# SKDAV GOVT.POLYTECHNIC ROURKELA

# DEPARTMENT OF CIVIL ENGINEERING

# LECTURE NOTES

**Year & Semester: 2ND Year, IV Semester**
**Subject Code/Name:**
**TH-2, HYDRAULICS & IRRIGATION ENGINEERING**

# HYDRAULICS & IRRIGATION ENGINEERING

## PART:A (HYDRAULICS)

### 1.HYDROSTATICS

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

**Fluid:-**

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

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

**1.1 PROPERTIES OF FLUID:-**

**Density:-**

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

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

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

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

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

**Specific Weight:-**

The specific weight of a fluid is designated by the Greek symbol $\gamma$(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 $\rho$ = density

*Note:* It is customary to use:

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

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

**Relative Density (Specific Gravity):-**

The relative density of any fluid is defined as the ratio of the density of that fluid to the density of the standard fluid. For liquids we take water as a standard fluid with density 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.

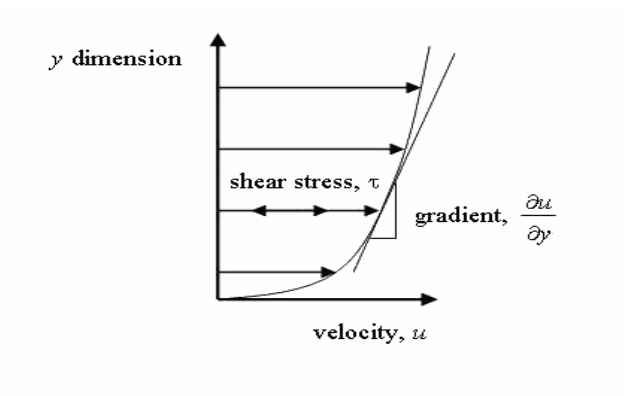



Fig. 1.1

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 ($\nu$, 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 $F=\int PA$

- 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:-**

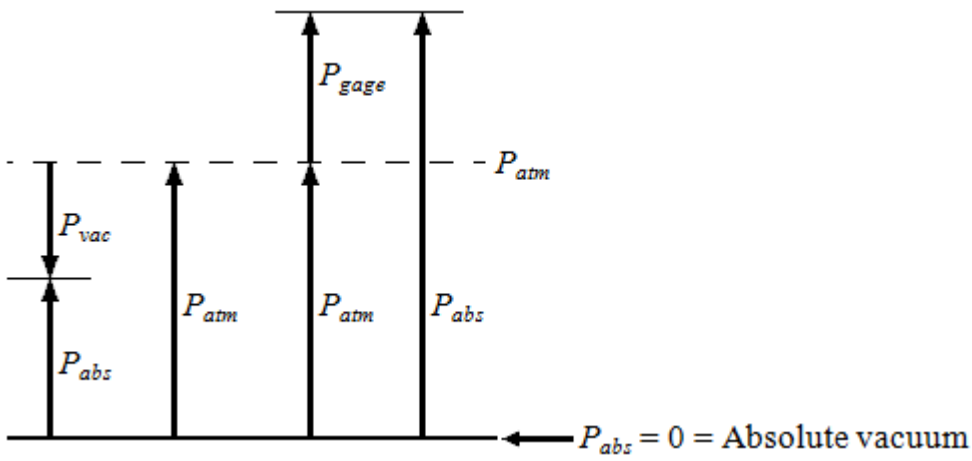



Fig. 1.2

- 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

$\psi$ is pressure head (Length, typically in units of m);
$p$ is fluid pressure (force per unit area, often as Pa units); and
$\gamma$ is the specific weight (force per unit volume, typically N/m$^3$ units)
$\rho$ 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 $\delta A$ 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**,

$P = p_1a_1 + p_2a_2 + p_3a_3 \ldots\ldots\ldots\ldots$

Where,

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

The position of an immersed surface may be,

- Horizontal
- Vertical
- Inclined

**Total Pressure On A Horizontal Immersed Surface**
Consider a plane horizontal surface immersed in a liquid as shown in figure 1.

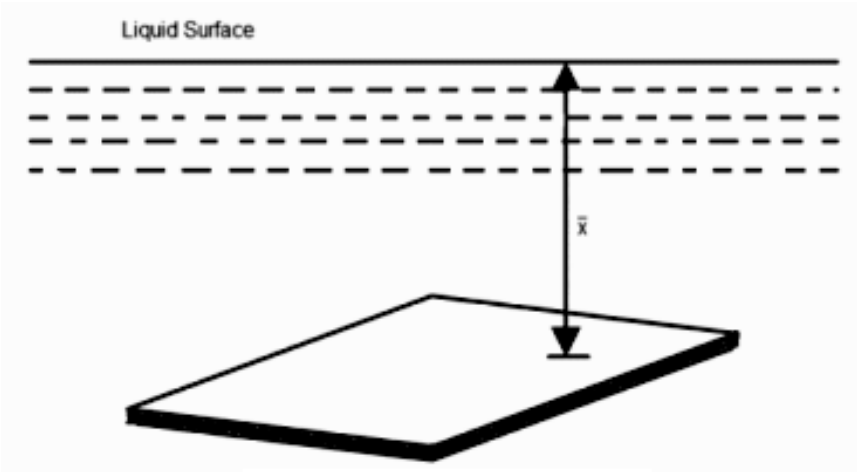



Fig. 1.3

- $\omega$ = Specific weight of the liquid
- $A$ = Area of the immersed surface in $in^2$
- $\chi$ = 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.

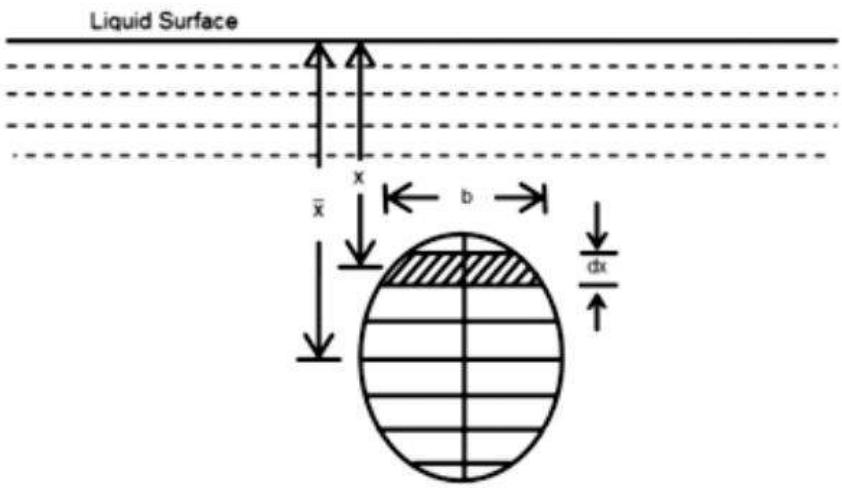



Fig. 1.4

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

Here,

- $\omega =$ Specific weight of the liquid
- A = Total area of the immersed surface
- $\chi$ = 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*

$\therefore$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 = $A\bar{x}$

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

# 2.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 energy; of a particle remains the same, while the particle moves from one point to another." This statement is based on the assumption that there are no "losses due to friction in the pipe. Mathematically,

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

where

Z = Potential energy,

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

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

Proof

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

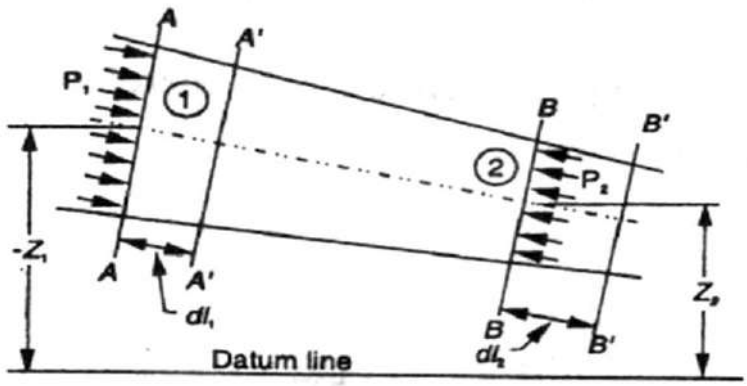



Fig. 2.1

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

Let

$Z_1$ = Height of AA above the datum,

$P_1$= Pressure at AA,

$V_1$ = Velocity of liquid at AA,

$A_1$ = Cross-sectional area of the pipe at AA, and

$Z_2, P_2, V_2, A_2$= Corresponding values at BB.

Let the liquid between the two sections AA and BB move to A' A' and B' B' through very small lengths $dl_1$ and $dl_2$ as shown in Fig. This movement of the liquid between AA and BB is

equivalent to the movement 'of the liquid between AA and A' A' to BB and B' B', the remaining liquid between A' A' and BB being uneffected.
Let W be the weight of the liquid between AA and A' A'. Since the flow is continuous, therefore
$W = wa_1dI_1 = wa_2dL_2$

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

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

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

We know that work done by pressure at AA, in moving the liquid to A' A'
= Force x Distance $= P_1. a_1 . dL_1$
Similarly, work done by pressure at BB, in moving the liquid to B' B'
$=-P_2a_2dl_2$
...(Minus sign is taken as the direction of $P_2$ is opposite to that of $P_1$)
:. Total work done by the pressure
$= P_1a1dl_1- P_2a_2dl_2$
$=P_1a1dl_1-p_{2a}1dl_1$
$\ldots(a_1dl_1=a_2dl_2)$

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

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.

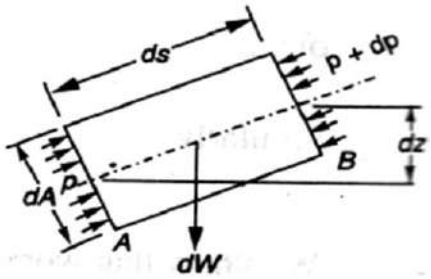
**Euler's Equation For Motion**

The "Euler's equation for steady flow of an ideal fluid along a streamline is based on the Newton's Second Law of Motion. The integration of the equation gives Bernoulli's equation in the form of energy per unit weight of the flowing fluid. It is based on the 'following assumptions:

1. The fluid is non-viscous (i.e., the frictional losses are zero).
2. The fluid is homogeneous and incompressible (i.e., mass density of the fluid is constant).
3. The flow is continuous, steady and along the streamline.
4. The velocity of flow is uniform over the section.
5. No energy or force (except gravity and pressure forces) is involved in the flow.

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

Let
dA = Cross-sectional area of the fluid element,
ds = Length of the fluid element,
dW = Weight of the fluid 5!1ement,
p = Pressure on the element at A,
p + dp = Pressure on the element at B, and
v = Velocity of the fluid element.




We know that the external forces tending to accelerate the fluid

element in the direction of the streamline

= p. dA - (p + dp) dA Fig. 2.2

= -dp.dA

We also know that the weight of the fluid element,

dW = $\rho$ 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

= - $\rho$ g . dA . ds cos$\theta$

= - $\rho$ g . dA . ds($\frac{dz}{ds}$) …cos$\theta$ =$\frac{dz}{ds}$

= - $\rho$ g. dA. dz

$\therefore$ *mass of the fluid element* = $\rho$ .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) - ($\rho$ g . dA . dz-) = $\rho$ . dA . ds $\times \frac{dv}{ds}$

$$\frac{dp}{\rho} + g.dz = v.dv$$ …(dividing both side by - $\rho dA$ )

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

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

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

$\frac{p}{\rho}$ + $g_z+v^2/2$=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.'Given: $d_1$= 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 $a_1.\ V_1 = a_2.\ V_2$

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

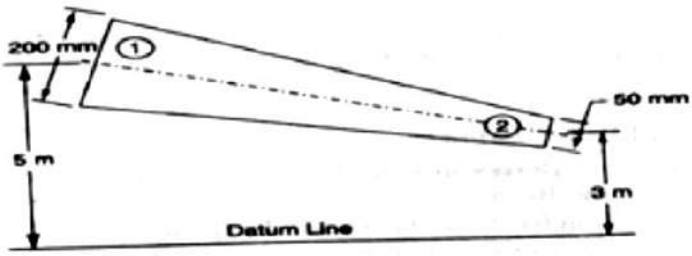



Fig. 2.3

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**

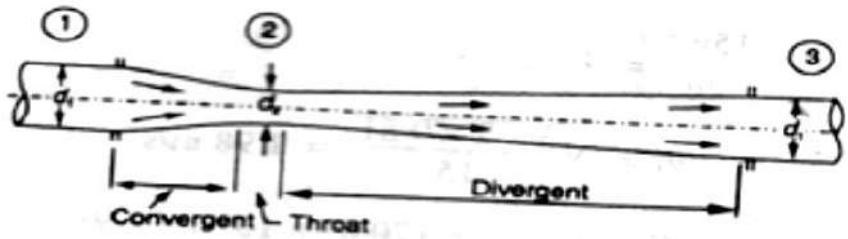



Fig. 2.4

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

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

**(a) Convergent cone**

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

**(b) Throat**

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

**(c) Divergent cone**

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

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

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

**Discharge through a Venturi meter**

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

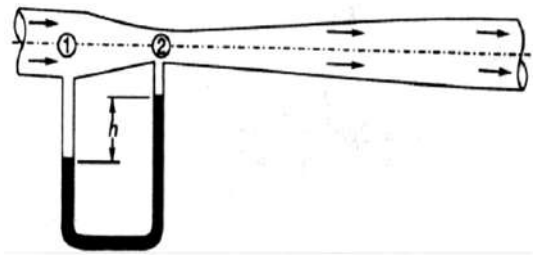



Fig. 2.5

Let

$P_1$ = Pressure at section 1,

$V_1$ = Velocity of water at section 1,

$Z_1$ = Datum head at section 1,

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

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

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

$Z_1 + v_1^2/2g + (p_1/w) = Z_2 + v_2^2/2g + (p_2/w)$ ……..(1)

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

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

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

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

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

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

$V_1^2 = a_2^2v_2^2/a_1^2$ ……..(3)

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

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

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

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

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

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

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

We know that the discharge through a venture meter,

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

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

**Example**

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

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

We know that the area at inlet,

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

and the area at throat,

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

We also know that the difference of pressure head,
H=0.2(13.6-0.9/0.9)=2.82 m of oil
and the discharge through the venturi meter,

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

$=63.9 \text{ X } 10^{-3}m^3/s=63.9lit/s$ 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).
Let
h = Reading of the mercury manometer,
$P_1$ = Pressure at inlet,
$V_1$ = Velocity of liquid at inlet,
$a_1$ = Area of pipe at inlet, and
$P_2,v_2,a_2$= Corresponding values at the throat.

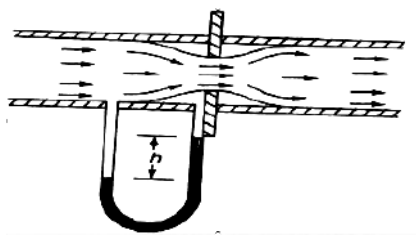



Fig. 2.6

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

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

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

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

Since the discharge is continuous, therefore $a_1.v_1 = a_2v_2$

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

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

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

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

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

**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,

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

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

**Pitot Tube.**

A Pitot tube is an instrument to determine the velocity of flow at the required point in a pipe or a stream. In its simplest form, a pitot tube consists of a glass tube bent a through 90° as shown in Fig.

The lower end of the tube faces the direction of the flow as shown in Fig. The liquid rises up in the tube due to the pressure exerted by the flowing liquid. By measuring the rise of liquid in the tube, we can find out the velocity of the liquid flow.

