small periods of short supplies.

Weir

The weir is a solid obstruction put across the river to raise its water level and divert the water in to the canal. If a weir also stores water for tiding over small periods of short supplies, it is called a storage weir.

The component parts of diversion headwork

- Weir or barrage
- Divide wall or divide groyne
- Fish ladder
- Head sluice or canal head regulator
- Canal off-takes
- Flood banks
- River training works.

Dam

A dam is a hydraulic structure constructed across a river to store the suppliy for a longer durationand release it through designed outlets.

Types of Dams

Based on Materials of Construction

- Rigid.
- Non-Rigid.

Based on Structural Behaviour

- Gravity Dam.
- Arch Dam.
- Buttress Dam.
- Embankment Dam.

Based on Functions

- Storage Dam.
- Detention Dam.
- Diversion Dam.
- Coffer dam.

Based on Hydraulic Behaviour

- Over Flow Dam.
- Non Over Flow Dam.

General Types

- Solid gravity dam (masonry, concrete, steel and timber)
- Arch dams
- Buttress dams
- Earth dams
- Rockfill dams
- Combination of rockfill and earth dams

Gravity dam

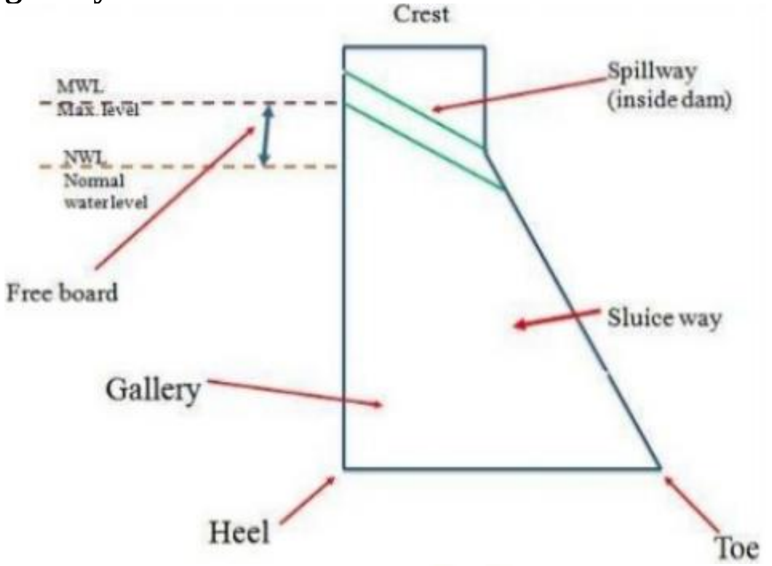
- A gravity dam is a structure so proportioned that its own weight resists the forces exerted upon it. It requires little maintenance and it is most commonly used.
- A Gravity dam has been defined as a “structure which is designed in such a way that its own weight resist the external forces”.
- This type of a structure is most durable and solid and requires very less maintenance.
- Such dams are constructed of masonry or Concrete.
- However, concrete gravity dams are preferred these days and mostly constructed.
- The line of the upstream face or the line of the crown of the dam if the upstream face is

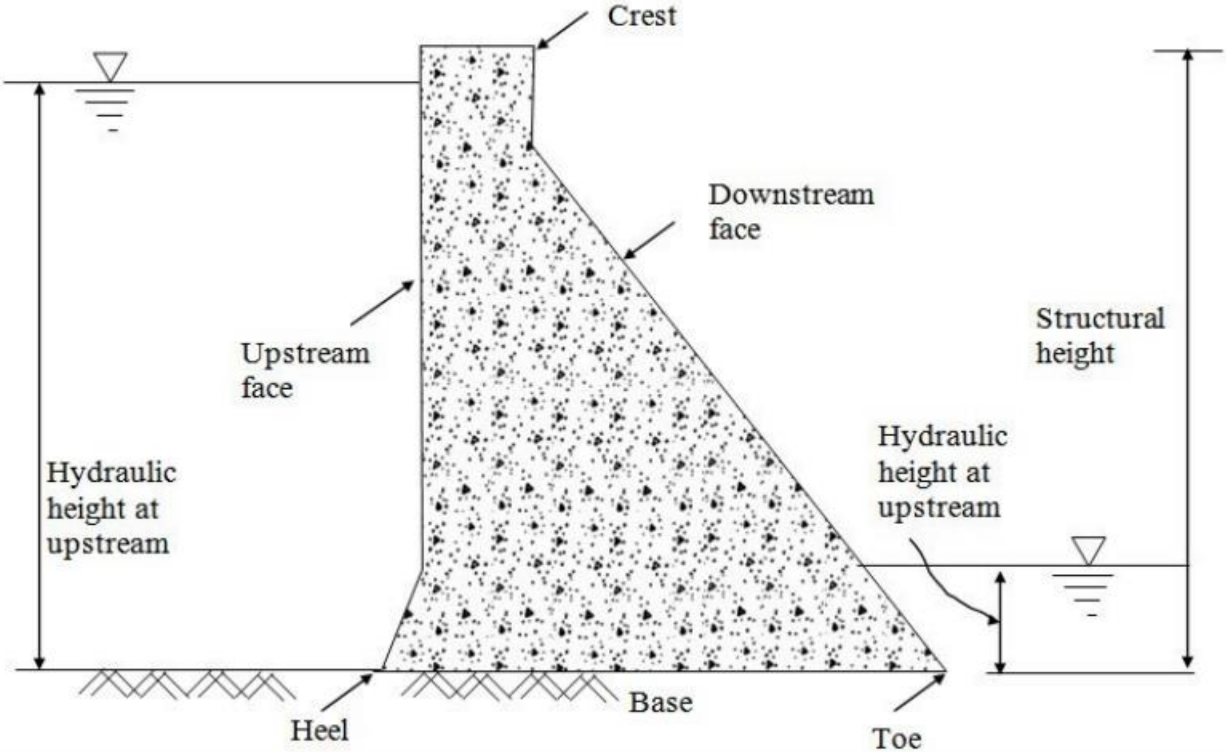
sloping, is taken as the reference line for layout purpose etc. and is known as the Base line of the dam or the “Axis of The Dam” When suitable conditions are available such dams canbe constructed up to great heights.

The different components of a solid gravity dam are

- Crest.
- Free Board.
- Heel.
- Toe.
- Sluice Way.
- Drainage Gallery.

Typical cross section of gravity Dam:





Heel: contact with the ground on the upstream side

Toe: contact on the downstream side

Abutment: Sides of the valley on which the structure of the dam rest

Galleries: small rooms like structure left within the dam for checking operations.

Diversion tunnel: Tunnels are constructed for diverting water before the construction of dam. This helps in keeping the river bed dry.

Spillways: It is the arrangement near the top to release the excess water of the reservoir to downstream side

Sluice way: An opening in the dam near the ground level, which is used to clear the silt accumulation in the reservoir side.

Forces Acting on Gravity Dam

The Various external forces acting on Gravity dam may be:

- Water Pressure
- Uplift Pressure
- Pressure due to Earthquake forces
- Silt Pressure
- Wave Pressure
- Ice Pressure
- The stabilizing force is the weight of the dam itself

Self weight of the Dam

Self weight of a gravity dam is main stabilizing force which counter balances all the external forces acting on it.

For construction of gravity dams the specific weight of concrete & stone masonry shouldn't be lessthan 2400 kg/m^3 & 2300 kg/m^3 respectively.

The self weight of the gravity dam acts through the centre of gravity of the.

Its calculated by the following formula – $W = \gamma_m \ X\ Volume$

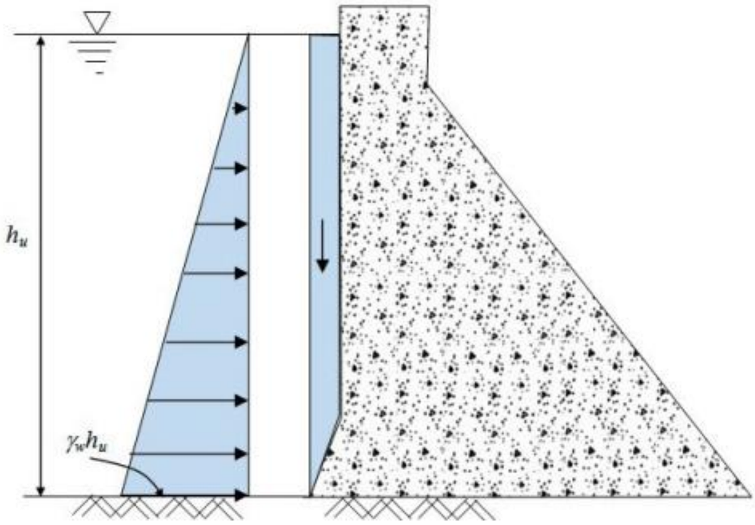
Where γ_m is the specific weight of the dam's material.

Water pressure

- Water pressure on the upstream side is the main destabilizing force in gravity dam.
- Downstream side may also have water pressure.
- Though downstream water pressure produces counter overturning moment, its magnitude is much smaller as compared to the upstream water pressure and therefore generally not considered in stability analysis.

- Water Pressure is the most major external force acting on a gravity dam.
- On upstream face pressure exerted by water is stored upto the full reservoir level. The upstream face may either be vertical or inclined.
- On downstream face the pressure is exerted by tail water. The downstream face is always inclined. It is calculated by the following formula – $P = \frac{1}{2} \gamma_w x h^2$

Where γ_w is the unit weight of water & h is the height of water.



Uplift water pressure

- The uplift pressure is the upward pressure of water at the base of the dam as shown in Figure 29.3. It also exists within any cracks in the dam.
- The water stored on the upstream side of the dam has a tendency to seep through the soil below foundation.
- While seeping, the water exerts a uplift force on the base of the dam depending upon the head of water.
- This uplift pressure reduces the self weight of the dam.
- To reduce the uplift pressure, drainage galleries are provided on the base of the dams.
- It is calculated by the following formula – $U = \frac{1}{2} \gamma_w x h x B$

Where 'B' is the width of the base of the dam.

Wave Pressure

When very high wind flows over the water surface of the reservoir, waves are formed which exert pressure on the upstream part of the dam.

The magnitude of waves depend upon –

- The velocity of wind.
- Depth of Reservoir.
- Area of Water Surface.

It is calculated by the following formula - $P_v = 2.4\, \gamma_w\, x h_w$

Where ‘h_w’ is the wave height.

WIND PRESSURE :

- The top exposed portion on the dam is small & hence the wind pressure on this portion of dam is negligible.
- But still an allowance should be made for the wind pressure at the rate of about 150 kg/m^2 for the exposed surface area of the upstream & downstream faces.

SEISMIC FORCES :

- Dams are subjected to vibration during earthquakes.
- Vibration affects both the body of the dam as well as the water in the reservoir behind the dam.
- The most danger effect occurs when the vibration is perpendicular to the face of the dam.
- Body Forces: Body force acts horizontally at the center of gravity and is calculated as: $P_{em} = \alpha\, x\, W$
- Water Force: Water vibration produces a force on the dam acting horizontally & calculated by: $P_{ew} = \frac{2}{3} C_e \alpha h^2$

ELEMENTRY PROFILE

- When water is stored against any vertical face, then it exerts pressure perpendicular to the face which is zero at top & maximum at bottom.
- The required top thickness is thus zero & bottom thickness is maximum forming a right angled triangle with the apex at top, one face vertical & some base width.
- Two conditions should be satisfied to achieve stability
  - **When empty** - The external force is zero & its self weight passes through C.G. of the triangle.
  - **When Full** - The resultant force should pass through the extreme right end of the middlethird.

The limiting condition is – $h = \frac{\sigma_c}{\gamma(1+S)}$

- where, σc=allowable compressive stressc=allowable compressive stress

Practical Profile

- Various parameters in fixing the parameters of the dam section are,

- Free Board –IS 6512, 1972 specifies that the free board will be 1.5 times the wave height above normal pool level.
- Top Width – The top width of the dam is generally fixed according to requirements of the roadway to be provided. The most economical top width of the dam is 14 % of its height.

- Base Width – The base width of the dam shall be safe against overturning, sliding & no tension in dam body.

For elementary profile –

- When uplift is considered, $B=\frac{h}{\sqrt{S}}$
- When uplift isn't considered $B=\frac{h}{\sqrt{S-1}}$

Low Gravity Dam

- A low gravity dam is designed on the basis if of elementary profile, where the resultant force passes through the middle-third of its base.
- The principal stress is given by – σc=allowable compressive stress = γ H (S – C + 1) Where, σc=allowable compressive stress=principal stress, γ=unit weight, S=Specific Gravity and C=A constant.
- The principal stress varies with 'H' as all other terms are constant. To avoid failure of the dam the value of 'σc=allowable compressive stress' shouldn't exceed allowable working stress(f). F = γ H (S – C + 1)

High Gravity Dam

- The high gravity is a complicated structure, where the resultant force may pass through a point outside the middle-third of the base.
- The section of the dam is modified by providing extra slope on the upstream and downstream side.
- The condition for the high gravity dam are $H>\frac{f}{w(S+1)}$

– Where, f=allowable working stress.

Failure of Gravity Dam

Failure of gravity dams are caused due to,

- Sliding – It may take place on a horizontal joint above formation, on the foundation. Sliding takes place when total horizontal forces are grater than the combined shearing resistance of the joint and the static friction induced by total vertical forces.
- Overturning – A dam fails in overturning when total horizontal forces acting on the dam section are quite great in comparison with total vertical forces. In such cases the resultant of two passes outside the limits of the dam.
- Dam may fail when tension is produced in the concrete.
- Dam may fail in crushing.

Precautions against Failure

- To prevent overturning, the resultant of all forces acting on the dam should remain within the middle-third of the base width of the dam.

- In the dam, the sliding should be fully resisted when the condition for no sliding exists in the dam section.

- In the dam section, the compressive stresses of concrete or masonry should not exceed the permissible working stress to avoid failure due to crushing.
- There should be no tension in the dam section to avoid the formation of cracks.
- The factor of safety should be maintained between 4 to 5.

Temperature Control

During setting of concrete heat of hydration is evolved producing internal temperature stresses resulting in development of internal cracks can get formed.

To control the temperature the following steps may be taken

1. Low heat cement may be used in concrete.
2. The water & coarse aggregates should be cooled down to 5°C by suitable means before mixing.
3. During laying the height of concrete blocks should not be more than 1.5 m. It helps radiate heat to the atmosphere more quickly.
4. The water is cooled by crushed ice before using it for the curing purpose.

Advantages

Gravity dams are more suitable in narrow valle**CROP WATER REQUIREMENT**

Need and classification of irrigation- historical development and merits and demerits of irrigation-types of crops-crop season-duty, delta and base period- consumptive use of crops- estimation of Evapotranspiration using experimental and theoretical methods.

Irrigation- Definition

- Irrigation is an artificial application of water to the soil.
- It is usually used to assist the growing of crops in dry areas and during periods of inadequate rainfall.

Need of the Irrigation

- India is basically an agricultural country, and all its resources depend on the agricultural.
- Water is evidently the most vital element in the plant life.
- Water is normally supplied to the plants by nature through rains.
- However, the total rainfall in a particular area may be either insufficient, or ill-timed.
- Systematic irrigation system – Collecting water during the period of excess rainfall & releasing it to the crop when it is needed.

Less rainfall:

- Artificial supply is necessary
- Irrigation work may be constructed at a place where more water is available & than convey

the water where there is less rainfall.

Non uniform rainfall:

- Rainfall may not be uniform over the crop period in the particular area.
- Rains may be available during the starting period of crop but no water may be available at end, with the result yield may be less or crop may be die.
- Collection of water during the excess rainfall & supplied to the crop during the period when there may be no rainfall.

Commercial crops with additional water:

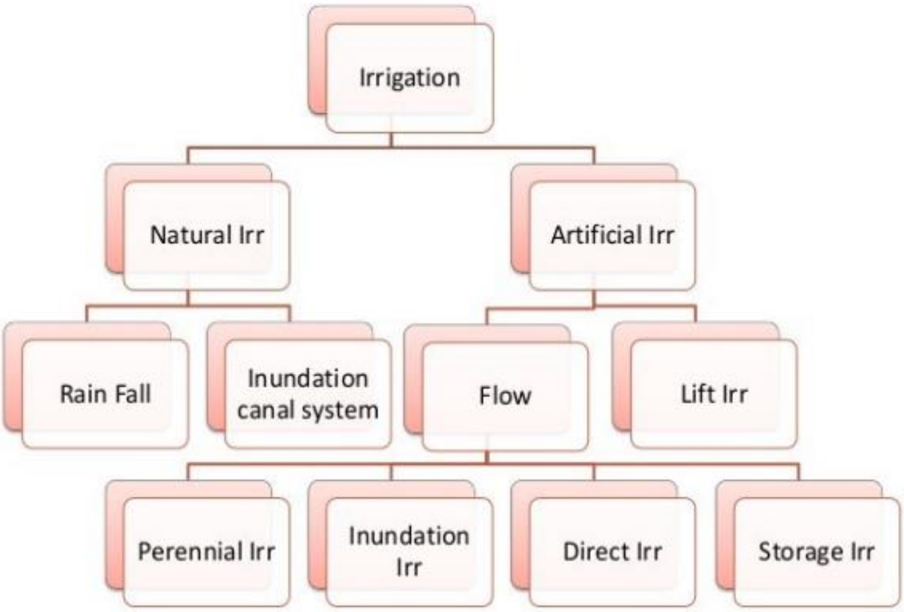
- Rainfall may be sufficient to raise the usual crop but more water may be necessary for raising commercial & cash crop . (Sugarcane, Tea, Tobacco, cotton, cardamom, & indigo)

Controlled water supply:

- Yield of the crop may be increased by the construction of proper distribution system

Benefits of Irrigation:

- Increase in food production
- Protection from famine
- Cultivation of cash crop (Sugarcane, Tobacco, & cotton)
- Addition to the wealth of the country
- Increase the prosperity of people
- Generation of hydro-electric power
- Domestic & industrial water supply
- Inland navigation
- Improvement of communication
- Canal plantations
- Improvement in the ground water storage
- General development of the country.



