LEARNING RESOURCE MATERIAL

COURSE CODE- Th 2

HYDRAULICS & IRRIGATION ENGINEERING

DEPARTMENT

OF

CIVIL ENGINEERING

S.K.D.A.V. GOVERNMENT POLYTECHNIC, ROURKELA

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COURSE OBJECTIVES

On completion of the course students will be able to -
1. Define common fluid properties and interpret results from pressure measuring instruments.
2. Realize the science behind fluid flow and compute fluid flow characteristics through notches, weirs, channels and pipes.
3. Realize the working principle of hydraulic pumps and evaluate their performance in general cases.
4. Comprehend the need of irrigation
5. Determine cause and effect of water logging
6. Comprehend the purpose of irrigation system components and elaborate on these

CONTENTS

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

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

ρ = 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 ρ=1000 kg/m^3. For gases we take air or O_2 as a standard fluid with density, ρ=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.

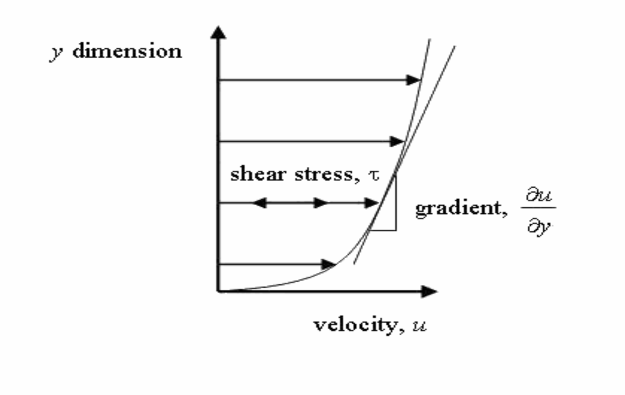


Fig. 1.1

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:-

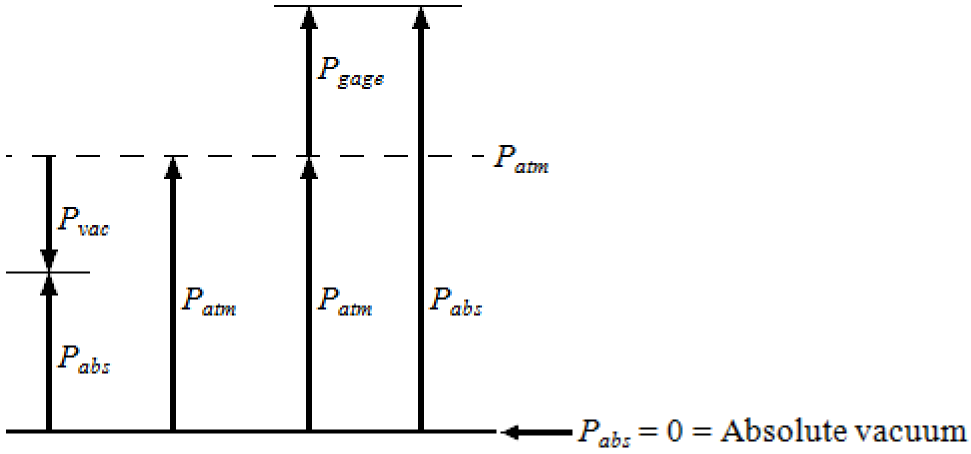


Fig. 1.2

- 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**,

$P=p_1a_1 + p_2a_2+ p_3a_3$…………

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.

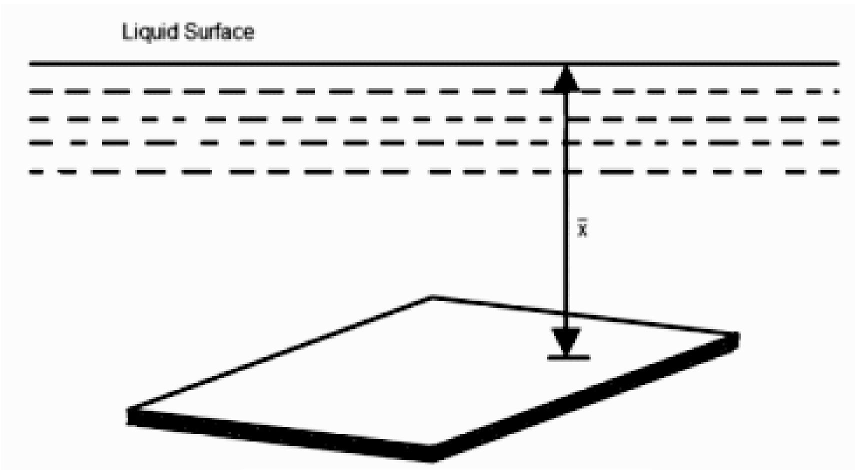


Fig. 1.3

- 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.

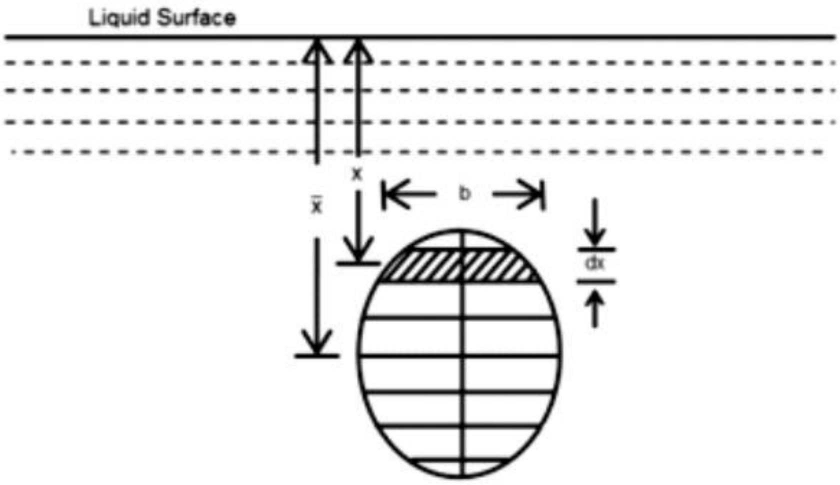


Fig. 1.4

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' ’rinciple 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' ’rinciple 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.

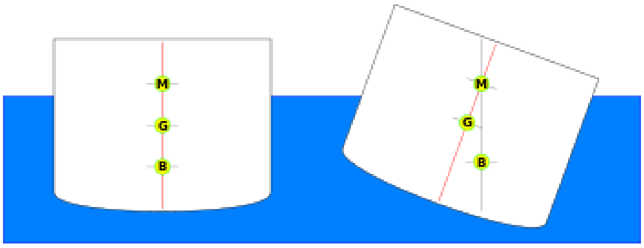


Fig. 1.5

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,

$Q_1 = a_1.v_1 = (17.67x10^{-3})x2.5 = 44.2 \times 10^{-3}\ m^3/s$

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

:. 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-

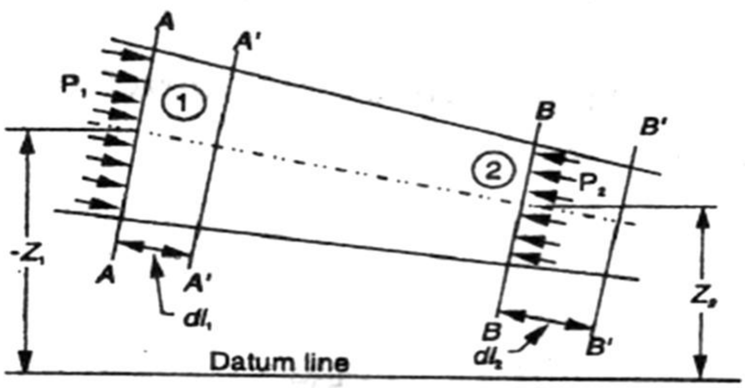


Fig. 2.1

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' '' 'nd B' '' 'hrough 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 'o' the liquid between AA and A' '' 'o BB and B' '','the remaining liquid between A' '' 'nd BB being un horoughlyLet W be the weight of the liquid between AA and 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' ''
' Force x Distance = $P_1 . a_1 . dL_1$
Similarly, work done by pressure at BB, in moving the liquid to B' ''
'-$P_2a_2dl_2$
..Iinus sign is taken as the direction of P_2 is opposite to that of P_1)
:. Total work done by the pressure
$= P_1a1dl_1 - P_2a_2dl_2$
$=P_1a1dl_1-p_{2a}1dl_1$
$\ldots(a_1dl_1=a_2dl_2)$

$= a_1.dl_1 (P_1- P_2) = \frac{W}{w} (P_1 - P_2) \ldots(a_1.dl_1 = \frac{W}{w})$

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 "E"ler'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 'f'llowing 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' 'low 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.

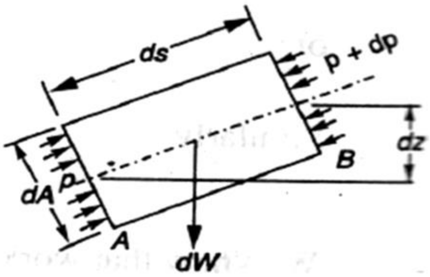


Fig. 2.2

We know that the external forces tending to accelerate the fluid element in the direction of the streamline
= p. dA - (p + dp) dA
= -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,
'

‘

$\frac{p}{\rho}+$ $g_z+v^2/_2$=constant

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

$\frac{p}{w}$+Z+ $v^2/2g$=constant (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.'G’~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$\times 10^{-3}$)$\times$ 1]/ 1.964$\times 10^{-3}$=16m/s

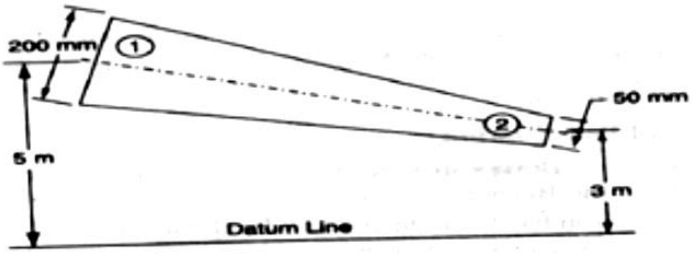


Fig. 2.3

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

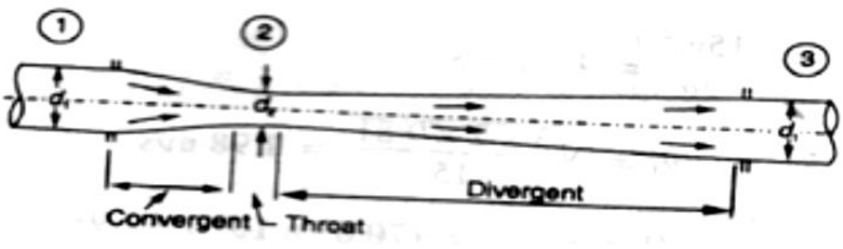


Fig. 2.4

A ve horougeter 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.
©Convergent 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 ve horougeter. 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 mi horoughthe 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.

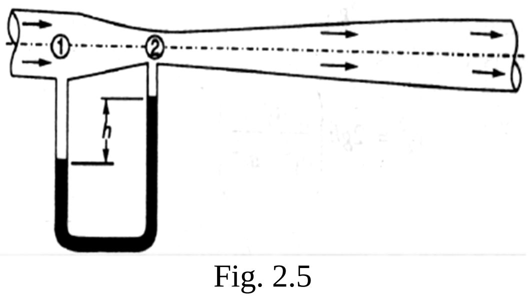


Fig. 2.5

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 ve horougeter 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 ve horougead and is denoted by h.

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

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

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

We know that the discharge through a venture meter,

Q = Coefficient of ve horougeter x $a_2\,v_2$

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

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 ve horougeter,

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

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

Orifice 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).

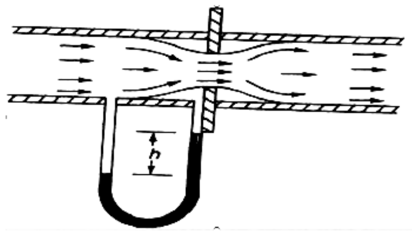


Fig. 2.6

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.

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 $a_1.v_1=a_2v_2$

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

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

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

$\therefore\ v_2^2=2gh[a_1^2/(a_1^2-a_2^2)]$ 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 $a_2\cdot v_2$

$=[C\sqrt{\ }a_1a_2/\sqrt{\ }(a_1^2-a_2^2)]\times\sqrt{2gh}$

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,

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

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

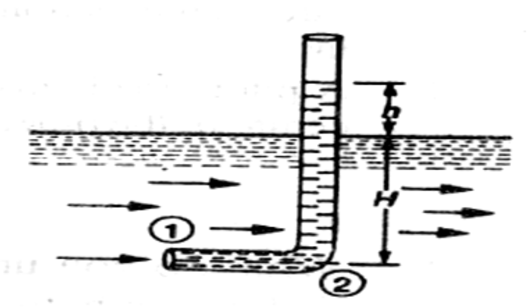


Fig. 2.7

H = Depth of tube in the liquid, and

v = Velocity of the liquid.

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

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

…..($z_1 = z_2$)

$h = v^2/2g$

$\therefore v = \sqrt{}2gh$

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,
$Q = m^2 \times m/s = m^3/s$ = cumecs
2.member 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.

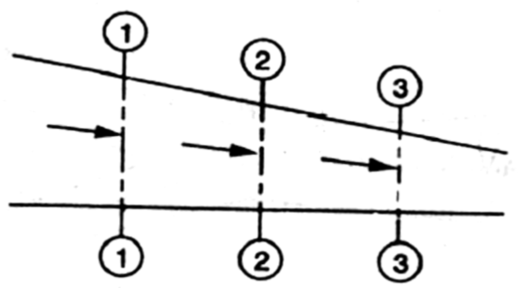


Fig. 2.8

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,

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

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

$$Q_2 = a_1.v_1 \qquad \text{……………………(ii)}$$

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

$$Q_3 = a_3.v_3 \qquad \text{……………………(iii)}$$

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 = $a_1.v_1$ = (7.854×10^{-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.

(b) Notch without end contraction or suppressed notch,

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.

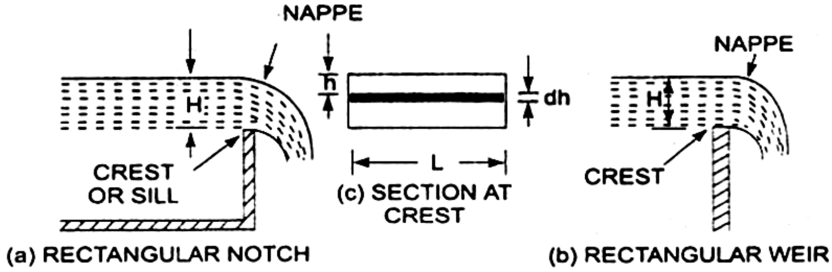


Fig. 2.9

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 - –

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 .

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

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

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

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

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

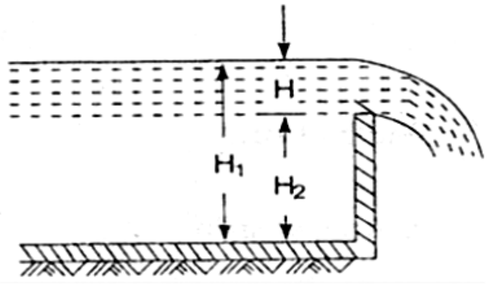


Fig. 2.10

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

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

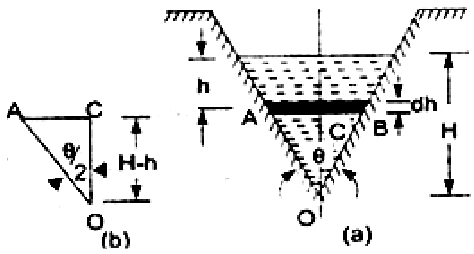


Fig. 2.11

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 $$Q_1 = \frac{2}{3} \times C_{d1} \times L \times \sqrt{2g} \times H^{3/2}$$

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 AE=1 m

Base width, CD=L=0.4 m

Head of water, H=0.20 m

For rectangular portion, C_{d1}=0.62

From ΔABC ,we have

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

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

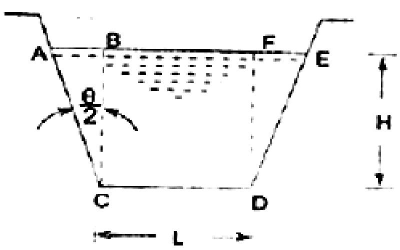


Fig. 2.12

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' ’hrough 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 3 res

C_d=Co-efficient of discharge for all notches

Total discharge $Q=Q_1+Q_2+Q_3$

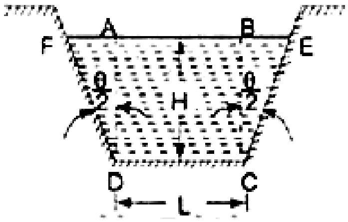
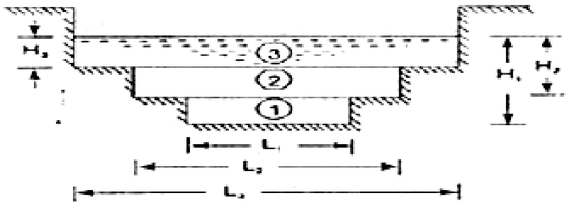


Fig. 2.12

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



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

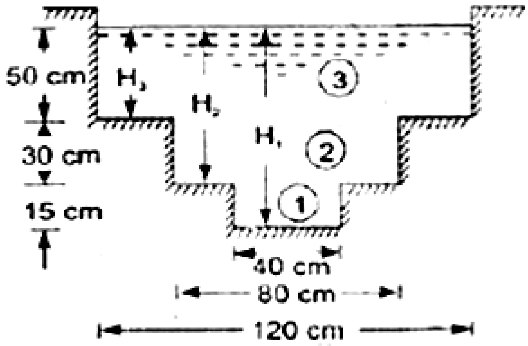
H_1 = 50 + 30 + 15 = 95 crn,

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

∴ $Q=Q_1+Q_2+Q_3$

=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,' ’hen 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 -d–scharge-by the cross-sectional area of the channel .on the' ’pstream 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 .

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

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 :-

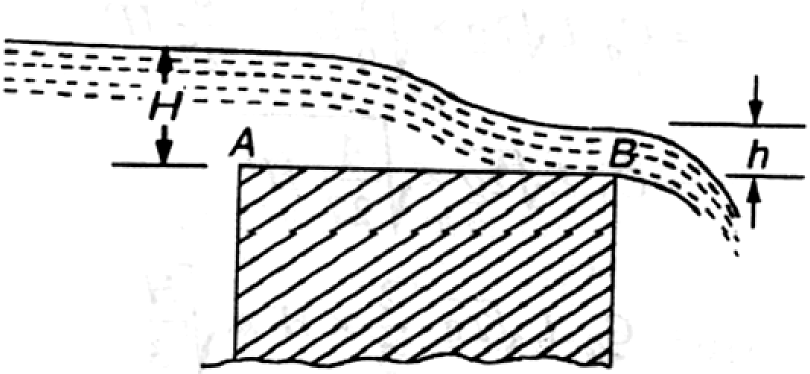


Fig. 2.15

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.

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

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 -l–ne 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.,

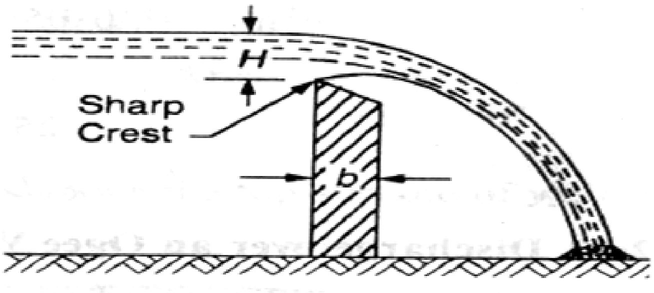


Fig. 2.16

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,

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

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

$$= 0.0073\ m^3/s = 7.3\ \text{litres/s.}$$ **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.,

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

Where C_d = Co-efficient of discharge and
L= Length of an ogee weir

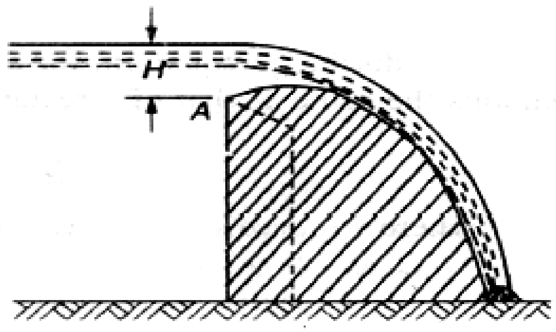


Fig. 2.17

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,

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

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

$= 7.323 \times 0.354 = 2.59 \text{ m}^3/\text{s} = 2590$ 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.