Let h = Height of the liquid in the pitot tube above the surface,

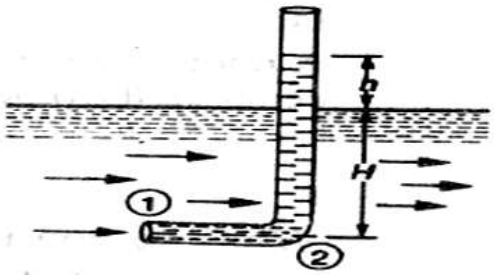



Fig. 2.7

H = Depth of tube in the liquid, and

v = Velocity of the liquid.

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

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

$.....(z_1 = z_2)$

$h = v^2/2g$

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

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

**Equation of Continuity of a Liquid Flow**

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

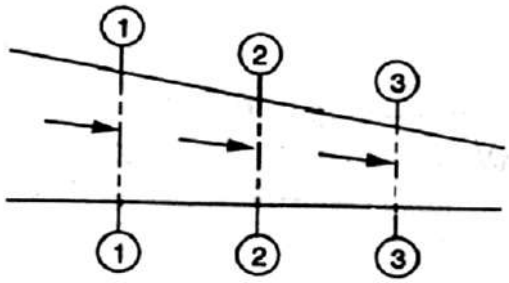



Fig. 2.8

**CONTINUITY OF A LIQUID FLOW**

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

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

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

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

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

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

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

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 = $a_1 v_1$ = $(7.854 \times 10^{-3}) \times 10 = 78.54 \times 10^{-3}\ m^3/s$
= 78.54 litres/s **Ans.**

*Velocity of water at the other end of the pipe*
We also know that cross-sectional area of the pipe at point 2,

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

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

## 2.2 Flow over Notches:-

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

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

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

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

**Classification Of Notches And Weirs:-**

The notches are classified as :

I. According to the shape of the opening:

(a) Rectangular notch,

(b) Triangular notch,

(c) Trapezoidal notch, and

(d) Stepped notch.

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

(a) Notch with end contraction.

lb) Notch without end contraction or suppressed notch e,

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

**Discharge Over A Rectangular Notch Or Weir**

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

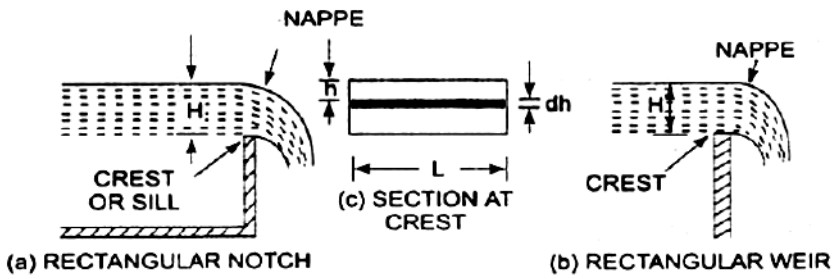



Fig. 2.9

**Rectangular notch and 'weir:-**

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

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

**Problem - 1**

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

Solution. Given:

Length of the notch, L=2.0m

Head over notch, H = 300 m = 0.30 m

$C_d$=0.06

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

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

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

**Problem 2**

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

Solution. Given:

Length of weir, L=6m

Depth of water, H1=1.8m

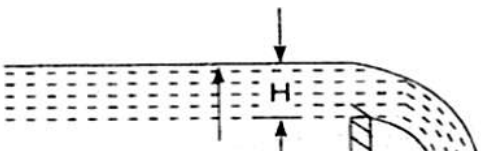
Discharge, Q = 2000 litIs = 2 m3/s

Cd/=0.6

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

The discharge over the weir is given by the equation .

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



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

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

$= H^{3/2}=\frac{2.0}{10.623}$ Fig. 2.10

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

Height of weir, H2 = H1- H

= Depth of water on upstream side - H

= 1.8 - .328 = 1.472 m. Ans.

**Discharge Over A Triangular Notch Or Weir:-**

The expression for the discharge over a triangular notch or weir is the same. It is derived as :
Let H = head of water above the V- notch
$\theta$ = 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, $Q=\frac{8}{15}\times 0.6\times 1\times\sqrt{2\times 9.81}\times H^{5/2}$
$=1.417\times H^{5/2}$

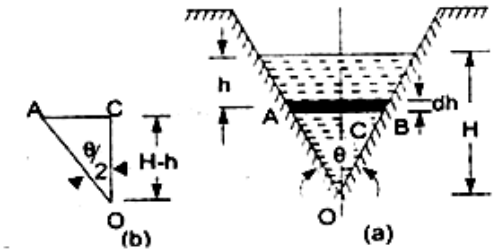


Fig. 2.11

**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.

$\theta$ = 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

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

$$0.10635=\frac{8}{15}\times 0.59 \times \frac{\tan 90}{2} \times \sqrt{2g} \times H_{1^{5/2}} \qquad \{\ \theta = 90^0 \text{ and } H=H_1\ \}$$

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

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

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

$H_1=(0.07631)^{0.4}=0.3572$ 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 $\Delta ABC$ ,we have

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

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

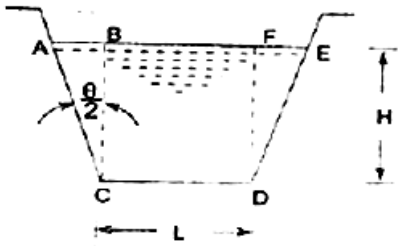



Fig. 2.12

Discharge through trapezoidal notch is given by equation

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

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

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

**Discharge Over A Stepped Notch:-**

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

Consider a stepped notch as shown in Fig.

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

$L_1$ = Length of notch 1,

$H_2$,$L_2$ and $H_3$,$L_3$ are corresponding values for notches 2 and 3 resp

$C_d$=Co-efficient of discharge for all notches

Total discharge $Q=Q_1+Q_2+Q_3$

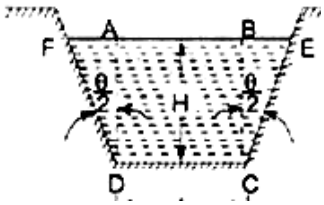
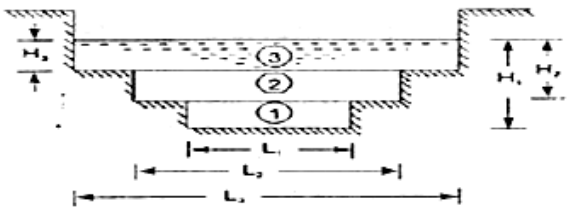



Fig. 2.12

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



**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

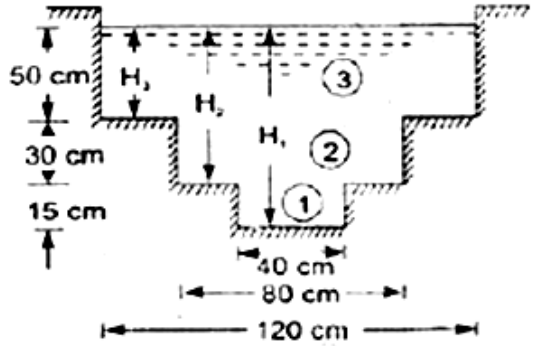



Fig. 2.14

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

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

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

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

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

=530141 cm$^3$/s

=530.144 lit/s

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

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

=776771 cm$^3$/s

=776.771 lit/s

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

=154.067+530.144+776.771

=1460.98 lit/s Ans.

**Velocity Of Approach**

Velocity of approach is defined as the velocity with which the water approaches or reaches the weir or notch before it flows over it. Thus if $V_a$ is the velocity of approach, then an additional head $h_a$ equal

to $V_a^2/2g$ due to velocity of approach, is acting on the water. flowing over the notch. Then initial height of water over the notch becomes $(H+h_a)$ and final height becomes equal to $h_a$,' Then all the formulae are

changed taking into consideration of velocity of approach.

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

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

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

Discharge over a rectangular weir, with velocity of approach

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

**Problem:-**

Water is flowing in a rectangular channel of 1 m wide and 0.75 m deep. Find the discharge over a rectangular weir of crest length 60 cm if the head of water over the crest of weir is 20 cm and water from channel flows over the weir. Take Cd = 0.62. Neglect end contractions. Take velocity of approach into consideration.

Solution. Given:

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

Length of weir, L = 60 cm = 0.6 m

Head of water, $H_1$ = 20 cm = 0.2 m

$C_d$ = 0.62

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

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

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

$= 0.0982\ m^3/s$

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

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

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

Then discharge with velocity of approach is given by equation

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

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

= 1.098 [0.09002- .00002566]

= 1.098 x 0.09017

=0.09881 $m^3/s$. Ans

**Types of Weirs :-**

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

1. Narrow-crested weirs,

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

**Discharge over a Narrow-crested Weir :-**

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

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

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

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

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 :-**

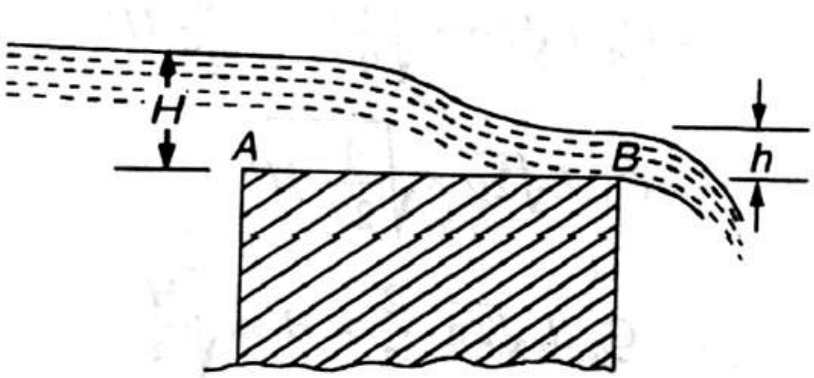



Fig. 2.15

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 -line and then springs up from the crest and falls as a trajectory as shown in Fig.

In a sharp-crested weir, the thickness of the weir is

kept less than half of the height of water on the weir. i.e.,

$$b<(H/2)$$

where , b = Thickness of the weir,

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

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

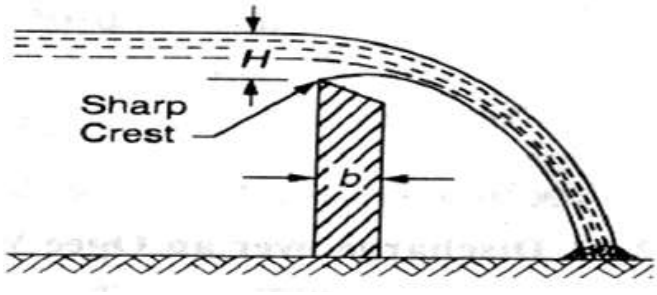



Fig. 2.16

**Sharp-crested weir :-**

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

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

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

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

We know that the discharge over the weir,

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

.

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

$$= 0.0073 \text{ m}^3/\text{s} = 7.3 \text{ litres/s. } \textbf{Ans.}$$

**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

**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,

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

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

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

**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.

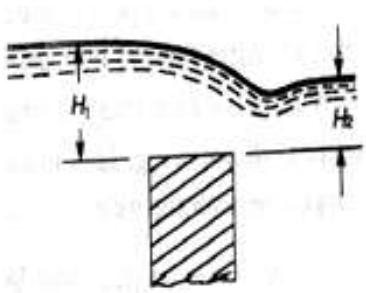



Fig. 2.17

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),

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

**Submerged weir :-**

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

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

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

**Example** A submerged sharp crested weir 0.8 metre high stands clear across a channel having vertical sides and a width of 3 meters. The depth of water in the channel of approach is 1.2

meter. And 10 meters downstream from the weir, the depth of water is 1 meter. Determine the discharge over the weir in liters per second. Take $C_d$ as 0.6.

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

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

of water on the upstream side,

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

$H_2$ = 1 - 0.8 = 0.2 m

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

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

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

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

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

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

## 2.5 Flow through the Open Channels

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 :-**

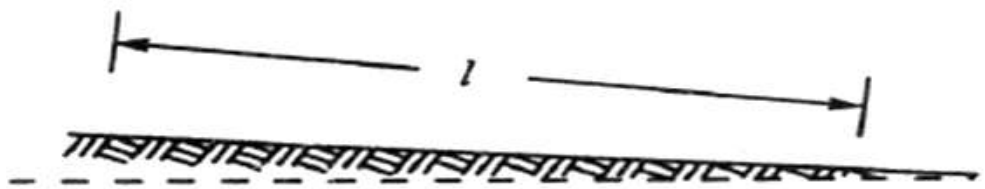



Fig. 2.18

**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 radius )

∴ 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)= 15$m^3$/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

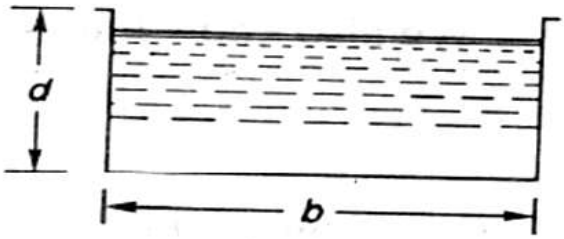



Fig. 2.20

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 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.

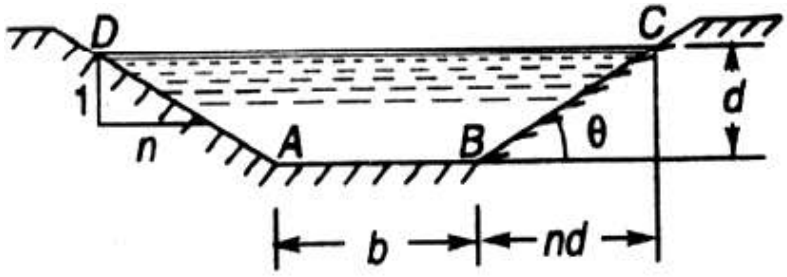



Fig. 2.21

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= 180$m^3$/min = 3$m^3$/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.$\sqrt{mi}$ =$1.866d^{5/2}$

∴ $d^{5/2}$ =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 'of circular section, discharging water under the atmospheric pressure shown in Fig.

Let r = Radius of the channel,

h = Depth of water in the channel, and

$2\theta$ = Total angle (in radians) subtended at the centre by the water

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

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

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

$A = r^2\theta - \frac{r2\sin 2\theta}{2} = r^2(\theta - \frac{\sin 2\theta}{2})$ …(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 $2\theta$ = 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.

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

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

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

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

And velocity of water v= $MXm^{2/3}Xi^{½}$ = 0.71m/s ANS

# 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:

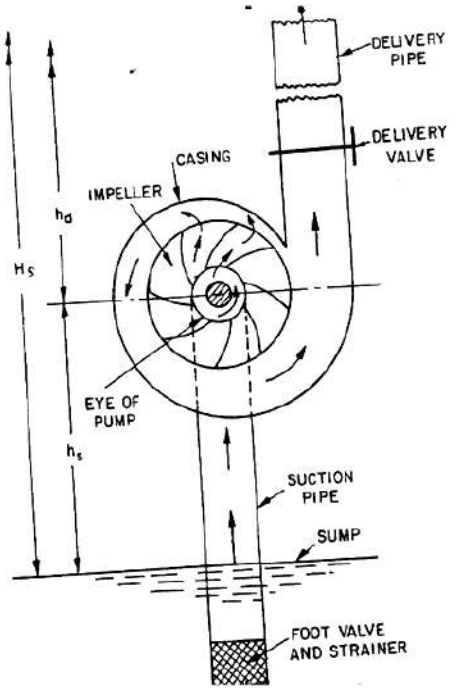
   a. Volute **casing** as shown in Fig.19.1

   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.