Types of Irrigation OR Classification of Irrigation:

Natural Irrigation

- No engineering structure is constructed.

3) **Rainfall Irrigation**

- Rainfall is only used for raising crops.

4) Inundation canal system

- Flood water is utilized for Irrigation purpose by properly direction flow of water.

Artificial Irrigation

- Properly designed engineering structure are constructed.

3) Flow irrigation

- Water flows to the irrigated land by gravity.
- Water sources is to be higher level than the irrigated land.

e) Perennial irrigation :

Water is supplied according to the requirements throughout the crop period throughstorage canal head works & Canal distribution system.

f) Inundation irrigation:

- Lands are submerged & throughly flooded when floods occur in the river.
- Lands are allowed to drain off & the crop are sown.
- Now the soil retains sufficient moisture for the crops to grow.

g) Direct irrigation :

- Water is directly diverted to the canal from the river is called Direct irrigation.
- Discharge in the river shall be higher than the water requirement during the crop period.
- A low diversion weir or a barrage is constructed across the river to rise the water level and divert the same to the canal.
- Direct irrigation can be adopted only where there is enough flow in the river to provide sufficient quantity of water required for irrigation throughout the crop period.

h) Storage Irrigation:

- River flow is not perennial or insufficient during crop period, Storage Irrigation is adopted.
- A dam is construction across the river to store water in the reservoir.
- In some area rain water that run off from a catchment area is stored in tanks and is used for irrigation during the crop period.

4) Lift or well Irrigation:

- Water is lifted up by mechanical such as pump etc or manual to supply for irrigation .
- Lift irrigation is adopted when the water source is lower than the level of lands to be irrigated.

Historical development of Irrigation

- Historically, civilizations have been dependent on development of irrigated agriculture.
- Archaeological investigation has identified evidence of irrigation in **Mesopotamia, Ancient Egypt & Ancient Persia** (at present Iran) as far back as the 6th millennium BCE.

- In the **“Zana”** valley of the Andes Mountain in **Peru**, archaeologists found remains of three irrigation canals radiocarbon dated from the 4th millennium BCE, the 3rd Millennium BCE & the 9th century CE, These canals are the earliest record of irrigation in the new world.
- The **Indus valley** civilization in Pakistan & North India (from 2600 BCE) also had an early canal irrigation system. Large scale agriculture was used for the purpose of irrigation.
- There is evidence of **ancient Egyptian** Pharaoh Amenemhet-III in the 12th dynasty (about 1800 BCE) using the natural lake of the Faiyum Oasis as a reservoir to store surpluses of water for use during the dry seasons, the lake swelled annually from flooding of the Nile.
- The irrigation works of **ancient Sri Lanka**, the earliest dating from about 300 BCe, in the reign of King Pandukabhaya & under conditions development for the next thousand years, were one of the most complex irrigation systems of the ancient world.
- In the Szechwan region **ancient China** the Dujiangyan Irrigation System was built in 250 BCE to irrigate a large area & it still supplies water today.
- In the **Americas**, extensive irrigation systems were created by numerous groups in prehistoric times. One example is seen in the recent archaeological excavations near the Santa Cruz River in Tucson, Arizona. They have located a village site dating from 4000 years ago.

Present status of Irrigation:

- In the middle of 20th century, the advent of diesel & electric motors led for the first time to system that could pump groundwater out of major aquifers faster than it was recharged.
- This can lead to permanent loss of aquifer capacity, decreased water quality, ground subsidence & other problems.
- The largest contiguous areas of high irrigation density are found in North India & Pakistan along the rivers Ganges & Indus, in the Hai He, Huang He & Yangtze basins in China, along the Nile River in Egypt & Sudan, in the Mississippi-Missouri river basin & in parts of California.

Developmental Aspects of Irrigation:

Irrigation is practiced to maintain the different developmental parameters. Those are:

9. To make up for the soil moisture deficit.
10. To ensure a proper & sustained growth of crops.
11. To make harvest safe.
12. To colonize the cultivable wasteland for horizontal expansion of cultivation.
13. To shift from seasonal cultivation.
14. To promote more intensive cultivation by multiple cropping.
15. To improve the level of agricultural productivity by acting as an agent for adoption of modern technology.
16. To lessen the regional & size-class inequalities in agricultural productivity that will reduce in turn socio-economic imbalances.

Advantages of irrigation

Advantages of irrigation can be direct as well as indirect.

III. Direct Benefits

- The grower has many choices of crops and varieties and can go for multiple cropping for cultivation
- Crop plants respond to fertilizer and other inputs and there by productivity is high.
- Quality of the crop is improved.
- Higher economic return and employment opportunities. It makes economy drought proof.
- Development of pisciculture and afforestation. Plantation is raised along the banks of canals and field boundaries.
- Domestic water supply, hydel power generation at dam site and means of transport where navigation is possible.
- Prevention of damage through flood.

IV. Indirect Benefits

- Increase in gross domestic product of the country, revenue, employment, land value, higher wages to farm labour, agro-based industries and groundwater storage.
- General development of other sectors and development of the country
- Increase of food production.
- Modify soil or climate environment – leaching.
- Lessen risk of catastrophic damage caused by drought.
- Increase income & national cash flow.
- Increase labor employment.
- Increase standard of living.
- Increase value of land.
- National security thus self sufficiency.
- Improve communication and navigation facilities.
- Domestic and industrial water supply.
- Improve ground water storage.
- Generation of hydro-electric power.

Disadvantages of Irrigation

The following are the disadvantages of irrigation.

- Water logging.

- Salinity and alkalinity of land.
- Ill aeration of soil.

- Pollution of underground water.
- Results in colder and damper climate causing outbreak of diseases like malaria.

Types of Crops:

7) **Wet crops-** which lands are irrigated and than crop are cultivation
8) **Dry crops-**which do not need irrigation.
9) **Garden crops-** which need irrigation throughout the year
10) **Summer crop (Kharif)-**which are sown during the south west monsoon & harvested in autumn.
11) **Winter crops(rabi)-**which are sown in autumn & harvested in spring.

S.No	Crop	Sown	Harvested
1	Summer season (Kharif crop)		
	Rice	June -July	Oct-Nov
	Maize	June -July	Sep-Oct
	Bajra	June -Aug	Sep-Oct
	Jowar	June -July	Oct-Nov
	Pulses	June -July	Nov-Dec
2	Winter season (Rabi Crops)		
	Wheat, Barley, peas	Oct-Nov	March - April
	Gram	Sep- Oct	March - April
	Tobacco	Feb-Mar	June
	Potato	Oct	Feb
3	Eight Months Crop cotton	May-June	Dec-Jan
4	Annual crop sugercane	Feb-March	Dec-march

12)
Cash crop – which has to be encased in the market. As it cannot be consumed directly by the cultivators.

Crop

Seasons:

- In north India the crop season is divided as Rabi & Kharif.

- Rabi crops are called as winter crops and kharif crops are called as summer crops.
- Kharif crops required more water than rabi crops.
- Rabi starts from 1 st oct and ends on 31 march
- In TamilNadu crops are classified as wet and dry crops.

Crops rotation:

Rotation of crops implies the nature of the crop sown in a particular field is changed year after year.

Necessity for rotation

- The necessity for irrigation when the same crop is grown again and again in the same field, the fertility of land gets reduced as the soil becomes deficient in plant foods favorable to that particular crop.
- If different crops were to be raised there would certainly be more balanced fooding and soil deficient in one particular type of nutrient is allowed to recouped.
- Crop diseases and insect pests will multiply at an alarming rate, if the same crop is to be grown continuously. Rotation will check the diseases.
- A leguminous crop (such as gram) if introduced in rotation will increase nitrogen content of soil thus increasing its fertility.
- The deep rooted and shallow rooted crops in rotation draw their food from different depths of soil. The soil will be better utilized.
- Rotation of crops is beneficial to the farmers as there would be rotation of cash crops, fooder and soil renovating crops.

General rotation of crops can be summarized as:

1. Wheat – great millet – gram.
2. Rice – gram
3. Cotton – wheat – gram.
4. Cotton – wheat – sugarcane
5. Cotton – great millet – gram.

Consumptive Use of Water

- Considerable part of water applied for irrigation is lost by evaporation & transpiration.
- This two processes being difficult to separate are taken as one and called Vapor- transpiration or Consumptive use of water.

Duty :

Delta:

Duty- Area of the crop irrigated/ Volume of water required.

- The depth of water required every time, generally varies depending upon the type of the crop.

- The total depth of water required a crop to nature is called delta.
- Crop period-the time from the instant of its sowing to the instant of harvesting.
- Base Period-time b/w the first supply of water to the land and the last watering before harvesting.

Factor affecting the duty:

9) Soil Moisture

- In clayey soil less water is required since its retentive capacity is more.
- Pervious soil it will be more.

10) Topography

- Uniform distribution depends on topography.
- If the area is sloping the lower portion will get more water than the flat portion, & hence Water requirement is increase.

11) Nature of rainfall

- If rainfall is high over the crop period water requirement becomes less, otherwise it will be more.

12) Nature of crop irrigated

- Dry crop required less water where as wed crop required more water.

13) Method of cultivation:

- If the fields are properly ploughed it will have high retentive capacity & the number of watering are reduced.

14) Season of crop

- Less irrigation water is required for rainy season crop and the duty increased.
- If the crop grown in summer, more irrigation water is required & the duty gets decreased

15) System of Irrigation

- In perennial irrigation, continuous supply of water is given & hence water table is kept high& percolation losses is minimized
- In inundation type wastage is more by deep percolation.

16) Canal Condition

- Well maintained canal will have more duty as the losses is less.

Improving Duty

6. The water losses can be reduced by having the irrigated area nearer to the head of thecanal.

7. Evaporation losses can be minimized by using the water as quickly as possible.

8. Water losses can be minimized by lining the canals.

9. The cultivators should be trained to use water economically without wasting.

10. The soil properties should be studied by establishing research stations in villages.

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**.

Duty and Delta of a Crop Delta:

The total quantity of water required by the crop for its full growth may be expressed in hectare-meter 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 is called delta (Δ).

Problem –1: If rice requires about 10 cm 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*:**

No. of watering required = 120/10 = 12

Total depth of water required in 120 days = $10 \times 12 = 120$ cm

Δ for rice = 120 cm

Problem –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*:**

No. of watering required = 140/28 = 5

Total depth of water required in 140 days = $7.5 \times 5 = 37.5$ cm

Δ for wheat = 37.5 cm

Duty:

- It may be defined as the number of hectares of land irrigated for full growth of a given crop by supply of 1 m^3/s of water continuously during the entire base of that crop.
- Simply we can say that, the area (in hectares) of land can be irrigated for a crop period, B (in days) using one cubic meter of water.

Factors on which duty depends:

1. Type of crop

2. Climate and season
3. Useful rainfall
4. Type of soil
5. Efficiency of cultivation method

Importance of Duty

- It helps us in designing an efficient canal irrigation system.
- Knowing the total available water at the head of a main canal, and the overall duty for all the crops required to be irrigated in different seasons of the year, the area which can be irrigated can be worked out.
- *Inversely, if we know the crops area required to be irrigated and their duties, we can work out the discharge required for designing the channel.*

Measures for improving duty of water:

The duty of canal water can certainly be improved by effecting economy in the use of water by resorting to the following precautions and practices:

(7) Proper Ploughing:

Ploughing should be done properly and deeply so that the moisture retaining capacity of soil is increased.

(8) Methods of supplying water:

The method of supplying water to the agriculture land should be decided according to thefield and soil conditions. For example,

- Furrow method For crops sown ion rows
- Contour method For hilly areas
- Basin For orchards
- Flooding For plain lands

(9) Canal Lining:

It is provided to reduce percolation loss and evaporation loss due to high velocity.

(10) Minimum idle length of irrigation Canals:

The canal should be nearest to the command area so that idle length of the canal is minimumand hence reduced transmission losses.

(11) Quality of water:

Good quality of water should be used for irrigation. Pollution en route the canal should beavoided.

(12) Crop rotation:

The principle of crop rotation should be adopted to increase the moisture retaining capacity and fertility of the soil.

Consumptive use of crops

Definition:

- It is the quantity of water used by the vegetation growth of a given area.
- It is the amount of water required by a crop for its vegetated growth to evapotranspiration and building of plant tissues plus evaporation from soils and intercepted precipitation.
- It is expressed in terms of depth of water. Consumptive use varies with temperature,humidity, wind speed, topography, sunlight hours, method of irrigation, moistureavailability.

Mathematically,

Consumptive Use = Evapotranspiration = Evaporation + transpiration

- It is expressed in terms of depth of water.

Factors Affecting the Consumptive Use of Water

Consumptive use of water varies with:

1. Evaporation which depends on humidity
2. Mean Monthly temperature
3. Growing season of crops and cropping pattern
4. Monthly precipitation in area
5. Wind velocity in locality
6. Soil and topography
7. Irrigation practices and method of irrigation
8. Sunlight hours

Types of Consumptive Water Use

Following are the types of consumptive use,

1. Optimum Consumptive Use
2. Potential Consumptive Use
3. Seasonal Consumptive Use

1. Optimum Consumptive Use:

It is the consumptive use which produces a maximum crop yield.

2. Potential Consumptive Use:

If sufficient moisture is always available to completely meet the needs of vegetation fullycovering the entire area then resulting evapotranspiration is known as Potential Consumptive Use.

3. Seasonal Consumptive Use:

The total amount of water used in the evapo-transpiration by a cropped area during the entire growing season.

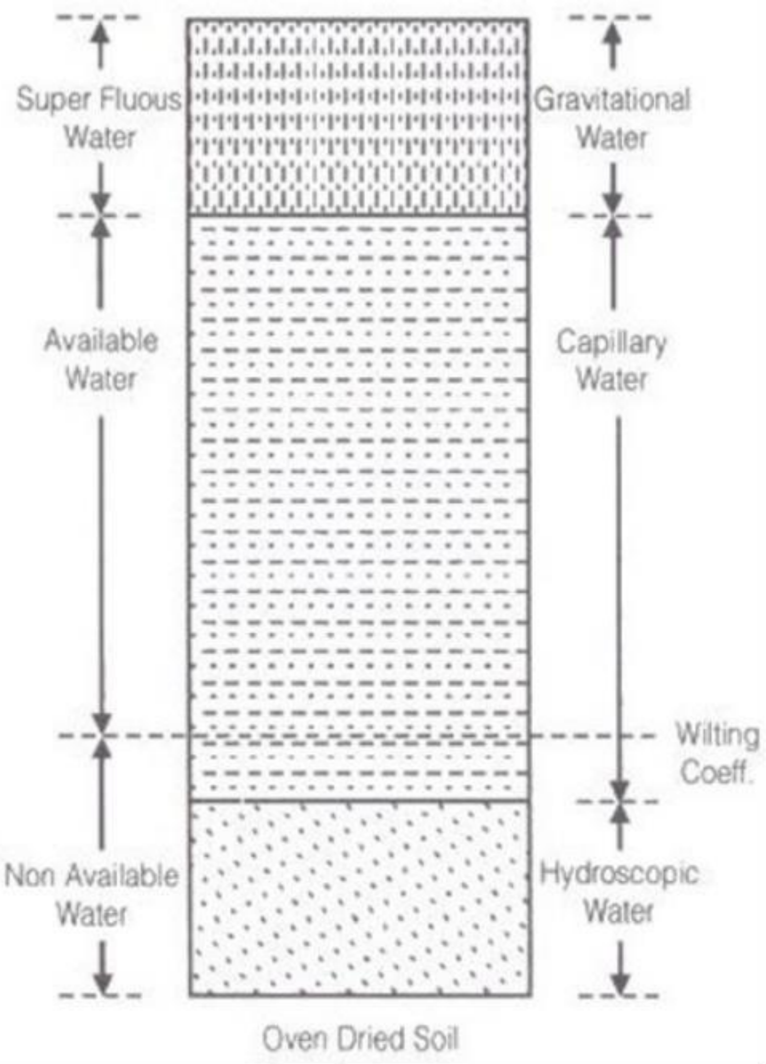
Crop Water Requirements

Soil moisture

Classes and availability of soil water

Water present in the soil may be to classified under three heads

1. Hygroscopic water
2. Capillary water
3. Gravitational water



Hygroscopic water

Water attached to soil particles through loose chemical bonds is termed hygroscopic water. This water can be removed by heat only. But the plant roots can use a very small fraction of this soil moisture under drought conditions.

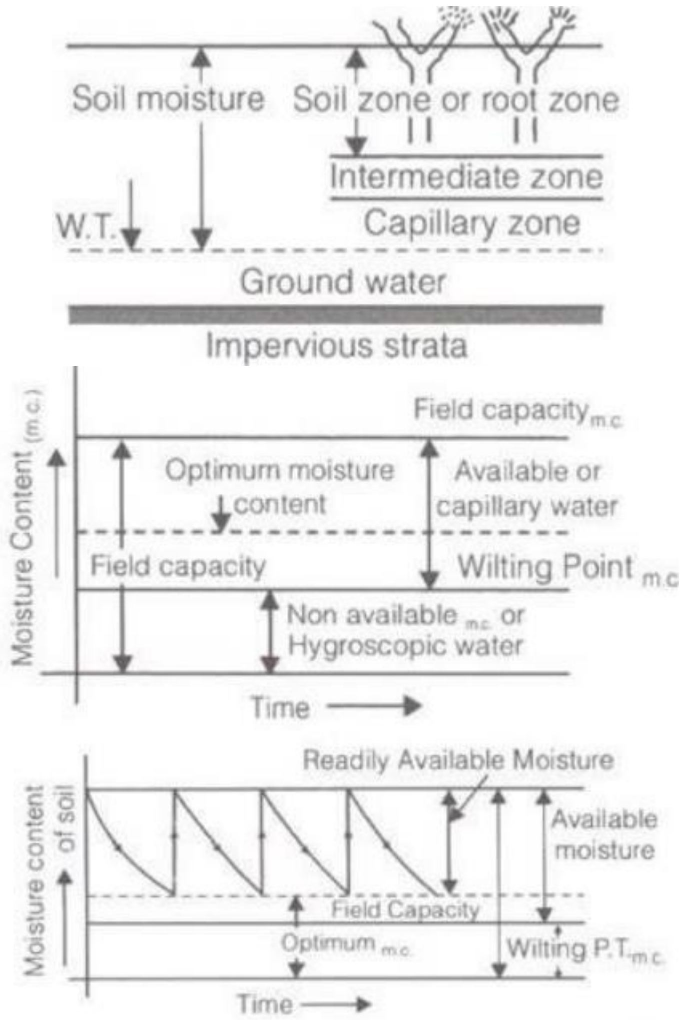
Capillary water

The capillary water is held within soil pores due to the surface tension forces (against gravity) which act at the liquid-vapour (or water-air) interface.