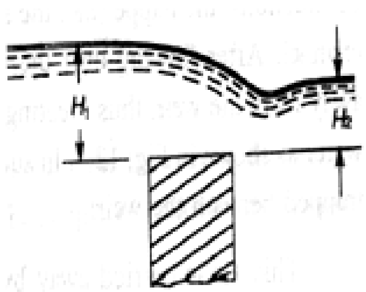


Fig. 2.18

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 - –.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 x 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 :-

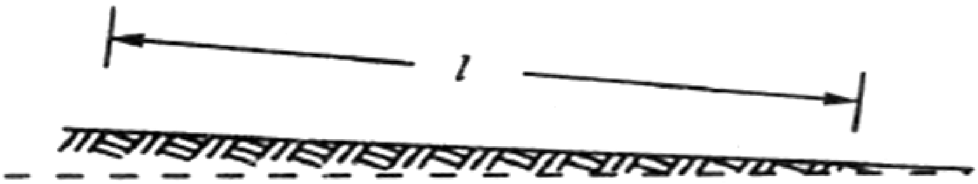


Fig. 2.19

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$\sqrt{\square}$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$\sqrt{\square}$mi = 9x50$\sqrt{\square}$ (1 X 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 $\sqrt{\square}$mi=M X $m^{2/3}$ X $i^{1/2}$

where

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

Now the discharge,

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

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

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

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

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

$=1.945\ m^3/s$ 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

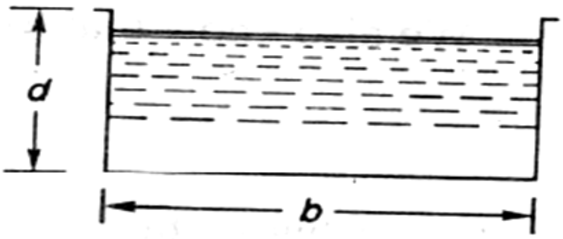


Fig. 2.20

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 $\sqrt{mi}$= 8 x 55$\sqrt{1 \text{ X } 0.001}$ 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.

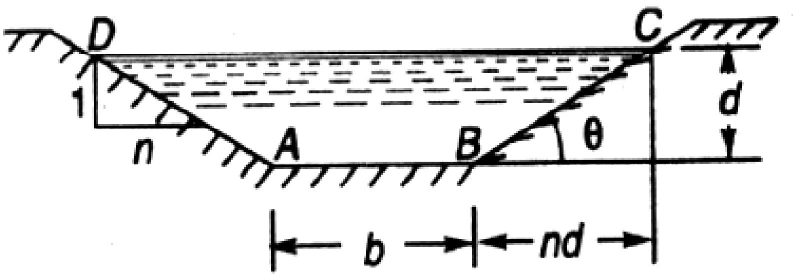


Fig. 2.21

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$d^{5/2}$

∴ $d^{5/2}$ =3/1.866 = 1.608

Or d = 1.21 m

∴ 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 'o' 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 = 2$r\,\theta$ ..I)

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

A = $r^2\,\theta$ - $\frac{r2 \sin 2\theta}{2}$ = r^2 (θ - $\frac{\sin 2\theta}{2}$) …(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** = $r^2 \left(\theta - \frac{\sin 2\theta}{2}\right)$ = $171 m^2$

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

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

And velocity of water v= $M X m^{2/3} X i^{1/2}$ = 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.

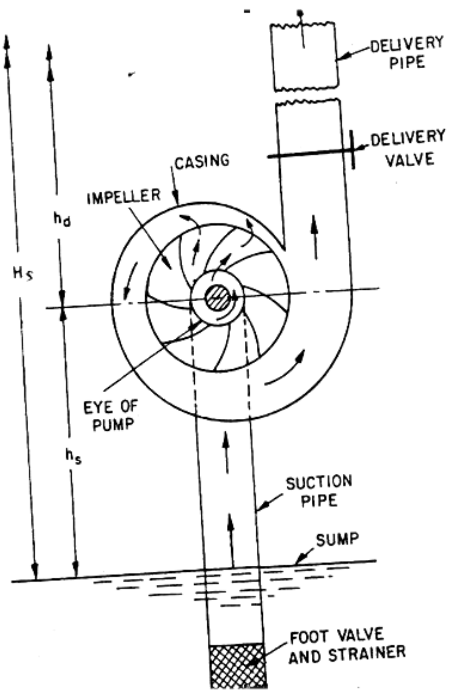
2. **Casing:** The casing of a centrifugal pump is similar to the casing of a reaction turbine. It is an air-tight passage surrounding the impeller & is designed in such a way that the kinetic energy of the water discharged at the outlet of the impeller is converted into pressure energy before the water 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.



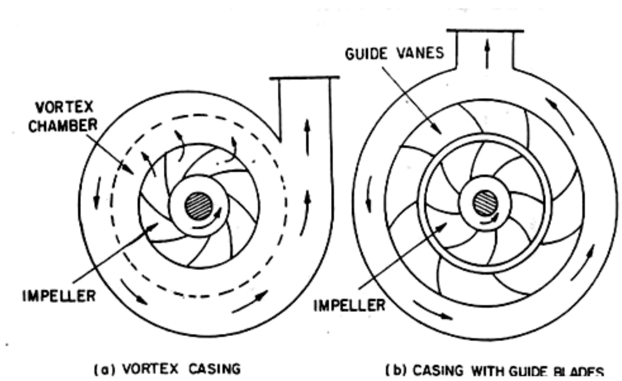
Main parts of a centrifugal pump

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 type of casing

Fig: 3.2

4.livery 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

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

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

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

η_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

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

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

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,

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

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, $u_2 = \dfrac{\pi D_2 N}{60} = \dfrac{\pi\times.30\times1000}{60} = 15.70m/s$

Now using equation

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

$$0.95 = \frac{9.81\times14.5}{V_{w2}\times15.70}$$

$$V_{w2} = \frac{0.95\times14.5}{0.95\times15.70} = 9.54m/s$$

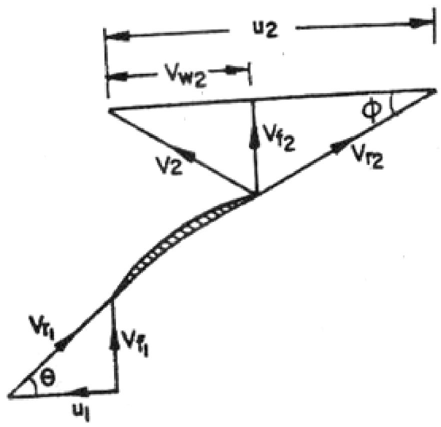


Fig. 3.3

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

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

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

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

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

$$= \pi\times0.30\times0.05\times3.556m^3/s = 0.1675m^3/s$$

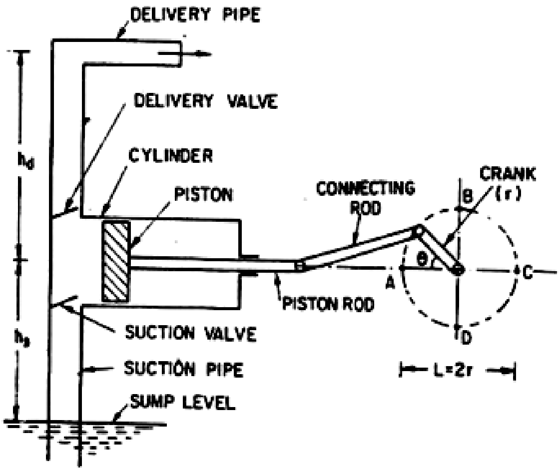
3.2 Reciprocating Pump:-

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

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

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 $(h_s + h_d)$= Total height through which water is lifted

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

Substituting the value of W in equation () we get

Work done per second =

$$\frac{\rho g ALN}{60}(h_s + h_d) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots$$

Power required to drive the pump, in kW $P = \frac{Work\ done\ per\ second}{1000}$ = $\frac{\rho \times g \times ALN(h_s + h_d)}{60 \times 1000}$

$$= \frac{\rho g ALN(h_s + h_d)}{60{,}000} kW \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots$$

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:

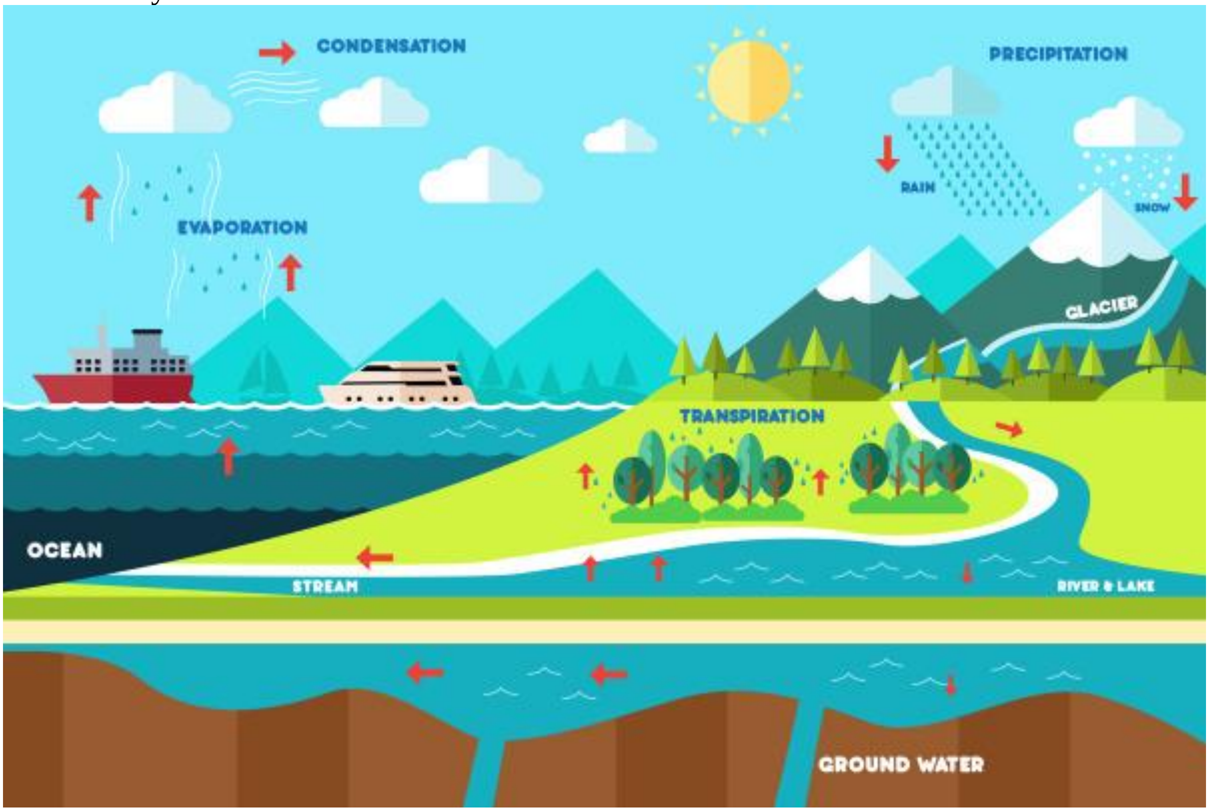
I. Single cylinder pump
II. Double cylinder pump
III. Triple cylinder pump

Chapter-IV

HYDROLOGY

Hydrological cycle

Hydrological cycle is also known as the “water cycle”; it is the normal water recycling system on Earth . Due to solar radiation, water evaporates, generally from the sea, lakes, etc. Water also evaporates from plant leaves through the mechanism of *transpiration*. As the steam rises in the atmosphere, it is being cooled, condensed, and returned to the land and the sea as precipitation. Precipitation falls on the earth as surface water and shapes the surface, creating thus streams of water that result in lakes and rivers. A part of the water precipitating penetrates the ground and moves downward through the incisions, forming aquifers. Finally, a part of the surface and underground water leads to sea. During this trip, water is converted in all phases: gas, liquid, and solid. As mentioned above, water always changes states between liquid, vapor and ice, with these processes happening in the blink of an eye and over millions of years.



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

(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 and hail.

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/hr

Heavy 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/cm3 and it is usual to assume an average density of 0.1 g /cm3 . 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 00C, 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 increases outwards. 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.

(cIticyclones: These are regions of high pressure, usually of large areal extent. The weather is 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.

(d) Convective precipitation: In this type of precipitation, a packet of air which is warmer than the surrounding 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.

(e) 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.

MEASUREMENT OF PRECIPITATION

1)infall Precipitation is expressed in terms of the depth to which rainfall water would stand on an area if all the rain were collected on it. Thus, 1cm of rainfall over a catchment area of 1km represents a volume of water equal to 100 cu m . 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 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 cm2 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.

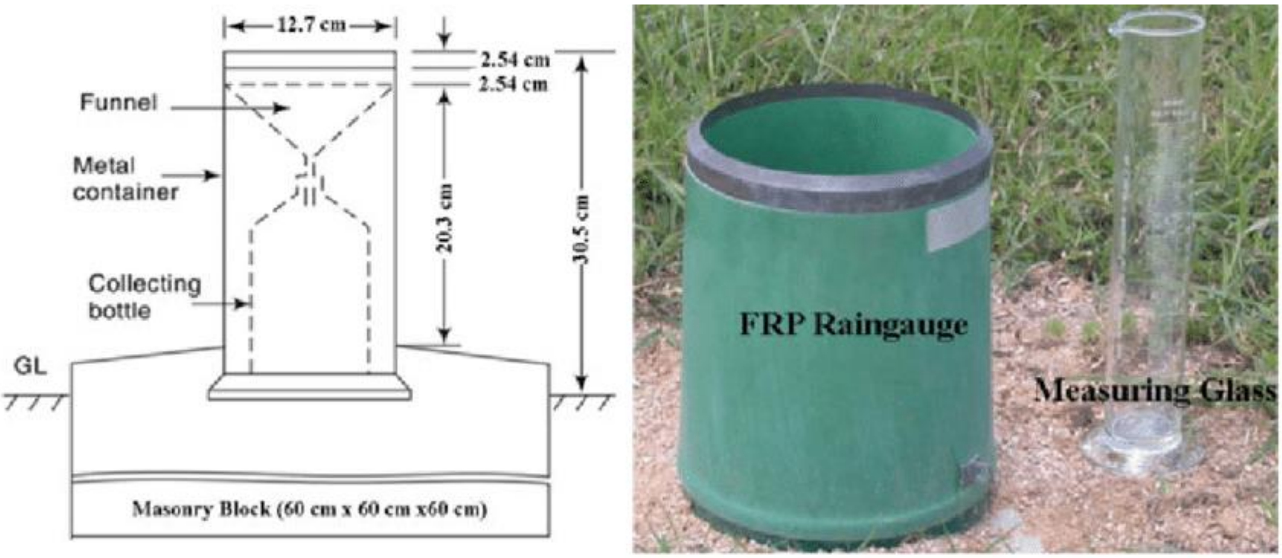


Fig- Symon's raingauge

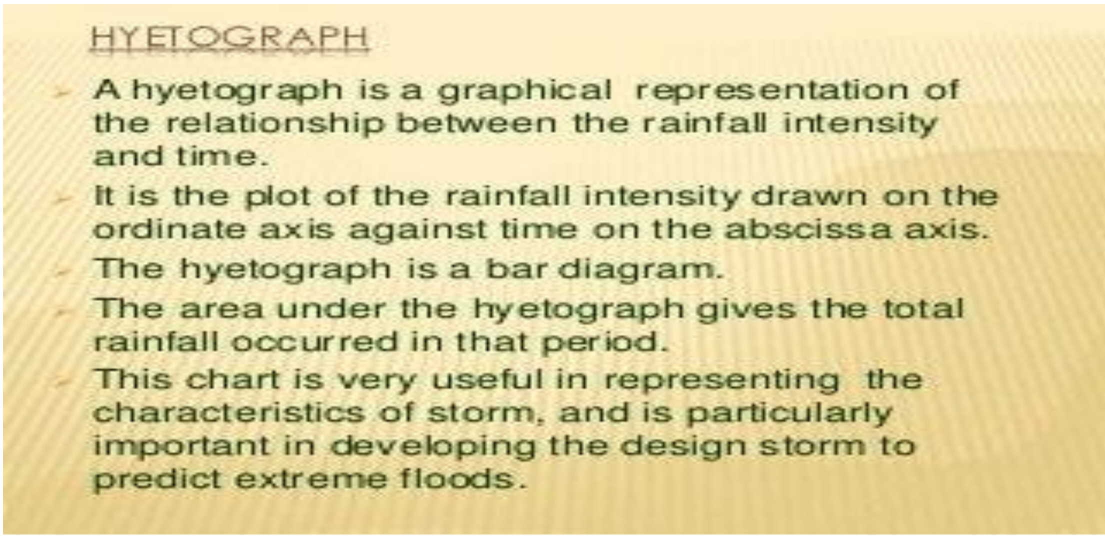
B. Recording Gauges Recording gauges produce a continuous plot of rainfall against time and provide valuable data of intensity and duration of rainfall for hydrological analysis of storms. The following are some of the commonly used recording rain-gauges.

(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 are so balanced that when 0.25 mm of rainfall collects in one bucket, it tips and brings the other one in position. The water from the tipped bucket is collected in a storage can. The tipping actuates an electrically driven pen to trace a record on the clockwork-driven chart. The water collected in the storage can is measured at regular intervals to provide the total rainfall and also serve as a check. It may be noted 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.

(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).



RUNOFF

It means the draining/flowing off of precipitation from catchment area through a surface channel. Otherwise it represents the output from the catchment in a basin unit of time.

BASIN: ¬ Area bounded by the highest contour called ridge line from where precipitated water is collected by surface and subsurface flows & drained out through the natural river.

The ridge line divides one basin from the other basin/catchment/watershed/drainage basin. Watershed discharge ‘Q’ can be related to the area ‘A’ as

$Q=x.A^y$

Where x, y = parameters (Depending on this values Q = peak flow min. or mean flow.

STREAM:- It is a natural flow channel in which water from a basin is collected and drained out to the water body.

OVERLOAD FLOW & SURFACE RUNOFF- After meeting all the losses, the excess rain water flows over the land surface in the form of a sheet of water to join the nearest stream and is called over land flow. The surface runoff is considered as overland flow so long as it does not join the nearest stream.

Lof = Length of overland flow

Ds = Stream density

SURFACE RUNOFF

The flow where it travels all the time over the surface is overland flow and through the channels as open channel flow and reaches the catchment outlet is called surface runoff.

SUBSURFACE RUNOFF

Otherwise known as interflow/ subsurface flow/ through flow/Storm seepage/ quick return flow. ϖThe part of precipitation which infiltrates into the ground and moves laterally/ horizontally in the soil and returns to the surface at some location away from the point of entry into the soil is called as interflow. Depending upon the time delay between the infiltration and outflow, Prompt (with least time log)

GROUND WATER FLOW

Another route for the infiltrated water is to undergo deep resolution and reach the ground water storage in the soil.

The part of runoff is called ground water runoff/ flow.

STREAM FLOW The total runoff consisting of surface flow, subsurface flow, ground water or base flow & the precipitation falling directly in the stream is the stream flow/ total runoff of a basin. Effluent stream (When the ground water table is higher than the water level of stream, then the stream receives water from ground water reservoir)

Influent stream (When the position of ground water table is lower than the water level of stream, stream water from the stream contributes to the ground water storage. e.G. In early part of rainy season all rivers of India.

Based on the time delay between the precipitation and the runoff, runoff are of two types Direct Runoff & Base flow

DIRECT RUNOFF

Direct/storm runoff is that part of stream flow occurring promptly as precipitation starts & contributes for an acceptable period after the storm ceases. Contribution from subsurface flow is considered constant during the period.

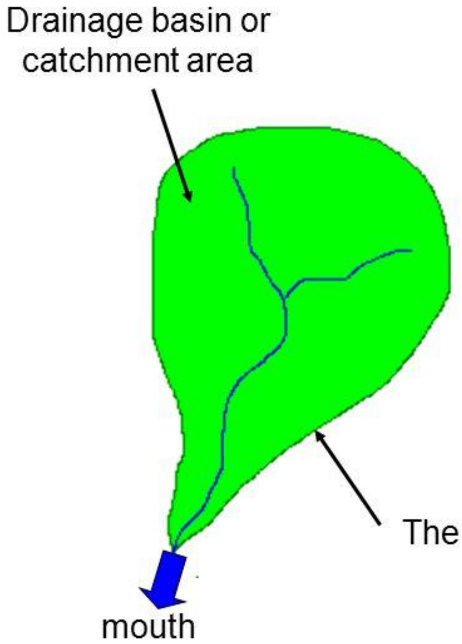
BASE FLOW

It is that part of stream flow available mainly from ground water reservoir and delayed sub-surface flow appearing during dry period. Direct runoff and base flow are distinguished by mainly on time of arrival of flow to the catchment.

CONCEPT OF CATCHMENT AREA

The word catchment (aka watershed) in hydrology is spoken with respect to a river/stream. It denotes all that area that drains into the river/stream under consideration. Any precipitation that falls on the catchment region of a river will flow down the land to join the river to contribute to its volume either through surface runoff or through baseflow (underground water flow). A catchment’s boundary is denoted by the ridge line

The drainage basin



An area of land (also called the *catchment area*) drained by a river and its tributaries. The boundary which divides one drainage basin from another Is called the watershed

EMPIRICAL METHODS

1) Dicken's formula :

The formula was developed for the north and central India.

$$Q = C\,A^{\frac{3}{4}}$$

Q = Maximum flood discharge

A = Area of the catchment

C = Dicken's coefficient

Ryve's formula

$$Q=CA^{2/3}$$

Where Q= flood discharge in cumec

A= catchment area of basin in sq.m

C= flood coefficient

constant C depends upon the catchment and may be obtained from table-

Location of catchment	C
Area within 24 km from the coast	6.75
Area within 24 km to 161km from the coast	8.45
Limited area near hills	10.1

CHAPTER V

WATER REQUIREMENT OF CROPS

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 inadequaterainfall.

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 conveythe 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 atend, 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 whenthere may be no rainfall.