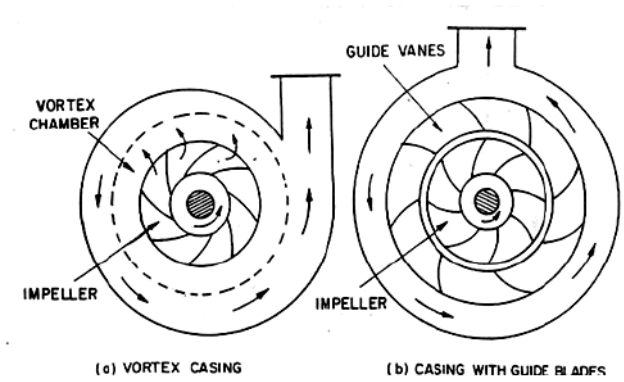

Main parts of a centrifugal pump

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

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

a. Manometric efficiencies $\eta_{man}$

b. Mechanical efficiencies $\eta_m$

c. Overall efficiencies $\eta_o$

a) **Manometric Efficiencies** $\eta_{man}$ **:** 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

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

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 = $\frac{Work\ done\ by\ impeller\ per\ second}{100_0} kW$

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

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

**b) Mechanical efficiencies:-**

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

$$\eta_m = \text{Power at the impeller/Power at the shaft}$$

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** $\eta_o$

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

**E**xternal Dia. Of impeller ,=$D_2$=400mm=0.40m

Speed N=1200r.p.m

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

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

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

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

Tangential velocity of impeller at inlet & outlet are,

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

From inlet velocity triangle,

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

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

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

Speed, N =1000r.p.m

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

Impeller diameter means the diameter of the impeller at outlet

Diameter, $D_2 = 300mm = 0.30m$

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

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

Tangential velocity of impeller at outlet, $u_2 = \frac{\pi D_2 N}{60} = \frac{\pi \times .30 \times 1000}{60} = 15.70 m/s$

Now using equation

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

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

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

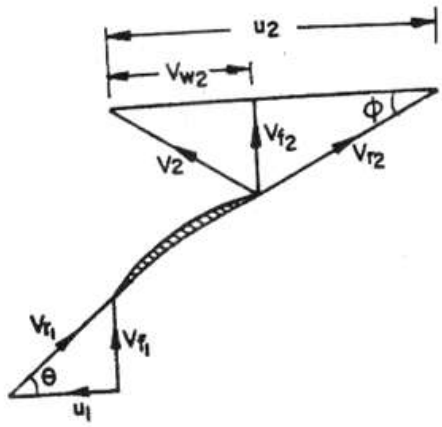



Fig. 3.3

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

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

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

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

$$Discharge = Q = \pi \times D_2 \times B_2 \times V_{f2}$$

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

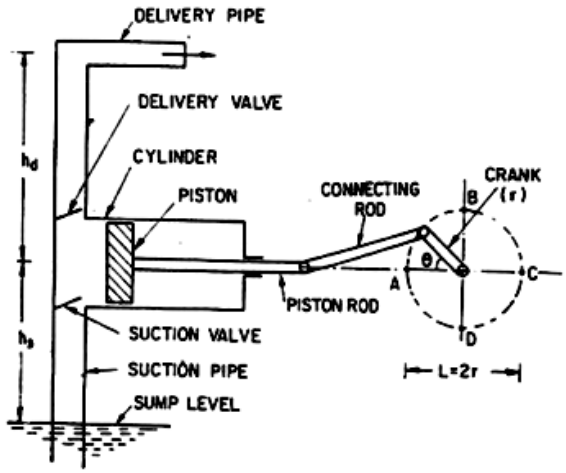
## 3.2 Reciprocating Pump:-

**Introduction:-**

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

**Main parts of a reciprocating pump:-**

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




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

Fig. 3.4

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

Let D= dia. Of the cylinder

A= C/s area of the piston or cylinder

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

r= Radius of crank

N=r.p.m of the crank

L=Length of the stroke=2*r

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

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

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

= Area * Length of stroke = A*L

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

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

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

Wt. of water delivered per second, W = $\rho g Q = \frac{\rho g A L N}{60}$ ……

**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 $W = \frac{\rho gALN}{60}$

Substituting the value of W in equation () we get

Work done per second = $\frac{\rho gALN}{60}(h_s + h_d)$

Power required to drive the pump, in kW

$$P = \frac{Work\ done\ per\ second}{100o}$$

$$= \frac{\rho gALN(h_s + h_d)}{60{,}000} kW$$

**Classification of reciprocating pumps:**

**T**he 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

# PART B (IRRIGATION ENGINEERING)

## INTRODUCTION

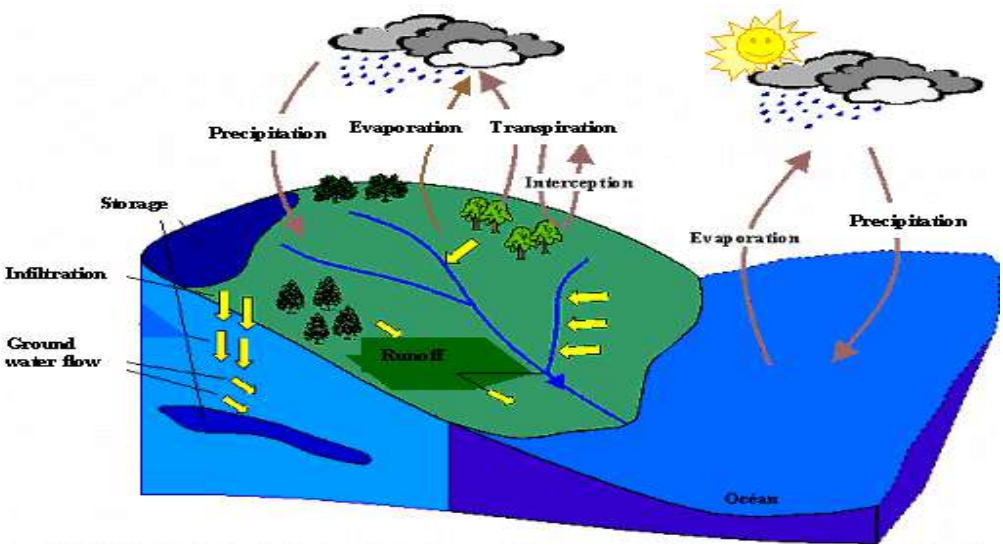
Water on the surface of earth is available in the atmosphere, the oceans, on land and within the soil and fractured rock of the earth’ s crust. Water molecules from one location to another are driven due to the solar energy transmitted to the surface of the earth from Sun. Moisture circulates from the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

## 1.Hydrology:

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

### 1.1Hydrological Cycle:-

The water cycle, also known as the hydrologic cycle or the hydrological cycle, describes the continuous movement of water on, above and below the surface of the Earth. The mass of water on Earth remains fairly constant over time but the partitioning of the water into the major reservoirs of ice, fresh water, saline water and atmospheric water is variable depending on a wide range of climatic variables. The water moves from one reservoir to another, such as from river to ocean, or from the ocean to the atmosphere, by the physical processes of evaporation, condensation, precipitation, infiltration, surface runoff, and subsurface flow. In doing so, the water goes through different forms: liquid, solid (ice) and vapour.




Precipitation

↑

Condensation

↑

Formation of Clouds

↑

Evaporation

↑

Ocean

**PRECIPITATION**

**INTRODUCTION:**

The term “precipitation” denotes all forms of water that reach the earth from the atmosphere. The usual forms are rainfall, snowfall, hail, frost and dew.The magnitude of precipitation varies with time and space. For precipitation to form: (i) the atmosphere must have moisture, (ii) there must be sufficient nuclei present to aid condensation , (iii) weather conditions must be good for condensation of water vapour to take place, and (iv) the products of condensation must reach the earth.

**FORMS OF PRECIPITATION:**

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

*(3)Anticyclones:*

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.

*(4) 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.

*(5) Orographic precipitation:*

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

**MEASUREMENT OF PRECIPITATION**

*1) Rainfall*

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 1km2 represents a volume of water equal to 104 m3. In the case of snowfall, an equivalent depth of water is used as the depth of precipitation. The precipitation is collected and measured in a *rain gauge.* Terms such pluviometer, ombrometer and hyetometer area also sometimes used to designate a rain gauge. A rain gauge essentially consists of a cylindrical vessel assembly kept in the open to collect rain. The rainfall catch of the rain gauge is affected by its exposure conditions. To enable the catch of rain gauge to accurately represent the area in the surrounding the rain gauge standard settings are adopted. For setting up a rain gauge the following considerations are important: The ground must be level and in the open and the instrument must present a horizontal catch surface. The gauge must be set as near the ground as possible to reduce wind effects but it must be sufficiently high to prevent splashing flooding etc. The instrument must be surrounded by an open fenced area of at least 5.5 m * 5.5m. 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 than10cm of rain and as such, in the case of heavy rainfall, the measurements must be done more frequently and entered. However, the last reading must be taken at 8.30 a.m. and the sum of the previous readings in the past 24 hours entered as the total of that day. Proper care, maintenance and inspection of rain-gauges, especially during dry weather to keep the instrument free from dust and dirt, is very necessary. The details of installation of non-recording rain-gauges and measurement of rain are specified in Indian Standard (IS:4986-1968). This rain-gauge can also be used to measure snowfall. When snow is expected, the funnel and receiving bottle are removed and the snow is allowed to collect in the outer metal container. The snow is then melted and the depth of resulting water measured. Antifreeze agents are sometimes used to facilitate melting of snow. In areas where considerable snowfall is expected, special snow-gauges with shields (for minimizing the wind effect) and storage pipes (to collect snow over longer durations) are used.

B. **Recording Gauges**

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

**(a)Tipping-Bucket Type**

This is a 30.5 cm size rain-gauge adopted for use by the US Weather Bureau. The catch from the funnel falls onto one of a pair of small buckets. These buckets 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.

**(c) Natural-Syphon Type**

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

**HYETOGRAPH**

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

**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.

**STREAM:-**

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

**OVERLAND 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.

**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.

 The 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.

**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.

**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.

**RAINFALL EXCESS:**

The part/percentage of precipitation which is equal to the vol. of direct runoff from a basin is called rainfall excess.

Effective rainfall = Direct runoff volume + subsurface runoff = Rainfall excess +Subsurface runoff

 Effective rainfall > Rainfall excess

## EMPERIAL FORMULAE

### Flood Peak area relationship

The simplest relationship is those which relate the flood peak to the drainage area. The max flood discharge Qp from a catchment area A is given by Qp = f (A)

### Dickens formula

Where $Qp = Cd\ A^{3/4}$

Qp= Max flood discharge (m3/s)

Cd = Dickens const

A = Catchment area (km2)

Dickens formula is used in the central and northern parts of the country.

### Ryves formula

$Qp = C_r A^{2/3}$

where Cr = Ryves coefficient

Ryves formula used in Tamilnadu and parts of Karnataka & Andhra Pradesh

# 2. Water Requirement of Crops

## 2.1 Water requirement of crops

For proper growth and maturity of the crops, water is of vital importance throughout the crop period. The water requirement may vary from crop to crop, from soil to soil and from period to period. Again, the total water requirement for a crop is not supplied at a time, but at a fixed interval so that the root zone of the crop may remain saturated throughout the crop period. Generally, the additional requirement is fulfilled by the irrigation system.

**Factor affecting the water requirement**

- Water Table
- Climate
- Ground Slope
- Intensity of Irrigation
- Type of Soil
- Method of Application of Water
- Method of Ploughing

**Crop Season**

The period during which some particular types of crops can be grown every year on the same land is known as crop season. The following are the main crop seasons.

a) Kharif Season

This season ranges from June to October. The crops are sown in the very beginning of monsoon and harvested at the end of autumn.
Ex- Rice, Maize

b) Rabi Season

This season ranges from October to March. The crops are sown in the very beginning of winter and harvested at the end of spring. Ex-Wheat, Gram etc.

**Crop Period:** The crop period is defined as the total period from the time of sowing a crop to the time of harvesting it. That means it is the period in which the crop remains in the field.

**Base Period (B):** When the base period is longer the water requirement will be more and the duty will be low and vice-versa. It defines that period from the first to the last watering of the crop just before its maturity.

**Delta (Δ):** Each crop requires certain amount of water per hectare for its maturity. If the total amount of water supplied to the crop is stored on the land without any loss, then there will be a thick layer of water standing on that land. This water layer depth is delta. Its unit is Cm.

**Duty (D):** The duty is defined as number of hectares that can be irrigated by constant supply of water at the rate of 1 cumec throughout the base period. Its unit is Hectare/cumec.

**Relationship between Duty, Delta and Base Period :**

$$\Delta = 864B/D$$

Where $\Delta$ in cm

B in Days

D in Hectare/cumec

**Consumptive Use of Water:** It is defined as the total quantity of water used for the growth of the plants by transpiration and the amount lost by evaporation. It expressed in hectare-meter/hectare.

**Frequency of Irrigation**: The irrigation water is applied to the field to raise the moisture content of the soil up to its filed capacity. The application of water is then stopped. The water content also reduces gradually due to transpiration and evaporation. If the moisture content is dropped below the requisite amount, the growth of the plants gets disturbed. So the moisture content requires to be immediately replenished by irrigation and it should be raised to the field capacity.

**TIME FACTOR:-** It is the ratio of number of days the canal has actually run to the number of days of irrigation Period.

**Crop Ratio:** It is defined as the ratio of area irrigated in Kharif season to Rabi season.

# Flow Irrigation

**Systems of irrigation:-**

Various types of irrigation techniques differ in how the water obtained from the source is distributed within the field. In general, the goal is to supply the entire field uniformly with water, so that each plant has the amount of water it needs, neither too much nor too little. The various irrigation techniques are as under:

i) Surface irrigation

ii) Sub-surface irrigation

**1) <u>Surface irrigation:-</u>**

a) Flow irrigation

b) Lift irrigation

When the water is available at a higher level and it is supplied to lower level by the mere action gravity, then it is called flow irrigation. But if the water is lifted up by some mechanical (or) manual means such as by pumps etc. and then supplied for irrigation then it is called Lift irrigation. Use of wells and tube wells for supplying water for irrigation falls under the category of Lift irrigation.

**Flow irrigation:-**

i. Perennial irrigation

ii.Flood irrigation

**<u>Perennial Irrigation:</u>**

In perennial system of irrigation constant and continuous water supply is assure to the crops in accordance with the requirements of the crop throughout the crop period. In this system of irrigation water is supplied through canal distribution system taking off above a weir or a reservoir. When irrigation is done by diverting the river runoff into the main canal by constructing a diversion weir or a barrage across the river then it is called direct irrigation but if a dam is constructed across a river to store water during monsoons so as to water in the off taking channel during periods of low flow.

**<u>Flood Irrigation:</u>**

Flood irrigation is also known as inundation irrigation. In this method of irrigation soil is kept submerged and thoroughly flooded with water so as to cause thorough saturation of the land. The moisture soaked by the soil when occasionally supplemented by natural rainfall (or) minor watering.

**<u>Sub surface irrigation:</u>**

It is termed as sub surface irrigation because in this type of irrigation water does not wet the soil surface. The underground water nourishes the plant roots by capillary. It may be divided into the following two types.