Gravitational water

Gravity water is that water which drains away under the influence of gravity. Soon after irrigation (or rainfall), this water remains in the soil and saturates the soil, thus, preventing circulation of airin the void spaces.

(1)



(4) Available moisture for the plant $= F_C - \phi$

(5) Readily available moisture for the plant = FC - MoHere FC= field capacity

φ = wilting point or wilting coefficient below plant can't survive.

$$= \frac{weight\,/\,readily\,available\ moisture\ depth}{consumptive\ use\ rate}$$

Mo= Readily available moisture content

(6) Frequency of Irrigation

(7) $$F_C = \frac{weight\ of\ water\ stored\ in\ soil\ of\ unit\ area}{weight\ of\ same\ soil\ of\ unit\ area}$$

where, weight of water stored in soil of unit area $= \gamma_w \cdot d_w \cdot 1 \cdot$ Weight of some soil of unit area $= \gamma \cdot d \cdot l$

dw= depth of water stored in root zone.

(8) $d_w = \frac{\gamma \cdot d}{\gamma_w} \cdot F_C \quad \gamma \rightarrow$ dry unit wt. of soil

(9) Available moisture depth to plant $d_w' = \frac{\gamma . d}{\gamma_w}(F_C - \phi)$

(10) Readily available moisture depth to plant $d_w' = \frac{\gamma . d}{\gamma_w}(F_C - m_o)$

(11) $F_C = n / G$ where, G = specific gravity and n = porosity

Duty and delta

Duty:

- The duty of water is the relationship between the volume of water and the area of the crop it matures.
- It is defined as the area irrigated per cumec of discharge running for base period B.
- The duty is generally represent by D.

Delta:

- It is the total depth of water required by a crop during the entire base period and is represented by the symbol Δ.

Relation between duty and delta

$$\Delta = \frac{8.64B}{D}$$

Where,

- Δ=Delta in meter

- D = Duty in Ha/cumec
- B = Base period in days

$\Delta = \frac{2B}{D}$ Also

Where,

- Δ=Delta in meter
- B = Base period in days
- D = Duty in acre/cures

Irrigation Requirements of crops

(1) Consumptive Irrigation Requirement (CIR)

CIR = Cu- P_{eff}

Where, Cu= total consumptive use requirementP_{eff}= Effective rainfall.

(2) Net Irrigation Requirement (NIR)

NIR = CIR + Leaching requirement

(3)

$$FIR = \frac{NIR}{\eta_a}$$

(4) Field irrigation requirement (FIR)

(5) Gross irrigation requirement, (GIR)

$$GIR = \frac{FIR}{\eta_c}$$

The equation for this method is,

U = 0.0015 H + 0.9 (Over specified)

U = Consumptive Use

H = Accumulated degree days during the growing season computed from maximum temperature above 32 °F

UNIT -3

DIVERSION AND IMPOUNDING STRUCTURES

Types of Impounding structures - Gravity dam – Forces on a dam -Design of Gravity dams; Earth dams, Arch dams- Diversion Head works - Weirs and Barrages.

Impounding structure

- Impounding structure or dam means a man-made device structure, whether a dam across a watercourse or other structure outside a watercourse, used or to be used to retain or store waters or other materials.
- The term includes: (i) all dams that are 25 feet or greater in height and that create an impoundment capacity of 15 acre-feet or greater, and (ii) all dams that are six feet or greater in height and that create an impoundment capacity of 50 acre-feet or greater.

Diversion headwork.

- Any hydraulic structure, which supplies water to the off-taking canal, is called a headwork.
- A diversion headwork serves to divert the required supply in to the canal from the river.

The purposes of diversion headwork.

1. It raises the water level in the river so that the commanded area can be increased.
2. It regulates the intake of water in to the canal.
3. It controls the silt entry in to the canal.
4. It reduces fluctuations in the level of supply in the river.
5. It stores water for tiding over small periods of short supplies.

Weir

The weir is a solid obstruction put across the river to raise its water level and divert the water in to the canal. If a weir also stores water for tiding over small periods of short supplies, it is called a storage weir.

The component parts of diversion headwork

- Weir or barrage
- Divide wall or divide groyne
- Fish ladder
- Head sluice or canal head regulator
- Canal off-takes
- Flood banks
- River training works.

Dam

A dam is a hydraulic structure constructed across a river to store the suppliy for a longer durationand release it through designed outlets.

Types of Dams

Based on Materials of Construction

- Rigid.
- Non-Rigid.

Based on Structural Behaviour

- Gravity Dam.
- Arch Dam.
- Buttress Dam.
- Embankment Dam.

Based on Functions

- Storage Dam.
- Detention Dam.
- Diversion Dam.
- Coffer dam.

Based on Hydraulic Behaviour

- Over Flow Dam.
- Non Over Flow Dam.

General Types

- Solid gravity dam (masonry, concrete, steel and timber)
- Arch dams
- Buttress dams
- Earth dams
- Rockfill dams
- Combination of rockfill and earth dams

Gravity dam

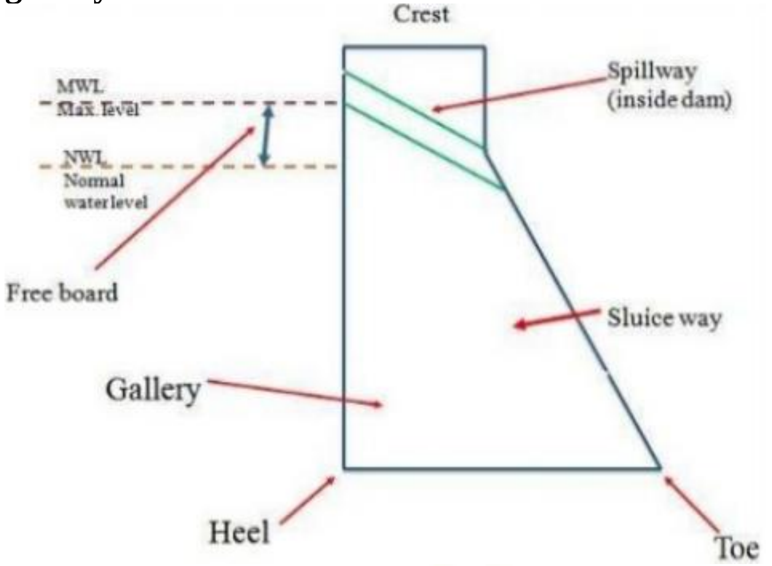
- A gravity dam is a structure so proportioned that its own weight resists the forces exerted upon it. It requires little maintenance and it is most commonly used.
- A Gravity dam has been defined as a “structure which is designed in such a way that its own weight resist the external forces”.
- This type of a structure is most durable and solid and requires very less maintenance.
- Such dams are constructed of masonry or Concrete.
- However, concrete gravity dams are preferred these days and mostly constructed.
- The line of the upstream face or the line of the crown of the dam if the upstream face is

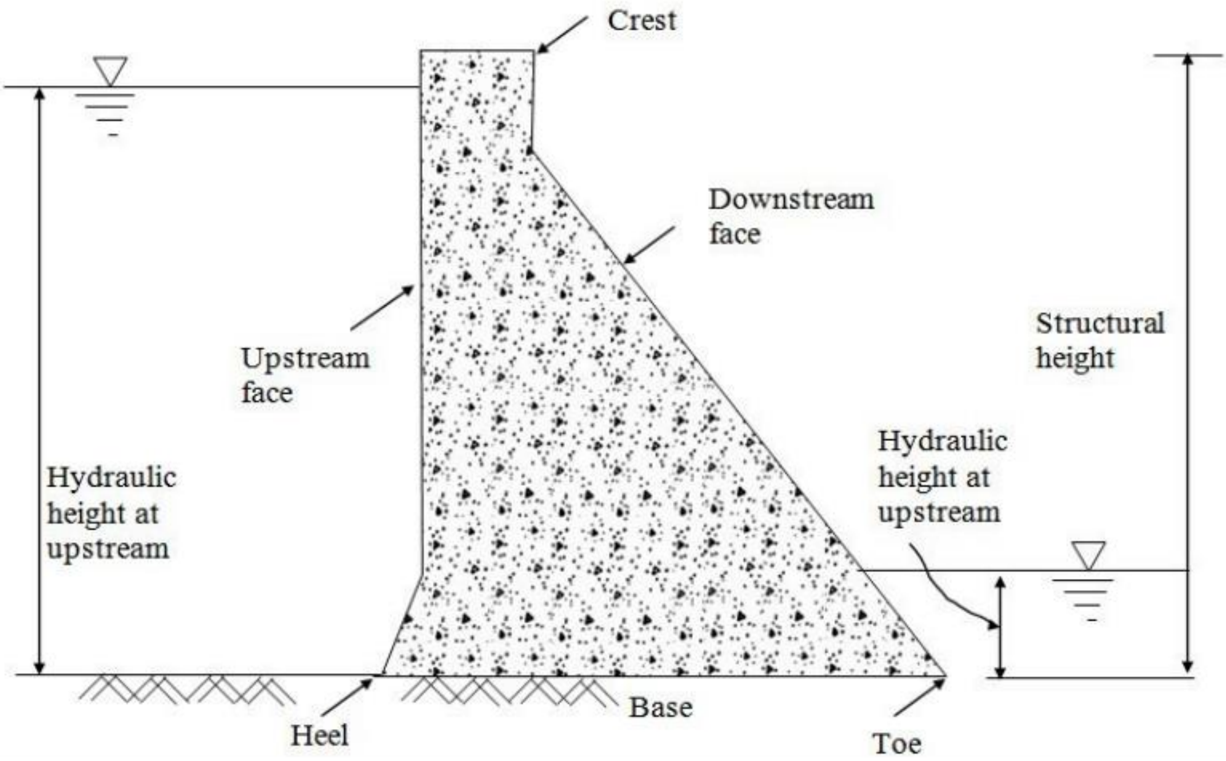
sloping, is taken as the reference line for layout purpose etc. and is known as the Base line of the dam or the "Axis of The Dam" When suitable conditions are available such dams canbe constructed up to great heights.

The different components of a solid gravity dam are

- Crest.
- Free Board.
- Heel.
- Toe.
- Sluice Way.
- Drainage Gallery.

Typical cross section of gravity Dam:





Heel: contact with the ground on the upstream side

Toe: contact on the downstream side

Abutment: Sides of the valley on which the structure of the dam rest

Galleries: small rooms like structure left within the dam for checking operations.

Diversion tunnel: Tunnels are constructed for diverting water before the construction of dam. This helps in keeping the river bed dry.

Spillways: It is the arrangement near the top to release the excess water of the reservoir to downstream side

Sluice way: An opening in the dam near the ground level, which is used to clear the silt accumulation in the reservoir side.

Forces Acting on Gravity Dam

The Various external forces acting on Gravity dam may be:

- Water Pressure
- Uplift Pressure
- Pressure due to Earthquake forces
- Silt Pressure
- Wave Pressure
- Ice Pressure
- The stabilizing force is the weight of the dam itself

Self weight of the Dam

Self weight of a gravity dam is main stabilizing force which counter balances all the external forces acting on it.

For construction of gravity dams the specific weight of concrete & stone masonry shouldn't be lessthan 2400 kg/m^3 & 2300 kg/m^3 respectively.

The self weight of the gravity dam acts through the centre of gravity of the.

Its calculated by the following formula – $W = \gamma_m \ X \ Volume$

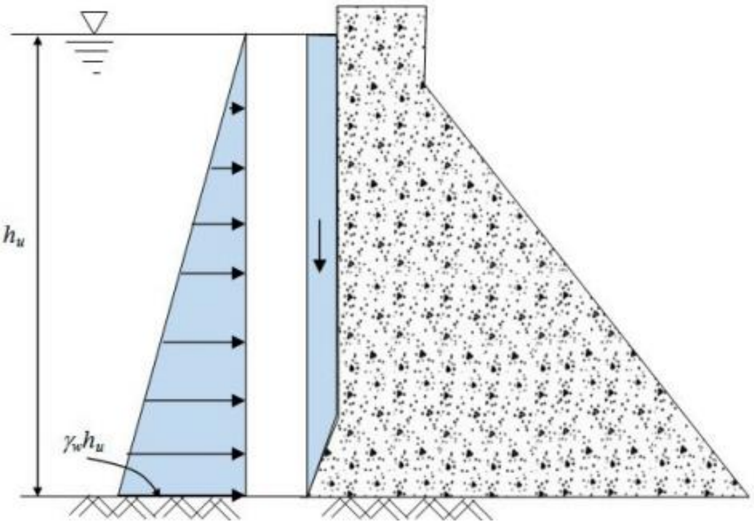
Where γ_m is the specific weight of the dam's material.

Water pressure

- Water pressure on the upstream side is the main destabilizing force in gravity dam.
- Downstream side may also have water pressure.
- Though downstream water pressure produces counter overturning moment, its magnitude is much smaller as compared to the upstream water pressure and therefore generally not considered in stability analysis.

- Water Pressure is the most major external force acting on a gravity dam.
- On upstream face pressure exerted by water is stored upto the full reservoir level. The upstream face may either be vertical or inclined.
- On downstream face the pressure is exerted by tail water. The downstream face is always inclined. It is calculated by the following formula – $P=\frac{1}{2}\gamma_w \times h^2$

Where γ_w is the unit weight of water & h is the height of water.



Uplift water pressure

- The uplift pressure is the upward pressure of water at the base of the dam as shown in Figure 29.3. It also exists within any cracks in the dam.
- The water stored on the upstream side of the dam has a tendency to seep through the soil below foundation.
- While seeping, the water exerts a uplift force on the base of the dam depending upon the head of water.
- This uplift pressure reduces the self weight of the dam.
- To reduce the uplift pressure, drainage galleries are provided on the base of the dams.
- It is calculated by the following formula – $U=\frac{1}{2}\gamma_w \times h \times B$

Where ‘B’ is the width of the base of the dam.

Wave Pressure

When very high wind flows over the water surface of the reservoir, waves are formed which exert pressure on the upstream part of the dam.

The magnitude of waves depend upon –

- The velocity of wind.
- Depth of Reservoir.
- Area of Water Surface.

It is calculated by the following formula - $P_v = 2.4\, \gamma_w\, x\, h_w$

Where 'h_w ' is the wave height.

WIND PRESSURE :

- The top exposed portion on the dam is small & hence the wind pressure on this portion of dam is negligible.
- But still an allowance should be made for the wind pressure at the rate of about 150 kg/m^2 for the exposed surface area of the upstream & downstream faces.

SEISMIC FORCES :

- Dams are subjected to vibration during earthquakes.
- Vibration affects both the body of the dam as well as the water in the reservoir behind the dam.
- The most danger effect occurs when the vibration is perpendicular to the face of the dam.
- Body Forces: Body force acts horizontally at the center of gravity and is calculated as: $P_{em} = a\, x\, W$
- Water Force: Water vibration produces a force on the dam acting horizontally & calculated by: $P_{ew} = \frac{2}{3} C_e\, a\, h^2$

ELEMENTRY PROFILE

- When water is stored against any vertical face, then it exerts pressure perpendicular to the face which is zero at top & maximum at bottom.
- The required top thickness is thus zero & bottom thickness is maximum forming a right angled triangle with the apex at top, one face vertical & some base width.
- Two conditions should be satisfied to achieve stability
  - **When empty** - The external force is zero & its self weight passes through C.G. of the triangle.
  - **When Full** - The resultant force should pass through the extreme right end of the middlethird.

The limiting condition is – $h = \frac{\sigma_c}{\gamma (1+S)}$

- where, σc=allowable compressive stressc=allowable compressive stress

Practical Profile

- Various parameters in fixing the parameters of the dam section are,

- Free Board –IS 6512, 1972 specifies that the free board will be 1.5 times the wave height above normal pool level.
- Top Width – The top width of the dam is generally fixed according to requirements of the roadway to be provided. The most economical top width of the dam is 14 % of its height.

- Base Width – The base width of the dam shall be safe against overturning, sliding & no tension in dam body.

For elementary profile –

- When uplift is considered, $B=\frac{h}{\sqrt{S}}$
- When uplift isn't considered $B=\frac{h}{\sqrt{S-1}}$

Low Gravity Dam

- A low gravity dam is designed on the basis if of elementary profile, where the resultant force passes through the middle-third of its base.
- The principal stress is given by – σc=allowable compressive stress = γ H (S – C + 1) Where, σc=allowable compressive stress=principal stress, γ=unit weight, S=Specific Gravity and C=A constant.
- The principal stress varies with 'H' as all other terms are constant. To avoid failure of the dam the value of 'σc=allowable compressive stress' shouldn't exceed allowable working stress(f). F = γ H (S – C + 1)

High Gravity Dam

- The high gravity is a complicated structure, where the resultant force may pass through a point outside the middle-third of the base.
- The section of the dam is modified by providing extra slope on the upstream and downstream side.
- The condition for the high gravity dam are $H>\frac{f}{w(S+1)}$

– Where, f=allowable working stress.