Commercial crops with additional water:

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

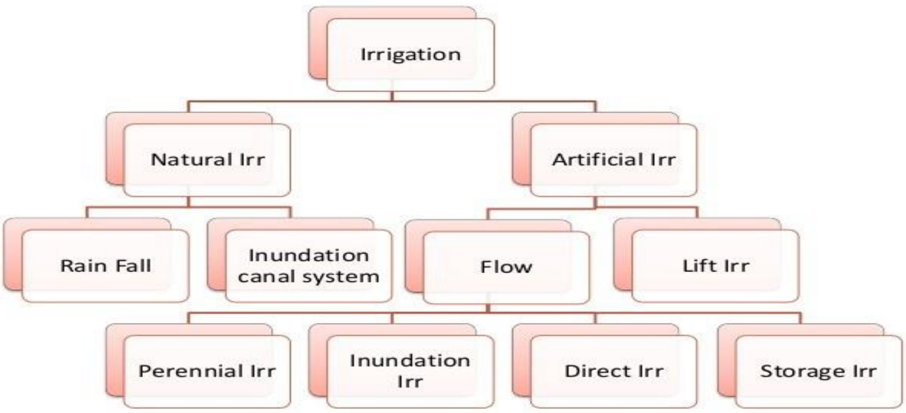
Controlled water supply:

- 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 through storage canal head works & Canal distribution system.

b) Inundation irrigation:

- Lands are submerged & th horoughlylooded 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.
6) **Cash crop** – which has to be encased in the market. As it cannot be consumed directly by the cultivators.

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

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 require 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 :

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

Delta:

- 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 the canal.
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 iscalled 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.

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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 the field 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 minimum and hence reduced transmission losses.

(5) Quality of water:

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

(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, moisture availability.

  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 fully covering 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.

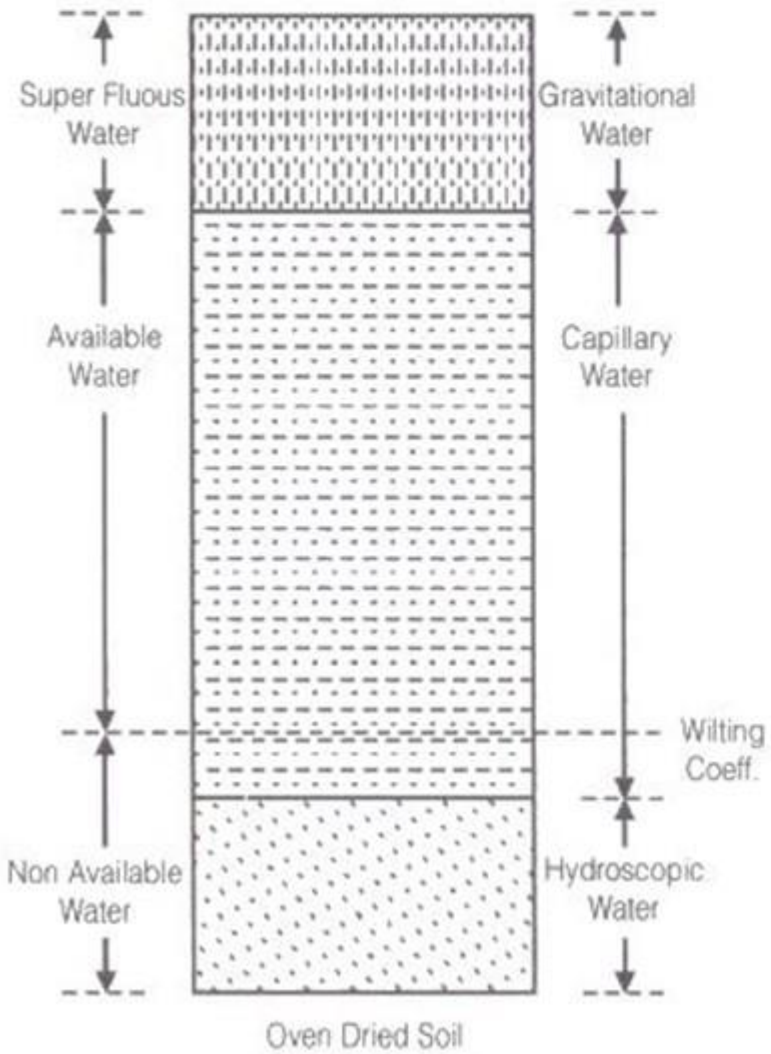
Crop Water Requirements

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.

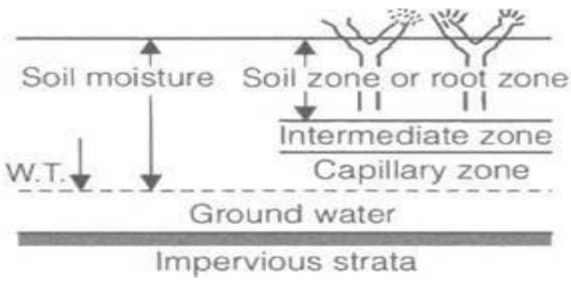
Capillary water

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.

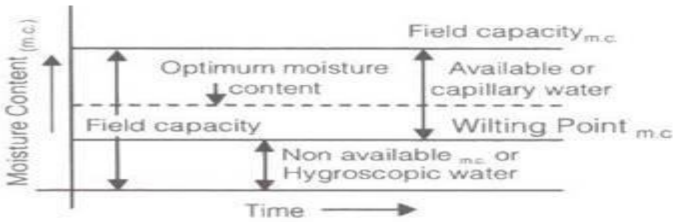
Gravitational water

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 air in the void spaces.

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

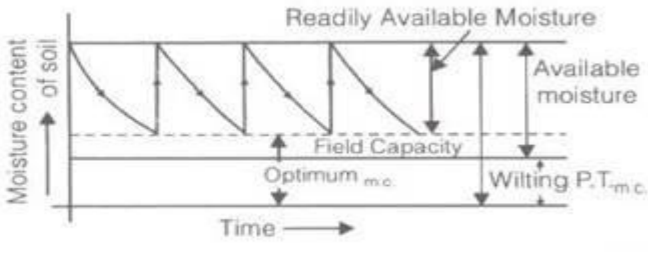


(2) Readily available moisture for the plant = FC - –o



Here FC= field capacity

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



Mo= Readily available moisture content

(3) Frequency of Irrigation $= \dfrac{weight\ /\ readily\ available\ moisture\ depth}{consumptive\ use\ rate}$

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

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.

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

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

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

(8) $F_C = n / G$ 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

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

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) Field irrigation requirement (FIR)

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

(4) Gross irrigation requirement, (GIR)

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

CHAPTER VI

FLOW IRRIGATION

Open canals

An open canal, channel, or ditch, is an open waterway whose purpose is to carry water from one place to another. Channels and canals refer to main waterways supplying water to one or more farms. Field ditches have smaller dimensions and convey water from the farm entrance to the irrigated fields.

i.Canal characteristics

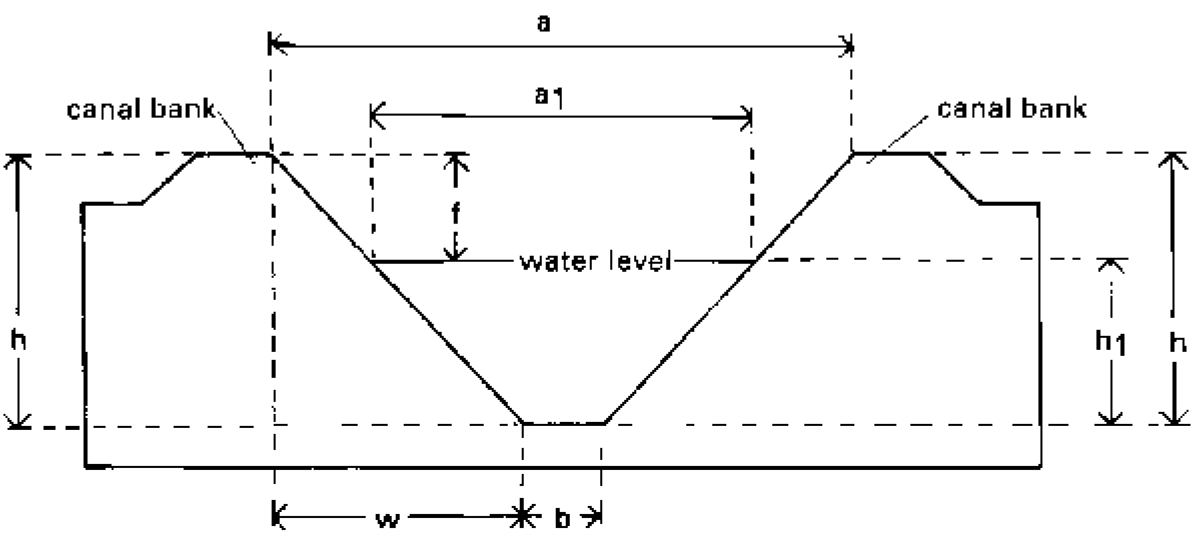
According to the shape of their cross-section, canals are called rectangular (a), triangular (b), trapezoidal (c), circular (d), parabolic (e), and irregular or natural

Some examples of canal cross-sections

The most commonly used canal cross-section in irrigation and drainage, is the trapezoidal cross-section. For the purposes of this publication, only this type of canal will be considered.

The typical cross-section of a trapezoidal canal is shown in Figure

Fig. A trapezoidal canal cross-section

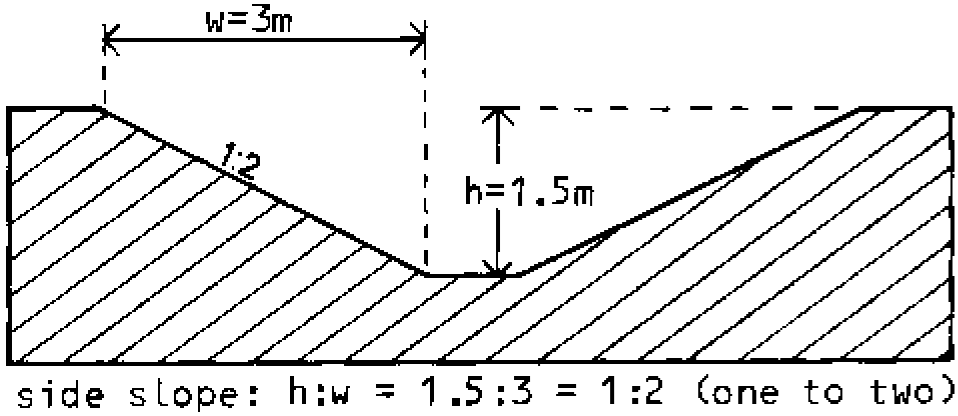


a = top width of the canal
a_1 = top width of the water level
h = height of the canal
h_1 = heigth or depth of the water in the canal
b = bottom width of the canal
h:w = side slope of the canal
f = free board (= $h-h_1$)

The freeboard of the canal is the height of the bank above the highest water level anticipated. It is required to guard against overtopping by waves or unexpected rises in the water level.

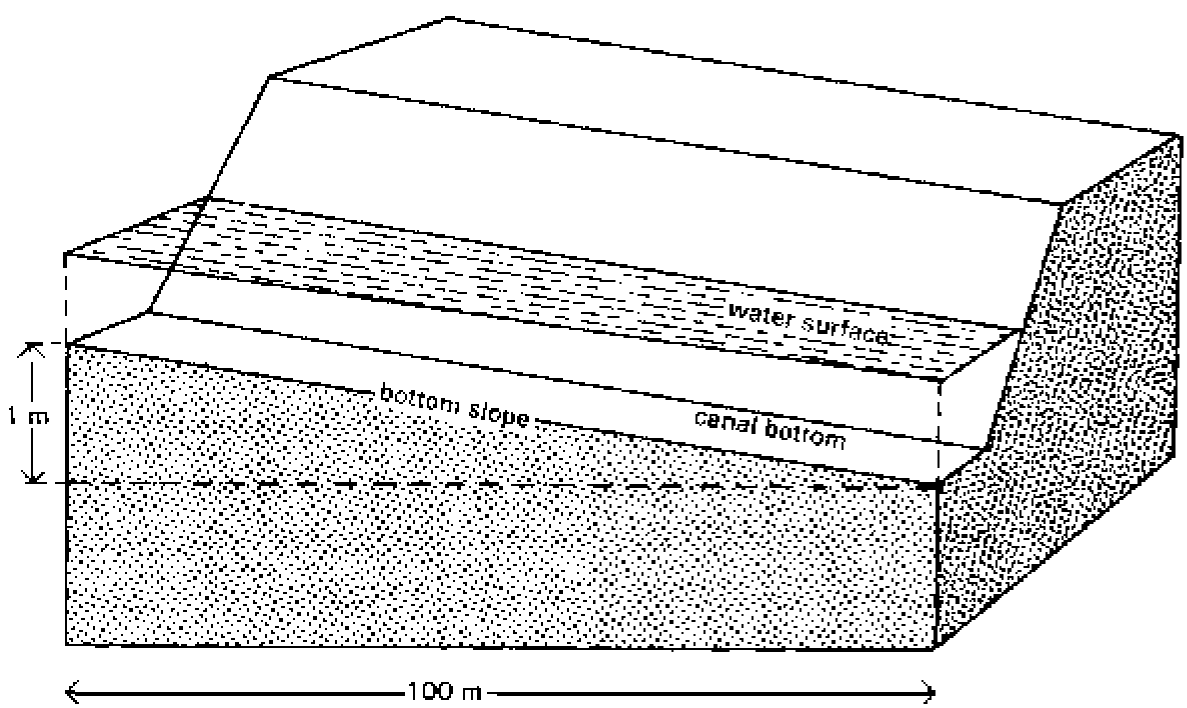
The side slope of the canal is expressed as ratio, namely the vertical distance or height to the horizontal distance or width. For example, if the side slope of the canal has a ratio of 1:2 (one to two), this means that the horizontal distance (w) is two times the vertical distance .

Fig. A side slope of 1:2 (one to two)



The bottom slope of the canal does not appear on the drawing of the cross-section but on the longitudinal section . It is commonly expressed in percent or per mil.

Fig A bottom slope of a canal



An example of the calculation of the bottom slope of a canal is given below

$$\text{the bottom slope } (\%) = \frac{\text{height difference (metres)}}{\text{horizontal distance (metres)}} \times 100 = \frac{1\text{ m}}{100\text{ m}} \times 100 = 1\%$$

or

$$\text{the bottom slope } (‰) = \frac{\text{height difference (metres)}}{\text{horizontal distance (metres)}} \times 1000 = \frac{1\text{ m}}{100\text{ m}} \times 1000 = 10‰$$

ii. Earthen Canals

Earthen canals are simply dug in the ground and the bank is made up from the removed earth.

Construction of an earthen canal

The disadvantages of earthen canals are the risk of the side slopes collapsing and the water loss due to seepage. They also require continuous maintenance in order to control weed growth and to repair damage done by livestock and rodents.

Maintenance of an earthen canal

iii. Lined Canals

Earthen canals can be lined with impermeable materials to prevent excessive seepage and growth of weeds .

Construction of a canal lined with bricks

Lining canals is also an effective way to control canal bottom and bank erosion. The materials mostly used for canal lining are concrete (in precast slabs or cast in place), brick or rock masonry and asphaltic concrete (a mixture of sand, gravel and asphalt).

The construction cost is much higher than for earthen canals. Maintenance is reduced for lined canals, but skilled labour is required.

5.2.2 Canal structures

The flow of irrigation water in the canals must always be under control. For this purpose, canal structures are required. They help regulate the flow and deliver the correct amount of water to the different branches of the system and onward to the irrigated fields.

There are four main types of structures: erosion control structures, distribution control structures, crossing structures and water measurement structures.

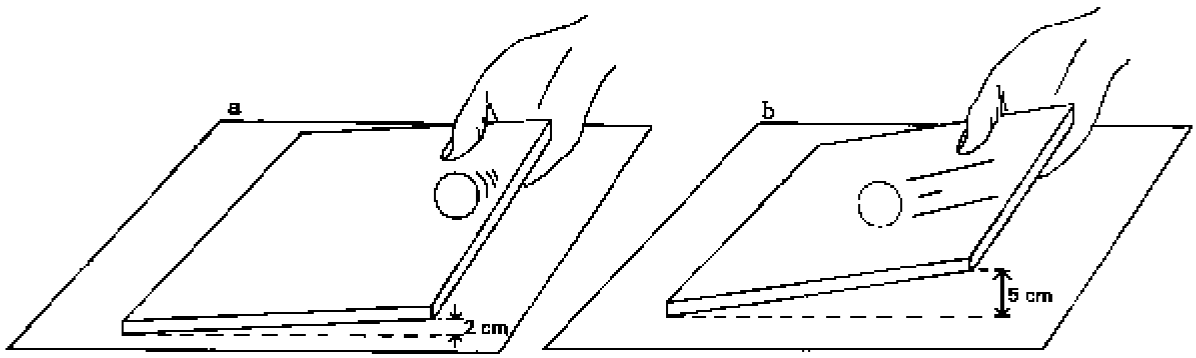
i. Erosion control structures

a. Canal erosion

Canal bottom slope and water velocity are closely related, as the following example will show.

A cardboard sheet is lifted on one side 2 cm from the ground . A small ball is placed at the edge of the lifted side of the sheet. It starts rolling downward, following the slope direction. The sheet edge is now lifted 5 cm from the ground , creating a steeper slope. The same ball placed on the top edge of the sheet rolls downward, but this time much faster. The steeper the slope, the higher the velocity of the ball.

The relationship between slope and velocity



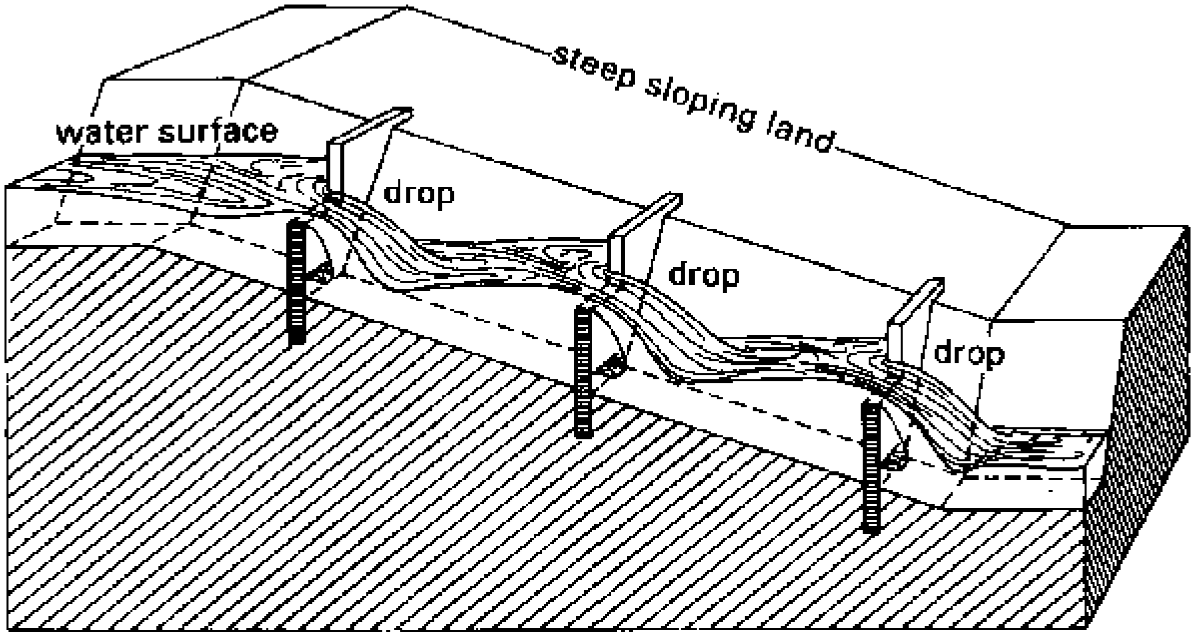
Water poured on the top edge of the sheet reacts exactly the same as the ball. It flows downward and the steeper the slope, the higher the velocity of the flow.

Water flowing in steep canals can reach very high velocities. Soil particles along the bottom and banks of an earthen canal are then lifted, carried away by the water flow, and deposited downstream where they may block the canal and silt up structures. The canal is said to be under erosion; the banks might eventually collapse.

b. Drop structures and chutes

Drop structures or chutes are required to reduce the bottom slope of canals lying on steeply sloping land in order to avoid high velocity of the flow and risk of erosion. These structures permit the canal to be constructed as a series of relatively flat sections, each at a different elevation .

Longitudinal section of a series of drop structures



Drop structures take the water abruptly from a higher section of the canal to a lower one. In a chute, the water does not drop freely but is carried through a steep, lined canal section. Chutes are used where there are big differences in the elevation of the canal.

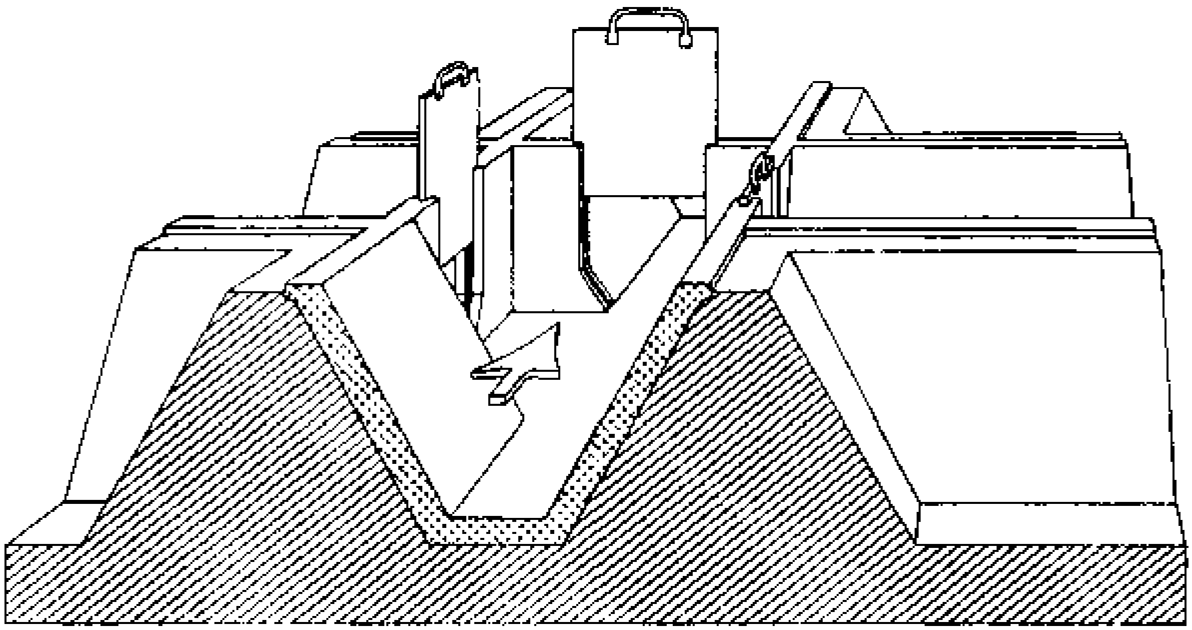
ii. Distribution control structures

Distribution control structures are required for easy and accurate water distribution within the irrigation system and on the farm.

a.Division boxes

Division boxes are used to divide or direct the flow of water between two or more canals or ditches. Water enters the box through an opening on one side and flows out through openings on the other sides. These openings are equipped with gates .