 Natural sub irrigation

 Artificial sub irrigation

Natural sub irrigation:

Leakage water from channels etc. goes underground and during passing through the sub soil it may irrigate crops sown on lower lands by capillary.

Artificial sub irrigation:

When a system of open joined drains is artificially laid below the soil so as to apply water to the crops by capillary then it is known as artificial sub irrigation

**Various techniques of distribution of water in the farms:-**

a. Flooding irrigation

b. Drip irrigation

c. Furrow irrigation

d. Sprinkler irrigation

**Flooding irrigation methods:**

 Free flooding method

 Basin flooding method

 Border flooding method

 Check flooding method

## CANAL IRRIGATION

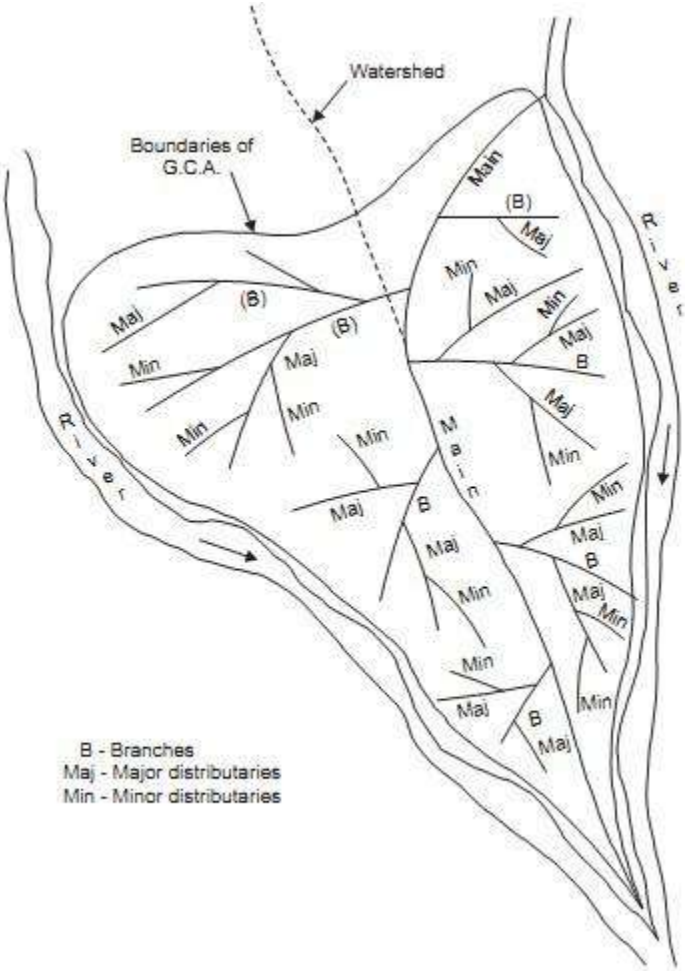
### CANALS:-

A conveyance subsystem for irrigation includes open channels through earth or rock formation, flumes constructed in partially excavated sections or above ground, pipe lines installed either below or above the ground surface, and tunnels drilled through high topographic obstructions. Irrigation conduits of a typical gravity project are usually open channels through earth or rock formations. These are called canals. A canal is defined as an artificial channel constructed on the ground to carry water from a river or another canal or a reservoir to the fields. Usually, canals have a trapezoidal cross-section. Canals can be classified in many ways. Based on the nature of source of supply, a canal can be either a permanent or an inundation canal. A permanent canal has a continuous source of water supply. Such canals are also called perennial canals. An inundation canal draws its supplies from a river only during the high stages of the river. Such canals do not have any head-works for diversion of river water to the canal, but are provided with a canal head regulator. Depending on their function, canals can also be classified as: (i) irrigation, (ii) navigation,

(iii) power, and (iv) feeder canals. An irrigation canal carries water from its source to agricultural fields. Canals used for transport of goods are known as navigation canals. Power canals are used to carry water for generation of hydroelectricity. A feeder canal feeds two or more canals. A canal can serve more than one function. The slope of an irrigation canal is generally less than the ground slope in the head reaches of the canal and, hence, vertical falls have often to be constructed. Power houses may be constructed at these falls to generate power and, thus, irrigation canals can be used for power generation also. Similarly, irrigation canals can also be utilised for the transportation of goods and serve as navigation canals. Inland navigation forms a cheap means of transportation of goods and, hence, must be developed. However, in India, inland navigation has developed only to a limited extent. This is mainly due to the fact that irrigation canals generally take their supplies from alluvial rivers and, as such, must flow with sufficient velocity to prevent siltation of the canal. Such velocities make upstream navigation very difficult. Besides, the canals are generally aligned on the watershed1 so that water may reach the fields on both sides by flow. This alignment may not be suitable for navigation which requires the canal to pass through the areas in the vicinity of industries.

An irrigation canal system consists of canals of different sizes and capacities. Accordingly, the canals are also classified as: (i) main canal, (ii) branch canal, (iii) major distributary,

(iv) minor distributary, and (v) watercourse.




**(Layout of an irrigation canal network)**

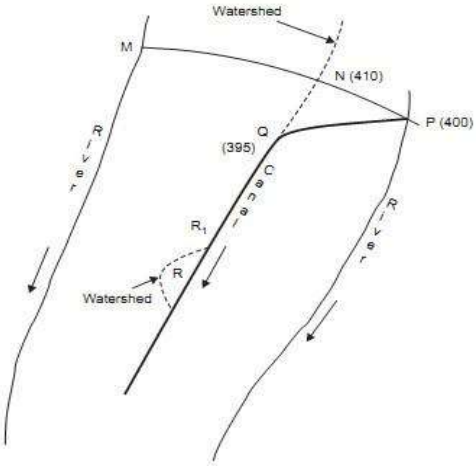
The main canal takes its supplies directly from the river through the head regulator and acts as a feeder canal supplying water to branch canals and major distributaries. Usually, direct irrigation is not carried out from the main canal. Branch canals (also called 'branches') take their supplies from the main canal. Branch canals generally carry a discharge higher than 5 m3/s and act as feeder canals for major and minor distributaries. Large branches are rarely used for direct irrigation. However, outlets are provided on smaller branches for direct irrigation. Major distributaries (also called 'distributaries' or rajbaha) carry 0.25 to 5 m3/s of discharge. These distributaries take their supplies generally from the branch canal and sometimes from the main canal. The distributaries feed either water courses through outlets or minor distributaries. Minor distributaries (also called 'minors') are small canals which carry a discharge less than 0.25 m3/s and feed the watercourses for irrigation. They generally take their supplies from major distributaries or branch canals and rarely from the main canals. A watercourse is a small channel which takes its supplies from an irrigation channel (generally distributaries) through an outlet and carries water to the various parts of the area to be irrigated through the outlet.

## ALIGNMENT OF IRRIGATION CANALS:-

Desirable locations for irrigation canals on any gravity project, their cross-sectional designs and construction costs are governed mainly by topographic and geologic conditions along different routes of the cultivable lands. Main canals must convey water to the higher elevations of the cultivable area. Branch canals and distributaries convey water to different parts of the irrigable areas.

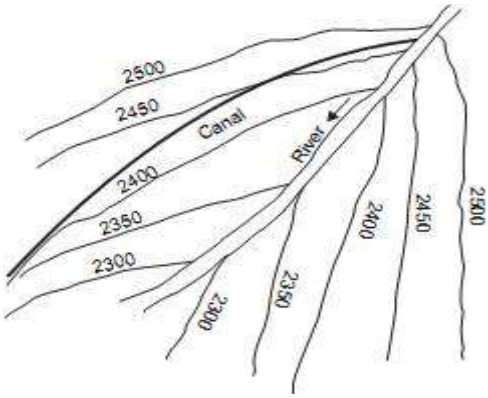
On projects where land slopes are relatively flat and uniform, it is advantageous to align channels on the watershed of the areas to be irrigated. The natural limits of command of such

irrigation channels would be the drainages on either side of the channel. Aligning a canal (main, branch as well as distributaries) on the watershed ensures gravity irrigation on both sides of the canal. Besides, the drainage flows away from the watershed and, hence, no drainage can cross a canal aligned on the watershed. Thus, a canal aligned on the watershed saves the cost of construction of cross-drainage structures. However, the main canal has to be taken off from a river which is the lowest point in the cross-section, and this canal must mount the watershed in as short a distance as possible. Ground slope in the head reaches of a canal is much higher than the required canal bed slope and, hence, the canal needs only a short distance to mount the watershed. This can be illustrated by Fig. 5.2 in which the main canal takes off from a river at P and mounts the watershed at Q. Let the canal bed level at P be 400 m and the elevation of the highest point N along the section MNP be 410 m. Assuming that the ground slope is 1 m per km, the distance of the point Q (395 m) on the watershed from N would be 15km. If the required canal bed slope is 25 cm per km, the length PQ of the canal would be 20 km. Between P and Q, the canal would cross small streams and, hence, construction of cross-drainage structures would be necessary for this length. In fact, the alignment PQ is influenced considerably by the need of providing suitable locations for the cross-drainage structures. The exact location of Q would be determined by trial so that the alignment PQ results in an economic as well as efficient system. Further, on the watershed side of the canal PQ, the ground is higher than the ground on the valley side (i.e., the river side). Therefore, this part of the canal can irrigate only on one side (i.e., the river side) of the canal.




**(Head reach of a main canal in plains)**

Once a canal has reached the watershed, it is generally kept on the watershed, except in certain situations, such as the looping watershed at R in. In an effort to keep the canal alignment straight, the canal may have to leave the watershed near R. The area between the canal and the watershed in the region R can be irrigated by a distributary which takes off at R1and follows the watershed. Also, in the region R, the canal may cross some small streams and, hence, some cross-drainage structures may have to be constructed. If watershed is passing through villages or towns, the canal may have to leave the watershed for some distance. In hilly areas, the conditions are vastly different compared to those of plains. Rivers flow in valleys well below the watershed or ridge, and it may not be economically feasible to take the channel on the watershed. In such situations, contour channels are constructed. Contour channels follow a contour while maintaining the required longitudinal slope. It continues like this and as river slopes are much steeper than the required canal bed slope the canal encompasses more and more area between itself and the river. It should be noted that the more fertile areas in the hills are located at lower levels only.




**(Alignment of main canal in hills)**

In order to finalise the channel network for a canal irrigation project, trial alignments of channels are marked on the map prepared during the detailed survey. A large-scale map is required to work out the details of individual channels. However, a small-scale map depicting the entire command of the irrigation project is also desirable. The alignments marked on the map are transferred on the field and adjusted wherever necessary. These adjustments are transferred on the map as well. The alignment on the field is marked by small masonry pillars at every 200 metres. The centre line on top of these pillars coincides with the exact alignment. In between the adjacent pillars, a small trench, excavated in the ground, marks the alignment.

## Lining of Irrigation Channels

Most of the irrigation channels in India are earthen channels. The major advantage of an earth channel is its low initial cost. The disadvantages of an earth channel are:

(i) the low velocity of flow maintained to prevent erosion necessitates larger cross-section of channels, (ii) excessive seepage loss which may result in water-logging and related problems such as salinity of soils, expensive road maintenance, drainage activities, safety of foundation structures, etc., (iii) favourable conditions for weed growth which further retards the velocity, and (iv) the breaching of banks due to erosion and burrowing of animals. These problems of earth channels can be got rid of by lining the channel.

A lined channel decreases the seepage loss and, thus, reduces the chances of water-logging. It also saves water which can be utilised for additional irrigation. A lined channel provides safety against breaches and prevents weed growth thereby reducing the annual maintenance cost of the channel. Because of relatively smooth surface of lining, a lined channel requires a flatter slope. This results in an increase in the command area. The increase in the useful head is advantageous in case of power channels also. The lining of watercourses in areas irrigated by tube wells assumes special significance as the pumped water supply is more costly. As far as practicable, lining should, however, be avoided on expansive clays. But, if the canal has to traverse a reach of expansive clay, the layer of expansive clay should be removed and replaced with a suitable non-expansive soil and compacted suitably. If the layer of expansive clay is too thick to be completely excavated then the expansive clay bed is removed to a depth of about 60 cm and filled to the grade of the underside of lining with good draining material. The excavated surface of expansive clay is given a coat of asphalt to prevent the entry of water into the clay.

The cost of lining a channel is, however, the only factor against lining. While canal lining provides a cost-effective means of minimising seepage losses, the lining itself may rapidly deteriorate and require recurring maintenance inputs if they are to be effective in controlling seepage loss. A detailed cost analysis is essential for determining the economic feasibility of lining a channel. The true cost of lining is its annual cost rather than the initial cost. The cost of lining is compared with the direct and indirect benefits of lining to determine the economic

feasibility of lining a channel. Besides economic factors, there might be intangible factors such as high population density, aesthetics, and so on which may influence the final decision regarding the lining of a channel.

## Types of Lining

Types of lining are generally classified according to the materials used for their construction. Concrete, rock masonry, brick masonry, bentonite-earth mixtures, natural clays of low permeability, and different mixtures of rubble, plastic, and asphaltic materials are the commonly used materials for canal lining. The suitability of the lining material is decided by:
(i) economy, (ii) structural stability, (iii) durability, (iv) reparability, (v) impermeability, (vi) hydraulic efficiency, and (vii) resistance to erosion (15). The principal types of lining are as follows:

(i)Concrete lining,
(ii)Shotcrete lining,
(iii)Precast concrete lining,
(iv) Lime concrete lining,
(v)Stone masonry lining,
(vi)Brick lining,
(vii)Boulder lining,
(viii)Asphaltic lining,
(ix)Earth lining.

### Concrete Lining

Concrete lining is probably the best type of lining. It fulfils practically all the requirements of lining. It is durable, impervious, and requires least maintenance. The smooth surface of the concrete lining increases the conveyance of the channel. Properly constructed concrete lining can easily last about 40 years. Concrete linings are suitable for all sizes of channels and for both high and low velocities. The lining cost is, however, high and can be reduced by using mechanised methods. The thickness of concrete depends on canal size, bank stability, amount of reinforcement, and climatic conditions. Small channels in warm climates require relatively thin linings.

Channel banks are kept at self-supporting slope (1.5H: 1V to 1.25H: 1V) so that the lining is not required to bear earth pressures and its thickness does not increase. Concrete linings are laid without form work and, hence, the workability of concrete should be good. Also, experienced workmen are required for laying concrete linings.

### Shotcrete Lining

Shotcrete lining is constructed by applying cement mortar pneumatically to the canal surface. Cement mortar does not contain coarse aggregates and, therefore, the proportion of cement is higher in shotcrete mix than in concrete lining. The shotcrete mix is forced under pressure through a nozzle of small diameter and, hence, the size of sand particles in the mix should not exceed 0.5 cm. Equipment needed for laying shotcrete lining is light, portable, and of smaller size compared to the equipment for concrete lining. The thickness of the shotcrete lining may vary from 2.5 to 7.5 cm. The preferred thickness is from 4 to 5 cm. Shotcrete lining is suitable for: (a) lining small sections, (b) placing linings on irregular surfaces without any need to prepare the subgrade, (c) placing linings around curves or structures, and (d) repairing

badly cracked and leaky old concrete linings. Shotcrete linings are subject to cracking and may be reinforced or unreinforced. Earlier, shotcrete linings were usually reinforced. A larger thickness of shotcrete lining was preferred for the convenient placement of reinforcement. The reinforcement was in the form of wire mesh. In order to reduce costs, shotcrete linings are not reinforced these days, particularly on relatively small jobs.