Failure of Gravity Dam

Failure of gravity dams are caused due to,

- Sliding – It may take place on a horizontal joint above formation, on the foundation. Sliding takes place when total horizontal forces are grater than the combined shearing resistance of the joint and the static friction induced by total vertical forces.
- Overturning – A dam fails in overturning when total horizontal forces acting on the dam section are quite great in comparison with total vertical forces. In such cases the resultant of two passes outside the limits of the dam.
- Dam may fail when tension is produced in the concrete.
- Dam may fail in crushing.

Precautions against Failure

- To prevent overturning, the resultant of all forces acting on the dam should remain within the middle-third of the base width of the dam.

- In the dam, the sliding should be fully resisted when the condition for no sliding exists in the dam section.

- In the dam section, the compressive stresses of concrete or masonry should not exceed the permissible working stress to avoid failure due to crushing.
- There should be no tension in the dam section to avoid the formation of cracks.
- The factor of safety should be maintained between 4 to 5.

Temperature Control

During setting of concrete heat of hydration is evolved producing internal temperature stresses resulting in development of internal cracks can get formed.

To control the temperature the following steps may be taken

1. Low heat cement may be used in concrete.
2. The water & coarse aggregates should be cooled down to 5°C by suitable means before mixing.
3. During laying the height of concrete blocks should not be more than 1.5 m. It helps radiate heat to the atmosphere more quickly.
4. The water is cooled by crushed ice before using it for the curing purpose.

Advantages

1. Gravity dams are more suitable in narrow valleys.
2. Maintenance cost is lower
3. Failure of these dams is not very sudden.
4. Gravity dams may be built to any height.
5. Loss of water by seepage in gravity dams is less

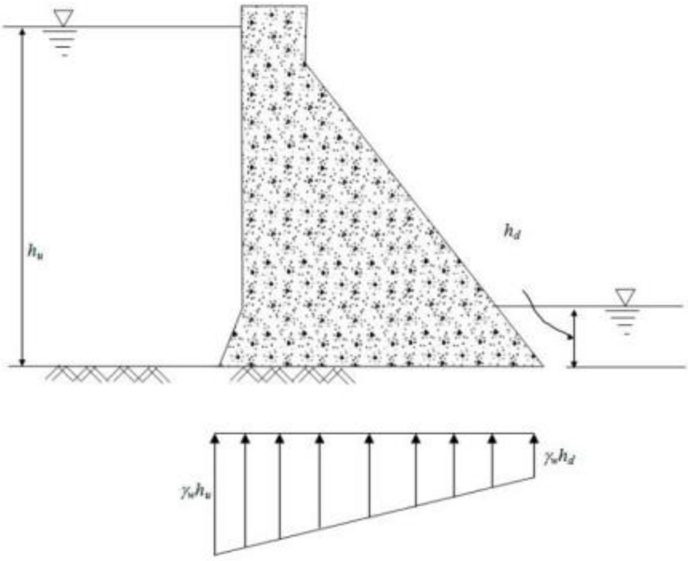
Disadvantages

1. Initial cost for construction of gravity dams is very higher.
2. Gravity dams of greater height can only be constructed on sound rock foundations.
3. Require skill labour for construction.
6. ys.
7. Maintenance cost is lower
8. Failure of these dams is not very sudden.
9. Gravity dams may be built to any height.
10. Loss of water by seepage in gravity dams is less

Disadvantages

4. Initial cost for construction of gravity dams is very higher.
5. Gravity dams of greater height can only be constructed on sound rock foundations.
6. Require skill labour for construction.

7.



8. Design of gravity dams is very complicated.

General Requirement for Stability

A gravity dam may fail in the following modes,

- Overturning
- Sliding
- Compression
- Tension

Therefore, the requirements for stability are,

- The dam should be safe against overturning.
- The dam should be safe against sliding.
- The induced stresses (either tension or compression) in the dam or in the foundation should not exceed the permissible value.

DESIGN OF GRAVITY DAM

Example 19.2. *Fig. 19.20 (a) shows the section of a gravity dam built of concrete. Examine the stability of this section at the base.*

The earthquake forces may be taken as equivalent to 0.1 g for horizontal forces and 0.05 g for vertical forces. The uplift may be taken as equal to the hydrostatic pressure at the either ends and is considered to act over 60% of the area of the section.

A tail water depth of 6 m is assumed to be present when the reservoir is full and there is no tail water when the reservoir is empty.

Also indicate the values of various kinds of stresses that are developed at heel and toe. Assume the unit wt. of concrete as 24 kN/m^3 ; and unit wt. of water = 10 kN/m^3.

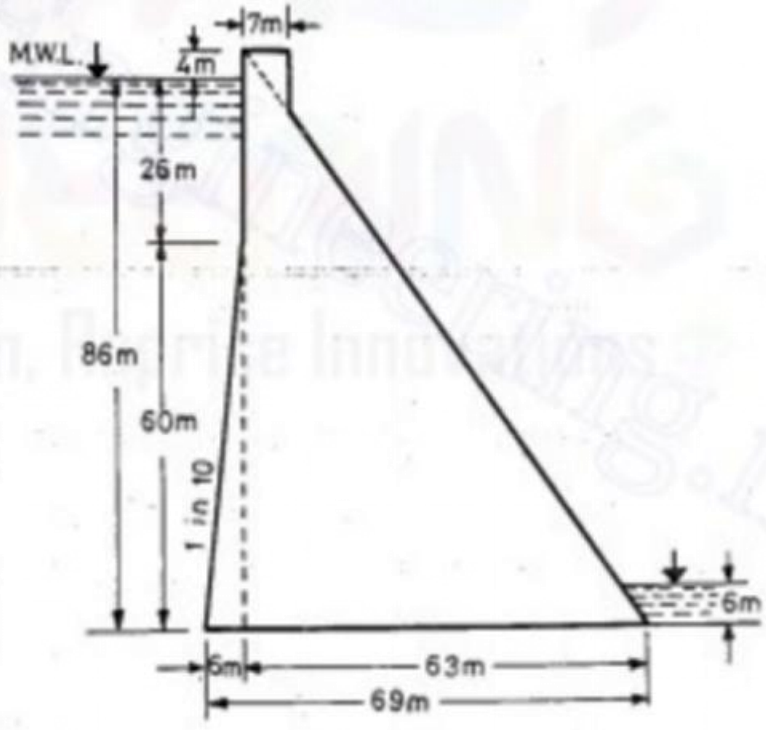


Fig. 19.20 (*a*)

Solution. The stability analysis shall be carried out for both the cases, *i.e.* (1) Reservoir Empty, and (2) Reservoir Full.

Case (I) Reservoir Empty. Consider 1 m length of the dam.

When the reservoir is empty, the various forces are worked out in Table 19.2 (*a*) with reference to Fig. 19.20 (*b*). Horizontal earthquake forces acting towards upstream are considered. Stability is examined for two sub-cases, *i.e.* (*a*) When vertical earthquake forces are additive to the weight of the dam ; (*b*). When vertical earthquake forces are subtractive to the dam weight.

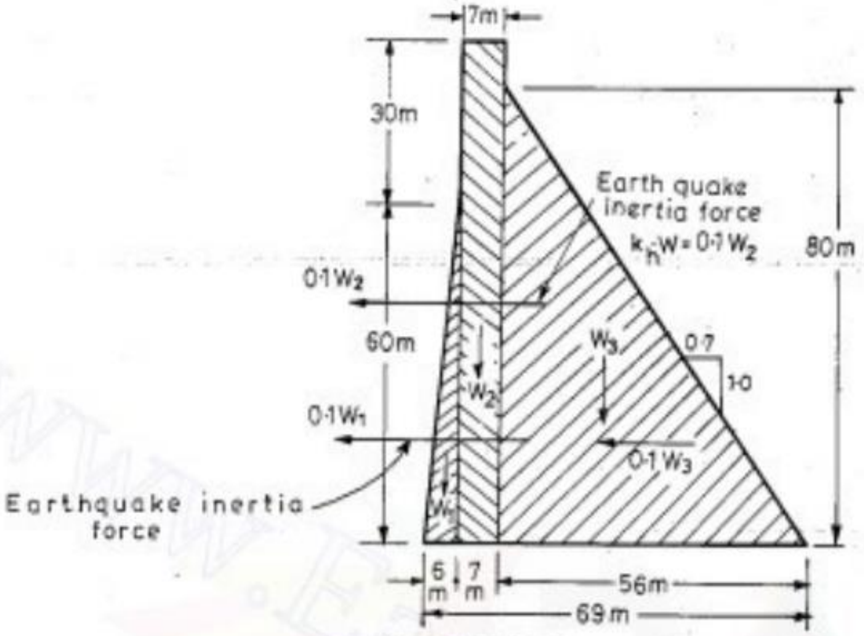


Fig. 19.20 (*b*). Reservoir empty case.

Table 19.2 (*a*)

Name of the force	*Designation if any*	*Magnitude of force in kN.*		*Lever arm m*	*Moments about the toe anti-clockwise (+ve) in kN.m.*
		Vertical	*Horizontal*		
Downward wt. of dam	W_1	(+) $\frac{1}{2} \times 6 \times 60 \times 24 = 4{,}320$		65.0	(+) 2,80,400
	W_2	(+) $7 \times 90 \times 24 = 15{,}110$		59.5	(+) 8,99,000
	W_3	(+) $\frac{1}{2} \times 56 \times 80 \times 24 = 53700$		37.33	(+) 20,00,000
		$\Sigma V_1 = 73{,}130$			$\Sigma M_1 = (+)\ 31{,}79{,}400$
Horizontal earthquake forces	P_{w_1}		$0.1\ W_1 = 0.1 \times 4320 = 432$	20.0	(+) 8640
	P_{w_2}		$0.1\ W_2 = 0.1 \times 1{,}511 = 1511$	45.0	(+) 68000
	P_{w_3}		$0.1\ W_3 = 0.1 \times 5{,}370 = 5370$	26.67	(+) 1,43,200
			$\Sigma H = 7313$		$\Sigma M_2 = 2{,}19{,}840$
Vertical earthquake forces		$\Sigma V_2 = 0.05 \times \Sigma V_1 = 0.05 \times 73130 = 3{,}657$			$\Sigma M_2 = 0.05 \times \Sigma M_1 = 0.05 \times 31{,}79{,}400 = 1{,}58{,}970$

Case (I). (*a*) *Reservoir empty and vertical earthquake forces are acting downward.*
From table 19.2 (*a*), we have $\Sigma M = \Sigma M_1 + \Sigma M_2 + \Sigma M_3$

$$= 31{,}79{,}400 + 2{,}19{,}840 + 1{,}58{,}970 = \mathbf{35{,}58{,}210\ kN \cdot m}$$

Also, $\Sigma V = \Sigma V_1 + \Sigma V_2 = 73{,}130 + 3{,}657 = 76{,}787$ kN

$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{35{,}58{,}210}{76{,}787} = 47.3 \text{ m}$$

$$e = \frac{B}{2} - \bar{x} = \frac{69}{2} - 46.3 = 34.5 - 46.3 = -\mathbf{11.8\ m} > \frac{B}{6}, \textit{ i.e. } 11.5 \text{ m}.$$

Resultant acts near the heel and slight tension will develop at toe.

$$p_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

$$\therefore \quad p_{max/min} = \frac{76{,}787}{69}\left[1 \pm \frac{6 \times 11.8}{69}\right] = 1114\ [1 \pm 1.026]$$

p_v at heel $= 1114 \times 2.026 = 2260$ kN/m^2; which is ≤ 3000 (safe)

p_v at toe $= 1114 \times (-0.026) = -29$ kN/m^2 ; which is < 420 (*safe*)

Average vertical stress

$$= \frac{\Sigma V}{B} = \frac{76787}{69} = 1114 \text{ kN/m}^2 \text{ ; which is} < 3000 \text{ (safe)}$$

Principal stress at toe,

$$\sigma = p_v \sec^2 \alpha \text{ ; } (\tan \alpha = 0.7)$$
$$= -29\,(1 + 0.49) = -29 \times 1.49 = -43 \text{ kN/m}^2 \text{ ; which is} < 420 \text{ (safe)}$$

Principle stress at heel

$\sigma_1 = p_{v \cdot (heel)} \sec^2 \phi$ where $\tan \phi = 0.1$

or $\sec^2 \phi = 1 + \tan^2 \phi = 1 + 0.01 = 1.01.$

or $\sigma_1 = 2260 \times 1.01 = \mathbf{2280\ kN/m^2}$; which is < 3000 (safe).

Shear stress at toe

$$\tau_{0(toe)} = p_{v(toe)} \tan \alpha$$
$$= -29 \times 0.7 = -20.3 \text{ kN/m}^2 \text{ ; which is} < 420 \text{ (safe)}$$

Shear stress at heel

$$\tau_{0(heel)} = p_{v \cdot (heel)} \tan \phi$$
$$= 2260 \times 0.1 = 226 \text{ kN/m}^2 \text{ ; which is} < 3000 \text{ (safe).}$$

Case I. (*b*) *Reservoir empty and vertical earthquake forces are acting upward.*

Then $\Sigma V = \Sigma V_1 - \Sigma V_3$

$$= 73{,}130 - 3657 = 69473 \text{ kN}$$

$$\Sigma M = \Sigma M_1 + \Sigma M_2 - \Sigma M_3$$
$$= 31{,}79{,}400 + 2{,}19{,}840 - 1{,}58{,}970 = 32{,}40{,}270 \text{ kN} \cdot \text{m}.$$

$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{32{,}40{,}270}{69473} = \mathbf{46.7\ m.}$$

$$e = \frac{B}{2} - \bar{x} = 34.5 - 46.7 = (-)\ 12.2 \text{ m} < \frac{B}{6}$$

[– ve sign shows that resultant lies near the heel and, therefore, tension will develop at toe.]

Average vertical stress

$$= \frac{\Sigma V}{B} = \frac{69{,}473}{69} = 1004 \text{ kN/m}^2$$

$$p_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

$$= \frac{69473}{69}\left[1 \pm \frac{6 \times 12.2}{69}\right] = 1004\,[1 \pm 1.06]$$

p_v at heel $= 1004 \times 2.06 = 2070 \text{ kN/m}^2 < 3000$ (safe)

p_v at toe $= (-)\ 1004 \times 0.06 = -\ 60.3 \text{ kN/m}^2 < 420$ (safe)

Principal stress at toe

$$= \sigma = p_{v(toe)} \sec^2 \alpha$$

$$= -\ 60.3\,(1 + 0.49) = -\ 60.3 \times 1.49 = 90 \text{ kN/m}^2$$

Shear stress at toe

$$= \tau_0 = p_{v(toe)} \tan \alpha = -\ 60.3 \times 0.7$$

$= -$ **42.21 kN/m²** ; which is < 420 (safe)

stresses at heel remain critical in this 1st case.

Case II. When the reservoir is full

Horizontal earthquake moving towards the reservoir causing upstream acceleration, and thus producing horizontal forces towards downstream is considered, as it is the worst case for this condition. Similarly, a vertical earthquake moving downward and thus, producing forces upward, *i.e.* subtractive to the weight of the dam is considered.

The uplift coefficient C is taken as equal to 0.6, as given in the equation, and thus uplift pressure diagram as shown in Fig. 19.20 (c), is developed.

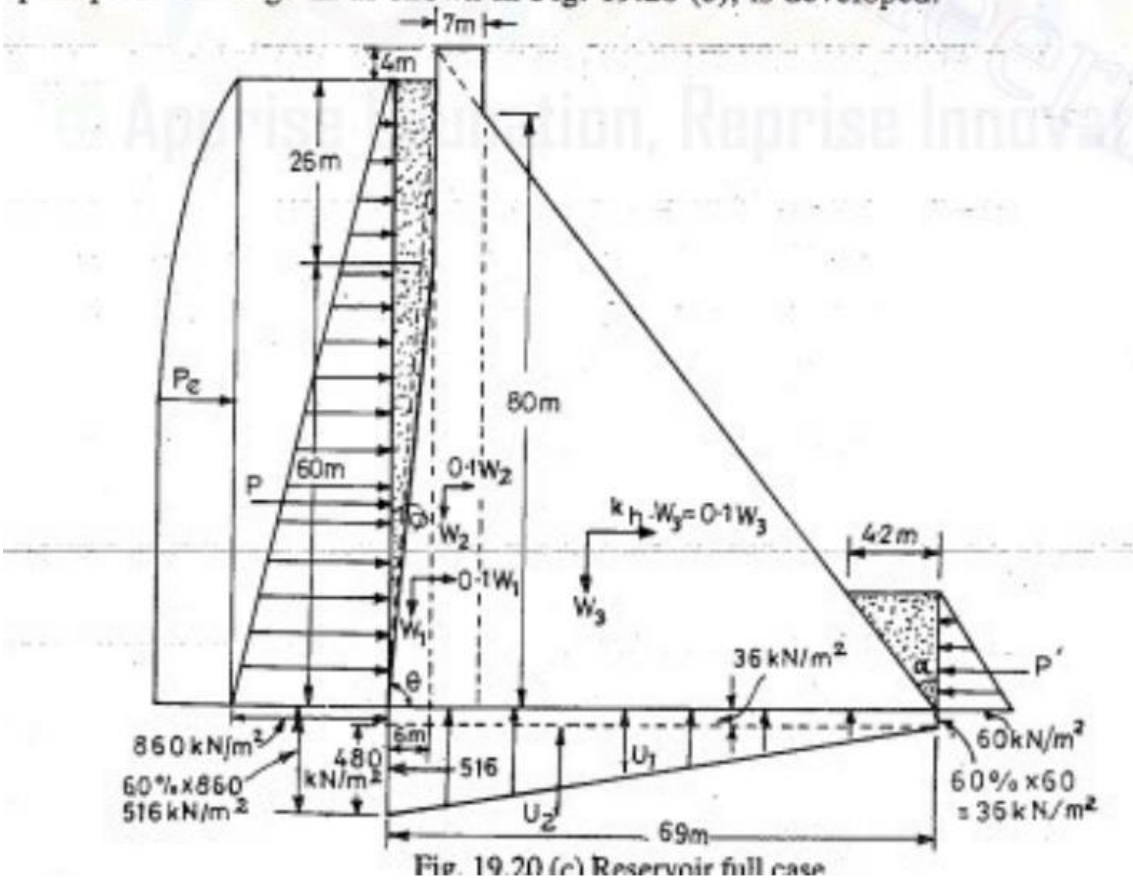


Fig. 19.20 (c) Reservoir full case

The various forces acting in this case are :

(*i*) Hydrostatic pressures P and P'.