A division box with three gates

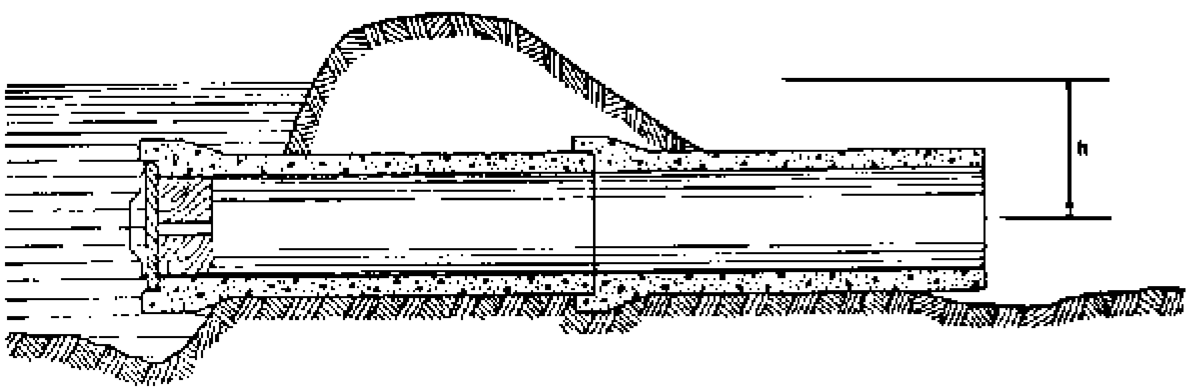


b. Turnouts

Turnouts are constructed in the bank of a canal. They divert part of the water from the canal to a smaller one.

Turnouts can be concrete structures , or pipe structures .

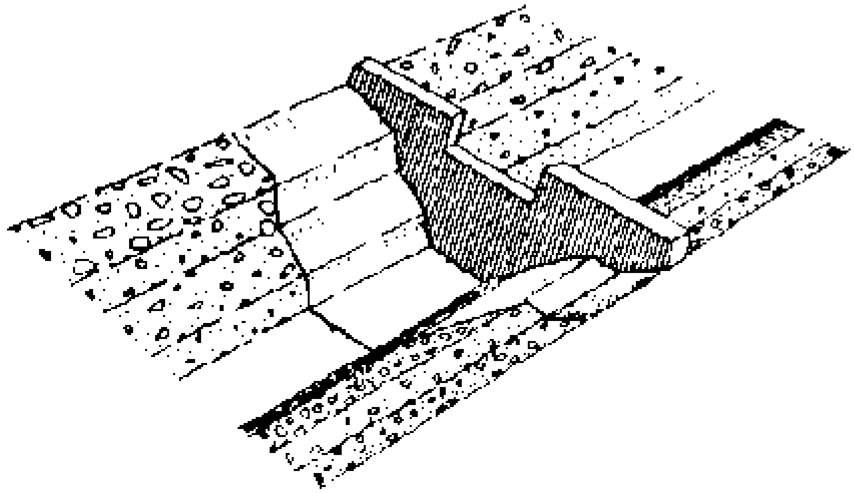
. A pipe turnout



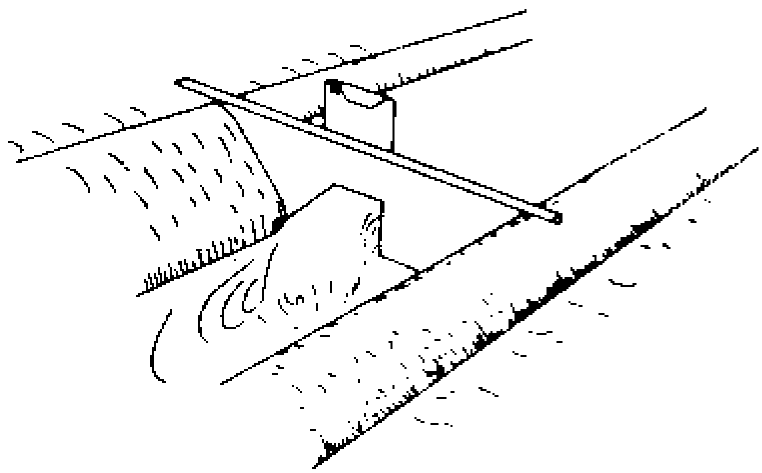
c.Checks

To divert water from the field ditch to the field, it is often necessary to raise the water level in the ditch. Checks are structures placed across the ditch to block it temporarily and to raise the upstream water level. Checks can be permanent structures or portable .

. A permanent concrete check



. A portable metal check



iii. Crossing structures

It is often necessary to carry irrigation water across roads, hillsides and natural depressions. Crossing structures, such as flumes, culverts and inverted siphons, are then required.

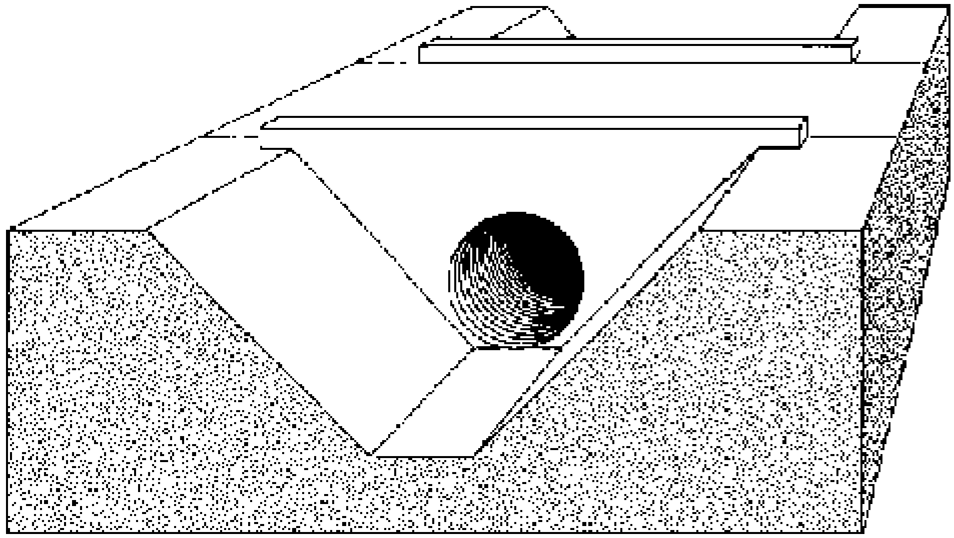
a.Flumes

Flumes are used to carry irrigation water across gullies, ravines or other natural depressions. They are open canals made of wood (bamboo), metal or concrete which often need to be supported by pillars .

b. Culverts

Culverts are used to carry the water across roads. The structure consists of masonry or concrete headwalls at the inlet and outlet connected by a buried pipeline .

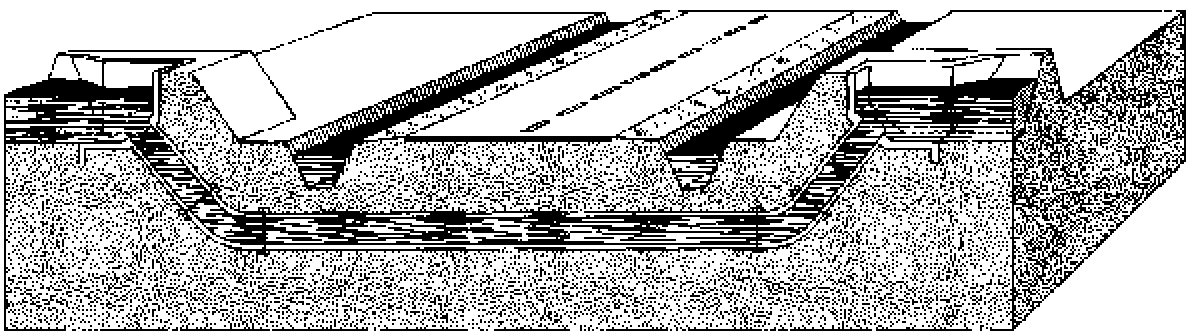
A culvert



c.inverted siphons

When water has to be carried across a road which is at the same level as or below the canal bottom, an inverted siphon is used instead of a culvert. The structure consists of an inlet and outlet connected by a pipeline . Inverted siphons are also used to carry water across wide depressions.

. An inverted siphon



iv. Water measurement structures

The principal objective of measuring irrigation water is to permit efficient distribution and application. By measuring the flow of water, a farmer knows how much water is applied during each irrigation.

In irrigation schemes where water costs are charged to the farmer, water measurement provides a basis for estimating water charges.

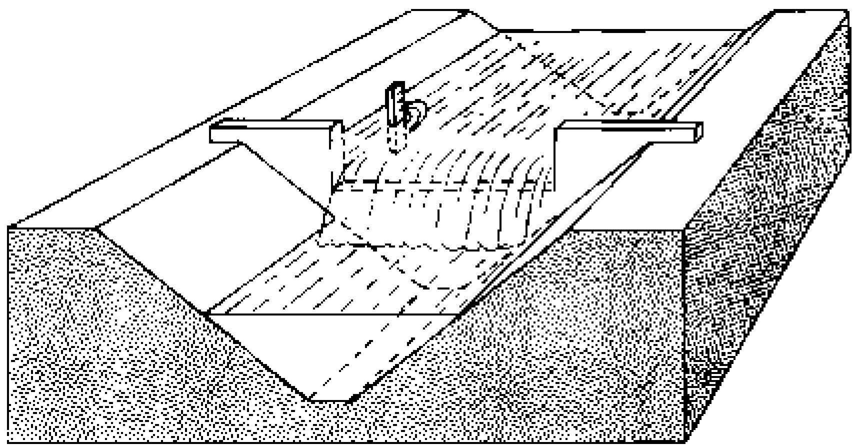
The most commonly used water measuring structures are weirs and flumes. In these structures, the water depth is read on a scale which is part of the structure. Using this reading, the flow-rate is then computed from standard formulas or obtained from standard tables prepared specially for the structure.

a.Weirs

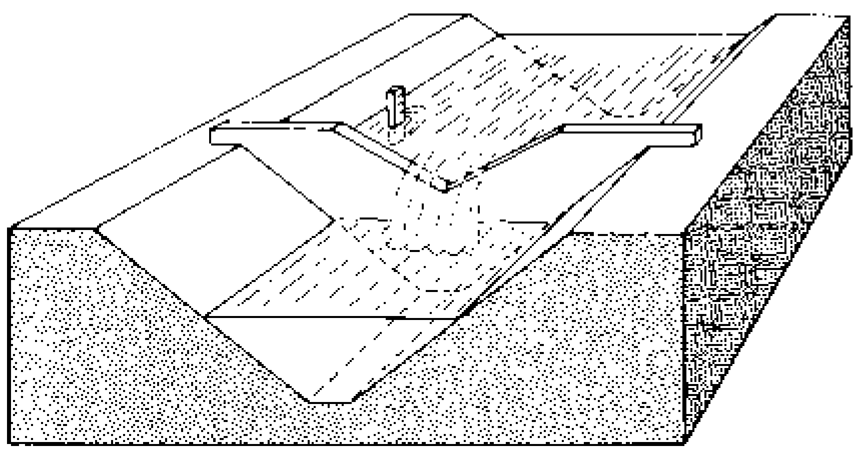
In its simplest form, a weir consists of a wall of timber, metal or concrete with an opening with fixed dimensions cut in its edge . The opening, called a notch, may be rectangular, trapezoidal or triangular.

. Some examples of weirs

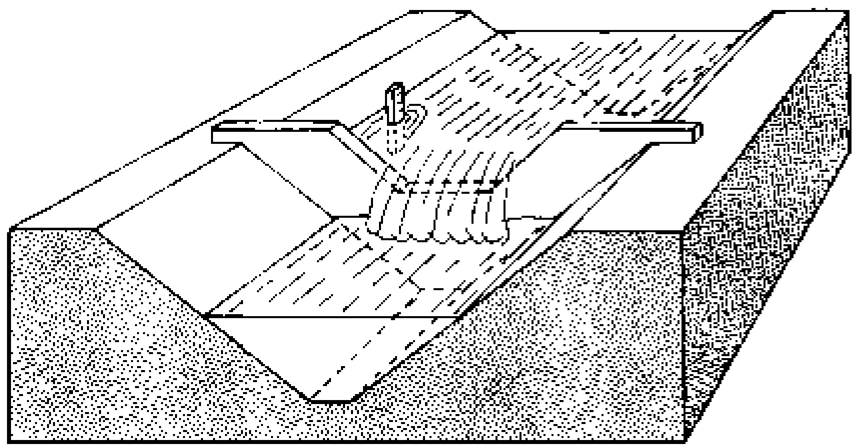
A RECTANGULAR WEIR



A TRIANGULAR WEIR



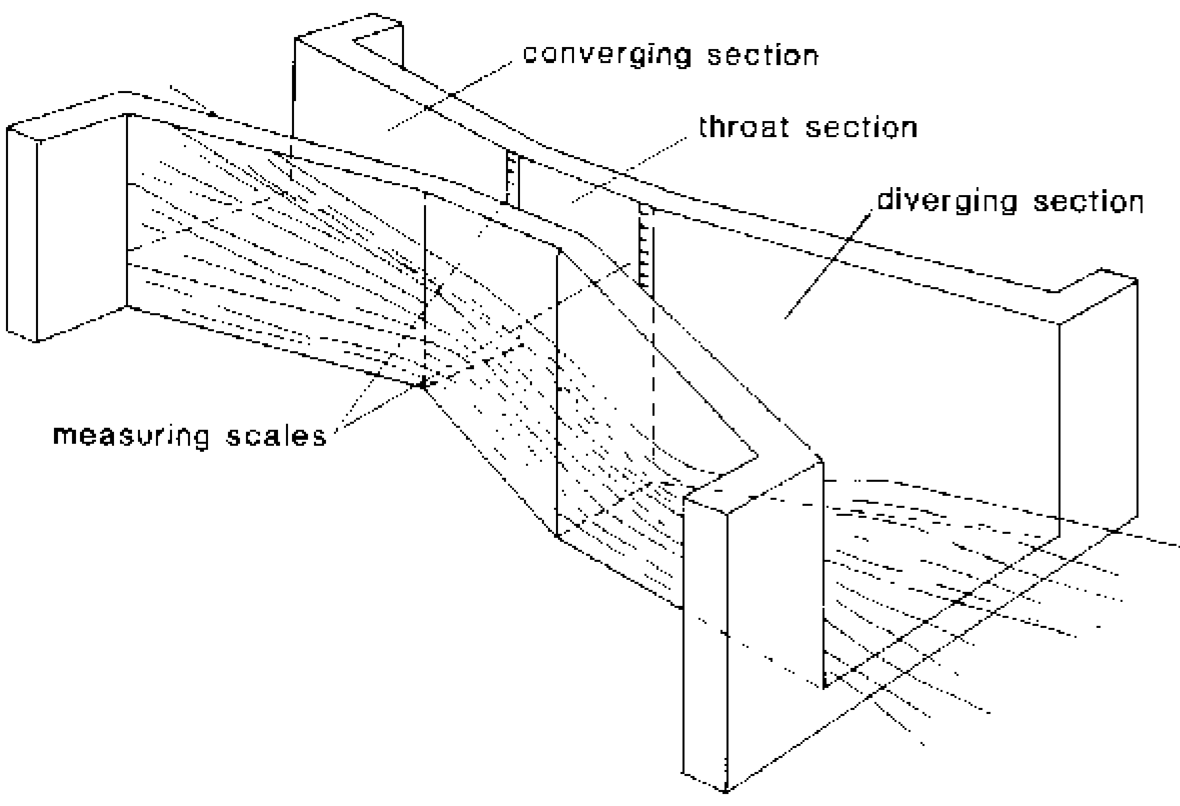
A TRAPEZOIDAL WEIR



b. Parshall flumes

The Parshall flume consists of a metal or concrete channel structure with three main sections: (1) a converging section at the upstream end, leading to (2) a constricted or throat section and (3) a diverging section at the downstream end .

. A Parshall flume

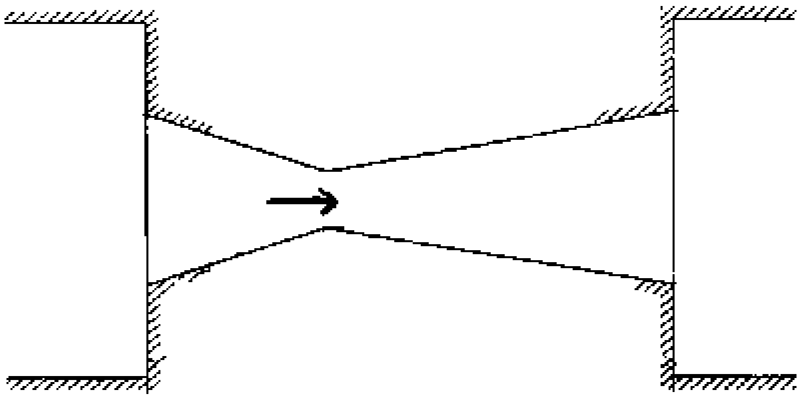


Depending on the flow condition (free flow or submerged flow), the water depth readings are taken on one scale only (the upstream one) or on both scales simultaneously.

c.cut-throat flume

The cut-throat flume is similar to the Parshall flume, but has no throat section, only converging and diverging sections . Unlike the Parshall flume, the cut-throat flume has a flat bottom. Because it is easier to construct and install, the cut-throat flume is often preferred to the Parshall flume.

. A cut-throat flume



Losses of Water in Canals (Canal Losses)

Losses of water in canals may be defined as the process in which a considerable amount of water is lost as it passes through the canal from the starting of the headwork to the final point where it is supplied to the agricultural field or elsewhere.

The loss of water in canals is also commonly referred to as **transit loss or transmission loss.**

The major losses in the canals result from evaporation, seepage, and transpiration.

According to the statistical data, the losses of water in canals can be as high as 20-25%. Such losses of water constitute a major part of the useable water. Hence, the losses of water in canals must be thoroughly studied and analyzed and must be considered during the design of the canal capacity.

Types of Losses of Water in Canals

The major types of canal losses are listed as follows:

1. Evaporation Losses in Canal

2. Seepage Losses in Canal

3. Transpiration Losses in Canal

Evaporation Losses in Canal

Evaporation loss in the canal is inevitable as the water flowing through any canal is exposed to the atmosphere.

According to the statistical records, evaporation losses constitute 0.25% to 1% of the total canal discharge and constitute 2 to 3% of the total water losses in the water canal.

Evaporation losses are less than the seepage losses in the canals.

The evaporation losses depend upon several factors such as temperature, humidity, wind, etc. But, the most dominant factor is temperature. On account of this, the evaporation losses are higher in summer than in winter.

However, the velocity of the wind also equally affects the rate of evaporation. Such losses are significant in the shallow depths of water.

Hence, this indicates that the evaporation losses in canals depend mostly upon the climatic conditions of the area. Such losses depend directly on the area of exposure of the surface of the water and inversely on the depth of water in the canal.

Thus, the major factors that affect the rate of losses due to evaporation in the canals can be listed as follows:

1. Temperature

2. Wind Velocity

3. Humidity

4. Area of water that is exposed to the atmosphere

b. Seepage Losses in Canal

The seepage loss in the canal is the most significant loss of water in the canals.

Such loss mainly depends upon the following factors:

1. The porosity of the soil.

2. Existing underground water table conditions.

3. The existing condition of the canal system.

4. Physical properties of the water canal such as turbidity of water.

5. Amount of silt carried by the water.

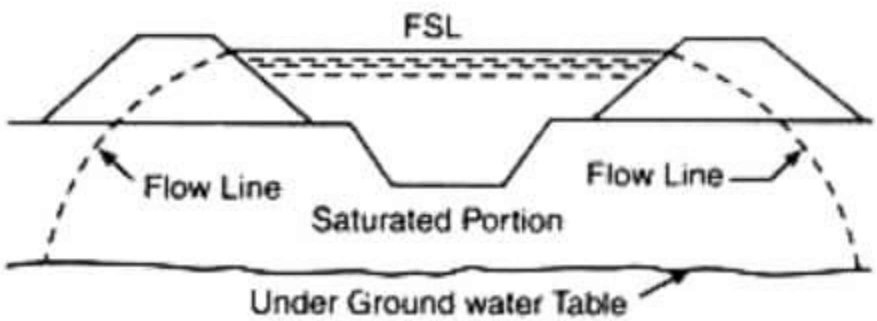
Types of seepage losses are:

Seepage Losses due to Percolation

It is the seepage loss in a canal in which the water is lost through a continuous zone that is formed between the canal and the water table. Such a zone that is formed consists of fully saturated soil that is capable of establishing continuity in the flow of water from the canal to the underground reservoir.