**Precast Concrete Lining**

Precast concrete slabs, laid properly on carefully prepared sub-grades and with the joints effectively sealed, constitute a serviceable type of lining. The precast slabs are about 5 to 8cm thick with suitable width and length to suit channel dimensions and to result in weights which can be conveniently handled. Such slabs may or may not be reinforced. This type of lining is best suited for repair work as it can be placed rapidly without long interruptions in canal operation. The side slopes of the Tungabhadra project canals have been lined with precast concrete slabs.

**Lime Concrete Lining**

The use of this type of lining is limited to small and medium size irrigation channels with capacities of up to 200 m3/s and in which the velocity of water does not exceed 2 m/s (16). The materials required for this type of lining are lime, sand, coarse aggregate, and water. The lime concrete mix should be such that it has a minimum compressive strength of about 5.00 kN/m2 after 28 days of moist curing. Usually lime concrete is prepared with 1 : 1.5 : 3 of kankar lime : kankar grit or sand : kankar (or stone or brick ballast) aggregate. The thickness of the lining may vary from 10 to 15 cm for discharge ranges of up to 200 m3/s. Lime concrete lining has been used in the Bikaner canal taking off from the left bank of the Sutlej.

**Stone Masonry Lining**

Stone masonry linings are laid on the canal surface with cement mortar or lime mortar. The thickness of the stone masonry is about 30 cm. The surface of the stone masonry may be smooth plastered to increase the hydraulic efficiency of the canal. Stone masonry linings are stable, durable, erosion-resistant, and very effective in reducing seepage losses. Such lining is very suitable where only unskilled labour is available and suitable quarried rock is available at low price. This lining has been used in the Tungabhadra project.

**Brick Lining**

Bricks are laid in layers of two with about 1.25 cm of 1:3 cement mortar sandwiched in between. Good quality bricks should be used and these should be soaked well in water before being laid on the moistened canal surface. Brick lining is suitable when concrete is expensive and skilled labour is not available. Brick lining is favoured where conditions of low wages, absence of mechanisations, shortage of cement and inadequate means of transportation exist. Brick linings have been extensively used in north India. The Sarda power channel has been lined with bricks. The thickness of the brick lining remains fixed even if the sub-grade is uneven. Brick lining can be easily laid in rounded sections without form work. Rigid control in brick masonry is not necessary. Sometimes reinforced brick linings are also used.

## Boulder Lining

Boulder lining of canals, if economically feasible, is useful for preventing erosion and where the ground water level is above the bed of the canal and there is a possibility of occurrence of damaging back pressures. The stones used for boulder linings should be sound, hard, durable, and capable of sustaining weathering and water action. Rounded or sub-angular river cobbles or blasted rock pieces with sufficient base area are recommended types of stones for boulder lining.

## Asphaltic Lining

The material used for asphaltic lining is asphalt-based combination of cement and sand mixed in hot condition. The most commonly used asphaltic linings are: (a) asphaltic concrete, and (b) buried asphaltic membrane. Asphaltic linings are relatively cheaper, flexible, and can be rapidly laid in any time of year. Because of their flexibility, minor movements of the sub- grade are not of serious concern. However, asphaltic linings have short life and are unable to permit high velocity of flow. They have low resistance to weed growth and, hence, it is advisable to sterilise the sub-grade to prevent weed growth. Asphaltic concrete is a mixture of asphalt cement, sand, and gravel mixed at a temperature of about 110°C and is placed either manually or with laying equipment. Experienced and trained workmen are required for the purpose. The lining is compacted with heavy iron plates while it is hot. A properly constructed asphaltic concrete lining is the best of all asphaltic linings. Asphaltic concrete lining is smooth, flexible, and erosion-resistant. Since asphaltic concrete lining becomes distorted at higher temperatures, it is unsuitable for warmer climatic regions. An asphaltic concrete lining is preferred to a concrete lining in situations where the aggregate is likely to react with the alkali constituents of Portland cement.

Buried asphaltic membrane can be of two types:

(a) Hot-sprayed asphaltic membrane, and
(b) Pre-fabricated asphaltic membrane.

## Earth Linings

Different types of earth linings have been used in irrigation canals. They are inexpensive but require high maintenance expenditure. The main types of earth linings are: (a) stabilised earth linings, (b) loose earth blankets, (c) compacted earth linings, (d) buried bentonite membranes, and (e) soil-cement linings.

Stabilised earth linings: Stabilised earth linings are constructed by stabilizing the Sub-grade. This can be done either physically or chemically. Physically stabilised linings are constructed by adding corrective materials (such as clay for granular subgrade) to the subgrade, mixing, and then compacting. If corrective materials are not required, the subgrade can be stabilised by scarifying, adding moisture, and then compacting. Chemically stabilised linings use chemicals which may tighten the soil. Such use of chemicals, however, has not developed much.

# Water logging

**Water logging**: It is a form of natural flooding that occurs with over irrigation and water that rises from underground levels to the surface.

**Causes:**

- Inadequate drainage of the overland runoff increases the rate of percolation and in turn helps in raising the water table.
- The water from rivers may infiltrate into the soil.
- Seepage of water from earthen canals also adds significant quantity of water to the underground reservoir.
- Sometimes sub soil does not permit free flow of subsoil water which may accentuate the process of raising the water table.

**Effects**:

- Creation of anaerobic condition in the crop root zone.
- Growth of water loving wild plants.
- Impossibility of Tillage operations.
- Accumulation of Harmful salts.
- Lowering of soil temperature.
- Reduction in time of maturity.

**Anti water-logging Measure:**

- Lining of channels
- Provision of surface drain for drainage of rain water
- Implementation of Tube well projects both extensive and local.

In order to harness the water potential of a river optimally, it is necessary to construct two types of hydraulic structures

1. **Storage structure**: Usually a dam, which acts like a reservoir for storing excess runoff of a river during periods of high flows (as during the monsoons) and releasing it according to a regulated schedule.

2. **Diversion structure**: It may be a weir or a barrage that raises the water level of the river slightly, not for creating storage, but for allowing the water to get diverted through a canal situated at one or either of its banks. Since a diversion structure does not have enough storage, it is called a run-of-the river scheme. The diverted water passed through the canal may be used for irrigation, industry domestic water needs or power generation.

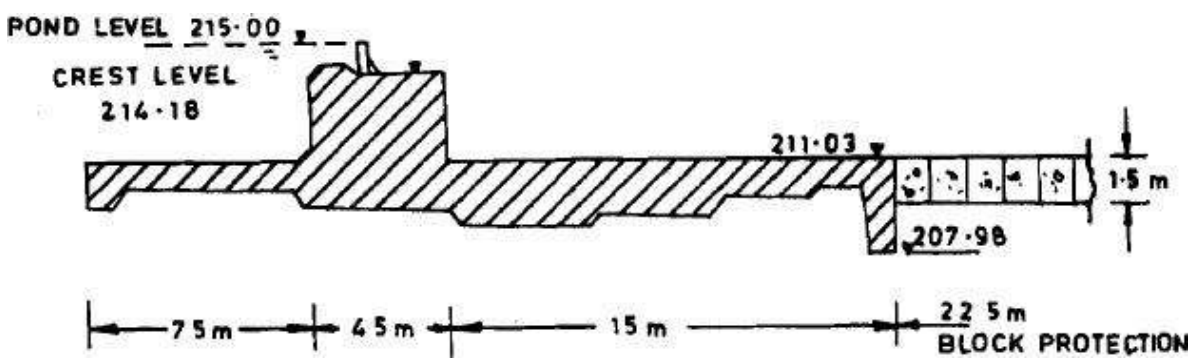
## DIVERSION HEAD WORKS

**Weir:** A low dam built across a river to raise the level of water upstream or regulate its flow.

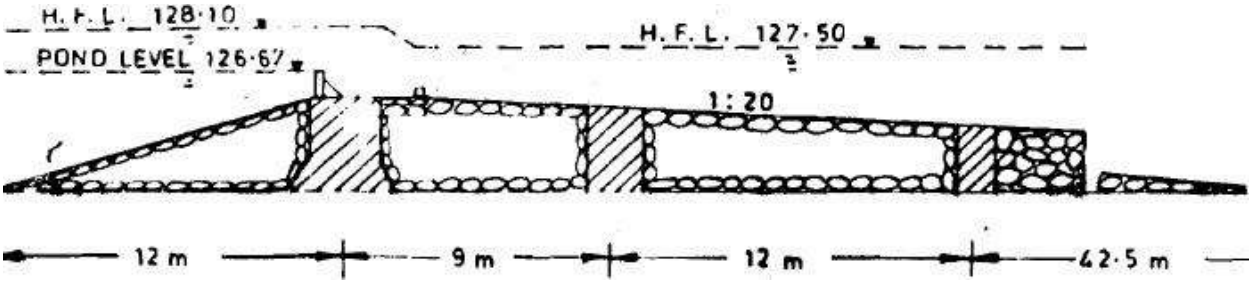
**Barrage:** An artificial barrier across a river or estuary to prevent flooding, aid irrigation or navigation, or to generate electricity by trial power.

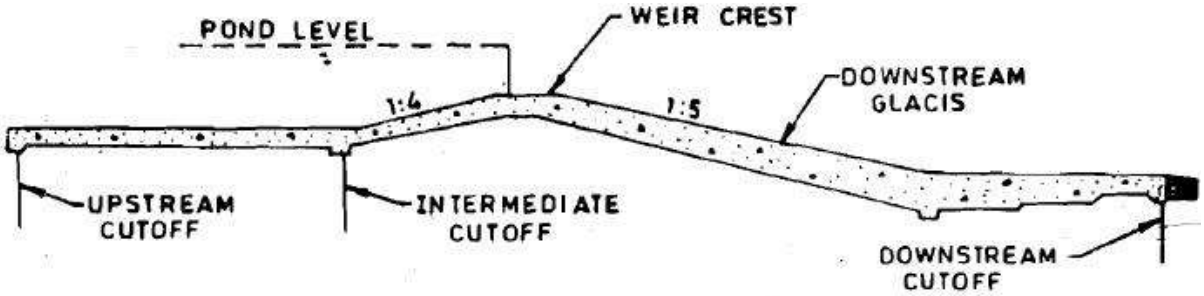
| Weir | Barrage |
|---|---|
| Low cost | High cost |
| Low control on flow | Relatively high control on flow and water levels by operation of gates |
| No provision for transport communication across the river | Usually, a road or a rail bridge can be conveniently and economically combined with a barrage wherever necessary |
| Chances of silting on the upstream is more | Silting may be controlled by judicial operation of gates |
| Afflux created is high due to relatively high weir crests | Due to low crest of the weirs (the ponding being done mostly by gate operation), the afflux during high floods is low. Since the gates may be lifted up fully, even above the high flood level. |

**Classification of Weir:**
There are mainly 3 types,



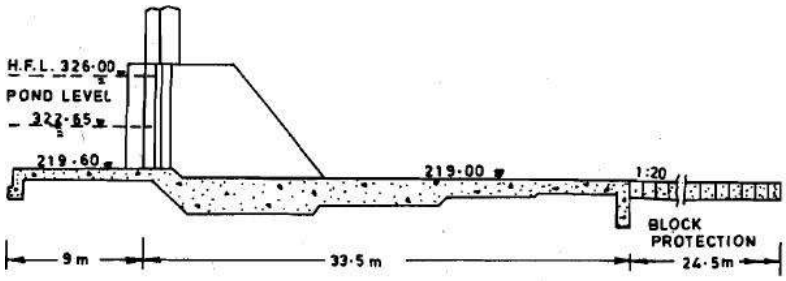

- Masonary wier with vertical drop
- Rockfill wier with sloping apron



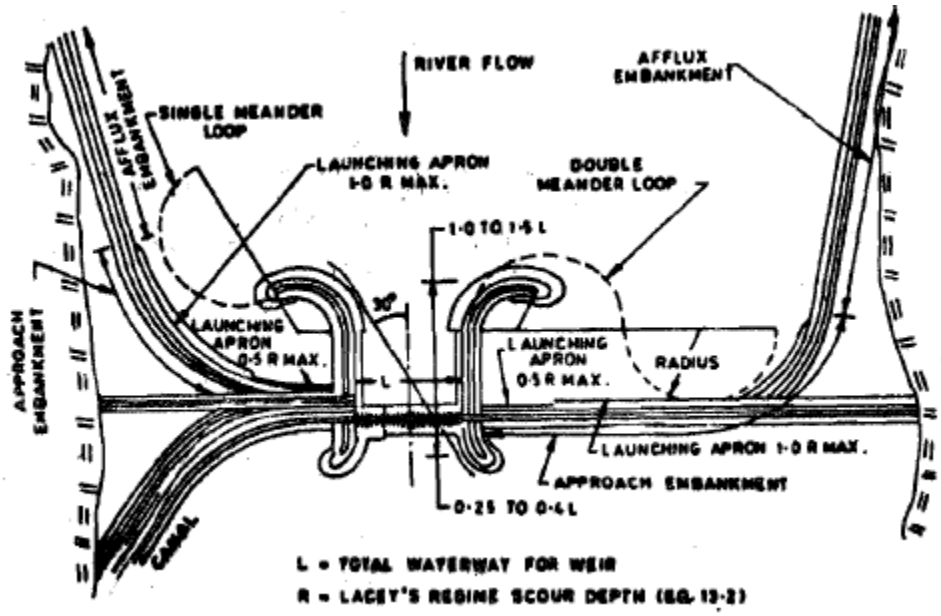

- Concrete weir with sloping glacis downstream

**Barrage: (**c/s of a barriage)




**Components of Weir:**

1. Weir /barrage divided into number of bays
2. Under sluices
3. Divide wall/groyne
4. Fish ladder
5. Canal head regulator
6. River training works




(Layout of canal head regulator with river training work)

1. **Spillway bays**:

This is the main body of the barrage for controlling the discharges and to raise the water level to the desired value to feed the canals. It is a reinforced concrete structure designed as a raft foundation supporting the weight of the gates, piers and the bridge above to prevent sinking into the sandy river bed foundation.

2. **Undersluice :** These low crested bays may be provided on only one flank or on both flanks of the river depending upon whether canals are taking-off from one or both sides. The width of theundersluice portion is determined on the basis of the following considerations.

- It should be capable of passing at least double the canal discharge to ensure good scouring capacity
- It should be capable of passing about 10 to 20 percent of the maximum flood discharge at high floods.

It should be wide enough to keep the approach velocities sufficiently lower than critical velocities to ensure maximum settling of suspended silt load.

**3. Divide wall:**

The divide wall is much like a pier and is provided between the sets of undersluice or river sluice or spill bays. The main functions of a divide wall:

- It separates the turbulent flood waters from the pocket in front of the canal head.
- It helps in checking parallel flow (to the axis of the barrage) which would be caused by the formation of deep channels leading from the river to the pocket in front of the sluices.

**4. Fish pass/ladder:**

Some barrages require providing special structures to allow migratory fishes to flow up and down the river through structures called Fish Passes or Fish Locks.

## 5. Canal Head Regulator:

The water that enters a canal is regulated through a Head Regulator. A typical cross section through a regulator is shown in Figure 9. As it is desirable to exclude silt as much as possible from the head regulator, the axis of the head regulator is laid out at 900-1100 an angle from to the barrage axis as recommended in Bureau of Indian Standards code IS : 6531(1972).