(*ii*) Hydrodynamic pressure P_e. (P_e' is neglected as it is very small and neglection is on conservative side.)

(*iii*) Uplift forces U_1 and U_2

(*iv*) Weight of the dam, W_1, W_2 and W_3.

(*v*) Horizontal inertial earthquake forces acting towards downstream, equal to 0.1 W_1, 0.1 W_2 and 0.1 W_3 at *c.gs.* of these weights W_1, W_2 and W_3 respectively.

(*vi*) A vertical force equal to 0.05 W or $(0.05\, \Sigma V_1)$ acting upward.

Calculation of $\mathbf{P_e}$

P_e and the moment due to this hydrodynamic force is calculated, and then all the forces and their moments are tabulated in Table 19.2 (*b*).

Calculation of $\mathbf{P_e}$ from Zanger's formulas

$$P_e = 0.726\, p_e\, H \qquad \text{...(19.3)}$$

$$\text{where } p_e = C_m \cdot K_h \cdot \gamma_w \cdot H \qquad \text{...(19.4)}$$

$$\text{and} \quad C_m \doteq 0.735 \frac{\theta}{90^\circ}$$

Since the u/s inclined face is extended for more than half the depth, the overall slope up to the whole height may be taken.

$$\therefore \quad \tan\theta = \frac{86}{6} = 14.33$$

$$\theta = 81.9^\circ$$

$$\therefore \quad C_m = 0.735 \times \frac{81.9^\circ}{90^\circ} = 0.668.$$

$$p_e = 0.668 \times 0.1 \times 10 \times 86 = 57.5$$

$$P_e = 0.726 \times 57.5 \times 86 = \mathbf{3580\ kN.}$$

$$M_e = 0.412 \cdot P_e \cdot H = 0.412 \times 3580 \times 86 = \mathbf{1{,}26{,}500\ kN.m.}$$

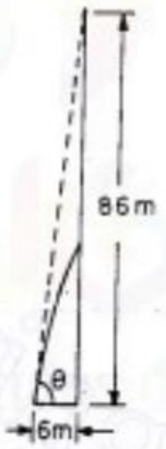


Fig. 19.20 (*d*)

Case 2 (*a*) **Reservoir full with all forces including uplift**

$$\Sigma M = [31{,}79{,}400 + 2{,}23{,}380 - 8{,}47{,}500 - 1{,}58{,}970 - 10{,}59{,}730 - 1{,}26{,}500 - 2{,}19{,}840]$$

$$= \mathbf{34{,}02{,}780 - 24{,}12{,}540 = 9{,}90{,}240\ kN/m.}$$

$$\Sigma V = 73130 + 3486 - 19030 - 3657 = 53929 \text{ kN}$$

$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{9{,}90{,}240}{53{,}929} = 18.36 \text{ m}$$

$$e = \frac{B}{2} - \bar{x} = 34.5 - 18.36 = 16.14 > \frac{B}{6}$$

The resultant is nearer the toe and tension is developed at the heel.

Average vertical stress

$$= \frac{\Sigma V}{B} = \frac{53929}{69} = 782 \text{ kN/m}^2.$$

$$p_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

Table 19.2 (*b*)

Name of force	Designation if any	Magnitude of force in kN: Vertical forces Downward = +ve Upward = −ve	Magnitude of force in kN: Horizontal forces Towards Upstream = +ve Towards Downstream = −ve	Lever arm in m	Moments about toe in kN. Anticlock wise (+ve) and clockwise (− ve) in kN.m
(1)	(2)	(3)	(4)	(5)	(6)
Weight of Dam	W_1	(+) $\frac{1}{2}\times 6\times 60\times 1\times 24 = 4320$		65.0	(+) 2,80,400
	W_2	(+) $7\times 90\times 1\times 24 = 15{,}110$		59.5	(+) 8,99,000
	W_3	(+) $\frac{1}{2}\times 56\times 80\times 1\times 24 = 53{,}700$		37.33	(+) 20,00,000
		$\Sigma V_1 = (+)\ 73{,}130$			$\Sigma M_1 = 31{,}79{,}400$
Weight of water supported on u/s slope	—	(+) $26\times 6\times 1\times 10 = 1560$		66.0	(+) 1,02,800
	—	(+) $\frac{1}{2}\times 60\times 6\times 1\times 10 = 1800$		67.0	(+) 1,10,400
water on d/s slope.	—	(+) $\frac{1}{2}\times 6\times 4.2\times 1\times 10 = 126$		1.4	(+) 180
		$\Sigma V_2 = (+)\ 3486$			$\Sigma M_2 = (+)\ 2{,}23{,}380$
Uplift forces	U_1	(−) $69\times 3.6\times 10 = 2{,}480$		34.5	(−) 85,500
	U_2	(−) $\frac{1}{2}\times 69\times 48\times 10 = 16{,}550$		46.0	(−) 7,62,000
		$\Sigma V_3 = (-)\ 19{,}030$			$\Sigma M_3 = (-)\ 8{,}47{,}500$
Upward vertical earthquake forces 0.05 W		$\Sigma V_4 = (-)\ 0.05\cdot\Sigma V_1$ $= (-)\ 0.05\times 73{,}130$ $= (-)\ 3{,}657$			$= (-)\ 0.05\cdot\Sigma M_1$ $= (-)\ 0.05\times 31{,}79{,}400$ $\Sigma M_4 = (-)\ 1{,}58{,}970$
Horizontal hydrostatic pressure	P		$(-)\frac{1}{2}\times 10\times 86\times 86\times 1$ $= (-)\ 36{,}980$	28.67	(−) 10,60,090
	P'		$(+)\frac{1}{2}\times 10\times 6\times 6\times 1 = (+)\ 180$	2.0	(−) 360
			$\Sigma H_1 = (-)\ 36{,}800$		$\Sigma M_5 = (-)\ 10{,}59{,}730$
Horizontal hydro-dynamic pressure	P_e		Calculated separately earlier : $= (-)\ 3{,}580$		$\Sigma M_e = (-)\ 1{,}26{,}500$ (calculated separately earlier
			$\Sigma H_2 = (-)\ 3{,}580$		
Horizontal inertia forces due to earthquake	P_{W_1}		(−) $0.1\ W_1 = (-)\ 432$	20.0	(−) 8,640
	P_{W_2}		(−) $0.1\ W_2 = (-)\ 1{,}511$	45.0	(−) 68,000
	P_{W_3}		(−) $0.1\ W_3 = (-)\ 5{,}370$	26.67	(−) 1,43,200
			$\Sigma H_3 = (-)\ 7{,}313$		$\Sigma M_7 = (-)\ 2{,}19{,}840$

$$\Sigma H = \Sigma H_1 + \Sigma H_2 + \Sigma H_3 = (-)36{,}800 - 3580 - 7313 = (-)\ 47{,}693$$

$$= \frac{53929}{69}\left[1 \pm \frac{6 \times 18.32}{69}\right] = 782\,[1 \pm 1.595]$$

p_v (at toe) $= 782 \times 2.595 = \mathbf{2030\ kN/m^2}$; which is < 3000 kN/m^2 (∴ Safe)

p_v (at heel) $= -782 \times 0.405$

$= \mathbf{-316.7\ kN/m^2}$; which is < 420 kN/m^2 (∴ Safe)

Since the tensile stress developed is less than the safe allowable value, the dam is safe even when examined with seismic forces, under reservoir full condition.

Principal stress at toe

$$= \sigma = p_v \cdot \sec^2 \alpha - p' \tan^2 \alpha \quad \text{i.e. Eq. (19.17)}$$

where $\tan \alpha = 0.7$, $p' = 60$ kN/m^2 ; $p_v = 2030$ kN/m^2

$$\sigma = 2030\,(1 + \tan^2 \alpha) - p' \tan^2 \alpha$$
$$= 2030\,(1 + 0.49) - 60 \times 0.49 = 2030 \times 1.49 - 29$$
$$= 3025 - 29 = \mathbf{2996\ kN/m^2} \textbf{ : which is } \mathbf{< 3000}$$ (just Safe)

Principal stress at heel is

$$\sigma_1 = p_{v(heel)} \sec^2 \phi - (p + p_e) \tan^2 \phi \quad \text{i.e. Eq. (19.19)}$$

where ϕ is the angle which the upstream face makes with the vertical

$\tan \phi = 0.1$

$$\therefore \quad \sigma_1 = -316.7\,[1 + (0.1)^2] - (860 + 57.5)\,(0.1)^2$$
$$= -316.7 \times 1.01 - 917.5 \times 0.01 = -319.9 - 9.2$$
$$= \mathbf{-329.1\ kN/m^2}$$; which is < 420 kN/m^2 (Hence, safe)

Shear stress at toe

$$\tau_{0(toe)} = (p_{v(toe)} - p') \tan \alpha = (2030 - 60)\,0.7$$
$$= 1970 \times 0.7 = \mathbf{1379\ kN/m^2}.$$

Shear Stress at heel

$$\tau_{0(heel)} = -[p_{v(heel)} - (p + p_e)] \tan \phi$$
$$= -[-329.1 - (860 + 57.5)]\,0.1$$
$$= -[-329.1 - 917.5]\,0.1 = +1246.6 \times 0.1 = \mathbf{124.7\ kN/m^2}$$

Factor of safety against overturning

$$= \frac{\Sigma M\,(+)}{\Sigma M\,(-)} = \frac{34{,}02{,}780}{24{,}12{,}540} = 1.41$$; which is < 1.5 (Hence, Unsafe)

Factor of safety against sliding

$$= \frac{\mu \cdot \Sigma V}{\Sigma H}$$

where $\mu = 0.7$

$\Sigma V = 53{,}929$

$\Sigma H = \Sigma H_1 + \Sigma H_2 + \Sigma H_3$

$= -36800 - 3580 - 7313 = -47{,}693$ kN

$$\textit{Sliding factor} = \frac{0.7 \times 53929}{47693} = 0.79, \text{ which is} < 1 \qquad \text{(Hence, Unsafe)}$$

Shear friction factor

$$\text{S.F.F.} = \frac{\mu \cdot \Sigma V + B \cdot q}{\Sigma H}$$

$$= \frac{0.7 \times 53929 + 69 \times 1400}{47693}$$

$$= 2.81 \text{ ; which is less than 3} \qquad \text{(Hence, slightly unsafe)}$$

Case 2 (*b*). Reservoir full, without uplift

Sometimes, values of stresses at toe and heel are worked out when there is no uplift, s the vertical downward forces are maximum in this case. For this case, we shall alculate ΣM and ΣV by ignoring the corresponding values of ΣV_3 and ΣM_3 caused by plift.

$$\therefore \quad \Sigma M = \Sigma M_1 + \Sigma M_2 + \Sigma M_4 + \Sigma M_5 + \Sigma M_6 + \Sigma M_7$$

$$= 31{,}79{,}400 + 2{,}23{,}380 - 1{,}58{,}970 - 10{,}59{,}730 - 1{,}26{,}500 - 2{,}19{,}840$$

$$= 34{,}02{,}780 - 15{,}65{,}040 = 18{,}37{,}740$$

$$\Sigma V = \Sigma V_1 + \Sigma V_2 + \Sigma V_4 = 73130 + 3486 - 3657 = 72{,}959 \text{ kN}$$

$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{18{,}37{,}740}{72{,}959} = 25.19 \text{ m}$$

$$e = \frac{B}{2} - \bar{x} = 34.5 - 25.9 = 9.31 \text{ m} > \frac{B}{6} \quad i.e. \quad \frac{69}{6} = 11.5 \text{ m}$$

Resultant is nearer the toe and no tension is developed any where.

$$p_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

$$= \frac{72{,}959}{69}\left[1 \pm \frac{6 \times 9.31}{69}\right] = 1057\,[1 \pm 0.81]$$

$$p_v \text{ at toe} = 1057 \times 1.81 = \mathbf{1913\ kN/m^2} < 3000 \qquad (\therefore \text{ Safe})$$

$$p_v \text{ at heel} = 1057 \times 0.19 = \mathbf{201\ kN/m^2} < 3000 \qquad (\therefore \text{ Safe})$$

$$\text{Principal stress at toe} = \sigma = p_v \cdot \sec^2 \alpha - p' \tan^2 \alpha \qquad \text{...(19.17)}$$

$$p' = 60, \tan \alpha = 0.7$$

$$\therefore \quad \sigma = 1913\,(1 + 0.49) - 60 \times 0.49 = 1913 \times 1.49 - 29 = 2821 \text{ kN/m}^2 < 3000$$

(Hence, Unsafe)

Principal stress at heel

$$\sigma_1 = p_{v(heel)} \sec^2 \phi - (p + p_e) \tan^2 \phi \qquad \text{...(19.19)}$$

where $\tan \phi = 0.1$

$$\therefore \quad \sigma_1 = 201(1 + 0.01) - (860 + 57.5) \times 0.01$$

$$= 203 - 9 = 194 \text{ kN/m}^2 < 420 \quad \text{(Safe)}$$

Shear stress at toe

$$\tau_0 = (p_v - p') \tan \alpha \qquad i.e. \text{ Eq. (19.20)}$$

$$= (1913 - 60)\ 0.7$$

$$= 1853 \times 0.7 = 1297 \text{ kN/m}^2 < 1400 \qquad (\therefore \text{ safe})$$

Note. Shear friction factor, etc. are not worked out here as they were more critical in the 1st case, *i.e.* in 'Reservoir full with uplift' case.

Conclusion. The dam is unsafe only in sliding and S.F.F., for which shear key etc. can be provided.

Example 19.3. *Examine the stability of the dam section given in the previous example, if there are no seismic forces acting on the dam. Also state the magnitude of maximum compressive stress and maximum shear stress that may develop under any conditions of loading in the dam and also state whether tension is developed anywhere or not.*

Solution. The figures calculated earlier in Table 19.2 (*a*) and (*b*) shall be used here.

Case I. When the reservoir is empty

$$\Sigma V = \Sigma V_1 \text{ from Table 10.2 } (a) = 73130$$

$$\Sigma M = \Sigma M_1 \text{ from Table 19.2 } (b) = 3179400$$

$$\therefore \qquad \bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{3179400}{73130} = 43.4 \text{ m}$$

$$e = \frac{B}{2} - \bar{x} = 34.5 - 43.4 = -8.9 \text{ m}$$

–ve sign means that the resultant is towards left side, *i.e.* nearer to the heel, and since $e < \frac{B}{6}$, no tension is develoned

$$p_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

$$= \frac{73130}{69}\left[1 \pm \frac{6 \times 8.9}{69}\right]$$

$$= 1060\,[1 \pm 0.774]$$

$$p_v \text{ at heel} = 1060(1 + 0.774) = 1060 \times 1.774 = \mathbf{1880\ kN/m^2}$$

$$p_v \text{ at toe} = 1060(1 - 0.774) = 1060 \times 0.226 = \mathbf{239\ kN/m^2}$$

Average vertical stress

$$= \frac{\Sigma V}{B} = \frac{73130}{69} = \mathbf{1060\ kN/m^2}$$

Principal stress at toe

$$\sigma = p_{v(toe)} \sec^2 \alpha$$

$$= 239(1 + 0.49) = 239 \times 1.49 = \mathbf{357\ kN/m^2}$$

Principal stress at heel,

$$\sigma = p_{v(heel)} \sec^2 \phi$$

where $\tan \phi = 0.1$

$$= 1880(1 + 0.01) = 1880 \times 1.01 = \mathbf{1896\ kN/m^2}$$

Shear stress at toe

$$\tau_0 = p_{v(toe)} \tan \alpha$$

$$= 239 \times 0.7 = 167.3 \text{ kN/m}^2$$

$$= 734\ (1 + 0.01) - 860 \times 0.01$$
$$= 742 - 9 = \mathbf{733\ kN/m^2}$$

Shear stress at toe

$$\tau_0 = [p_{v(toe)} - p'] \tan \alpha$$
$$= (1490 - 60)\ 0.7 = 1430 \times 0.7 = 1001\ \text{kN/m}^2$$

Shear stress at heel

$$= -[p_{v(heel)} - p] \tan \phi$$
$$= -[734 - 860] \times 0.7 = 126 \times 0.7 = \mathbf{88.2\ kN/m^2}$$

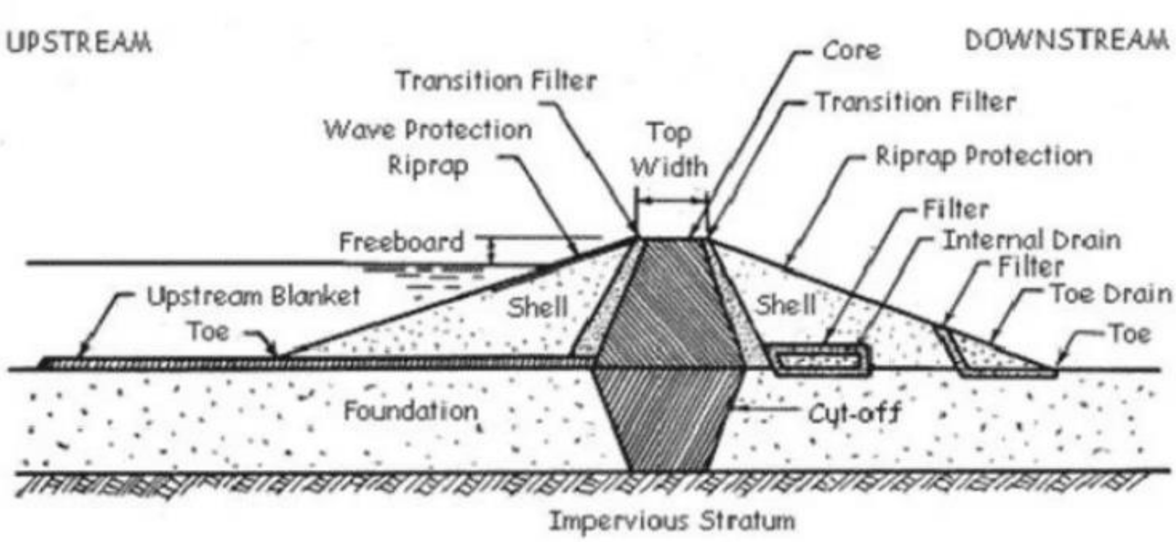
Conclusions. We find that the dam is safe throughout except that the S.F.F. is equal to 3.72, while generally it should be between 4 to 5. The dam thus remains slightly unsafe in S.F.F. even when the seismic forces are not considered.