The loss of water due to percolation is depicted in the figure below.

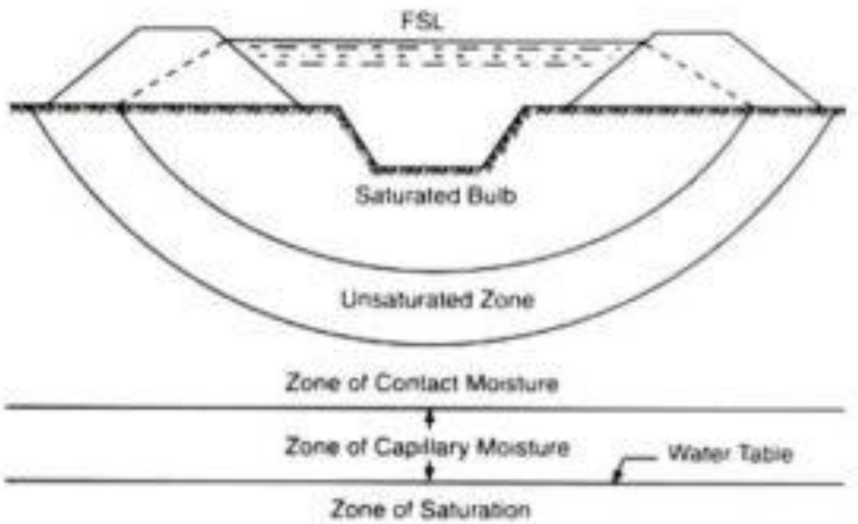
The loss of water through percolation is greatly affected by the difference in level between the topwater surface level of the channel to the water table level.



b. Seepage Loss due to Absorption

In the canals, usually, a zone of saturation is present below the canal that is accompanied by a zone of soil with decreasing saturation which is further bounded by a zone that is saturated by the capillary action of water rising from the adjacent water table level.

As shown in the figure, two zones of saturated soil bind the unsaturated zone of the soil.



Let H be the seepage head, h be the distance between the water surface level of the canal and the bottom of the saturated zone, h_c be the capillary head, then the rate of loss of water due to absorption depends upon $(h + h_c)$.

Methods Of Determining The Losses Due To Seepage

1. By the use of seepage meters.

2. By ponding method.

3. By inflow and outflow method.

Transpiration Losses in Canal

Some amount of water flowing through the canals is lost by the process of transpiration.

The plants, grasses, weeds, or other vegetation that grow on the banks of the canal undergo transpiration thereby resulting in loss of water from the canal.

Transpiration losses are very less in comparison to the other two losses in the canal.

Preventive Measures to Reduce Canal Losses

Some of the preventive measures to reduce canal losses are:

a.Canal Lining

Canal lining is the process of making the bed and sides of the canal impervious to reduce canal losses.

b. Increasing the height of the canal despite width. This reduces evaporation and percolation losses.

c. Provision of proper slope gradient to the canal to increase the velocity of the water.

d. Removal of vegetation from the canal and its sides.

Types of Canal based on Various Factors

There are 6 types of canal-based on various factors.

1. Based on the nature of the supply source

2. Based on functions

3. Based on the type of boundary surface soil

4. Based on the financial output

5. Based on discharge

6. Based on canal alignment

A brief explanation of these types of the canal is given below.

Based on the Nature of Supply Source

a.Permanent Canal:
It is a type of canal in which water is accessible throughout the year.

This type of canal is generally run from a permanent source of supply water.

Several Permanent hydraulic structures are built in this type of canal for water distribution and regulation.

A Permanent canal is also known as a **perennial canal.**

b. Inundation Canal:
It is a type of canal in which water is accessible only during the flood.

These types of canals are carried off from rivers to control the high water level in rivers during flood periods.

A head regulator is provided to regulate the flow of water into the canal.

2.Based on Functions of Canal

a.Irrigation canal:

The canal which is used to supply water to the cultivation field for agricultural purposes is called Irrigation Canal.

Fig: Irrigation Canal

b. Power canal:

The canal that is used for the generation of hydraulic power is called a power canal.

Hydraulic power is also called hydroelectricity.

Simply these canals are constructed in micro-hydropower projects.

Fig: Power Canal

c.feeder canal:
A feeder canal is build to supply two or more other canals or branch canals.

Fig: Feeder Canal

d.Carrier canal:
A carrier canal is a multi-purpose canal that consists of the features of both the irrigation canal and feeder canal.

It means that the carrier canal feeds the branch canal as well as provides water for direct irrigation.

Fig: Carrier Canal

e.Navigation canal:
A canal that is built mostly for navigational purposes is known as a navigation canal.

The water level & width required in a navigation canal is usually a lot higher to facilitate the navigation of large boats, ships, etc.

Fig: Navigation Canal

3. Based on Type of Boundary Surface of Canal:

a.Alluvial canal:
If the canal is constructed by digging in alluvial soils such as silt, sand, gravel, etc. then it is called an alluvial canal.

Fig: Alluvial Canal

b. Non-alluvial canal:
If the surface of the canal is non-alluvial soils such as loam, clay, rock, etc. then it is called a non-alluvial canal.

Fig: Non-alluvial Canal

c.Rigid Surface canal:

Rigid surface canals come under non-alluvial canals but the boundary surface of the canal is lined with a hard layer of lining material such as cement, concrete, stones, etc.

Fig: Rigid-Surface Canal

4.Based on Financial Output

a.Protective Canal:
Protective canals are constructed to save a particular area from the shortage of water.

The main aim of a protective canal is to fulfill the needs of cultivators during the time of famine.

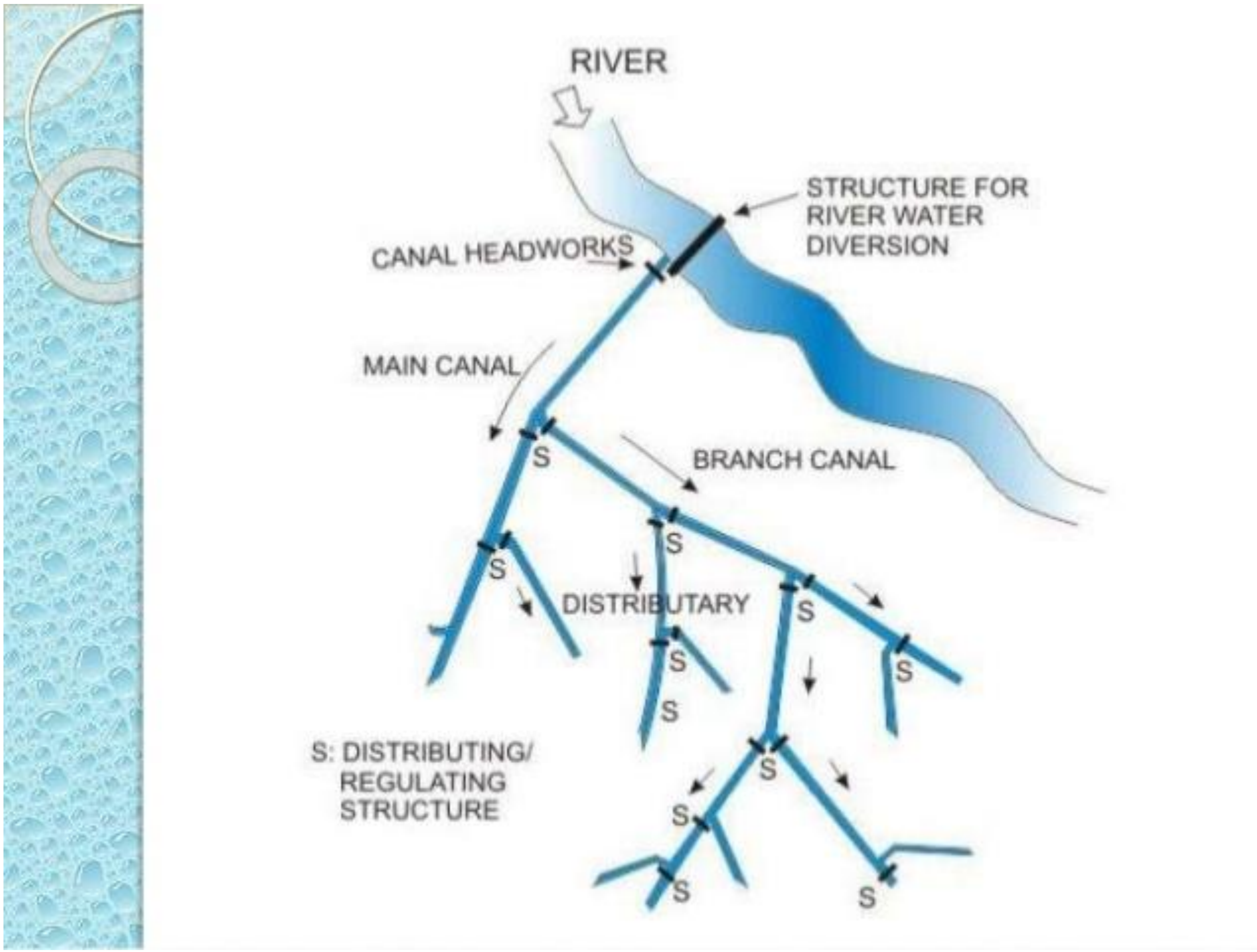
Fig: Protective Canal

b. Productive Canal
Productive canals are those which will produce more revenue for their maintenance and running costs and also to recover the initial investment done on the construction of the canal.

It is good if it recovers 6% or more of its initial investment per annum.

Fig: Productive Canal

5.Based on Discharge



a.Main canal:
Water in the main canal takes off directly from a river or **reservoir**.

Main Canal feeds the branch canals.

Due to the conveying of more discharge through the main canal, it is not suggested to do direct irrigation from it.

b. Branch Canal:
Water in the branch canal takes off from the main canals at regular intervals.

Branch canals supply water to major and minor distributary canals.

The discharge of the branch canal is usually over 5 m^3/sec.

Direct irrigation by branch canal is not suggested unless the discharge through it is not low.

c.Major Distributary Canal:
Water in Major Distributary Canal takes off from the branch canal or in few cases from the main canal.

This canal supply water to the minor distributaries and field channels.

A canal is called to be a major distributary when its discharge lies between 0.25 to 5 m^3/sec.

d.Minor Distributary Canal:
Water in Minor Distributary Canal takes off from major distributaries and directly from branch canals depending upon the discharge of canals.

Their discharge is usually below 0.25 m^3/sec. These canals provide water to the field channels.

e.Field Channels:
Field channels are small water channels that are excavated by cultivators in the irrigation field.

It is also called watercourses.

These channels are feed by the distributary canals and branch canals through canal outlets.

6.Based on Canal Alignment

a.Ridge Canal:
The canal that is aligned along the ridgeline or watershed line of an area is called the ridge canal or watershed canal.

Since this canal lies at a high altitude, irrigation on both sides of the canal is possible.

It is also possible to irrigate a larger area.

There is an absence of interception of natural drains on ridgelines due to which there is no requirement of cross drainage work.

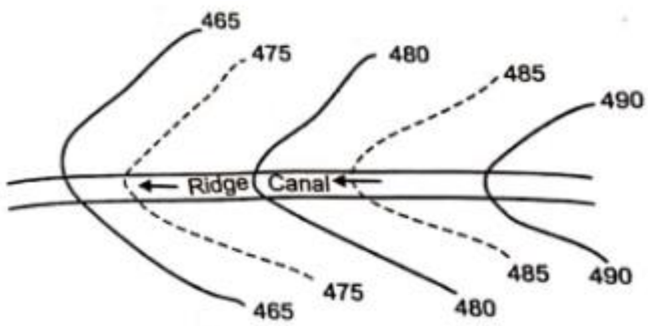


Fig: Ridge Canal

b. Contour Canal:
A canal that is aligned roughly parallel to the contours of the area is known as the contour canal.

This type of canal can be observed in hilly regions.

Irrigation is only possible in a single direction only. It has to pass the drainage for which there should be the provision of cross drainage works.

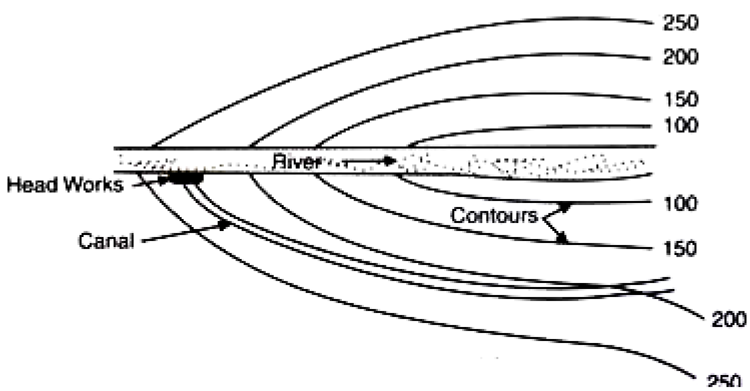


Fig: Contour Canal

c.Side-slope Canal:
A canal that is aligned nearly perpendicular to the contour of the area is called a side-slope canal.

It is situated exactly in between ridgeline and valley line.

It is parallel to the natural drainage line and hence no cross drainage works are needed.

The bed slope of this canal is steep.

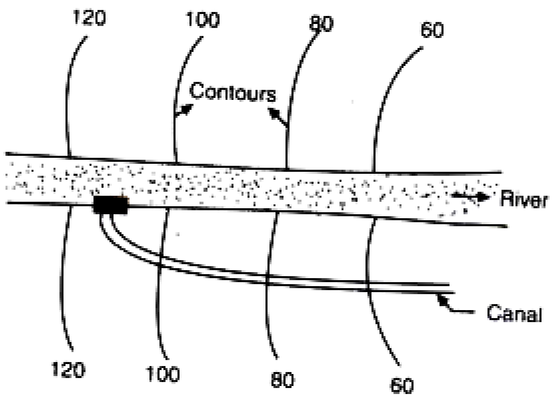


Fig: Side Slope Canal

CHAPTER VII
Water Logging and Drainage

Definition of water logging:-

The natural process of making saturation of soil and unproductive due to excessive in its pore is known as the water logging.

That area where it occurred is called water logging area.

Equation:-

I = O+S

Where,

I= inflow of water

O = outflow of water

S= storage of water

Cause of water logging:-

Topographic condition: - when the surface of soil is not uniform, then the rain water collects and make small pound. So due to permeability of soil, this water increases the water table, hence water logging occurred.

Inadequate drainage:-

If the drainage facility is not adequate then, it will increase the chance of water logging by increasing the water table.

Canal seepage:-

Seepage of water from the earthen canal also raises the water table and hence water logging occurred.

Over irrigation:-

Irrigation water if used in excess over the field's .then it will raises the water table and hence water logging occurred.

Rainfall and flood:-

Excess rainfall or flood will also cause the increasing in water table. By which water logging occurred.

Effects of water logging:-

Anchorage problems of plants and trees:-

If the soil becomes saturated then, the plant roots become very shallow and are easily uprooted when wind blows.

Growth of water loving wild plants:-

When the soil is water- logged water loving plant grow up which are harmful to the crop growth.

Increase in harmful salts:-

When the water table rise, then the upward moment of water brings harmful salts in the crop root zone. After evaporation, the water leaves behind salt, which reduce the production of crops.

Lowering of soil temperature:-

If moisture is present in the soil pores, the temperature of soil lowers down. Then bacteria activity retarded, which affects growth of crops badly.

Reduction of time for maturity:-

When the underwater is logged, then the crop period shortens which reduces the crop's maturity time.

Detection of water logged area:-

1. The water logged area can be easily detected by knowing the intensity of rainfall and the amount of runoff. So to calculate the quantity of rain water which infiltrate into the sub soil and help in raising the level of water table.

2. If the above information is no available then, a hole is made to be drilled to determine the level of underwater below the surface. If the water is available nearer to the root zone of the crops, then the area is called waterlogged area.

Preventive measures:-

1.Co**ntrolling seepage from the canals: -**

By following measures should be adopted to reduce seepage from the canals.

a) lowering the F.S.L of the canal:-

When the full supply level of canal is lowered then the loss of water's seepage loss is reduced, and hence water logging is also reduced.

b) **lining of canal :-**

The bed and sides of the canal should be lined by protective materials so that seepage loss is reduced and hence water logging is also reduced.

Disposal of rain water:-

Rain water as soon as, it falls on the earth's surface should be disposed of as soon as possible otherwise it will increase water logging.

Reducing the intensity of irrigation:-

The intensity should be reduced in this area where water logging occurred. Irrigation should be done rotation wise in different seasons.

REMEDIAL MEASURES:-

The followings methods are adopted to reclaim the water logged areas.

Installation of lift irrigation system:-

When a tube well systems are introduce, then the level of underground water goes down and hence water logging is reduced.

Implementation of drainage scheme:-

Area is reclaimed by introducing overland and subsurface drainage schemes. Surface drainage may be of:-

1) Providing seepage drain

2) By providing storm water or surface drain.

3) By providing lining of canal.

4) Implementation of tube well in fields or water logging area.

Surface drain: -

Surface drains are that construction may be natural of artificial which remove surplus water from any area and placed over the surface of the soil. While aligning surface drains followings points which are given below, should be considered.

1) Drain should follow lowest contour in a natural drainage line.
2) The total alignment of drain should be straight, so that length of the drain is reduced and all this reduces the cost of construction of that drain.
3) The drain should not pass through any ponds and it also should not cross irrigation canals

Sub surface drains:-

When the depth of the surface drain increases, then the drain scheme becomes uneconomical. In this situation subsurface drain is used. Subsurface drain is pipe drain laid in permeable stratum below a ground water table. These drains are circular pipes made of vitrified clay. The trench is excavated in the ground up to the required depth and tile line is laid on 15 cm sand bed.

Ground water recharge:-

It is a process where water moves downward from surface water to ground water. This process usually below plant root. The water table recharge occurs naturally of artificially.

Natural Ground water is recharge by rain water, by melting of snow, by river and lake through permeability of soil

Artificial ground water is recharge by making a pond, reservoirs and by storing rain watered.

CHAPTER VIII

Diversion Head Works And Regulatory Structures

DIVERSION HEADWORKS:

Selection of site and layout, parts of diversion head-works, types of weirs and barrages, design of weirs on permeable foundations, control of silt entry into canal, silt excluders and different types of silt ejectors.

1.1 Definition

A structure constructed at the junction of the source (river, dam, canal) and the off taking canal.

1.2 Types of Headworks

The different types of headworks are as follows:

1.2.1 Diversion Headworks

- diverts the required supply from the source channel to the off taking channel
- water level in the source channel raised to the reqd. level
- reduces the need of excavation in the head reach
- command area is served better by flow irrigation
- should be capable of regulating the supplies into the off taking channel; all supplies when demand is keen & supplies are less
- control sediment entry

1.2.2 Storage Headworks

- fulfill requirements of the diversion headworks
- in addition, store excess water when available and release when demand exceeds supplies

1.2.3 Temporary Headworks

- bunds constructed across the river every year after floods
- replaced with permanent headworks when demand of water increases

1.2.4 Permanent Headworks - all important headworks

1.3 Location of Headworks on Rivers

Rivers have four stages: (i) rocky, (ii) boulder, (iii) trough (or alluvial), and (iv) delta

Rocky stage: far away from command arca; length and, therefore, cost of main canal increases,so is unsuitable for headworks

Delta Stage: irrigation demand is less and, also, nature of river (flat slopes, braiding) poses other problems, hence unsuitable

Boulder and alluvial stages - both suitable

The favorable features of boulder stage for headworks are:

- initial cost is less
- local availability of stones
- smaller width of river (therefore, weir)
- smaller scour depths (reduce the depth of cut-off)
- close proximity of higher banks (less training works)
- canal will have number of falls – can be utilized for power generation
- construction of temporary bunds (for initial period) possible
- will require large number of cross-drainage structures
- considerable loss of water through subsoil flow in the river bed and from the head reach of the main canal - crucial during periods of short supply

Hilly regions usually have wet climate and, therefore, irrigation demand is, generally, small to begin with and may increase later. In alluvial regions the demand for irrigation is high right fromthe beginning.