## 6. River training works :

The river training works for barrages are required to achieve the following;

1.Prevent out flanking of the structure

2.Minimize cross flows through the barrage

3.Prevent flooding by the river lands upstream

4.Provide favorable curvature of flow at the head regulator from the point of sediment entry into the canal, and Guide the river to flow axially through the barrage or weir

# Cross Drainage Work

**Cross Drainage Work**: A cross drainage work is a structure carrying the discharge from a natural stream across a canal intercepting the stream. Canal comes across obstructions like rivers, natural drains and other canals. The various types of structures that are built to carry the canal water across the above mentioned obstructions or vice versa are called cross drainage works. It is generally a very costly item and should be avoided by diverting one stream into another. The changing the alignment of the canal so that it crosses below the junction of two streams.

## Types of cross drainage works:

Depending upon levels and discharge, it may be of the following types,

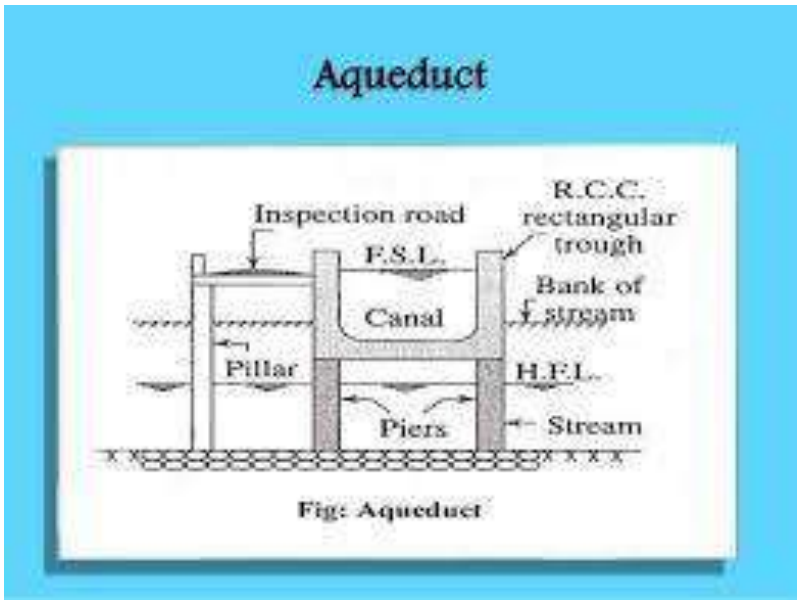
Cross drainage works carrying canal across the drainage the structures that fall under this type are:

**1. Aqueduct**

**2. Siphon Aqueduct**:

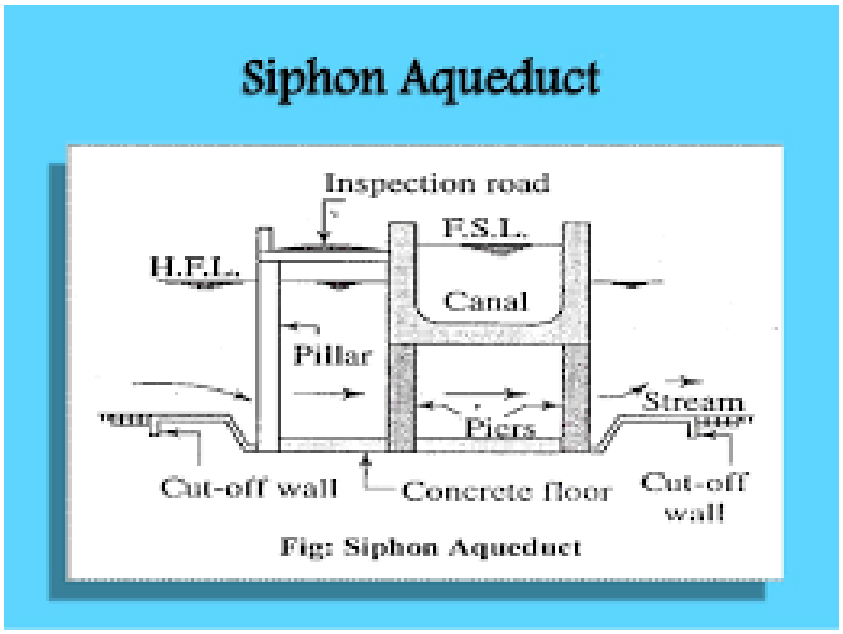
When the HFL of the drain is sufficiently below the bottom of the canal such that the drainage water flows freely under gravity, the structure is known as Aqueduct.

- In this, canal water is carried across the drainage in a trough supported on piers.
- Bridge carrying water
- Provided when sufficient level difference is available between the canal and natural and canal bed is sufficiently higher than HFL.



**Siphon Aqueduct:**

In case of the siphon Aqueduct, the HFL of the drain is much higher above the canal bed, and water runs under siphonic action through the Aqueduct barrels. The drain bed is generally depressed and provided with pucca floors, on the upstream side, the drainage bed may be joined to the pucca floor either by a vertical drop or by glacis of 3:1. The downstream rising slope should not be steeper than 5:1. When the canal is passed over the drain, the canal remains open for inspection throughout and the damage caused by flood is rare. However during heavy floods, the foundations are susceptible to scour or the waterway of drain may get choked due to debris, tress etc.

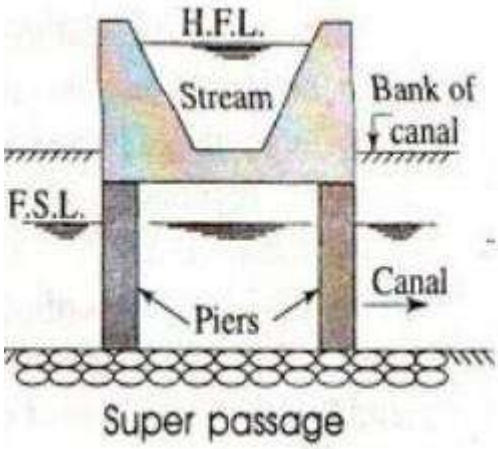



**Cross drainage works carrying drainage over canal**. The structures that fall under this type are:

1. **Super passage**
2. **Canal siphon or siphon**

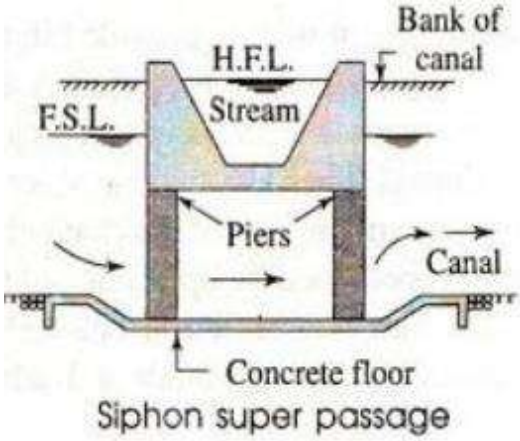
**Super passage:** The hydraulic structure in which the drainage is passing over the irrigation canal is known as super passage. This structure is suitable when the bed level of drainage is above the flood surface level of the canal. The water of the canal passes clearly below the drainage

1. A super passage is similar to an aqueduct, except in this case the drain is over the canal.
2. The FSL of the canal is lower than the underside of the trough carrying drainage water. Thus, the canal water runs under the gravity.
3. Reverse of an aqueduct.




**Canal Syphon:**

- If two canals cross each other and one of the canals is siphoned under the other, then the hydraulic structure at crossing is called "canal siphon". For example, lower Jhelum canal is siphoned under the Rasul-Qadirabad (Punjab, Pakistan) link canal and the crossing structure is called "L.J.C siphon"
- In case of siphon the FSL of the canal is much above the bed level of the drainage trough, so that the canal runs under the siphonic action.
- The canal bed is lowered and a ramp is provided at the exit so that the trouble of silting is minimized.
- Reverse of an aqueduct siphon
- In the above two types, the inspection road cannot be provided along the canal and a separate bridge is required for roadway.



# DAMS

## CLASSIFICATION OF DAMS

Dams may be classified into a number of different categories, depending upon the purpose of the classification. For the purposes of this manual, it is convenient to consider three broad classifications: Dams are classified according to their use, their hydraulic design, or the materials of which they are constructed.

Classification According to Use.-Dams may be classified according to the broad function they serve, such as storage, diversion, or detention. Refinements of these classifications can also be made by considering the specific functions involved. Storage dams are constructed to impound water during periods of surplus supply for use during periods of deficient supply. These periods may be seasonal, annual, or longer. Many small dams impound the spring runoff for use in the dry summer season.

Storage dams may be further classified according to the purpose of the storage, such as water supply, recreation, fish and wildlife, hydroelectric power generation, irrigation, etc. The specific purpose or purposes to be served by a storage dam often influence the design of the structure and may establish criteria such as the amount of reservoir fluctuation expected or the amount of reservoir seepage permitted. Figure 4-l shows a small earth-fill storage dam, and figure 4-2 shows a concrete gravity structure serving both diversion and storage purposes.

Diversion dams are ordinarily constructed to provide head for carrying water into ditches, canals, or other conveyance systems. They are used for irrigation developments, for diversion from a live stream to an off-channel-location storage reservoir, for municipal and industrial uses, or for any combination of the above.

Detention dams are constructed to retard flood runoff and minimize the effect of sudden floods. Detention dams consist of two main types. In one type, the water is temporarily stored and released through an outlet structure at a rate that does not exceed the carrying capacity of the channel downstream.

In the other type, the water is held as long as possible and allowed to seep into pervious banks or into the foundation. The latter type is sometimes called a water-spreading dam or dike because its main purpose is to recharge the underground water supply. Some detention dams are constructed to trap sediments; these are often called debris dams.

Although it is less common on small projects than on large developments, dams are often constructed to serve more than one purpose. Where multiple purposes are involved, a reservoir allocation is usually made to each distinct use. A common multipurpose project combines storage, flood control, and recreational uses.

**Classification by Hydraulic Design.-**

Dams may also be classified as overflow or non-overflow dams.Overflow dams are designed to carry discharge over their crests or through spillways along the crest. Concrete is the most common material used for this type of dam. Non-overflow dams are those designed not to be overtopped. This type of design extends the choice of materials to include earth-fill and rock-fill dams.

Often the two types are combined to form a composite structure consisting of, for example, an overflow concrete gravity dam with earth-fill dikes.

**Classification by Materials.-**The most common classification used for the discussion of design procedures is based upon the materials used to build the structure. This classification also usually recognizes the basic type of design, for example, the "concrete gravity" dam or the "concrete arch" dam.

This text is limited in scope to consideration of the more common types of dams constructed today; namely, earth-fill, rock-fill, and concrete gravity dams. Other types of dams, including concrete arch, concrete buttress, and timber dams, are discussed briefly with an explanation of why their designs are not covered in this text.

**Earth-fill Dams.-**Earth-fill dams are the most common type of dam, principally because their construction involves the use of materials from required excavations and the use of locally available natural materials requiring a minimum of processing. Using large quantities of required excavation and locally available borrow are positive economic factors related to an earth-fill dam. Moreover, the foundation and topographical requirements for earth-fill dams are less stringent than those for other types. It is likely that earth fill dams will continue to be more prevalent than other types for storage purposes, partly because the number of sites favorable for concrete structures is decreasing as a result of extensive water storage development. This is particularly true in arid and semiarid regions where the conservation of water for irrigation is a fundamental necessity.

Although the earth-fill classification includes several types, the development of modern excavating, hauling, and compacting equipment for earth materials has made the rolled-fill type so economical as to virtually replace the semi-hydraulic-fill and hydraulic-fill types of earth-fill dams. This is especially true for the construction of small structures, where the relatively small amount of material to be handled precludes the establishment of the large plant required for efficient hydraulic operations.

**Rock-fill Dams.-**Rock-fill dams use rock of all sizes to provide stability and an impervious membrane to provide water tightness. The membrane may be an upstream facing of impervious soil, a concrete slab, asphaltic-concrete paving, steel plates, other impervious elements, or an interior thin core of impervious soil. Like the earth embankments, rock-fill dams are subject to damage or destruction by the overflow of water and so must have a spillway of adequate capacity to prevent overtopping. An exception is the extremely low diversion dam where the rock-fill facing is designed specifically to withstand overflows.

Rock-fill dams require foundations that will not be subject to settlements large enough to rupture the watertight membrane. The only suitable foundations, therefore, are rock or compact sand and gravel.

The rock-fill type dam is suitable for remote locations where the supply of good rock is ample, where the scarcity of suitable soil or long periods of high rainfall make construction of an earth-fill dam impractical, or where the construction of a concrete dam would be too costly. Rock-fill dams are popular in tropical climates because their construction is suitable for long periods of high rainfall.

**Concrete Gravity Dams.-**Concrete gravity dams are suitable for sites where there is a reasonably sound rock foundation, although low structures may be founded on alluvial foundations if adequate cutoffs are provided. They are well suited for use as overflow spillway crests and, because of this advantage, are often used as spillways for earth-fill or rock-fill dams or as overflow sections of diversion dams.

Gravity dams may be either straight or curved in plan. The curved dam may offer some advantage in both cost and safety. Occasionally the dam curvature allows part of the dam to be located on a stronger foundation, which requires less excavation. The concept of constructing concrete dams using RCC (roller-compacted concrete) has been developed and implemented. Several RCC dams have been constructed in the United States and in other countries. The technology and design procedures, however, are not presented in this manual because procedures and approaches are relatively new and are still being developed.

**Concrete Arch Dams.-**Concrete arch dams are suitable for sites where the ratio of the width between abutments to the height is not great and where the foundation at the abutments is solid rock capable of resisting arch thrust. Two types of arch dams are defined here: the single and the multiple arch dam. A single arch dam spans a canyon as one structure and is usually limited to a maximum crest length to height ratio of 10:1. Its design may include small thrust blocks on either abutment, as necessary, or a spillway somewhere along the crest. A multiple arch dam may be one of two distinct designs. It may have either a uniformly thick cylindrical barrel shape spanning 50 feet or less between buttresses, such as Bartlett Dam in Arizona, or it may consist of several single arch dams supported on massive buttresses spaced several hundred feet on centers. The dam's purpose, whether it be a permanent major structure with a life expectancy of 50 years or a temporary cofferdam with a useful life of 5 years, will directly influence the time for design and construction, the quality of materials in the dam and foundation, the foundation treatment, and the hydraulic considerations. Structural and economic aspects prohibit the design of an arch dam founded on stiff soil, gravel, or cobblestones.

Uplift usually does not affect arch dam stability because of the relative thinness through the section, both in the dam and at the concrete rock contact.

**Concrete Buttress Dams.-**Buttress dams are comprised of flat deck and multiple arch structures.

They require about 60 percent less concrete than solid gravity dams, but the increased formwork and reinforcement steel required usually offset the savings in concrete. A number of buttress dams were built in the 1930"s, when the ratio of labor costs to material costs was comparatively low. The cost of this type of construction is usually not competitive with that of other types of dams when labor costs are high.

The design of buttress dams is based on the knowledge and judgment that comes only from specialized experience in that field. Because of this fact and because of the limited application for buttress dams under present-day conditions, their design is not covered in this text.

**Other Types. -**Dams of types other than those mentioned above have been built, but in most cases they meet some unusual local requirement or are of an experimental nature. In a few instances, structural steel has been used both for the deck and for the supporting framework of a dam. And before 1920, a number of timber dams were constructed, particularly in the Northwest. The amount of labor involved in the timer dam, coupled with the short life of the structure, makes this type of structure uneconomical for modern construction.