The results of stability analysis are given below :

The maximum shear stress developed in dam	= 1001 kN/m^2.	**Ans.**
Maximum compressive stress developed in dam	= 2191 kN/m^2	
No tension is developed anywhere.		
Factor of safety against sliding	= 1.10	
S.F.F.	= 3.72	
Factor of safety against overturning	= 1.78	

EARTHEN DAMS

- An earthen embankment is a raised confining structure made from compacted soil.
- The purpose of an earthen embankment is to confine and divert the storm water runoff. It can also be used for increasing infiltration, detention and retention facilities.
- Earthen embankments are generally trapezoidal in shape and most simple and economic in nature. They are mainly built with clay, sand and gravel, hence they are also known as earth fill dams or earthen dams.
- They are constructed where the foundation or the underlying material or rocks are weak to support the masonry dam or where the suitable competent rocks are at greater depth.
-



• They are relatively smaller in height and broader at the base.

Components of an Earthen Dam

1. **Shell, Upstream Fill, Downstream Fill or Shoulder:** These components of the earthen dam are constructed with pervious or semi-pervious materials upstream or downstream of the core. The upstream fill is called the upstream shell and the downstream portion is the downstream shell.
2. **Upstream Blanket:** It is a layer of impervious material laid on the upstream side of an earthen dam where the substratum is pervious, to reduce seepage and increase the path of flow. The blanket decreases both the seepage flow and excess pressure on the downstream side of the dam. A natural blanket is a cover of naturally occurring soil material of low permeability.
3. **Drainage Filter:** It is a blanket of pervious material constructed at the foundation to the downstream side of an earthen dam, to permit the discharge of seepage and minimize the possibility of piping failure.
4. **Cutoff Wall or Cutoff:** It is a wall, collar or other structure intended to reduce percolation of water through porous strata. It is provided in or on the foundations.
5. **Riprap:** Broken stones or rock pieces are placed on the slopes of embankment particularly the upstream side for protecting the slope against the action of water, mainly wave action and erosion.
6. **Core Wall, Membrane or Core:** It is a centrally provided fairly impervious wall in the dam. It checks the flow of water through the dam section. It may be of compacted puddled clay, masonry, or concrete built inside the dam.
7. **Toe Drain:** It is a drain constructed at the downstream slope of an earthen dam to collect and drain away the seepage water collected by the drain filters.
8. **Transition Filter:** It is a component of an earthen dam section which is provided with core and consists of an intermediate grade of material placed between the core and the shells to serve as a filter and prevent lateral movement of fine material from the core.

Advantages

1. Design procedures are straightforward and easy.
2. Local natural materials are used.
3. Comparatively small establishment and equipment are required.
4. Earth fill dams resist settlement and movement better than more rigid structures and can be more suitable for areas where earth movements are common.

Disadvantages

1. An earthen embankment is easily damaged or destroyed by water flowing on, over or against it. Thus, a spillway and adequate upstream protection are essential for any earthen dam.
2. Designing and constructing adequate spillways is usually the most technically difficult part of any dam building work. Any site with a poor quality spillway should not be used.

3. If it is not adequately compacted during construction, the dam will have weak structure prone to seepage.

4. Earthen dams require continual maintenance to prevent erosion, tree growth, subsidence, animal and insect damage and seepage.

Types of Earthen Dam

1. Based on the method of construction:

(a) Rolled Fill Earthen Dams:

- In this type of dams, successive layers of moistened or damp soils are placed one above the other.
- Each layer not exceeding 20 cm in thickness is properly consolidated at optimum moisture content maintained by sprinkling water.
- It is compacted by a mechanical roller and only then the next layer is laid.

(b) Hydraulic Fill Earthen Dam:

In this type of dams, the construction, excavation and transportation of the earth are done by hydraulic methods.

Outer edges of the embankments are kept slightly higher than the middle portion of each layer.

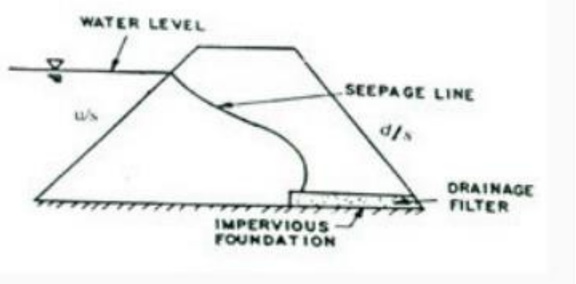
During construction, a mixture of excavated materials in slurry condition is pumped and dischargedat the edges.

This slurry of excavated materials and water consists of coarse and fine materials. When it is discharged near the outer edges, the coarser materials settle first at the edges, while the finer materials move to the middle and settle there.

Fine particles are deposited in the central portion to form a water tight central core. In this method, compaction is not required.

2. Based on the mechanical characteristics of earth materials used in making the section of dam:

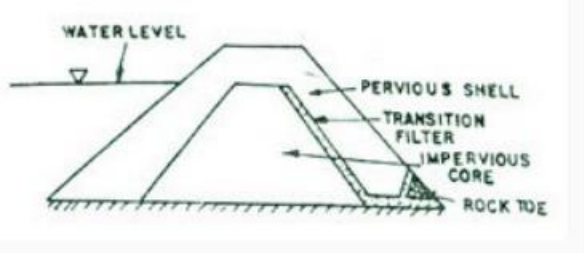
(a) Homogeneous Earthen Dams:

- It is composed of one kind of material (excluding slope protection).
- 

The material used must be sufficiently impervious to provide an adequate water barrier, andthe slopes must be moderately flat for stability and ease of maintenance

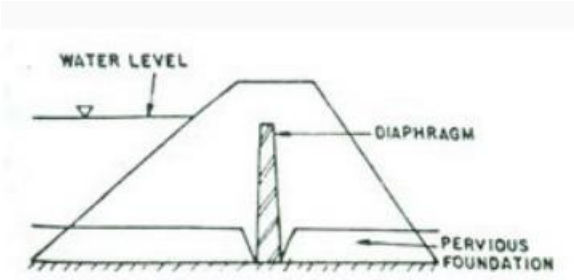
(b) Zoned Earthen Dams:

- It contains a central impervious core, surrounded by zones of more pervious material, called shells.
- These pervious zones or shells support and protect the impervious core.



(c) Diaphragm Earthen Dam:

- This type of dam is a modified form of homogenous dam which is constructed with pervious materials, with a thin impervious diaphragm in the central part to prevent seepage of water.
- The thin impervious diaphragm may be made of impervious clayey soil, cement concrete or masonry or any impervious material.
- The diaphragm can be constructed in the central portion or on the upstream face of the dam.
-



- The main difference in zoned and diaphragm type of dams depends on the thickness of the impervious core or diaphragm. The thickness of the diaphragm is not more than 10 m.

Design Criteria

Following main design criteria may be laid down for the safety of an earth dam:

1. To prevent hydraulic failures the dam must be so designed that erosion of the embankment isprevented. For this purpose, the following steps should be followed:
    a) Spillway capacity is sufficient to pass the peak flow.
    b) Overtopping by wave action at maximum water level is prevented.
    c) The original height of structure is sufficient to maintain the minimum safe freeboard after settlement has occurred.
    d) Erosion of the embankment due to wave action and surface runoff does not occur.
    e) The crest should be wide enough to withstand wave action and earthquake shock.
2. To prevent the failures due to seepage:
    a) Quantity of seepage water through the dam section and foundation should be limited.
    b) The seepage line should be well within the downstream face of the dam to prevent sloughing.
    c) Seepage water through the dam or foundation should not remove any particle or in other

words cause piping.

d) There should not be any leakage of water from the upstream to the downstream face. Such leakage may occur through conduits, at joints between earth and concrete sections or through holes made by aquatic animals.

3. To prevent structural failures:

- The upstream and downstream slopes of the embankment should be stable under all loading conditions to which they may be subjected including earthquake.
- The foundation shear stresses should be within the permissible limits of shear strength of the material.

Design of Earthen Dam

The preliminary design of earthen dam is done on the basis of past experiences. For designing purpose several parameters, given below should be considered.

1. Top Width
2. Free Board
3. Settlement Allowance
4. Casing or Outer Shell
5. Cut-off Trench
6. Downstream Drainage System

1. Top Width:

- Minimum top width (W) should be such that it can enhance the practicability and protect it against the wave action and earth wave shocks.
- Sometimes it is also used for transportation purposes.
-

$$W = \frac{H}{5} + 3 \quad \text{(for very low dam)}$$

$$W = 0.55\sqrt{H} + 0.2H \quad (H \leq 30)$$

$$W = 1.65\sqrt[3]{H + 1.5} \quad (H \geq 30)$$

- It depends upon the height of the earthen dam and can be calculated as follows:

where H = the height of the dam (m), for Indian conditions it should not be less than 6 m.

2. Free board:

- It is the vertical distance between the top of the dam and the full supply level of the reservoiror the added height.
- It acts as a safety measure for the dam against high flow condition that is waves and runoff from storms greater than the design frequency from overtopping the embankment.

- The Recommended values of free board for different heights of earthen dams, given by U.S.B.R., are given in Table.

Recommended Values of Free Board given by U.S.B.R.

Nature of spillway	Height of dam	Free board
Free	Any	Minimum 2 m and maximum 3 m over the maximumflood level
Controlled	< 60 m	2.5 m above the top of the gate
Controlled	> 60 m	3 m above the top of the gate

If fetch length or exposure is given then the free board can also be calculated by Hawksley's formula: $h_w = 0.014\, D_m^{0.5}$ where, = wave height (m); Dm = fetch or exposure (m).

2. **Settlement Allowance:**
   - It is the result of the settlement of the fill and foundation material resulting in the decrease of dam storage.
   - It depends upon the type of fill material and the method and speed of construction.
   - It varies from 10% of design height for hand compacted to 5% for machine compacted earthfill.
3. **Casing or Outer Shell:**
   - Its main function is to provide stability and protection to the core.
   - Depending upon the upstream and downstream slopes, a recommendation for the casing and outer shell slopes for different types of soils given by Terzaghi is presented in Table

Recommended Slopes of Earthen Dam (Sources: S.K. Garg, 2008)

Sl. No.	Types of material	u/s slope	d/s slope
1.	Homogenous well graded material	$2\frac{1}{2}:1$	$2:1$
2.	Homogenous coarse silt	$3:1$	$2\frac{1}{2}:1$
3.	Homogenous silty clay or clay a) Height less than 15 m b) Height more than 15 m	 $2\frac{1}{2}:1$ $3:1$	 $2:1$ $2\frac{1}{2}:1$
4.	Sand or sand and gravel with clay core	$3:1$	$2\frac{1}{2}:1$
5.	Sand or sand and gravel with R.C. core wall	$2\frac{1}{2}:1$	$2:1$

Cutoff Trench:

- It is provided to reduce the seepage through the foundation and also to reduce the piping in the dam.

- It should be aligned in a way that its central line should be within the upstream face of the impervious core.
- Its depth should be more than 1 m. Bottom width of cutoff trench (B) is calculated as:

$B = h - d$ where h = reservoir head above the ground surface (m); and d = depth of cutofftrench below the ground surface (m).

4. Downstream Drainage System:

- It is performed by providing the filter material in the earthen dam which is more pervious than the rest of the fill material.
- It reduces the pore water pressure thus adding stability to the dam.
- Three types of drains used for this purpose are:
    1. Toe Drains
    2. Horizontal Blanket
    3. Chimney Drains.

Causes of Failure

1.Hydraulic Failures 40%

2. Seepage Failures 30%

3. Structural Failures 30%

a) Piping

b) Sloughing

a) Overtopping

b) Wave Erosion

c) Toe Erosion

d) Gullying

a) Upstream slope failure due to sudden drawdown

b) Failure by excessive pore pressure

c) Downstream slope failure by sliding

d) Failure due to settlement of foundation

e) Failure by sliding of foundation

f) Failure by spreading

i. Overtopping:

- The dam is overtopped when the volume of incoming flow into the reservoir is more than the actual storage capacity of the reservoir, or the capacity of spillway is not sufficient.
- Sometimes, the faulty operation of spillway also leads to the overtopping problem.

- Similarly, insufficient free board or settlement of foundation as well as embankment also cause the overtopping problem in earthen dam.

ii. Wave Erosion:

- Wave action removes the soil particles from the unprotected part of upstream face of the clam, continuously.
- This is one of the effective factors to cause the hydraulic type failure in earthen dam.

Toe erosion in the earth fill dam, mainly occurs due to following reasons:

1. **Erosion caused by the tail water**; and

- (ii) Erosion due to cross-currents produced by the storage water, spillway bucket or from the outlet, create the problem of hydraulic failure.
- This type of failure can be overcome by providing a thick layer of stone riprap on the downstream face upto the height of tail water level.

iv. Gullying:

- Development of gully in earthen dam is the result of heavy down pour. Such type of failures can be eliminated by providing a proper size of berm, turf or good drainage system towards down-stream side of the dam.

2. Seepage Type Failures:

Failure of earthen dam due to seepage phenomena may be due to following two reasons:

i. Piping; and

ii. Sloughing.

i. Piping:

The continuous flow of seepage water through the body as well as foundation of the dam is the main reason of piping. It causes catastrophic failures in the earth fill dams.

The flow of seepage water through the body of earth dam develop following four effects:

a) The flow of seepage water generates an erosive force, which tends to dislodge the soil particles from the dam section. The dislodged particles are migrated into the voids of the filter materials, down-stream side; and thus clogged them, as result the drainage system gets failed.
b) The seepage flow develops differential pore pressure which tends to lift up the soil mass, causing boiling effect in the dam.
c) Piping is also the result of internal erosion of the soil mass due to seepage flow through the earth dam.
d) The pore pressure developed in the soil reduces the soil strength, which makes the soil mass weak, as result there is failure of dam due to shear force.

Sometimes, the leakage from earthen dam also generates the piping type failure. Furthermore, it is also observed that, the piping type failure is most prominent in those dams, which are poorly constructed. Generally, this is due to poor compaction surrounding the concrete outlets or other parts of the structure etc.

ii. Sloughing:

- Failure of earthen dam due to sloughing is closely related to the water level in the reservoir.
- In full reservoir condition the downstream toe of the dam becomes fully saturated, which is failure by producing a small slump or miniature slide.
- Under miniature slide the saturated steep face of the dam is dislodged.
- This process is continued till the remaining portion of the dam is being very weak to withstand against pore water pressure.

3. Structural Failures:

i. Structural failure mainly caused by the following reasons:

ii. Upstream and downstream slope failures due to formation of excessive pore pressure.

iii. Upstream failure due to sudden drawdown in the reservoir water level.

iv. Downstream failure at the time of full reservoir.

v. Foundation slide.

vi. Failure of dam due to earthquake.

vii. Failure of dam due to unprotected side slope.

viii. Failure due to damage caused by burrowing animals.

ix. Failure due to damage caused by water soluble materials.

i. Upstream and Downstream Slope Failure due to Pore Pressure:

- Development of pore pressure in the body of earthen dam, is mainly due to poor compressibility of the soil.
- This occurrence is more susceptible, when dam is constructed with relatively impervious compressible soils, in which drainage of seepage water is extremely low, which causes the development of pore pressure in the soil.
- The compressibility of soil is related to the permeability.
- It has been observed that, when permeability of soil is less than 10–6 cm/s, then there is no substantial drop in pore pressure in the central part of the dam by the end of construction.
- A pore pressure equal to 140% of total weight of soil develops a very crucial situation regarding dam stability. In this condition the slope of dam is likely to failed.

ii. Failure of Upstream Slope due to Sudden Draw down in the Reservoir Water level:

- Failure of upstream slope due to sudden draw down in reservoir water level is a critical condition.
- During this stage, the hydrostatic pressure acting along the upstream slope is suddenly removed, as result the face of the dam gets slide.
- In this failure the upstream side slope did not get complete failure, because when slide takes place due to sudden draw down in reservoir water level, the pore pressure acting along the

sliding surface is reduced to a large extent. In this way, the tendency to continue the processof sloughing and sliding of upstream face of the dam, is checked.

iii. Downstream Slope Slide during Full Reservoir Condition:

- When the reservoir is in full condition, then there happens maximum percolation/seepage loss through the dam section.
- This results into reduction of stability of the dam, which causes the downstream slope gets collapse.
- In this case, the failure of downstream slope generally takes place in-following two types of slide:

(a) Deep Slide:

- Deep slide generally takes place in the clay foundations.
- In deep slide the magnitude of free board given to the dam is reduced due to extending of upstream face beyond its edge of the crest.
- In this type of slide the pore pressure does not decrease, and the unstable vertical face tendsto slough or slide again and again, until to breach the entire dam.