1.4 Site Selection for Headworks

River reach should be:

- straight and narrow
- well-defined and non-erodible high banks
- preferably deep channels on both banks and shallow channel at the centre

Based on considerations of suitability of site for different components of headworks following points are important:

- weir (barrage) - minimum length for economy, uniform flow for proper functioning
- under sluices - presence of deep channel to ensure adequate supply to the off-taking canal
- canal alignment - capable of serving its command area without much excavation
- sediment – off taking channel sited on the downstream end of the outer side of the bend

1.5 Diversion Headwork Components

- Weir (or barrage)
- Undersluices
- Divide Wall
- Fish Ladder
- Canal Head Regulator
- Sediment Excluder and Sediment Ejector
- River Training Works

1.6 Layout of Headworks

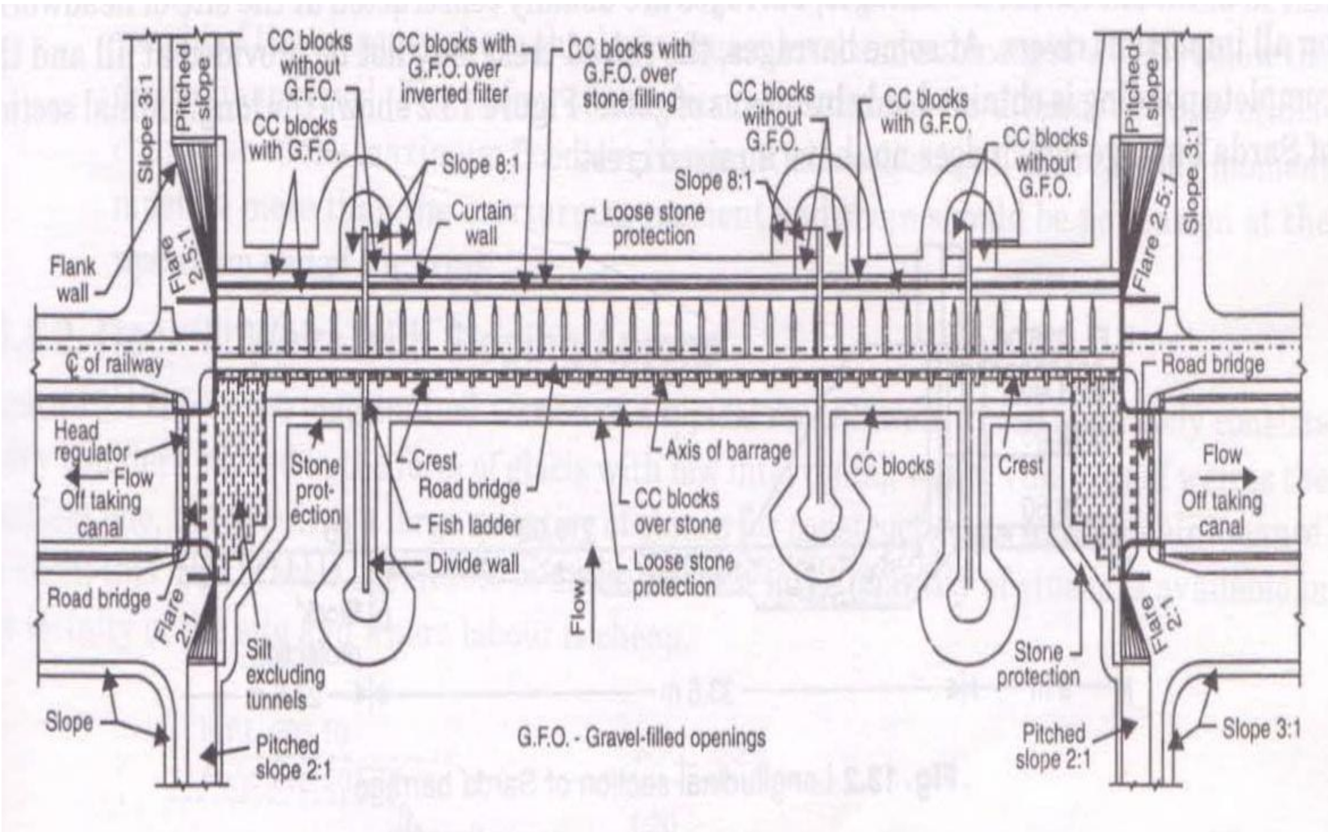


Figure 1. Typical layout of headworks

1.7 Weir (or Barrage)

1.7.1 Weir

- ungated barrier across a river
- raises water level in the river and diverts water into an off taking canal - on one or both banks of the river - just u/s of the weir
- usually aligned at right angle to the direction of flow - results in minimum length & normal uniform flow through all weir bays which minimizes the chances of shoal formation and oblique flow
- crest is raised above the river bed to raise the water level
- shutters at the top of the crest for further raising of water level and controlling pond level (difficult when pond level is higher than 2 m above the crest)
- provide gate-controlled weir - barrage

1.7.2 Barrage

- a gate-controlled weir with its crest at a lower level
- ponding up of the river for diversion is by means of gates
- offer better control on outflow and discharge in the offtaking canal
- afflux is small due to lower crest level of the barrage
- possible to provide a roadway across the river at small cost
- better control over sediment entry into canal
- Therefore, barrages are very common on all important headworks at times no raised crest as in Sarda barrage - ponding is by gates only design procedure is similar to that of weir

1.7.3 Types of Weirs

1.7.3.1 Masonry weirs with vertical d/s face

- masonry floor with a masonry crest on top of which shutters for ponding
- shutters dropped during floods to reduce afflux
- stability of crest examined for water level upto the top of shutter with no flow d/s when shutters dropped and water is on both sides of the crest

1.7.3.2 Rockfill weirs with sloping aprons

- simplest but requires large qty. of stones for constn. & maintenance

1.7.3.3 Concrete weirs (or barrages) with glacis

- on pervious foundation, only concrete weirs these days
- excess energy dissipated by means of hyd. Jump formed on glacis
- design based on Khosla's method and requires the knowledge of
- max. flood discharge & corresponding level around the weir site
- the stage - disch. Curve at the weir site
- the X-section of the river at the site

Based on the site conditions, general & economic considerations, decide

- afflux
- pond level
- min. waterway
- weir crest level

1.8 Undersluices (Sluice Ways or Scouring Sluices)

- Undersluices help in flushing the sediment deposited u/s of the canal head regulator on account of ponding up of water due to construction of weir across river
- gate-controlled openings in continuation of the weir with their crests at lower level
- located on the same side as the offtaking canal

- useful for passing low floods after meeting the requirements of the offtaking canal
- shutters (or gates) operated only for passing high floods during monsoon
- design procedure is similar to that of weir (use model analysis for major headworks)

Crest level of undersluice:

- generally coincides with the lowest cold weather level of the river bed at the weir site
- at least 1.2 m (2 m if sediment Excluder is provided) below that of the head regulator so that the sediment deposited u/s of the regulator does not enter the off taking canal. If needed, the crest level of the regulator is raised.

Discharge capacity of the undersluices is maximum of the following:

- twice the canal discharge - to ensure sufficient scouring capacity
- 10 - 20 % of the max. flood discharge - to reduce the length of the weir
- enough capacity to pass off low floods with w/s in the reservoir at pond level to avoid gate operation

1.9 Afflux & Waterway

- HFL u/s of weir rises due to construction of weir across river, this rise is afflux
  Afflux = u/s TEL - d/s TEL
- initially, the afflux is confined to a short reach u/s of weir but, gradually extends very far u/s in case of alluvial rivers - due to continued deposition
- afflux governs top levels of guide banks & marginal bunds & length of bunds
- waterway & afflux are interdependent

    larger afflux results in lesser waterway
    increases the discharge intensity q which, in turn,
    increases the scour &, hence, cost of protection
    works d/s
    higher afflux also increases the risk of failure of river training works

- For alluvial rivers:

afflux = 1m in upper and middle reaches of river
= 0.3m in lower reaches with flat gradients

waterway = 1.1 to 1.2 times Lacey's regime perimeter for the design discharge
= 0.8 to 0.9 times regime perimeter in rivers with coarser bed material

- lesser waterway increases afflux & cost of protection & river training works
- larger waterway is uneconomical & causes oblique flow and silting in part of waterways

Loosenes factor:

- overall length of weir /min. stable width from regime criterion

Pond level:

- water level which must be maintained in the under sluice pocket (i.e. u/s of the canal head regulator) so as to maintain FSL in the canal when full supply discharge is fed into it

□ = FSL + (1.0 to 1.2 m) so that sufficient working head is available even whenhead reach of canal has silted up or when canal is to be fed excess water

- limit on pond level : FSL = pond level - working head (1.0 to 1.2 m)
- maintained by keeping the weir crest at the pond level or by keeping the weir crest atlower level and provide shutters/gates

Retrogression:

- d/s of weir due to degradation; d/s HFL lowered; exit gradient increases during high floods : 0.3 to 0.5 m due to large qty. of sediment; during low floods : 1.25 to 2.25m due to relatively clear water for design flood retrogression is assumed 0.3 to 0.5 m

d/s TEL = u/s TEL - head loss (= afflux + retrogression)

1.10 Divide Wall

- parallel to canal head regulator
- separates main weir bays (and floor) from undersluice bays (and floor)
- extends on both sides of weir upto the end of the floor or loose apron
- on d/s it avoids cross flow causing scour near the structure
- isolates canal head regulator from main
  river flowcreates still pond in
  front of the regulator
  helps in deposition of sediment and relatively clear water in canalimproves scouring of undersluices by ensuring straight approach
- additional divide walls if possibility of cross currents exist
- generally of strong masonry with top width of about 1.5 to 2.25m and nose end sloping at 3(V):1(H); slight divergence of 1 in 10 advisable; extending upto about the u/s end of the canal or half to 2/3rd the length of the regulator

1.10 Fish ladder

- fish of various kinds
- migrate to d/s in the beginning of winter in search of warmth
- return u/s before the monsoon for clear water
- a narrow opening (Fish ladder) between the divide wall and the undersluices where wateris always present

- baffles to reduce the velocity to less than 3.0 m/s
- these openings are called as fish ladder or fishways or fish pass
- should take into account the requirements of the fish of the river

CHAPTER IX

Cross Drainage Works

Cross - Drainage Works:

In an irrigation project, when the network of main canals, branch canals, distributaries, etc are provided, then these canals may have to cross the natural drainages like rivers, streams, nallahs, etc. at different points within the commanded area of the project. The crossing of canals with such obstacle cannot be avoided. Therefore, suitable structures must be constructed at the crossing point for the easy flow of water of the canal and drainage in the respective directions. These structures are called cross – drainage works. Thus, a cross – drainage work is a structure carrying the discharge from a natural stream across a canal intercepting the stream.

Figure 1. Canal crossing a natural drainage

Necessity of Cross - Drainage Works:

The cross – drainage work is required to dispose of the drainage water so that the canal supply remains uninterrupted. The canal at the cross – drainage work is generally taken either over or below the drainage. However, it can also be at the same level as the drainage. As we know that, canals are usually aligned on the watershed so that there are no drainage crossings. However, it is not possible to avoid the drainages in the initial reach of a main canal because it takes off from a diversion headworks (or storage works) located on a river which is a valley. The canal, therefore, requires a certain distance before it can mount the watershed (or ridge). In this initial reach, the canal is usually a contour canal and it intercepts a number of natural drainages flowing from the watershed to the river.

After the canal has mounted the watershed, no cross-drainage work will normally be required, because all the drainage originate from the watershed and flow away from it. However, in some cases, it may be necessary for the canal to leave the watershed and flow away from it. In that case, the canal intercepts the drainages which carry the water of the pocket between the canal and the watershed and hence the cross-drainage works are required.

A cross-drainage work is an expensive structure and should be avoided as far as possible. The number of cross-drainage works can be reduced to some extent by changing the alignment of the canal. However, it may increase the length and hence the cost of the canal. Sometimes it is possible to reduce the number of cross-drainage works by diverting the small drainages into large drainages or by constructing the cross-drainages work below the confluence of two drainages by shifting the alignment. However, the suitability of the site for the construction of the structure should also be considered while deciding the location of the cross-drainage works.

Types of Cross-Drainages Works:

Depending upon the relative positions of the canal and the drainage, the cross-drainage works may be classified into 3 categories as:

1. Canal over the drainage

(a) Aqueduct: An aqueduct is a structure in which the canal flows over the drainage and the flow of the drainage below is open channel flow. An aqueduct is similar to an ordinary road bridge (or railway bridge) across drainage, but in this case, the canal is taken over the drainage instead of a road (or a railway). A canal trough is to be constructed in which the canal water flows from upstream to downstream. This canal trough is to be rested on a number of piers. An aqueduct is provided when the canal bed level is higher than the H.F.L. of the drainage.

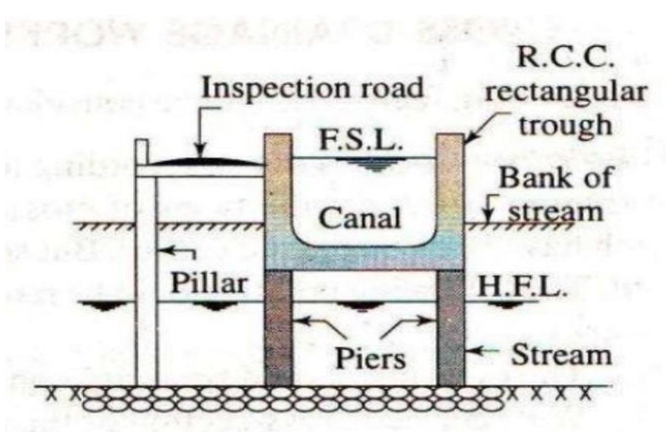


Figure 2. Aqueduct

(b) Syphon aqueduct: In a syphon aqueduct also the canal is taken over the drainage, but the flow in the drainage is pipe flow (i.e. the drainage water flows under syphonic action and there is no atmospheric pressure in the drainage). A syphon aqueduct is constructed when the H.F.L. of the drainage is higher than the canal bed level. When sufficient level difference is not available between the canal bed and the H.F.L. of the drainage to pass the drainage water, the bed of the drainage may be depressed below its normal bed level. The drainage is provided with an impervious floor at the crossing and thus a barrel is formed between the piers to pass the drainage water under pressure. Syphon aqueducts are preferred than Aqueducts, though costlier.

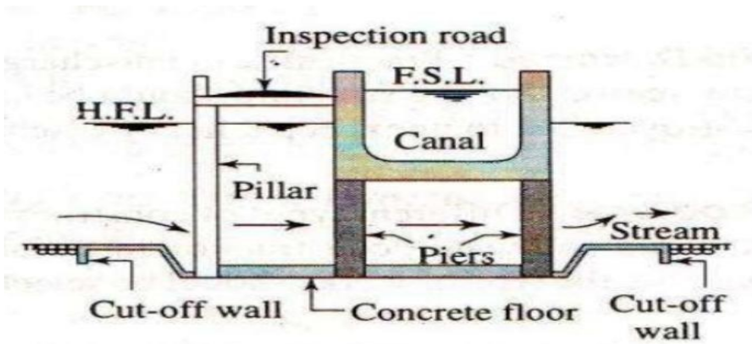


Figure 3. Syphon aqueduct

2. Canal below the drainage

(a) Superpassage: In a superpassage, the canal is taken below the drainage and the flow in the channel is open channel flow. A superpassage is thus reverse of an aqueduct. A superpassage is required when canal F.S.L is below the drainage bed level. In this case, the drainage water is taken in a trough supported over the piers constructed on the canal bed. The water in the canal flows under gravity and possess the atmospheric pressure.

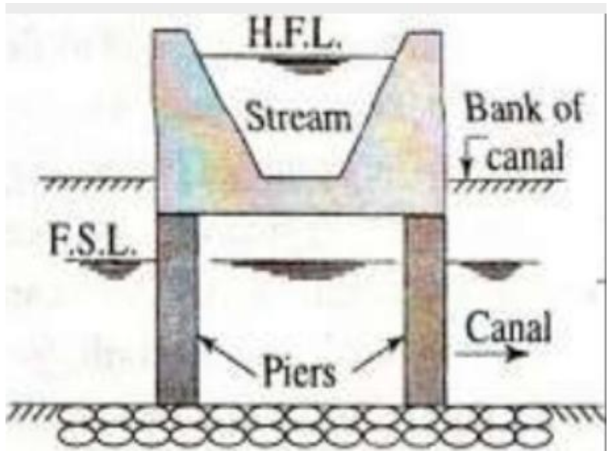


Figure 4. Superpassage

(b) Canal syphon: A canal syphon (or Simply a syphon) is a structure in which the canal is taken below the drainage and the canal water flows under symphonic action and there is no presenceof atmospheric pressure in the canal. It is thus the reverse of a syphon aqueduct.

A canal syphon is constructed when the F.S.L. of the canal is above the drainage bed level. Because some loss of head invariably occurs when the canal flows through the barrel of the canal syphon, the command of the canal is reduced. Moreover, there may be silting problem in the barrel. As far as possible, a canal Syphon should be avoided.

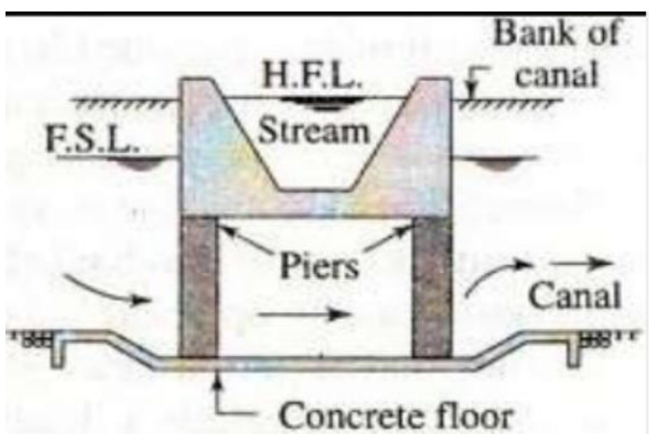


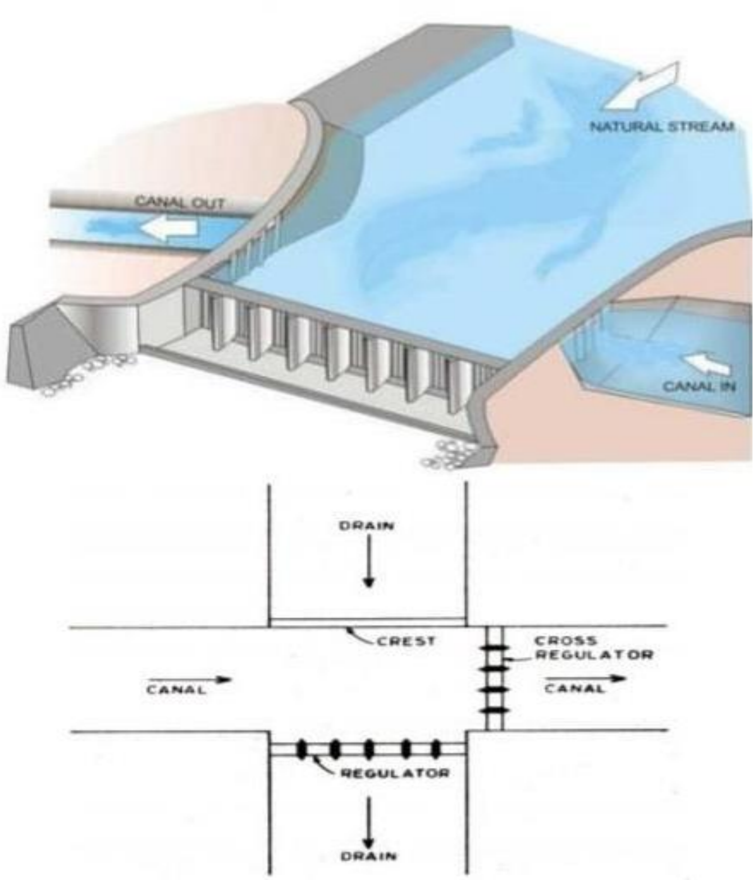
Figure 5. Canal Syphon

3. Canal at the same level as drainage

(a) Level crossing: A level crossing is provided when the canal and the drainage are practically at the same level. In a level crossing, the drainage water is admitted into the canal at one bank and is taken out at the opposite bank.

A level crossing usually consists of a crest wall provided across the drainage on the upstream of the junction with its crest level at the F.S.L. of the canal. The drainage water passes over the crest and enters the canal whenever the water level in the drainage rises above the F.S.L. of the canal. There is a drainage regulator on the drainage at the d/s or the junction and a cross-regulator on the canal at the d/s of the junction for regulating the outflows.

Figure 6. Level crossing



(b) Inlet and outlet: An inlet-outlet structure is provided when the drainage and the canal are almost at the same level, and the discharge in the drainage is small. The drainage water is admitted into the canal at a suitable site where the drainage bed is at the F.S.L. of the Canal. The excess water is discharged out the canal through an outlet provided on the canal at some distance downstream of junction. There are many disadvantages in use of inlet and outlet structure, because the drainage may pollute canal water and also the bank erosion may take place causing the deterioration of the canal structure so that maintenance costs are high.Hence, this type of structure is rarely constructed.

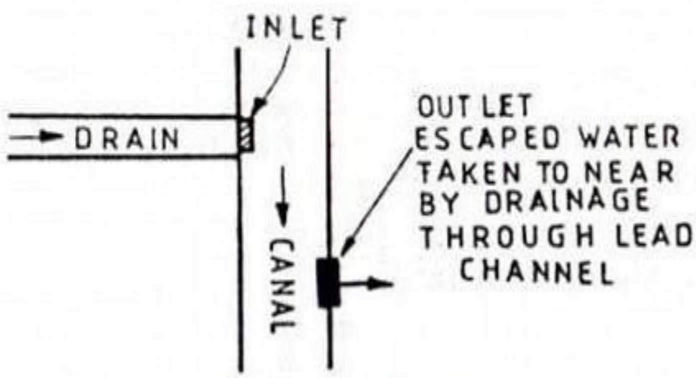


Figure 7. Inlet and Outlet

Selection of a suitable type of cross-drainage work:

The following points should be considered while selecting the most suitable type ofcross – drainage work:

1. Relative levels and discharges: The relative levels and discharges of the canal and of the drainage mainly affect the type of cross – drainage work required. The following are the broad outlines:

- If the canal bed level is sufficiently above the H.F.L of the drainage, an aqueduct may be provided.
- If the F.S.L. of the canal is sufficiently below the bed level of the drainage, a super-passage is provided.
- If the canal bed level is only slightly below the H.F.L. of the drainage, and the drainage is small, a syphon aqueduct is provided.
- If the F.S.L. of the canal is slightly above the bed level of the drainage and the canal is of small size, a canal syphon is provided.
- If the canal bed and the drainage bed are almost at the same level, a level crossing is provided when the discharge in the drainage is large, and an inlet-outlet structure is provi1ded when the discharge in the drainage is small.