## Concrete Gravity Dams

**Origin and Development** : A concrete gravity dam is proportioned so that its own weight provides the major resistance to the forces exerted upon it. If the foundation is adequate and the dam is properly designed and constructed, the concrete dam will be a permanent structure that requires little maintenance. Gravity dams of un-cemented masonry were built several thousand years B.C. Evidence found in archeological sites indicate dam base widths as much as four times the height. With the passing of centuries, various types of mortar have been used to bind the masonry, thereby increasing stability and water tightness and permitting steeper slopes to be used. Concrete and cement mortar were used in the construction of cyclopean masonry dams, the forerunners of the modern mass concrete gravity dams.

As an alternative to the conventional method of placing block upon block of mass concrete, RCC (roller-compacted concrete) is fast becoming an accepted method of constructing concrete gravity dams. An RCC dam is constructed in much the same way as an embankment dam. Zero slump concrete is placed, spread, and compacted with vibratory rollers in l- to 2-foot-thick lifts that are continuous between abutments. Because the RCC construction method is quicker and requires less labor, it is more cost efficient than conventionally placed mass concrete. Some of the concerns associated with the RCC construction method are bond strength and permeability along lift surfaces, cooling requirements, and incorporating transverse contraction joints. Because experience with RCC is still limited, improvements and changes are anticipated.

### Stability Analysis of Gravity Dam:

The stability analysis of gravity dams may be carried out by various methods, of which the gravity method is described here. In this method, the dam is considered to be made up of a number of vertical cantilevers which act independently for each other. The resultant of all horizontal and vertical forces including uplift should be balanced by an equal and opposite reaction at the foundation consisting of the total vertical reaction and the total horizontal shear and friction at the base and the resisting shear and friction of the passive wedge, if any. For the dam to be in static equilibrium, the location of this force is such that the summation of moments is equal to zero. The distribution of the vertical reaction is assumed as trapezoidal for convenience only. Otherwise, the problem of determining the actual stress distribution at the base of a dam is complicated by the horizontal reaction, internal stress relations, and other theoretical considerations. Moreover, variation of foundation materials with depth, cracks and fissures which affect. stresses and foundation pressures should be computed both with and without uplift to determine the worst condition.

The stability analysis of a dam section is carried out to check the safety with regard to

1. **Rotation and overturning**

2. **Translation and sliding**

3. **Overstress and material failure**

**1.Stability against overturning**

Before a gravity dam can overturn physically, there may be other types of failures, such as cracking of the upstream material due to tension, increase in uplift, crushing of the toe material and sliding. However, the check against overturning is made to be sure that the total stabilizing moments weigh out the de-stabilizing moments. The factor of safety against overturning may be taken as 1.5. As such, a gravity dam is considered safe also from the point of view of overturning if there is no tension on the upstream face.

**2.Stability against sliding**

Many of the loads on the dam act horizontally, like water pressure, horizontal earthquake forces, etc. These forces have to be resisted by frictional or shearing forces along horizontal or nearly-horizontal seams in foundation. The stability of a dam against sliding is evaluated by comparing the minimum total available resistance along the critical path of sliding (that is, along that plane or combination of plans which mobilizes the least resistance to sliding) to the total magnitude of the forces tending to induce sliding.

**3.Failure against overstressing:**

A dam may fail if any of its part is overstressed and hence the stresses in any part of the dam must not exceed the allowable working stress of concrete. In order to ensure the safety of a concrete gravity dam against this sort of failure, the strength of concrete shall be such that it is more than the stresses anticipated in the structure by a safe margin. The maximum compressive stresses occur at heel (mostly during reservoir empty condition) or at toe (at reservoir full condition) and on planes normal to the face of the dam. The strength of concrete and masonry varies with age, the kind of cement and other ingredients and their proportions in the work can be determined only by experiment.

## Earth Dams

Earthen dams are still cheaper as they utilize the locally available materials and les skilled labor is required for them, as they build with the natural materials with a minimum processing and primitive equipments.

Types of Earthen Dams:

- **Homogeneous embankment type**
- **Zoned embankment type**
- **Diaphragm type**

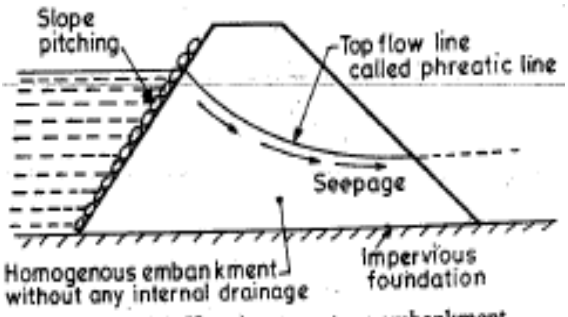


Fig. 20.1 (*a*). Homogeneous type embankment.

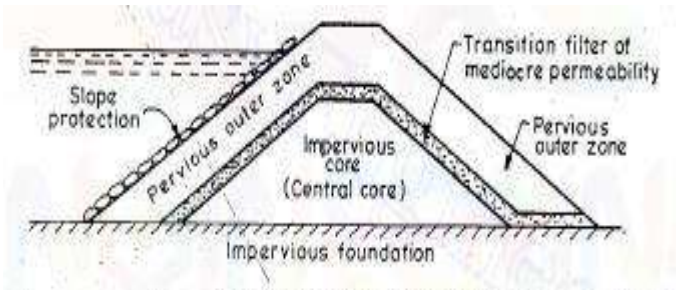


Fig. 20.2. Zoned type embankment.

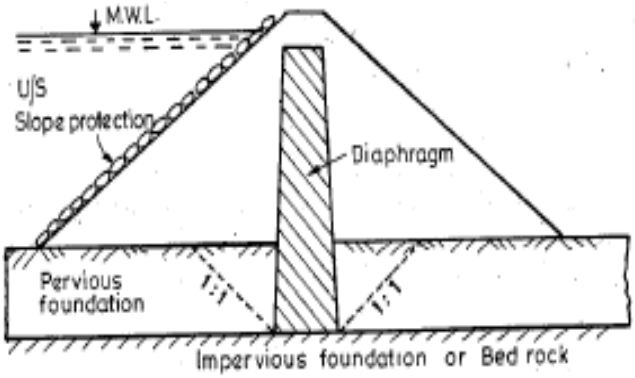


Fig. 20.3. Diaphragm type embankment.

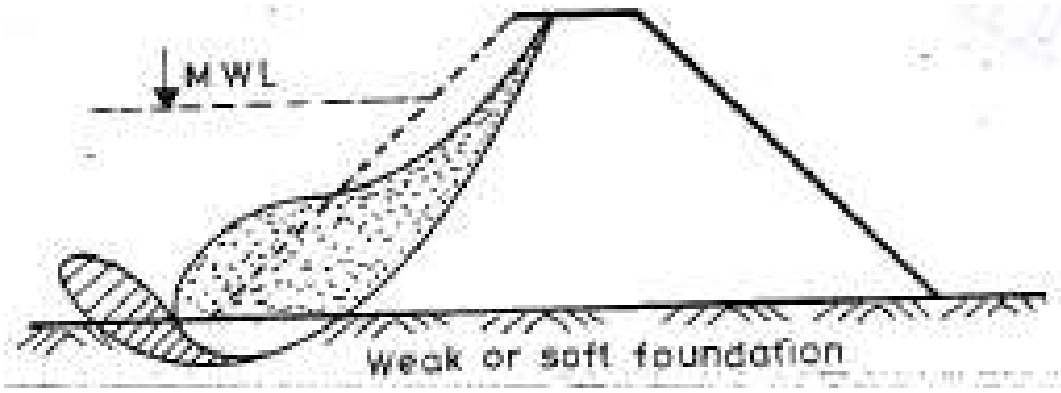
**Causes of Failure of Earthen Dams:**

Earth dams are less rigid and hence more susceptible to failure.

1. **Hydraulic Failures**
    - By over topping
    - Erosion of upstream face
    - Cracking due to frost action
    - Erosion of downstream face by gully formation
2. **Seepage Failures**
    - Piping through foundations
    - Piping through the dam body
    - Sloughing of downstream toe

**3. Structural Failures**

- Foundation slide




- Slide in embankment:

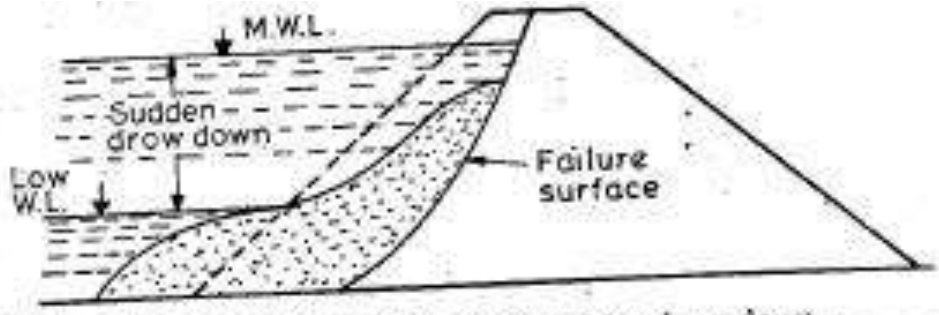



Fig. 20.9. U/S slope slide due to sudden draw-down.

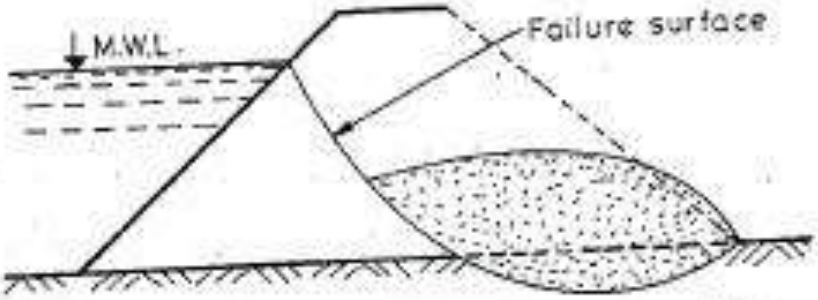



Fig. 20.10. D/S. slope slide during full reservoir condition.

# Spillways

A spillway is a passage constructed either within a dam or in the periphery of the reservoir to safely pass the excess of the flood water during flood flows effectively from upstream to downstream. Depending upon the inflow rate, water will start rising above the normal pool level, at same water release by the spillways also. If water level increases over the maximum flood level, it ultimately overtopped the dam by causing failure of dam. So, spillway is an essential for safety concern.

The surface of the spillway should also be such that it is able to withstand erosion or scouring due to the very high velocities generated during the passage of a flood through the spillway. At the bottom of the channel, where the water rushes out to meet the natural river, is usually provided with an *energy dissipation device* that kills most of the energy of the flowing water. These devices, commonly called as *Energy Dissipators*, are required to prevent the river surface from getting dangerously scoured by the impact of the out falling water.

Usually, spillways are provided with gates, which provide a better control on the discharges passing through. The capacity of a spillway is usually worked out on the basis of a flood routing study and depends on following major factors,

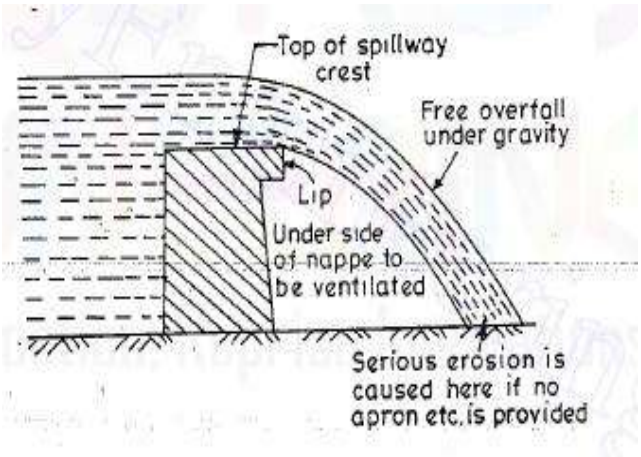
- The inflow flood
- The volume of storage provided by the reservoir
- Crest height of the spillway
- Gated or ungated

## Types of spillways:

1. Free Overfall (Straight Drop) Spillway
2. Overflow (Ogee) Spillway
3. Chute (Open Channel/Trough) Spillway
4. Side Channel Spillway
5. Shaft (Drop Inlet/Morning Glory) spillway
6. Tunnel (Conduit) spillway
7. Siphon spillway

### 1-Free Overfall / Straight drop Spillway:

In this type of spillway, the water freely drops down from the crest, as for an arch dam (Figure 1) also for a decked over flow dam with a vertical or adverse inclined downstream face (Figure 2). Flows may be free discharging, as will be the case with a sharp-crested weir or they may be supported along a narrow section of the crest. Water freely falls from crest under *the action of gravity*. Since vacuum is created in the under-side portion of the falling jet, sufficient ventilation of nappe is required in order to avoid *pulsating and fluctuating effects of the jet*.

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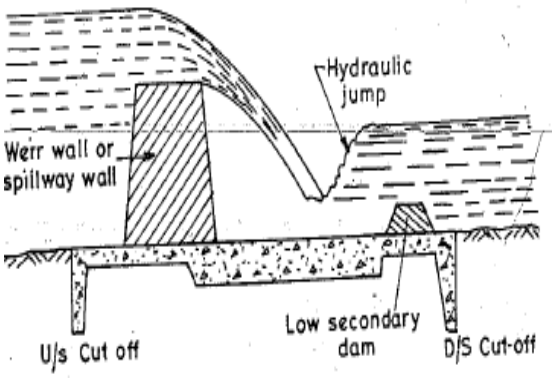


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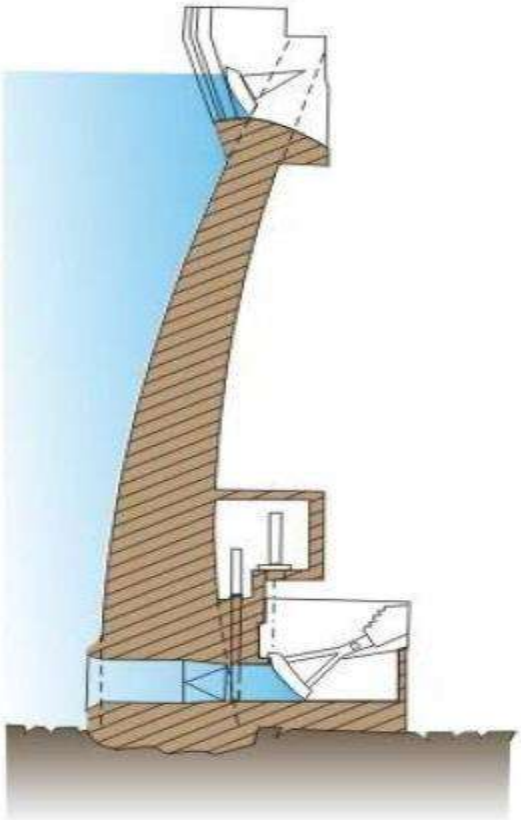
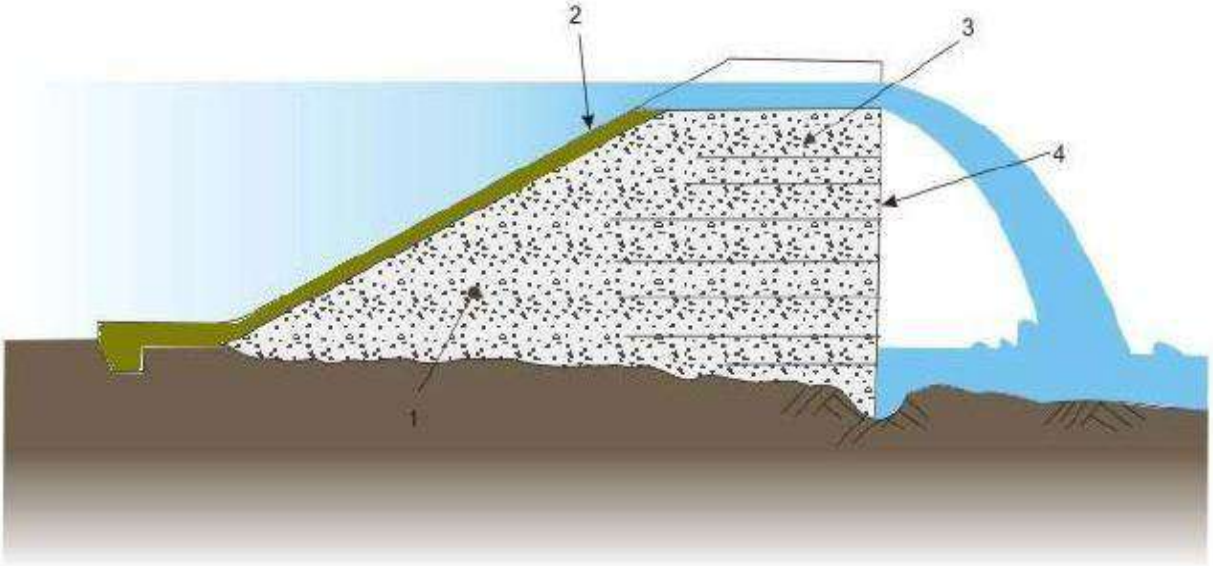