(b) Shallow Slide:

- The shallow slide extends in the dam section not more than 2 m in the direction normal to the slope.

iv. Failure due to Foundation Slide:

- This type of failure of earthen dam generally takes place, when foundation is constructed, using fine silt or soft soil materials.
- Sometimes, when soft and weak clayey soil exists under foundation, then dam also tends to get slide.
- Similarly, excess water pressure in confined sand and silt is also developed in the foundation, which causes the failure of dam due to creation of unbalanced condition.

v. Failure of Dam due to Earthquake:

It generally takes place due to following reasons:

1. Earthquake develops cracks in the body of dam; and thus leading to flow of water, which ultimately causes to failure the dam.
2. It compresses the foundation and embankment, both, thereby the total free board provided to the dam gets reduce and thus, increasing the chances of overtopping problem.
3. It shakes the bottom of the reservoir, as result there develop wave action, which causes the problem of failure of dam due to overtopping and wave erosion.
4. It generates an additional force on the face of embankment that can lead to develop shear slide of dam slope.

5. Earthquake is also responsible for sliding the top of dam, which may cause overtopping; and thus damaging the structure.

vi. Failure of Earthen Dam due to Slope Protection:

- Generally, slopes are protected by rip-rap or revetment using a layer of gravel or filter blanket.
- When a heavy storm occurs, then water wave beats the dam slope repeatedly above the reservoir level.
- This action of wave produces the following two effects:
- The wave enters the voids of the rip-rap and washout the filter layer from the dam face. This causes the embankment to get expose to the wave action; and
- If rip-rap is not done by heavy rocks, then there is greater chance of their removal by the forces generated from water waves.

vii. Failure due to Damage Caused by Burrowing:

- Burrowing develops piping type failure in earthen dam. Generally, the animals like muskats burrow the embankment section, either to make shelter for their living or to make a direct passage for running from one end to another.
- If several muskats involved together to make the hole, then their holes may extremely weaken the dam section.

viii. Failure due to Water Soluble Materials:

- Based on several observations on this aspect of failure of earthen dams, it has been found that the leaching of natural water soluble materials such as zypsum etc. from the dam tends to create water leakage problem through the dam section.
- In this condition, the foundation also gets settle down, and thus creates the problem of overtopping and ultimately the dam reaches to the point of its failure.

DESIGN OF EARTHEN DAM

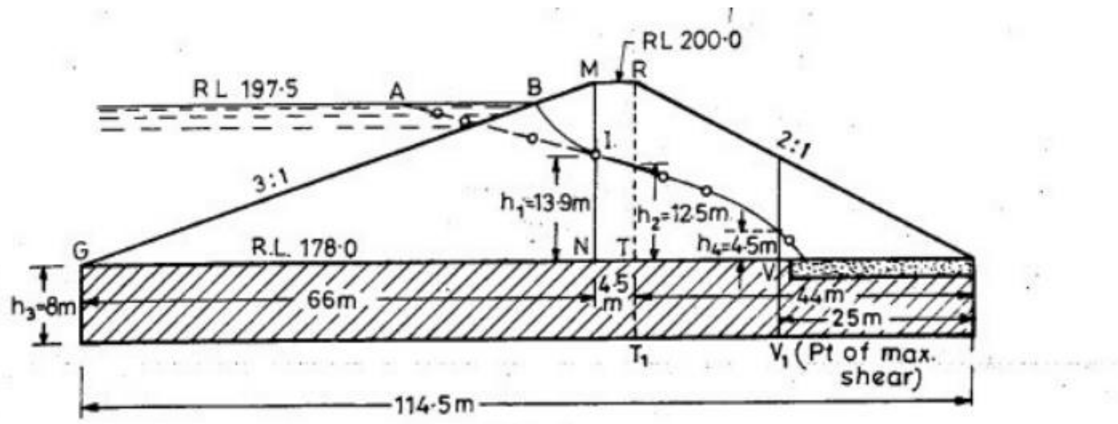


Fig. 20.29 (*a*)

The total area of dam section $= (114.5 + 4.5)\,\dfrac{22}{2} = 1{,}409$ sq. m

The area above the seepage line is measured and is approximately found to be 380 m^2. (In the absence of a planimeter, graph can be used).

$\therefore$ Area below the seepage line $= 1{,}409 - 380 = 1{,}029$ sq. m

Now

Weight of the dry portion of the dam section

$$= (380\ \text{m}^2 \times 1\ \text{m} \times 18\ \text{kN/m}^3 = 6830\ \text{kN}.$$

Weight of the submerged portion of the dam section

$$= 1029\ \text{m}^2 \times 1\ \text{m} \times 12\ \text{kN/m}^3 = 12{,}350\ \text{kN}$$

Total weight of dam (called average weight)

$$= 6{,}830 + 12{,}350 = 19{,}180\ \text{kN}$$

Example 20.4. *An earthen dam made of homogeneous material has the following data :*

Level of the top of the dam	= 200.00 m
Level of deepest river bed	= 178.0
H.F.L. of reservoir	= 197.5 m
Width of top of dam	= 4.5 m
Upstream slope	= 3 : 1
Downstream slope	= 2 : 1
Length of the horizontal filter from d/s toe, inwards	= 25 m
Cohesion of soil of dam	= 24 kN/ m^2
Cohesion of soil of foundation	= 54 kN/ m^2
Angle of internal friction of soil in the dam	= 25°
Angle of internal friction of soil in the foundation	= 12°
Dry weight of the soil in the dam	= 18 kN/ m^3
Submerged weight of the soil in the dam	= 12 kN/ m^3
Dry unit weight of the foundation soil	= 18.3 kN/ m^3
Coefficient of permeability of soil in the dam	= 5×10^{-6} m/sec.

The foundation soil consists of 8 m thick layer of clay, having negligible coefficient of permeability. Check the stability of the dam and its foundations.

Solution.

(1) **Overall stability of the dam section as a whole**

We will consider 1 m length of the dam. The section of the dam and the phreatic line is first of all drawn, as given in example 20.3 and shown in Fig. 20.29 (*a*). The dam section, etc. is generally drawn on a graph sheet so as to facilitate in measuring the areas above and below the seepage line, if planimeter is not available.

line at a point, the height of which is measured as $h_1 = 13.6$ m above the base of the dam.

Horizontal force (P_u) acting on the ΔGMN is given by equation (20.27) as :

$$P_u = \left[\frac{\gamma_1 h^2}{2} \tan^2\left(45 - \frac{\phi}{2}\right) + \gamma_w \cdot \frac{{h_1}^2}{2}\right]$$

where γ_1 = the weighted density at the centre of triangular shoulder upstream (ΔGMN) and is given by equation (20.28) as :

$$\gamma_1 = \frac{\gamma_{sub} \cdot h_1 + \gamma_{dry}(h - h_1)}{h}$$

$$= \frac{12 \times 13.9 + 18\,(22.0 - 13.9)}{22.0}$$

$$= 14.7 \text{ kN/m}^3$$

$$\therefore \quad P_u = \frac{14.7 \times (22.0)^2}{2} \tan^2\left(45° - \frac{25°}{2}\right) + 9.81 \times \frac{(13.9)^2}{2} = 2391 \text{ kN}$$

Shear resistance R_u of the u/s slope portion of dam developed at the base GN is given by equation (20.29) as :

$$R_u = C + W \tan \phi$$

$= c\,(B_u \times 1) + (\gamma_{sub} \frac{1}{2} B_u\, h) \tan \phi$; neglecting the small dry soil area BMI, as it is very small and this neglection is on a safer side.

$$B_u = 66 \text{ m}$$

$$\therefore \quad R_u = 24 \times 66 + (12 \cdot \tfrac{1}{2} \cdot 66 \cdot 22.0) \tan 25°$$

$$= 1584 + 4062 = 5646 \text{ kN}$$

Shear resistance of the dam at the base

$$= C + W \tan \phi$$

where C = Total cohesive strength of the soil at the base

$= c \times B \times 1 = (24 \times 114.5 \times 1)$ kN

B = Total base width = 114.5 m

$W \tan \phi = 19{,}180 \tan 25°$

$\therefore$ Shear resistance at base,

$$R = 24 \times 114.5 \times 1 + 19180 \tan 25° = 11690 \text{ kN}$$

Horizontal force = Horizontal pressure of water.

$$= P = \tfrac{1}{2} \gamma_w h^2 = \tfrac{1}{2} \cdot 9.81\,(19.5)^2 = 1865 \text{ kN}$$

Factor of safety against failure due to horizontal shear at base

$$= \frac{11690}{1865} = 6.27 > 1.3 \; (\therefore \text{ Safe})$$

(2) **Stability of the u/s slope portion of dam (Under sudden drawdown)** *horizontal shear along the base under the u/s slope of dam*

Draw a vertical through the u/s extremity of the top width of dam [*i.e.* point M, Fig. 20.29 (a)] so as to cut the base of the dam at point N. This vertical MN cuts the seepage

Factor of safety against horizontal shear along base under u/s slope

$$= \frac{R_u}{P_u} = \frac{5646}{2391} = 2.36 > 2.0 \qquad (\therefore \text{ safe})$$

Horizontal shear stress induced in the u/s slope portion of dam at base.

$$\therefore \quad \tau_{av} = \frac{P_u}{B_u \times 1} = \frac{2391}{66} \text{ kN/m}^2 = 36.23 \text{ kN/m}^2$$

$$\tau_{max} = \text{Maximum shear}$$

$$= 1.4\, \tau_{av} = 1.4 \times 36.23 = 50.72 \text{ kN/m}^2$$

The maximum shear is developed at a point $0.6\, B_u$

$$= 0.6 \times 66 = 39.6 \text{ m away from point } G$$

The unit shear resistance developed at this point

$$\tau_f = c + 0.6\, \gamma_{sub} \tan \phi$$

$$= 24 + 0.6 \times 22.0 \times 12 \tan 25° = 97.9 \text{ kN/m}^2$$

$$\therefore \quad \text{F.S.} = \frac{\tau_f}{\tau_{max}} = \frac{97.9}{50.72} = 1.93 > 1 \qquad (\therefore \text{ safe}).$$

(3) **Stability of d/s portion of dam.** *Horizontal shear along base under the d/s slope of dam.*

Draw a vertical through the d/s extremity of the top width of dam (*i.e.* point R) to cut the base at point T [Fig. 20.29 (*a*)]. Let this vertical cut the seepage line in a point, the height of which from the base is measured as $h_2 = 12.5\ m$.

Horizontal force P_d acting on the portion of downstream dam (RTS) during steady seepage is given by equation (20.35) as :

$$P_d = \left[\frac{\gamma_2 h^2}{2} \tan^2\left(45° - \frac{\phi}{2}\right) + \gamma_w \frac{h_2^2}{2}\right]$$

where γ_2 is the weighted density at the centre of the triangular shoulder RTS and given by equation (20.36) as :

$$\gamma_2 = \frac{\gamma_{sub}\, h_2 + \gamma_{dry}\,(h - h_2)}{h}$$

$$= \frac{12 \times 12.5 + 18 \times (22.0 - 12.5)}{22.0}$$

$$= 14.6 \text{ kN/m}^3$$

$$P_d = \frac{14.6\,(22)^2}{2} \tan^2\left(45° - \frac{25°}{2}\right) + 9.81 \cdot \frac{(12.5)^2}{2} = 2200 \text{ kN}$$

Shear resistance R_d of the d/s slope portion of dam developed at base TS is given as :

$$R_d = C + W \tan \phi$$

The area A_1 of the dry soil within the ΔRTS above the seepage line ≈ 300 sq. m (from graph or planimeter).

The total area of the $\Delta RTS = \frac{1}{2} \times 44 \times 22 = 484 \text{ m}^2$

$\therefore$ Area of submerged soil

$$A_2 = 484 - 300 = 184 \text{ sq. m}$$
$$R_d = cB_d + \left[\gamma_{dry} A_1 + \gamma_{sub} A_2\right] \tan\phi$$
$$= 24 \times 44.0 + [18 \times 300 + 12 \times 184] \tan 25° = 4604 \text{ kN.}$$

F.S. against horizontal shear along base under d/s slope

$$= \frac{R_d}{P_d} = \frac{4604}{2200} = 2.09 > 2 \ (\therefore \text{ Safe})$$

Average shear induced at base

$$= \frac{P_d}{B_d} = \frac{2200}{44} = 50 \text{ kN/m}^2$$

Maximum shear stress induced

$$\tau_{max} = 1.4 \times 50 = 70 \text{ kN/m}^2$$

The maximum shear stress is developed at a point 0.6 B_d

$$= 0.6 \times 44 = 26.4 \text{ m away from toe}$$

This unit shear resistance developed at this point

$$\tau_f = c + 0.6h\, \gamma_{sub} \tan\phi$$

(assuming the entire height as submerged as it will give safer results)

$$= 24 + 0.6 \times 22 \times 12 \tan 25° = 97.9 \text{ kN/m}^2$$

$$\therefore \quad \text{F.S.} = \frac{\tau_f}{\tau_{max}} \frac{97.9}{70} = 1.40 > 1 \qquad (\therefore \text{ Safe})$$

(4) **Stability of the foundation soil**

Average compressive stress on foundation soil

$$= \frac{\text{Weight of dam}}{\text{Base area on which it acts}}$$

Since the compressive stress is maximum when the entire dam soil is dry, therefore, we will first calculate the dry weight of the dam.

Area of section of dam

$$= 1{,}409 \text{ sq. m (calculated earlier)}$$

Dry weight of dam section

$$= 18 \times 1{,}409 = 25{,}362 \text{ kN}$$

Average compressive stress at base

$$= \frac{25362}{114.5} = 221.5 \text{ kN/m}^2$$

Shear stress induced at base

The total horizontal shear force (P) under the d/s slope of the dam (which is the worst case, *i.e.* the steepest slope) is given by equation (20.39) as :

$$P = \gamma_{eq}\left[\frac{(h+h_3)^2 - h_3^2}{2}\right]\left[\tan^2\left(45° - \frac{\phi_1}{2}\right)\right]$$

where γ_{eq} is the equivalent weight of dry soil in foundation and dam

$$\gamma_{eq} = \frac{18h + 18.3\,h_3}{h + h_3}$$

[$\because$ Unit wt. of foundation soil of thickness $h_3 = 18.3\ \text{kN/m}^3$]

where $h = 22$ m

$h_3 = 8$ m.

$$\therefore \quad \gamma_{eq} = \frac{18 \times 22 + 18.3 \times 8}{22 + 8} = 18.1\ \text{kN/m}^3$$

ϕ_1 is given by equation (20.41) as :

$$\gamma_{eq}\,(h + h_3)\tan\phi_1 = c_f + \gamma_{eq}\,(h + h_3)\tan\phi_f$$

or $\quad 18.1\,(22 + 8)\tan\phi_1 = 54 + 18.1\,(22 + 8)\tan 12°$

or $\quad \tan\phi_1 = 0.312$

or $\quad \phi_1 = 17.3°$

$$\therefore \quad P = 18.1\left[\frac{(22+8)^2 - (8)^2}{2}\right][\tan^2(45° - 8.65°)]$$

$$= \frac{18.1}{2}\,[900 - 64]\,[(0.737)^2] = 4100\ \text{kN}$$

Average shear stress induced at base of d/s slope

$$\tau_{av} = \frac{4100}{44} = 93.2\ \text{kN/m}^2$$

Maximum shear stress induced at $0.6 \times 44 = 26.4$ m away from the d/s toe inwards at point V_1 is given by

$$= \tau_{max} = 1.4 \times 93.2 = 130.4\ \text{kN/m}^2$$

Shear resistance of the foundation soil below the d/s slope portion of dam

Unit shear resistance τ_{f_1} below the toe at point S_1

$$= [c_f + \gamma_f \times h_3 \tan\phi_f]$$

$$= 54 + 18.3 \times 8 \times \tan 12°$$

$$\therefore \quad = 85.1\ \text{kN/m}^2$$

Unit shear resistance τ_{f_2} at point T_1

$$= c_f + \gamma_3\,(h + h_3)\tan\phi_f$$

where $\quad \gamma_3 = \dfrac{\gamma_{sub\,for\,dam} \times h_2 + \gamma_{dry\,for\,dam} \times (h - h_2) + \gamma_f\,h_3}{h + h_3}$

$$= \frac{12 \times 12.5 + 18 \times 9.5 + 18.3 \times 8}{30} = 15.6\ \text{kN/m}^3$$

$$\therefore \quad \tau_{f_2} = 54 + 15.6\,(22 + 8) \times \tan 12° = 153.5\ \text{kN/m}^2$$

The average unit shear resistance developed at foundation level in a length equal to $T_1S_1 = 44$ m, is given by

$$\tau_f = \frac{\tau_{f_1} + \tau_{f_2}}{2} = \frac{85.1 + 153.5}{2} = 119.3 \text{ kN/m}^2$$

Over all F.S. against shear

$$= \frac{\tau_f}{\tau_{av}} = \frac{119.3}{93.2} = 1.28 < 1.5 \quad \textbf{(Hence, unsafe)}$$

The foundation soil is thus weaker to carry the load and hence the d/s slope will have to be flattened.

Shear resistance at the point of maximum shear, *i.e.* at point V_1 is given as :

$$(\tau_f)_{max} = c_f + (0.6h + h_3)\,\gamma_4 \tan \phi_f$$

$$\gamma_4 = \frac{12 \times 4.5 + 18\,(0.6 \times 22 - 4.5) + 18.3 \times 8}{0.6 \times 22 + 8} = 16.8 \text{ kN/m}^3$$

$$(\tau_f)_{max} = 54 + [0.6 \times 22 + 8]\; 16.8 \tan 12° = 129.8 \text{ kN/m}^2$$

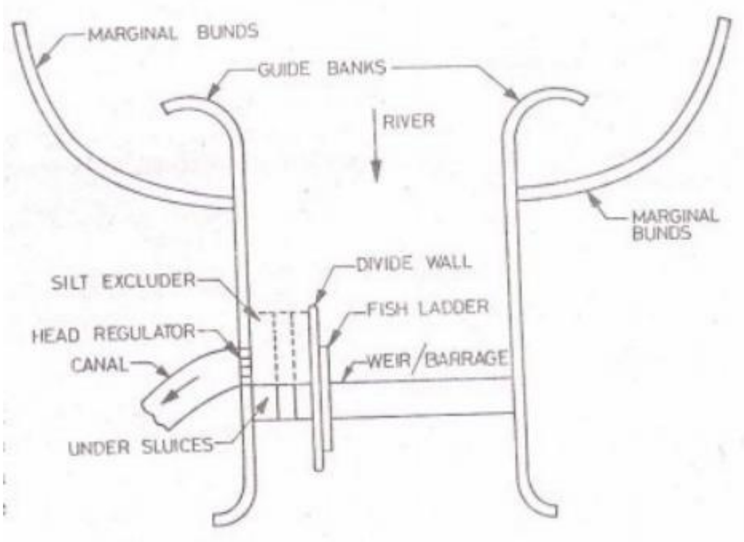
$$\text{F.S.} = \frac{[\tau_f]_{max}}{\tau_{max}} = \frac{129.8}{130.4} = 0.995 < 1.0 \qquad \text{(Hence, Unsafe)}$$

The foundation shear and F.S. can also be calculated below the u/s portion of dam soil, in the same manner as has been done for d/s slope portion, if required.