2. Performance: As far as possible, the structure having an open channel flow should be preferred to the structure having pipe flow. Therefore, an aqueduct should be preferred to a

syphon aqueduct. Similarly, a super-passage should be preferred to a canal syphon. Theperformance of inlet-outlet structures is not good and should be avoided.

3. Provision of road: A aqueduct is better than a super-passage because in the former, a road bridge can easily be provided along with the canal trough at a small extra cost, whereas in the latter, a separate bridge is required.

4. Size of drainage: When the drainage is of small size, a syphon aqueduct will be preferred to an aqueduct as the latter involves high banks and long approaches. However, if the drainage is of large size, an aqueduct is preferred.

5. Cost of earthwork: The type of cross-drainage work which does not involve a large quantity of earthwork should be preferred.

6. Foundation: The type of cross-drainage work should be selected depending upon the foundation available at the site.

7. Material of construction: Suitable types of material of construction in sufficient quantity should be available near the site for the particular type of cross – drainage work selected.

8. Cost of construction and overall cost: The cost of construction of cross-drainage work should not be

9. Subsoil water table: If the subsoil water table is high, the types of cross – drainage works which require deep excavation should be avoided.

10. Permissible loss of head: The cross-drainage works should be selected based on the permissible loss of head. Where the head loss cannot be permitted, a canal syphon should be avoided.

11. Canal alignment: The canal alignment is sometimes changed to achieve a better type of cross-drainage work. By changing the alignment, the type of cross-drainage work can be altered. The canal alignment is generally finalized after fixing the sites of the major cross – drainage works.

Selection of site of a cross-drainage work:

The following points should be considered while selecting the site of a cross-drainage work:

1. At the site, the drainage should cross the canal alignment at right angles. Such a site providesgood flow conditions and also the cost of the structure is usually a minimum.

2. The stream at the site should be stable and should have stable banks.

3. For economical design and construction of foundations, a firm and strong sub-stratum shouldexist below the bed of the drainage at a reasonable depth.

4. The site should be such that long and high approaches of the canal are not required.

5. The length and height of the marginal banks and guide banks for the drainage should besmall.

6. In the case of an aqueduct, sufficient headway should be available between the canal troughand the H.F.L of the drainage.

7. The water table at the site should not be high, because it can create dewatering problems forlaying foundations.

8. As far as possible, the site should be selected downstream of the confluence of two streams,thereby avoiding the necessity of construction of two structures.

9. The possibility of diverting one stream into another stream upstream of the canal crossingshould be considered, if found feasible.

10. A cross-drainage work should be combined with a bridge, if required. If necessary, thebridge site can be shifted to a cross-drainage structure or vice-versa.

CHAPTER 10

DAMS

GRAVITY DAMS-EARTH DAMS

A **gravity dam** is a dam constructed from concrete or stone masonry and designed to hold back waterby primarily utilizing the weight of the material alone to resist the horizontal pressure of water pushing against it. Gravity dams are designed so that each section of the dam is stable, independent of any other dam section

FORCES ACTING ON GRAVITY DAM:

In the design of a dam, the first step is the determination of various forces which acts on the structure and study their nature. Depending upon the situation, the dam is subjected to the following forces:

1. Water pressure
2. Earthquake forces
3. Silt pressure
4. Wave pressure

Ice pressure

6. Self weight of the dam.

The forces are considered to act per unit length of the dam.

For perfect and most accurate design, the effect of all the forces should be investigated. Out of these forces, most common and important forces are water pressure and self weight of the dam.

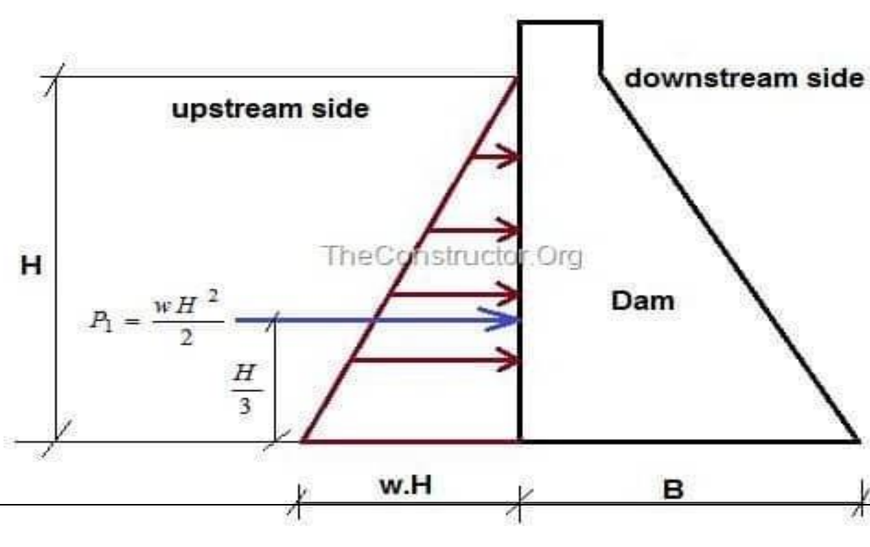
1. Water Pressure

Water pressure may be subdivided into the following two categories:

I) External water pressure:

It is the pressure of water on the upstream face of the dam. In this, there are two cases:

(I) Upstream face of the dam is vertical and there is no water on the downstream side of the dam (figure 1).



The total pressure is in horizontal direction and acts on the upstream face at a height $\frac{H}{3}$ from the bottom. The pressure diagram is triangular and the total pressure is given by $P_1 = \frac{wH^2}{2}$

Where w is the specific weight of water. Usually it is taken as unity.

H is the height upto which water is stored in m.

(ii) Upstream face with batter and there is no water on the downstream side (figure 2).

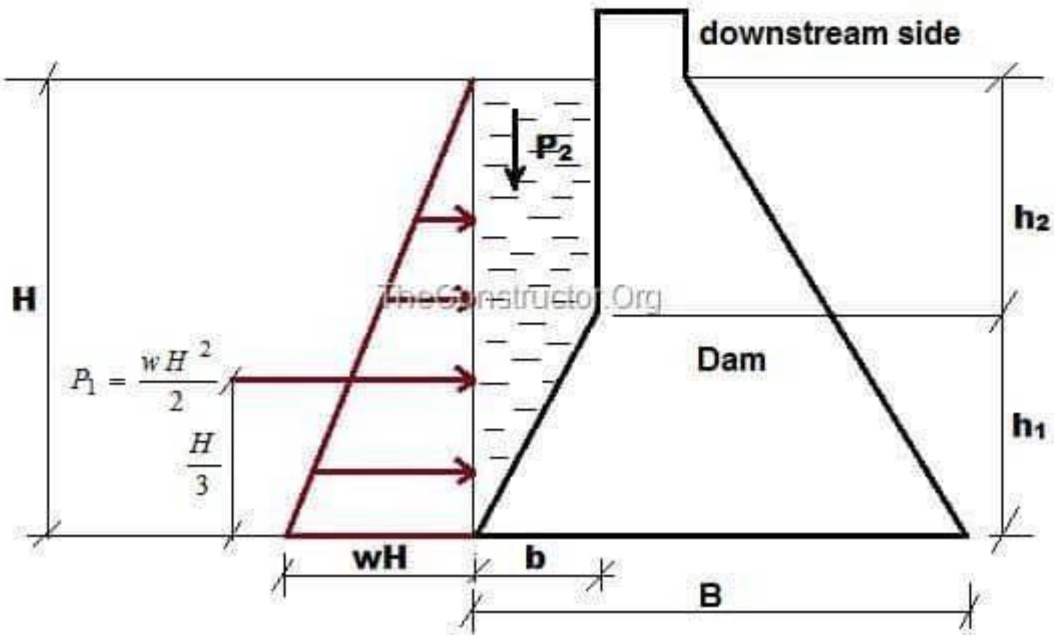


Figure 2

Here in addition to the horizontal water pressure P_1 as in the previous case, there is verticalpressure of the water. It is due to the water column resting on the upstream sloping side.

The vertical pressure P_2 acts on the length 'b' portion of the base. This vertical pressure is given by

$$P_2 = (b \times h_2 \times w) + \left(\frac{1}{2} b \times h_1 \times w\right)$$

Pressure P_2 acts through the centre of gravity of the water column resting on the sloping upstreamface.

If there is water standing on the downstream side of the dam, pressure may be calculated similarly. The water pressure on the downstream face actually stabilizes the dam. Hence as an additional factor of safety, it may be neglected.

II) Water pressure below the base of the dam or Uplift pressure

When the water is stored on the upstream side of a dam there exists a head of water equal to the height upto which the water is stored. This water enters the pores and fissures of the foundation material under pressure. It also enters the joint between the dam and the foundation at the base and the pores of the dam itself. This water then seeps through and tries to emerge out on the downstream end. The seeping water creates hydraulic gradient between the upstream and downstream side of the dam. This hydraulic gradient causes vertical upward pressure. The upward pressure is known as uplift. Uplift reduces the effective weight of the structure and consequently the restoring force is reduced. It is essential to study the nature of uplift and also some methods will have to be devised to reduce the uplift pressure value.

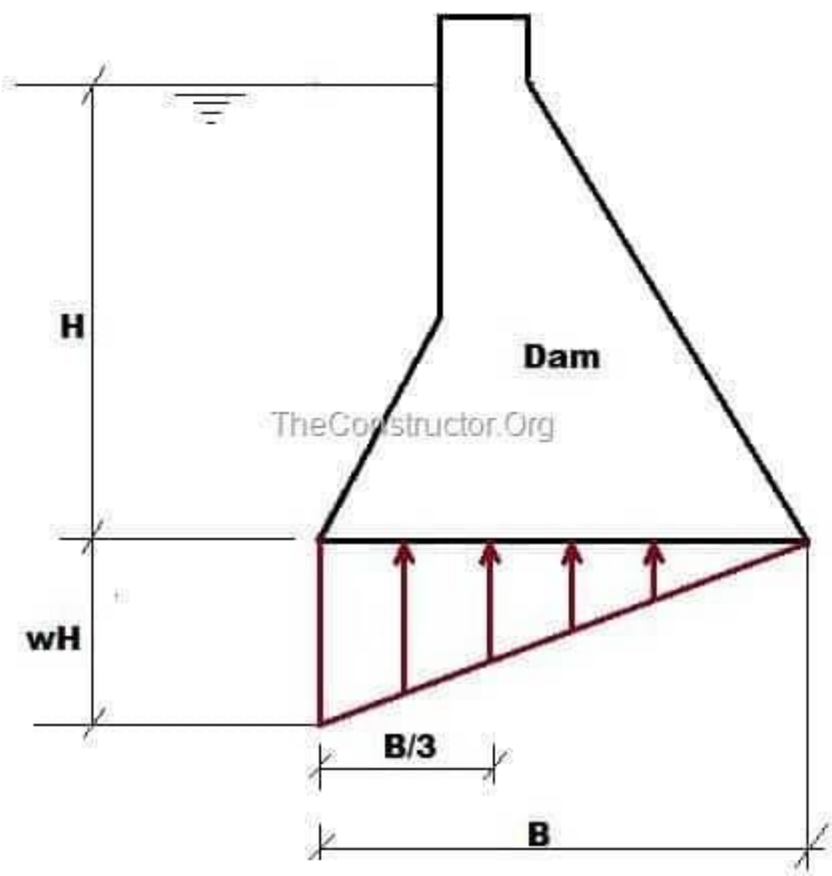


Figure 3

With reference to figure 3, uplift pressure is given by $P_u = \frac{wH \times B}{2}$

Where P_u is the uplift pressure, B is the base width of the dam and H is the height upto which wateris stored.

This total uplift acts at $\frac{B}{3}$ from the heel or upstream end of the dam.

Uplift is generally reduced by providing drainage pipes or holes in the dam section.

Self weight of the dam is the only largest force which stabilizes the structure. The total weight of the dam is supposed to act through the centre of gravity of the dam section in vertically downward direction. Naturally when specific weight of the material of construction is high, restoring force will be more. Construction material is so chosen that the density of the material is about 2.045 gram per cubic meter.

2. Earthquake Forces

The effect of earthquake is equivalent to acceleration to the foundation of the dam in the direction in which the wave is travelling at the moment. Earthquake wave may move in any direction and for design purposes, it is resolved into the vertical and horizontal directions. On an average, a value of
0.1 to 0.15g (where g = acceleration due to gravity) is generally sufficient for high dams in seismic zones. In extremely seismic regions and in conservative designs, even a value of 0.3g may sometimes by adopted.

Vertical acceleration reduces the unit weight of the dam material and that of water is to $(1 - k_v)$ times the original unit weight, where k_v the value of g accounted against earthquake forces, i.e. 0.1 is when 0.1g is accounted for earthquake forces. The horizontal acceleration acting towards the reservoir causes a momentary increase in water pressure and the foundation and dam accelerate towards the reservoir and the water resists the movement owing to its inertia. The extra pressure exerted by this process is known as hydrodynamic pressure.

3. Silt Pressure

If h is the height of silt deposited, then the forces exerted by this silt in addition to the external waterpressure, can be represented by Rankine formula
$P_{silt} = \frac{1}{2}\gamma_s h^2 k_a$ acting at $\frac{h}{3}$ from the

base. Where,

k_a = coefficient of active earth pressure of silt = $\frac{1 - \sin\phi}{1 + \sin\phi}$
ϕ = angle of internal friction of soil, cohesion neglected.
γ_s = submerged unit weight of silt material.

h = height of silt deposited.

4. Wave Pressure

Waves are generated on the surface of the reservoir by the blowing winds, which exert a pressure onthe downstream side. Wave pressure depends upon wave height which is given by the equation

$h_w = 0.032\sqrt{PV} + 0.763 - 0.271 \times (F)^{1/4}$ for F < 32 km, and

$h_w = 0.032\sqrt{VF}$ for F > 32 km

Where h_w is the height of water from the top of crest to bottom of trough in

meters. V – wind velocity in km/hour

F – fetch or straight length of water expanse in km.

The maximum pressure intensity due to wave action may be

$P_w = 2.4\gamma_w h_w$ given by and acts at $\frac{h_w}{2}$ meters above the still

water surface.

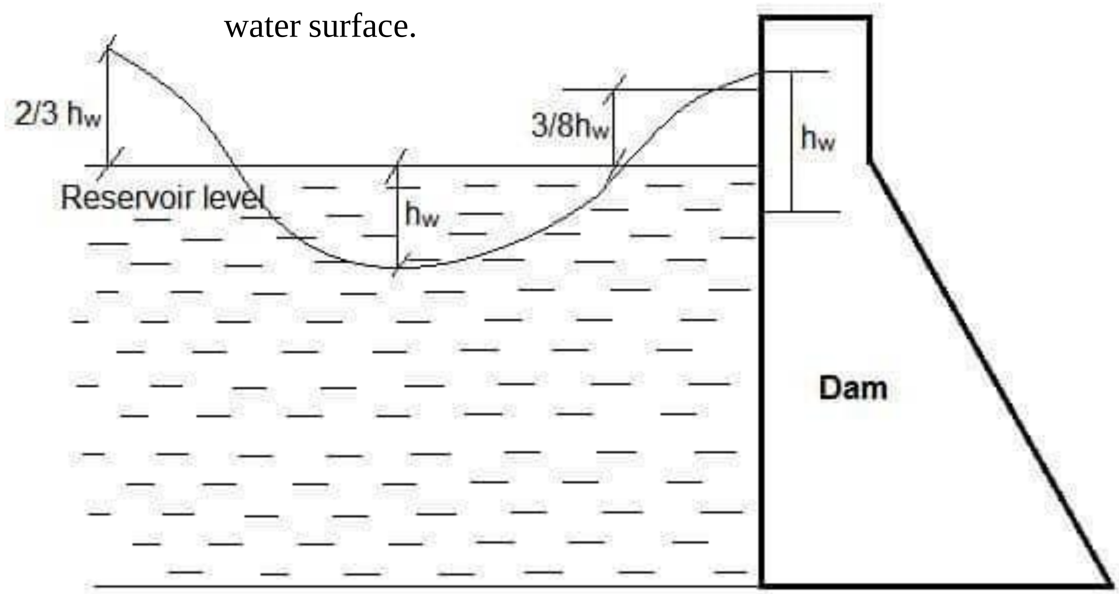


Figure 4

The pressure distribution may be assumed to be triangular of height $\frac{5h_w}{3}$ as shown in figure 4. Hence total force due to wave action P_w

= $\frac{1}{2} \times (2.4 \gamma_w h_w) \times \frac{5}{3} h_w$ acting at $\frac{3}{8} h_w$ above the reservoir surface.

5. Ice Pressure

The ice which may be formed on the water surface of the reservoir in cold countries may sometimes melt and expand. The dam face is subjected to the thrust and exerted by the expanding ice. This force acts linearly along the length of the dam and at the reservoir level. The magnitude of this force varies from 250 to 1500 kN/sq.m depending upon the temperature variations. On an average, a value of 500 kN/sq.m may be taken under ordinary circumstances.

6. Weight of dam

The weight of dam and its foundation is a major resisting force. In two dimensional analysis of dam

FAILURES OF GRAVITY DAM

Failure of gravity dam occurs due to overturning, sliding, tension and compression. A gravity dam is designed in such a way that it resists all external forces acting on the dam like water pressure, wind pressure, wave pressure, ice pressure, uplift pressure by its own self-weight. Gravity dams are constructed from masonry or concrete. However, concrete gravity dams are preferred these days and mostly constructed.

The advantage of gravity dam is that its structure is most durable and solid and requires very less maintenance.
Causes of failure of a Gravity Dam:

A gravity dam may fail in following modes:

1. Overturning of dam about the toe
2. Sliding – shear failure of gravity dam
3. Compression – by crushing of the gravity dam
4. Tension – by development of tensile forces which results in the crack in gravity dam.

Overturning Failure of Gravity Dam:

The horizontal forces such as water pressure, wave pressure, silt pressure which act against the gravity dam causes overturning moments. To resist this, resisting moments are generated by the self-weight of the dam.

If the resultant of all the forces acting on a dam at any of its sections, passes through toe, the dam will rotate and overturn about the toe. This is called overturning failure of gravity dam. But, practically, such a condition does not arise and dam will fail much earlier by compression.

The ratio of the resisting moments about toe to the overturning moments about toe is called the factor of safety against overturning. Its value generally varies between 2 and 3.

Factor of safety against overturning is given by

FOS = sum of overturning moments/ sum of resisting moments

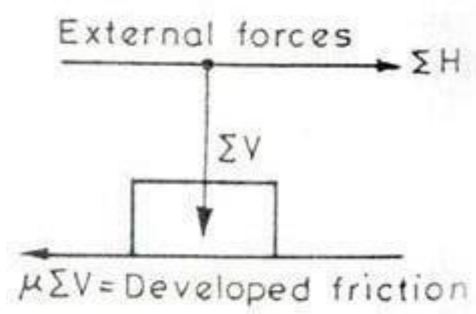


Fig:sum of external horizontal forces greater than vertical self-weight of dam (overacting, sliding occurs)

Sliding Failure of Gravity Dam: When the net horizontal forces acting on gravity dam at the base exceeds the frictional resistance (produced between body of the dam and foundation), The failure occurs is known as sliding failure of gravity dam.

In low dams, the safety against sliding should be checked only for friction, but in high dams, for economical precise design, the shear strength of the joint is also considered

Factor of safety against sliding can be given based on Frictional resistance and shear strength of the dam

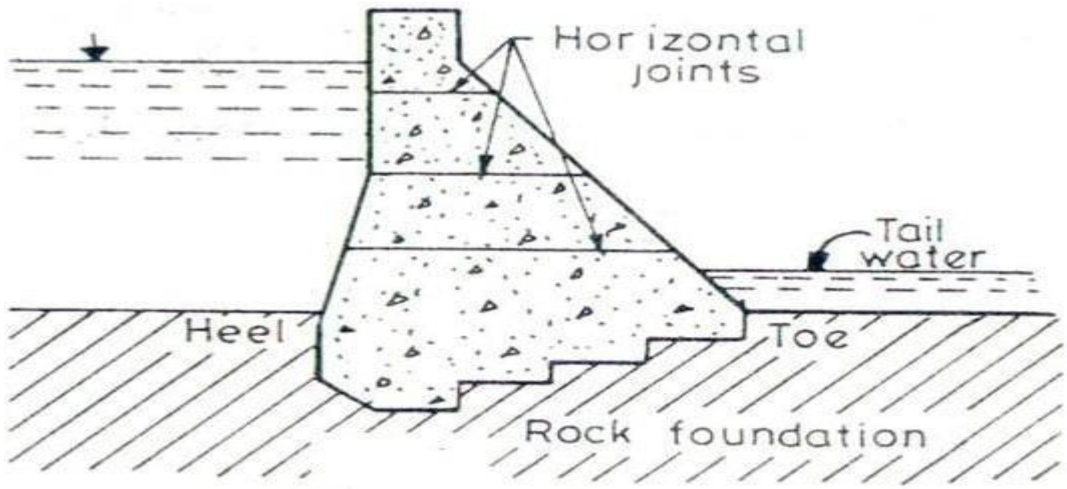
Factor of safety based on frictional resistance:

$$\text{FOS against sliding} = \text{FOS} = \frac{\mu \sum V}{\sum H}$$

μ=co-efficient of friction between two surfaces

$\sum V$ = sum of vertical forces acting on dam

$\sum H$ = sum of vertical forces acting on dam



Gravity Dam Failure due to Tension Cracks: Masonry and concrete are weak in tension. Thus masonry and concrete gravity dams are usually designed in such a way that no tension is developed anywhere. If these dams are subjected to tensile stresses, materials may develop tension cracks. Thus the dam loses contact with the bottom foundation due to this crack and becomes ineffective and fails. Hence, the effective width B of the dam base will be reduced. This will increase pmax at the toe. Hence, a tension crack by itself does not fail the structure, but it leads to the failure of the structure by producing excessive compressive stresses.