COMPONENT PARTS OF A DIVERRSION HEADWORK

A diversion headwork consist of the following component parts

1. Weir or barrage
2. Undersluices
3. Divide wall
4. Fish ladder
5. Canal head regulator
6. pocket or approach channel
7. Silt excluders/ Silt prevention devices/
8. River training works (Marginal bunds and guide banks)

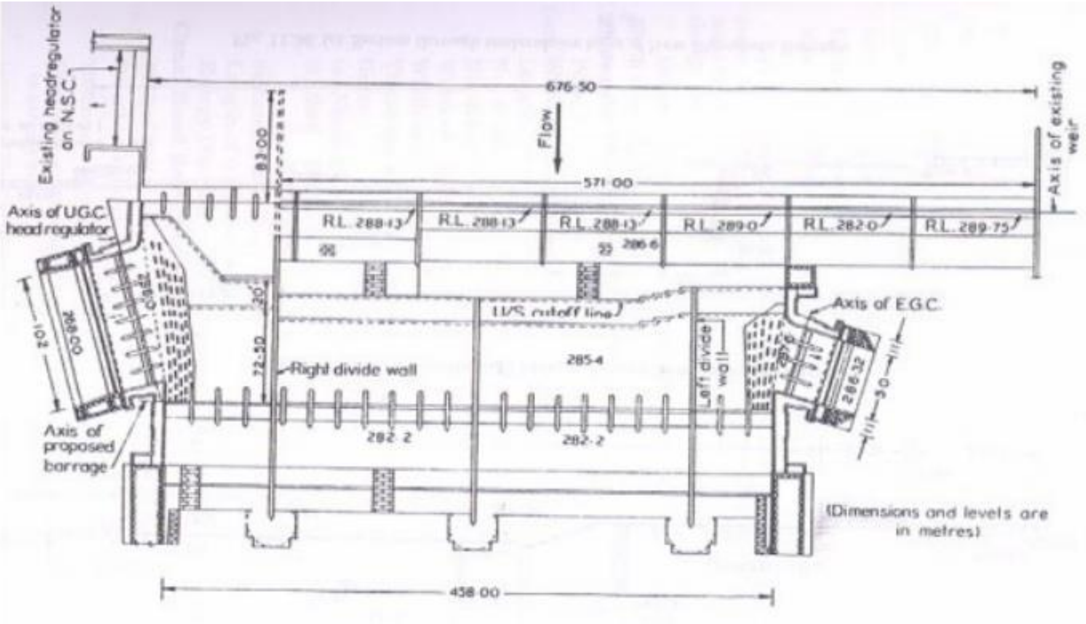


Undersluices

- Undersluice sections are provided adjacent to the canal head regulators.
- The undersluices should be able to pass fair weather flow for which the crest shutters on the weir proper need not be dropped.
- The crest level of the undersluices is generally kept at the average bed level of the river.

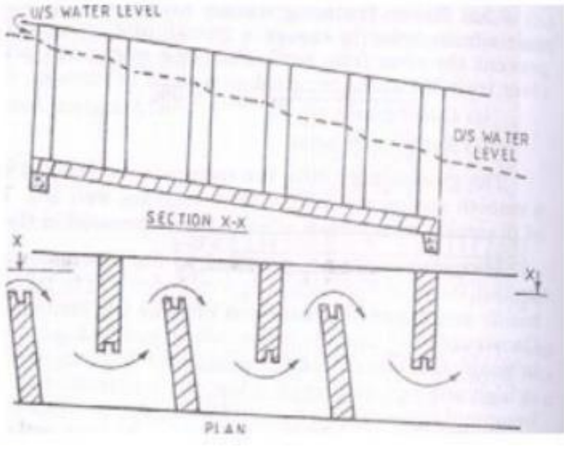
Divide Wall

- A divide wall is a wall constructed parallel to the direction of flow of river to separate the weir section and the undersluices section to avoid cross flows.
- If there are undersluices at both the sides, there are two divide walls.



Fish Ladder

- A fish ladder is a passage provided adjacent to the divide wall on the weir side for the fish to travel from the upstream to the downstream and vice versa.
- Fish migrate upstream or downstream of the river in search of food or to reach their sprawling places.
- In a fish ladder the head is gradually dissipated so as to provide smooth flow at sufficiently low velocity.
- Suitable baffles are provided in the fish passage to reduce the flow velocity.



Canal Head Regulator

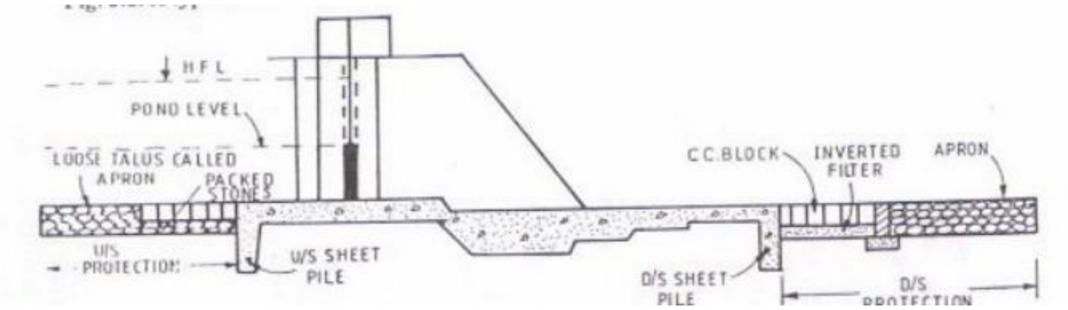
- A canal head regulator is provided at the head of the canal offtaking from the diversion headworks.
- It regulates the supply of water into the canal, controls the entry silt into the canal, and prevents the entry of river floods into canal.

Silt Excluder

- A silt excluder is a structure in the undersluices pocket to pass the silt laden water to the downstream so that only clear water enters into the canal through head regulator.
- The bottom layer of water which are highly charged with silt pass down the silt excluder an escape through the undersluices.

Guide Banks and Marginal Bunds

- Guide banks are provided on either side of the diversion headworks for a smooth approach and to prevent the river from outflanking.
- Marginal bunds are provided on either side of the river upstream of diversion headworks to protect the land and property which is likely to be submerged during ponding of water in floods.
- **Weir or Barrage**
- A diversion head works is a structure constructed across a river for the purpose of raising water level in the river so that it can be diverted into the offtaking canals.
- A weir is a raised concrete crest wall constructed across the river.



- It may be provided with small shutters (gates) on its top. In the case of weir, most of the raising of water level or ponding is done by the solid weir wall and little with by the shutters.
- A

barrage has a low crest wall with high gates. As the height of the crest above the river bed is lowmost of the ponding is done by gates. During the floods the gates are opened so afflux is very small.

- A weir maintains a constant pond level on its upstream side so that the water can flow into the canals with the full supply level (F.S.L.). If the difference between the pond level and the crest level is less than 1·5 m or so, a weir is usually constructed.
- On the other hand, if this difference is greater than 1·50 m, a gate-controlled barrage is

generally more suitable than a weir. In the case of a weir, the crest shutters are dropped during floods so that the water can pass over the crest.

- During the dry period, these shutters are raised to store water upto the pond level. Generally, the shutters are operated manually, and there is no mechanical arrangement for raising or dropping the shutters.
- On the 'other hand, in the case of a barrage, the control of pondage and flood discharge is achieved with the help of gates which are mechanically operated

ADVANTAGES AND DISADVANTAGES OF WEIRS AND BARRAGES

1. Weirs Advantages: The initial cost of weirs is usually low.

Disadvantages:

i. There is a large afflux during floods which causes large submergence.

ii. Because the crest is at high level, there is great silting problem

iii. The raising and lowering of shutters on the crest is not convenient. Moreover, it requires considerable time and labour.

iv. The weir lacks an effective control on the river during floods.

2. Barrages Advantages

i. The barrage has a good control on the river during floods. The outflow can be easily regulated by gates.

ii. The afflux during floods is small and, therefore, the submerged area is less.

iii. There is a good control over silt entry into the canal.

iv. There is a good control over flow conditions, shoal formations and crosscurrents on the upstream of the barrage.

v. There are better facilities for inspection and repair of various structures.

vi. A roadway can be conveniently provided over the structure at a little additional cost.

Disadvantages:

The initial cost of the barrage is quite high.

Conclusion: A barrage is generally better than a weir. Most of the diversion headworks these days usually consist of barrages.

TYPES OF WEIRS

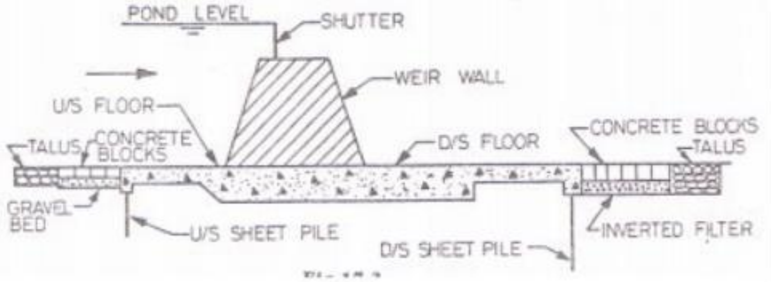
The weirs may be broadly divided into the following types

1. Vertical drop weirs.
2. Rockfill weirs.
3. Concrete glacis or sloping weirs.

1. Vertical drop weirs

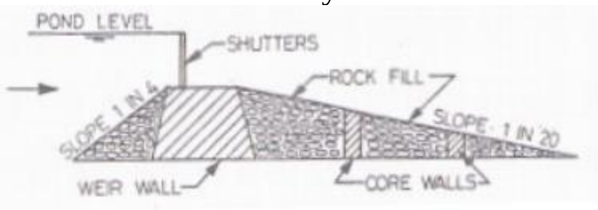
- A vertical drop weir consists of a masonry wall with a vertical (or nearly vertical) downstream face and a horizontal concrete floor.

- The shutters are provided at the crest, which are dropped during floods so as to reduce afflux. The water is ponded upto the top of the shutters during the rest of the period.
- Vertical drop weirs were quite common in early diversion headworks, but these are now becoming more or less obsolete.
- The vertical drop weir is suitable for hard clay foundation as well as consolidated gravel foundations, and where the drop is small.
- The upstream and downstream cutofIwalls (or piles) are provided upto the scour depth. The weir floor is designed as a gravity section.



2. Rockfill weirs:

- In a rockfill type weir, in addition to the main weir wall, there are a number of core walls. The space between the core walls is filled with the fragments of rock (called rockfill).
- A rockfill weir requires a lot of rock fragments and is economical only when a huge quantity of rockflll is easily available near the weir site.
- It is suitable for fine sand foundation. The old Okhla Weir across the Yamuna river is a rockfill weir.
- Such weirs are also more or less obsolete these days.

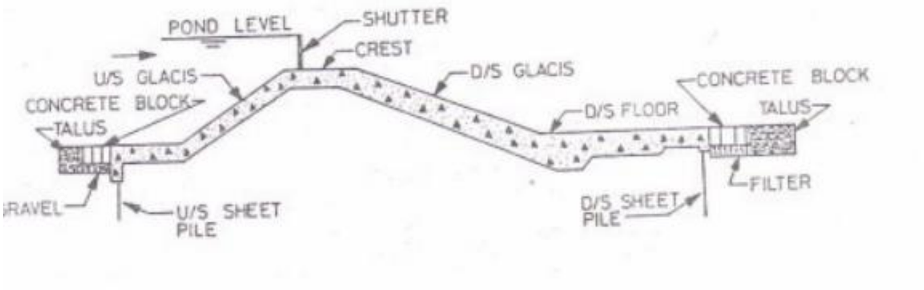


3. Concrete sloping weir :

- Concrete sloping weirs (or glacis weirs) are of relatively recent origin.
- The crest has glacis (sloping floors) on upstream as well as downstream.
- There are sheet piles (or cut off walls) driven upto the maximum scour depth at the upstream and downstream ends of the concrete floor.
- Sometimes an intermediate pile is also driven at the beginning of the upstream glacis or at the end of downstream glacis.

- The main advantage of a sloping weir over the vertical drop weir is that a hydraulic jump is formed on the d/s glacis for the dissipation of energy.

- Therefore, the sloping weir is quite suitable for large drops.



Modes of Failure :

- Irrigation structures (or hydraulic structures) for the diversion and distribution works are weirs, barrages, head regulators, distributary head regulators, cross regulators, cross-drainage works, etc.

- These structures are generally founded on alluvial soils which are highly pervious. Moreover, these soils are easily scoured when the high velocity water passes over the structures.

- The failures of weirs constructed on the permeable foundation may occur due to various causes, which may be broadly classified into the following two categories:

  1. Failure due to- subsurface flow

  2. Failure due to surface flow

1. Failure due to subsurface flow:

The failure due to subsurface flow may occur by piping or by rupture of floor due to uplift.

(a) Failure by piping:

- Piping (or undermining) occurs below the weir if the water percolating through the foundation has a large seepage force when it emerges at the downstream end of theimpervious floor.

- When the seepage force exceeds a certain value, the soil particles are lifted up at the exit point of the seepage.

- With the removal of the surface soil particles, there is further concentration of flow in the remaining portion and more soil particles are removed.

- This process of backward erosion progressively extends towards the upstream side, and a pipe-like hollow formation occurs beneath the floor.

- The floor ultimately subsides in the hollows so formed and fails. This type of failure is known as piping failure.

(b) Failure by rupture of floor:

- The water percolating through the foundation exerts an upward pressure on the impervious floor, called the uplift pressure.
- If the weight of the floor is not adequate to counterbalance the uplift pressure, it may fail by rupture.

2. Failure due to surface flow

The failure due to surface flow may occur by suction pressure due to hydraulic jump or by scouring of the bed.

(a) Failure by suction pressure :

- In the glacis type of weirs, a hydraulic jump is formed on the d/s glacis. In this case, the water surface profile in the hydraulic jump trough is much lower than the subsoil H.G.L.
- Therefore uplift pressure occurs on the glacis. This uplift pressure is known as the suction pressure. If the thickness of floor is not adequate, the rupture of floor may occur.

(b) Failure by scour :

- During floods, scouring occurs in the river bed. The bed of the river may be scoured to a considerable depth.
- If no suitable measures are adopted, the scour may cause damage to the structure and may lead to the failure.

Design aspects

The basic principles for the design of all irrigation structures on pervious foundations are as follows:

(a) Subsurface flow

1. The structure should be designed such that the piping failure does not occur due to subsurface flow.
2. The downstream pile must be provided to reduce the exit gradient and to prevent piping.
3. An impervious floor of adequate length is provided to increase the path of percolation and to reduce the hydraulic gradient and the seepage force.
4. The seepage path is increased by providing piles and impervious floor to reduce the uplift pressure.
5. The thickness of the floor should be sufficient to resist the uplift pressure due to subsurface flow. The critical section is d/s of the weir/crest wall.

6. A suitably graded inverted filter should be provided at the downstream end of the impervious floor to check the migration of soil particles along with water. The filter layer is loaded with concrete blocks. Concrete blocks are also provided at the upstream end.

(b) Surface flow

1. The piles (or cutoff walls) at the upstream and downstream ends of the impervious floor should be provided upto the maximum scour level to protect the main structure against scour.

2. The launching aprons should be provided at the upstream and downstream ends to provide a cover to the main structure against scour.

3. A device is required at the downstream to dissipate energy. For large drops, hydraulic jump is used to dissipate the energy.

4. Additional thickness of the impervious floor is provided at the point where the hydraulic jump is formed to counterbalance the suction pressure.

5. The floor is constructed as a monolithic structure to develop bending resistance (or beam action) to resist the suction pressure.

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CANAL IRRIGATION

Canal regulations – direct sluice - Canal drop – Cross drainage works-Canal outlets – Design of prismatic canal-canal alignments-Canal lining - Kennedy's and Lacey's Regime theory-Design of unlined canal

CANAL:-

A canal is an artificial channel generally trapezoidal in shape constructed on the ground to carry water to the field either from the river of from a reservoir.

Canal regulations

Any structure constructed to regulate the discharge, full supply level or velocity in a canal is knownas Regulation Work.

Types & Location:

1. →Head Regulator or Head Sluice at Barrage/Weir, Dam
2. →Cross Regulator on Parent Canal
3. →Distributory Head Regulator on Off-take Canal
4. →Canal Fall along Parent Canal or Off-take Canal
5. →Canal Escape on any type of canal
6. →Canal Outlet on Distributing Canal

Types & Purpose:

1. →Head Regulator or Head Sluice to divert water to parent channel from a barrage or weir
2. →Cross Regulator to head up water in the parent channel to divert some of it through an offtake channel or distributory canal
3. →Distributory Head Regulator to control the amount of water flowing in to off take channel
4. →Canal Fall to lower the water level of the canal

5. → Canal Escape to allow release of excess water from the canal system

6. → Canal Outlet to take out water for delivery to the field channel or water courses