For high gravity dams, certain amount of tension is permitted under severest loading conditions in order to achieve economy in design. This is permitted because the worst condition of loads may occur only momentarily and may not occur frequently.

Gravity Dam Failure due to Compression:A gravity dam may fail by the failure of its material, i.e. the compressive stresses produced may exceed the allowable stresses, and the dam material may get crushed.

STABILITY ANALYSIS OF GRAVITY DAMS

General Selection of the method of analysis should be governed by the type and configuration of the structure being considered. The gravity method will generally be sufficient for the analysis of most structures, however, more sophisticated methods may be required for structures that are curved in plan, or structures with unusual configurations. 3-4.2 Gravity Method The gravity method assumes that the dam is a 2 dimensional rigid block. The foundation pressure distribution is assumed to be linear. It is usually prudent to perform gravity analysis before doing more rigorous studies. In most cases, if gravity analysis indicates that the dam is stable, no further analyses need be done.

Stability Analysis Assumptions:

1. The dam is considered to be composed of a number of Cantilevers, each of which is 1 m thick and each of which acts independently of the other.
2. No load is transferred to the abutments by beam action
3. The foundation and the dam behave as a single unit, the joints being perfect.
4. The material in the foundation and the body of the dam are isotropic and homogeneous.
5. The stresses developed in the foundation and the body of the dam is isotropic and homogeneous.
6. No movements of dams are caused by the Transfers of loads.

Stability Analysis Procedure

Two dimensional analysis can be carried out analytically or graphically

Analytical Method

1. Consider unit length of the dam
2. Work out the magnitude and direction of all the vertical forces acting on the dam and their algebraic sum i.e. $\sum V$
3. Similarly, work out all the horizontal forces and their algebraic sum, i.e.. $\sum H$
4. Determine the level arm of all these forces about the toe
5. Determine the moments of all these forces about the toe and find out the algebraic sum of all those moments i.e.. $\sum M$

Graphical method

In the graphical method, the entire dam section is divided into number of horizontal sections at somesuitable interval. Particularly at the place where the slope changes.

1. For each section, the sum of the vertical forces $\sum V$ and the sum of all the horizontal forces $\sum H$ acting above that particular section, are worked out and the resultant is drawn, graphically
2. This is done for each section and a line joining all the points where the individual resultants cut the individual sections, is drawn.
3. This line represents the resultant force and should lie within the middle third, for no tensionto develop.
4. The procedure should be repeated for reservoir full as well reservoir empty case.

Profile of A Dam from Practical Considerations

• The elementary profile of a gravity dam, (i.e.. triangle with maximum water surface at apex) is only a theoretical profile. Certain changes will have to be made in this profile in order to cater to the practical needs.

These needs are,

(i) Providing a straight top width for road construction over the top of the dam

(ii) Providing a free-board above the top water surface, so that water may spill over the top of the dam due to wave action, etc.

The addition of these two provisions, will cause the resultant force to shift towards the heel. The resultant force, when the reservoir is empty, was earlier passing through the inner middle third point. This will, therefore, shift more towards the heel, crossing the inner middle third point and consequently, tension will be developed at the toe. In order to avoid the development of this tension, some masonry will have to be added to the upstream side., which shows the typical section along with the possible dimensions that can be adopted for a low gravity dam section. It should however, be checked for stability analysis.

Galleries in Gravity Dams

Galleries are the horizontal or sloping openings or passages left in the body of the dam. • They may run longitudinally (i.e. parallel to dam axis) or traversely (i.e. normal to the dam axis) and are provided at various elevations. All the galleries are interconnected by steeply sloping passages or by vertical shafts fitted with stairs or mechanical lifts.

Function and types of galleries in Dams

(i) Foundation Gallery

A gallery provided in a dam may serve one particular purpose or more than one purpose. For example, a gallery provided near the rock foundation, serves to drain off the water which percolates through the foundations. This gallery is called a foundation gallery or a drainage gallery.

1. It runs longitudinally and is quite near to the upstream face of the dam. Drain holes are drilled from the floors of this gallery after the foundation grouting has been completed. Seepages is collected through these drain holes.
2. Besides draining off seepage water, it may be helpful for drilling and grouting of the foundations, when this can not be done from the surface of the dam.

(ii)Inspection Galleries

The water which seeps through the body of the dam is collected by means of a system of galleries provided at various elevations and interconnected by vertical shafts, etc. All these galleries, besides draining off seepage water, serves inspection purpose. They provide access to the interior of the dam and are, therefore, called inspection purposes. They generally serve other purposes along with this purpose.

1. They intercept and drain off the water seeping through the dam body
2. They provide access to dam interior for observing and controlling the behavior of the dam.
3. They provide enough space for carrying pipes, etc. during artificial cooling of concrete
4. They provide access to all the outlets and spillway gates, valves, etc. by housing their electrical and mechanical controls. All these gates, valves, etc, can hence be easily controlled by men, from inside the dam itself.

5. They provide space for drilling and grouting of the foundations, then it cannot be done from the surface of the dam.

EARTH DAMS

An **embankment dam** is a large artificial dam. It is typically created by the placementand compaction of a complex semi-plastic mound of various compositions of soil, sand, clay, or rock. It has a semi-pervious waterproof natural covering for its surface and a dense, impervious core. This makes such a dam impervious to surface or seepage erosion.[1] Such a dam is composed of fragmented independent material particles. The friction and interaction of particles binds the particles together into a stable mass rather than by the use of a cementing substance.

Embankment dams come in two types: the **earth-filled dam** (also called an earthen dam or terrain dam) made of compacted earth, and the **rock-filled dam**. A cross-section of an embankment dam shows a shape like a bank, or hill. Most have a central section or core composed of an impermeable material to stop water from seeping through the dam. The core can be of clay, concrete, or asphalt concrete. This dam type is a good choice for sites with wide valleys. They can be built on hard rock or softer soils. For a rock-fill dam, rock-fill is blasted using explosives to break the rock. Additionally, the rock pieces may need to be crushed into smaller grades to get the right range of size for use in an embankment dam

The building of a dam and the filling of the reservoir behind it places a new weight on the floor and sides of a valley. The stress of the water increases linearly with its depth. Water also pushes against the upstream face of the dam, a nonrigid structure that under stress behaves semiplastically, and causes greater need for adjustment (flexibility) near the base of the dam than at shallower water levels. Thus the stress level of the dam must be calculated in advance of building to ensure that its break level threshold is not exceeded.

Overtopping or overflow of an embankment dam beyond its spillway capacity will cause its eventual failure. The erosion of the dam's material by overtopping runoff will remove masses of material whose weight holds the dam in place and against the hydraulic forces acting to move the dam. Even a small sustained overtopping flow can remove thousands of tons of overburden soil from the mass of the dam within hours. The removal of this mass unbalances the forces that stabilize the dam against its reservoir as the mass of water still impounded behind the dam presses against the lightened mass of the embankment, made lighter by surface erosion. As the mass of the dam erodes, the force exerted by the reservoir begins to move the entire structure. The embankment, having almost no elastic strength, would begin to break into separate pieces, allowing the impounded reservoir water to flow between them, eroding and removing even more material as it passes through. In the final stages of failure the remaining pieces of the embankment would offer almost no resistance to the flow of the water and continue to fracture into smaller and smaller sections of earth or rock until these would disintegrates into a thick mud soup of earth, rocks and water.

Therefore, safety requirements for the spillway are high, and require it to be capable of containing a maximum flood stage. It is common for its specifications to be written such that it can contain a five hundred year flood. Recently a number of embankment dam overtopping protection systems have been developed. These techniques include the concrete overtopping protection systems, timber cribs, sheet-piles, riprap and gabions, reinforced earth, minimum energy loss weirs, embankment overflow stepped spillways and the precast concrete block protection systems.

The two principal types of embankment dams are earth and rock-fill dams, depending on the predominant fill material used. Some generalized sections of earth dams showing typical zoning for different types and quantities of fill materials When practically only one impervious material is available and the height of the dam is relatively low, a homogeneous dam with internal drain. The inclined drain serves to prevent the downstream slope from becoming saturated and susceptible to piping and/or slope failure and to intercept and prevent piping through any horizontal cracks traversing the width of the embankment.

Earth Dam with impervious cores, are constructed when local borrow materials do not provide adequate quantities of impervious material. A vertical core located near the center of the dam is preferred over an inclined upstream core because the former provides higher contact pressure between the core and foundation to prevent leakage, greater stability under earthquake loading, and better access for remedial seepage control. An inclined upstream core allows the downstream portion of the embankment to be placed first and the core later and reduces the possibility of hydraulic fracturing. However, for high dams in steep-walled canyons the overriding consideration is the abutment topography. The objective is to fit the core to the topography in such a way to avoid divergence, abrupt topographic discontinuities, and serious geologic defects. For dams on pervious foundations,seepage control is necessary to prevent excessive uplift pressures and piping through the foundation.

The methods for control of under seepage in dam foundations are horizontal drains, cutoffs (compacted backfill trenches, slurry walls, and concrete walls), upstream impervious blankets, downstream seepage berms, toe drains, and relief wells.A rock-fill dam with steep slopes requires better foundation conditions than an earth dam, and a concrete dam (or roller-compacted concrete dam) requires better foundation conditions than a rock-fill dam.

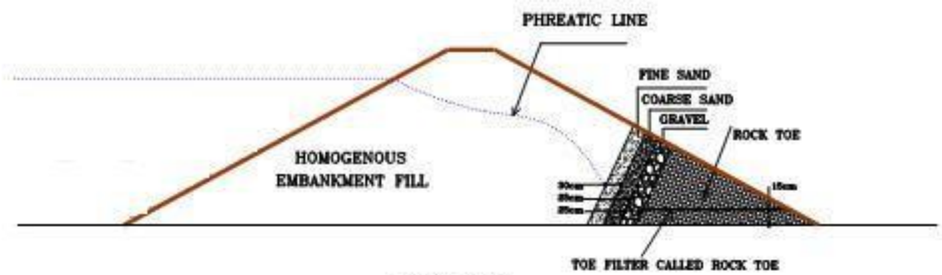
An earth dam is composed of suitable soils obtained from borrow areas or required excavation and compacted in layers by mechanical means. Following preparation of a foundation, earth from borrows areas and from required excavations is transported to the site, dumped, and spread in layers of required depth. The soil layers are then compacted by tamping rollers; sheep foot rollers, heavy pneumatic-tired rollers, vibratory rollers, tractors, or earth-hauling equipment. One advantage of an earth dam is that it can be adapted to a weak foundation, provided proper consideration is given to thorough foundation exploration, testing, and design.

Phreatic Line and Horizontal Drain In Earth fill Dams

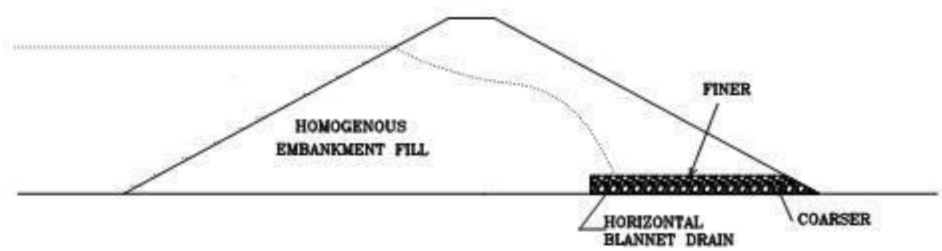
Earth dams are generally built of locally available materials in their natural state with a minimum of processing. Homogeneous earth dams are built whenever only one type of material is economically available.

The material must be sufficiently impervious to provide an adequate water barrier and slopes must be relatively flat to make it safe against piping and sloughing.

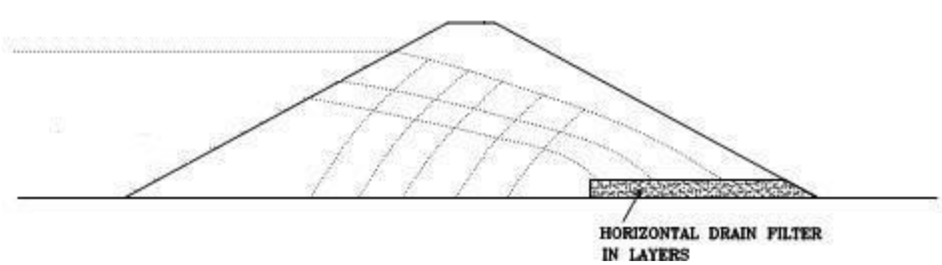
The general design procedure is to make a first estimate on the basis of experience with similar dams and then to modify the estimate as required after conducting a stability analysis except where there is a surplus of material.



Rock Toe



Horizontal Fillter



The upstream slopes of most of the earth dams in actual practice usually vary **from 2.0 (horizontal):1 (vertical) to 4:1** and the downstream slopes are generally between **2:1 and 3:1** (USBR 2003). Free board depends on the height and action of waves. USBR (2003) recommends normal free-board about 1.5 to 3 m depending on the fetch. The width of the dam crest is determined by considering the nature of embankment materials, height and importance of structure, possible roadways requirements, and practicability of construction. A majority of dams have the crest widths varying between 5 and 12 m.

About 30% of dams had failed due to the seepage failure, viz piping and sloughing. Recent comprehensive reviews by Foster et al. (2000a,b) and Fell et al. (2003) show that internal erosion and piping are the main causes of failure and accidents affecting embankment dams; and the proportion of their failures by piping increased from 43% before 1950 to 54% after 1950. The sloughing of the downstream face of a homogeneous earth dam occurs under the steady-state seepage condition due to the softening and weakening of the soil mass when the top flow line or phreatic line intersects it. Regardless of flatness of the downstream slope and impermeability of soil, the phreatic line intersects the downstream face to a height of roughly one-third the depth of water . It is usual practice to use a modified homogeneous section in which an internal drainage system in the form of a horizontal blanket drain or a rock toe or a combination of the two is provided. The drainage system keeps the phreatic line well within the body of the dam. Horizontal filtered drainage blankets are widely used for dams of moderate height.USBR constructed the 50 m high Vega dam, which is one of the highest with a homogenous section and a horizontal downstream drain.

The minimum length of the horizontal blanket drain required to keep the phreatic line within the body of the dam by a specified depth and also equations for maximum downstream slope cover and minimum and maximum effective lengths of the downstream filtered drainage system.

The position of the phreatic line influences the stability of the earth dam because of potential piping due to excessive exit gradient and sloughing due to the softening and weakening of the soil mass as if it touches the downstream slope or intersects it. When the dam embankment is homogeneous or when the downstream zone is of questionable permeability, a horizontal drainage blanket is provided to keep the phreatic line well within the dam body, to allow adequate embankment and foundation drainage, and to eliminate piping from the foundation and the embankment.

As the dams are made of fine-grained soil, saturation may occur due to the capillary rise above the phreatic surface so it is necessary to account for capillary rise while calculating the minimum length of the downstream filtered drainage. Though the suction head in the soil matrix above the phreatic surface within the dam body due to capillary rise generally improves the stability of the downstream slope, once the capillary fringe intersects the downstream slope the pressure changes from negative (suction) to atmospheric and the downstream face may become a seepage face leading to its failure. Hence the phreatic line should not intersect the downstream slope and it should be a distance greater than capillary rise below the sloping face so that the chances of the sloughing or piping may be nullified.

Stability and Failure of Earth Filled Dams

Failure of earth dams may be:

1. Hydraulic Failure
2. Seepage Failure
3. Structural Failure

1. Hydraulic Failure:

1. Overtopping of dams
2. Erosion of the Upstream Surface
3. Erosion of the Downstream Surface
4. Erosion of the Downstream toe

i. Overtopping of dams:

This type of dam is made up of only one type of material. Usually porous materials is used. These dams are easy and cheap to construct but cannot be used to make multipurpose large dams. For large multipurpose dams zoned type method is used. Over topping failures result from the erosive action of water on the embankment. Erosion is due to un-controlled flow of water over, around, and adjacent to the dam. Earth embankments are not designed to be over-topped and therefore are particularly susceptible to erosion. Once erosion has begun during over-topping, it is almost impossible to stop. A well vegetated earth embankment may withstand limited over topping if its crest is level and water flows over the crest and down the face as an evenly distributed sheet without becoming concentrated. The owner should closely monitor the reservoir pool level during severe storms.

ii. Erosion of the Upstream Surface:

Here zones of different materials are made.
Shell is used to give support and stability to the structure of dam. It is made of coarse materials andis pervious in nature.
Core is used to make the dam water tight and to reduce the seepage. Fine material is used here.Used in large dams.

iii. Erosion of the Downstream Surface:

Due to rainfall, snow and winds the downstream surface of the dam also erodes. By providing a section of coarse materials here, this erosion can be reduced or prevented.

2. Seepage Failure:

All earth dams have seepage resulting from water permeating slowly through the dam and its foundation. Seepage must be controlled in both velocity and quantity. If uncontrolled, it can progressively erode soil from the embankment or its foundation, resulting in rapid failure of the dam. Erosion of the soil begins at the downstream side of the embankment, either in the dam proper or the foundation, progressively works toward the reservoir, and eventually develops a direct connection to the reservoir. This phenomenon is known as "piping." Piping action can be recognized by an increased seepage flow rate, the discharge of muddy or discolored water, sinkholes on or near the embankment, or a whirlpool in the reservoir. Once a whirlpool (eddy) is observed on the reservoir surface, complete failure of the dam will probably follow in a matter of minutes. As with over topping, fully developed piping is virtually impossible to control and will likely cause failure. Seepage can cause slope failure by creating high pressures in the soil pores or by saturating the slope. The pressure of seepage within an embankment is difficult to determine without proper instrumentation. A slope which becomes saturated and develops slides may be showing signs of excessive seepage pressure.

Seepage failure of the dams is of the following types

1. Piping through the dam
2. Piping through the foundation
3. Conduit Leakage

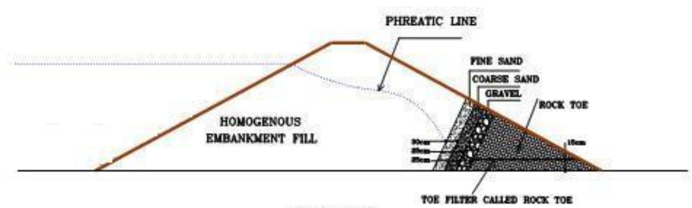
1. **Piping thorugh the dam:** There are two kinds of forces acting on the downstream face of the dam:
    1. Weight of the material
    2. Seepage Force

If the seepage force exceeds the weight of the material the water washes away the soil from theplate and creates a hole in the ground. This hole deepens as more and more mateial is taken away from it and extends longitudinally, making a pipe hole called "Piping in the dam".

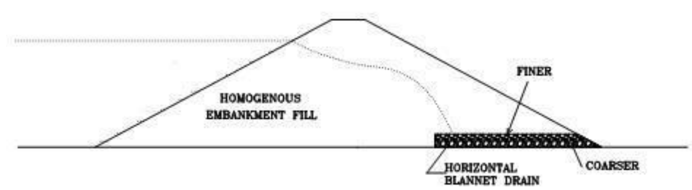
3. Structural Failure:

Structural failures can occur in either the embankment or the appurtenances. Structural failure of a spillway, lake drain, or other appurtenance may lead to failure of the embankment.

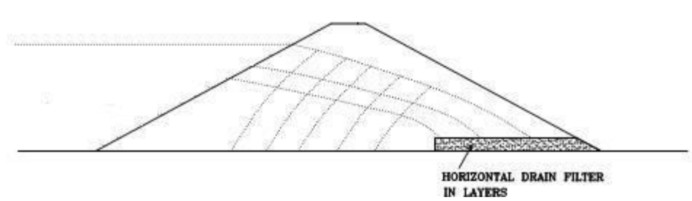
Cracking, settlement, and slides are the more common signs of structural failure of embankments. Large cracks in either an appurtenance or the embankment, major settlement, and major slides will require emergency measures to ensure safety, especially if these problemsoccur suddenly. If this type of situation occurs, the lake level should be lowered, the appropriate state and local authorities notified, and professional advice sought. If theobserver is uncertain as to the seriousness of the problem, the Division of Water should be contacted immediately. The three types of failure previously described



Rock Toe



Horizontal Fillter



are often interrelated in a complex manner. For example, uncontrolled seepage may weaken the soil and lead to a structural failure. A structural failure may shorten the seepage path and lead to a piping failure. Surface erosion may result in structural failure.

1. Failure of downstream face during steady seepage conditions
2. Failure of upstream face during sudden draw down
3. Failure due to sliding of foundation
4. damage due to burrowing animals
5. Failure of dam due to earthquake

1. Usually upper part of the dam is dry and the lower is saturated with water which gives rise to pore water pressure within the voids. Dam body is saturated - All pores / voids are filled with water, pore water pressure is induced. Effective pressure reduces and shear strength of soil decreases
2. When water is suddenly withdrawn or in other words if the level of water in the reservoir reduces suddenly, the soil on the upstream face of the dam body may be highly saturated and has pore water pressure that tries to destabilise the dam and if this force is high enough, it can fail the dam.
3. If the shear strength of the soil on which the foundation is built is weak though the foundation itself may be strong but due to weakness of the soil foundation may slide on the sides and in some cases the foundation itself may be not able to resist the shear force that may have increased from normal due to any reason.
4. Burrowing animals - Small animals living in the holes and pits may have dug theri holes anywhere in te dam body which may widen with the passage of time and can be dangerous.
5. Earthquake

Minor defects such as cracks in the embankment may be the first visual sign of a major problem which could lead to failure of the structure. The seriousness of all deficiencies should be evaluated by someone experienced in dam design and construction. A qualified professional engineer can recommend appropriate permanent remedial measures.