FIGURE 1. Free over fall spillway for an arch dam




LEGEND

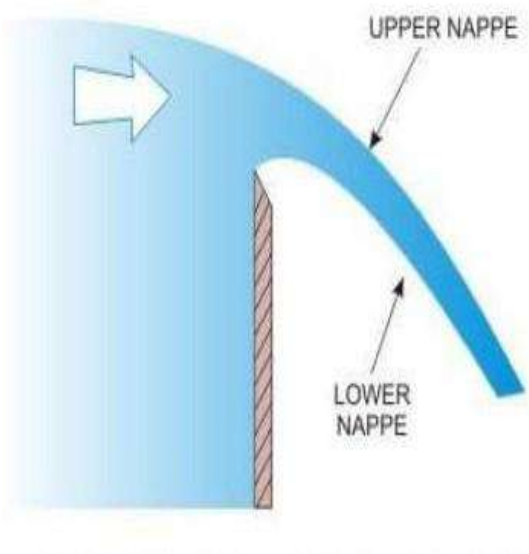
1. RANDOM FILL 2. WATERTIGHT MEMBRANE 3. STEEL TENDONS 4. CONCRETE SLABS (1.5 M X 1.5 M).

FIGURE 2. Free over fall spillway for a decked embankment dam

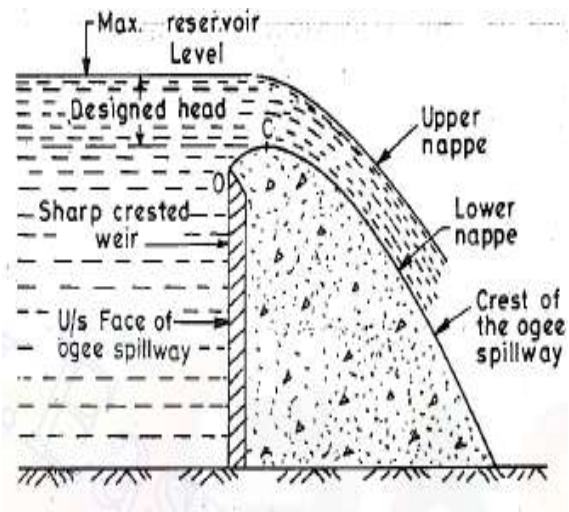
## 2- Overflow (Ogee) Spillway:

This type of spillway is the most common type adopted in the field. It divides naturally into three zones i.e. Crest, spillway face and the toe. The concept evolves from replacing the lower nappy of the flow over thin plate weir by solid boundary. The overflow type spillway has a crest shaped in the form of *an ogee or S-shape*. The upper curve of the ogee is made to conform closely to the profile of the lower nappy of a ventilated sheet of water falling from a sharp crested weir (figure 3). Flow over the crest of an overflow spillway is made to adhere to the face of the profile by preventing access of air to the underside of the sheet of flowing water.

Naturally, the shape of the overflow spillway is designed according to the shape of the lower nappe of a free flowing weir conveying the discharge flood any discharge higher than the design flood passing through the overflow spillway would try to shoot forward and get detached from the spillway surface, which reduces the efficiency of the spillway due to the presence of negative pressure between the sheet of water and spillway surface. For discharges at designed head, the spillway attains near-maximum efficiency.



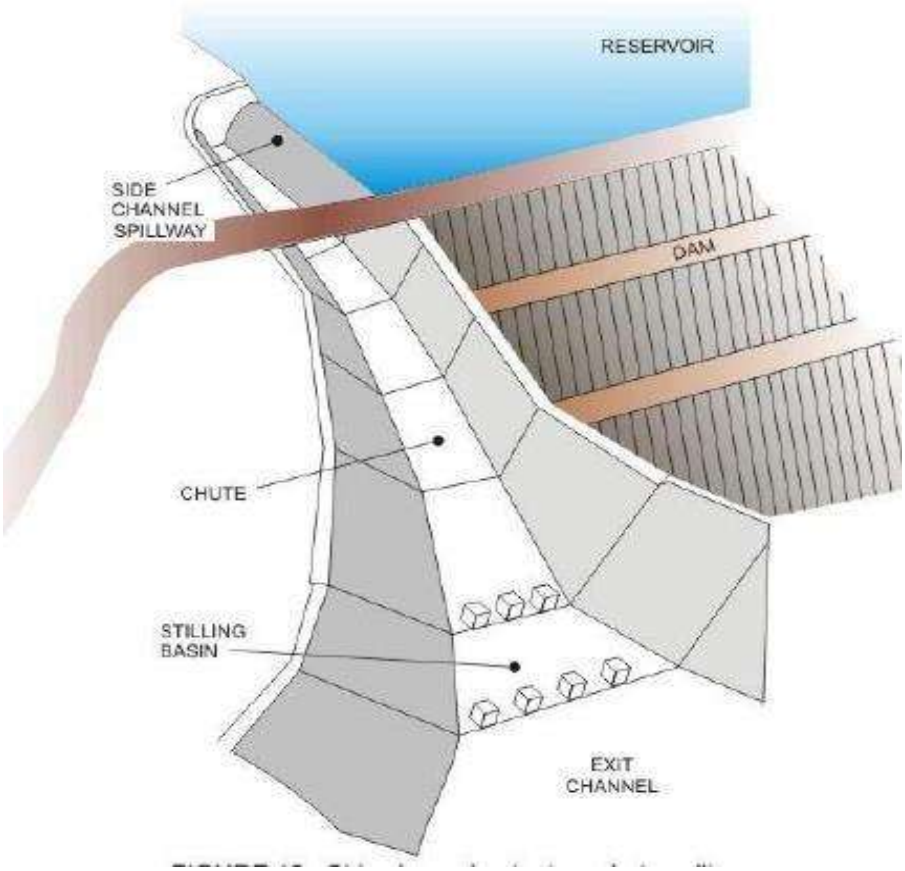

(Fig-3 Outflow from a freely falling weir, properly ventilated from below)




(Fig -4 Section of an Ogee spillway with vertical u/s face)

## 3. Chute (Open Channel/Trough) Spillway:

A chute spillway, variously called as open channel or trough spillway, is one whose discharge is conveyed from the reservoir to the downstream river level through an open channel, placed either along a dam abutment or through a saddle. The control structure for the chute spillway need not necessarily be an overflow crest, and may be of the side-channel type, as has been shown in Figure 5. Generally, the chute spillway has been mostly used in conjunction with embankment dams, like the Tehri dam. Chute spillways are simple to design and construct and have been constructed successfully on all types of foundation materials, ranging from solid rock to soft clay. Chute spillways ordinarily consist of an entrance channel, a control structure, a discharge channel, a terminal structure, and an outlet channel.

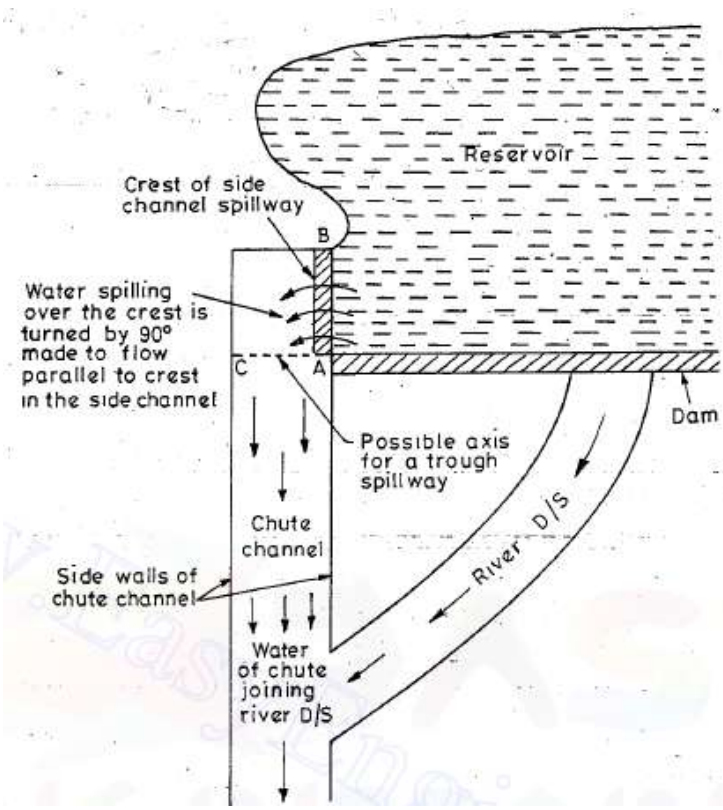



(Fig-5 side channel entry to a Chute spillway)

## 4. Side Channel Spillway:

A side channel spillway is one in which the control weir is placed approximately parallel to the upper portion of the discharge channel, as may be seen in fig 6. The flow over the crest falls into a narrow trough opposite to the weir, turns an approximate right angle, and then continues into the main discharge channel. The side channel design is concerned only with the hydraulic action in the upstream reach of the discharge channel and is more or less independent of the details selected for the other spillway components.

Discharge characteristics of a side channel spillway are similar to those of an ordinary overflow spillway and are dependent on the selected profile of the weir crest. Although the side channel is not hydraulically efficient, nor inexpensive, it has advantages which make it adoptable to spillways where a long overflow crest is required in order to limit the afflux (surcharge held to cause flow) and the abutments are steep and precipitous.

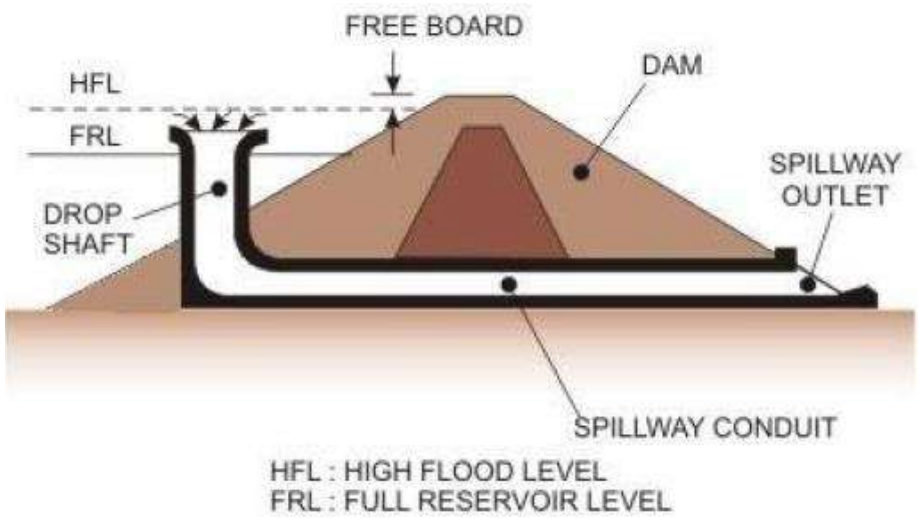



(Fig-6 sketch of a side-channel spillway)

## 5. Shaft (Drop Inlet/Morning Glory) spillway:

A Shaft Spillway is one where water enters over a horizontally positioned lip, drops through a vertical or sloping shaft, and then flows to the downstream river channel through a horizontal or nearly horizontal conduit or tunnel. A drop inlet spillway can be used advantageously at dam sites that are located in narrow gorges where the abutments rise steeply.

Discharge characteristics of the drop inlet spillway may vary with the range of head. The head increases, the flow pattern would change from the initial weir flow over crest to tube flow and then finally to pipe flow in the tunnel. This type of spillway attains maximum discharging capacity at relatively low heads. However, there is little increase in capacity beyond the designed head, should a flood larger than the selected inflow design flood occur.

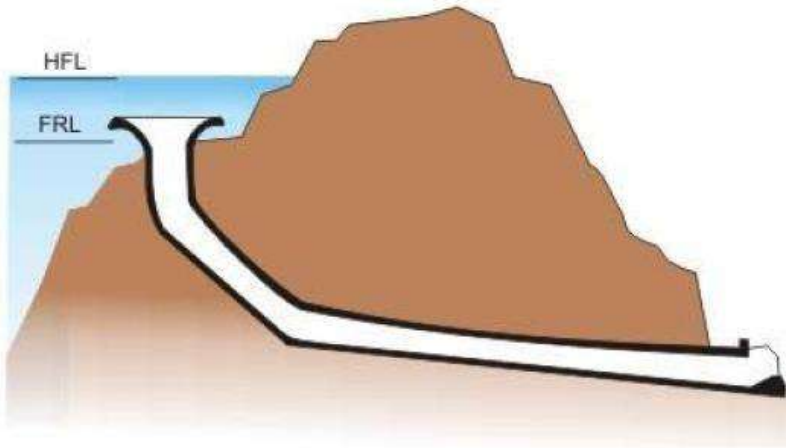



(Fig-7 Section through a shaft spillway)

## 6. Tunnel (Conduit) spillway:

Where a closed channel is used to convey the discharge around a dam through the adjoining hill sides, the spillway is often called a tunnel or conduit spillway. The closed channel may take the form of a vertical or inclined shaft, a horizontal tunnel through earth or rock, or a conduit constructed in open cut and backfilled with earth materials. Most forms of control structures, including overflow crests, vertical or inclined orifice entrances, drop inlet entrances, and side channel crests,

can be used with tunnel spillways. Tunnel spillways are advantageous for dam sites in narrow gorges with steep abutments or at sites where there is danger to open channels from rock slides from the hills adjoining the reservoir. Conduit spillways are generally most suited to dams in wide valleys as in such cases the use of this types of spillway would enable the spillway to be located under the dam very close to the stream bed.




(Fig-8 Tunnel spillway with a morning glory entrance)

## 7. Siphon spillway:

A siphon spillway is a closed conduit system formed in the shape of an inverted U, positioned so that the inside of the bend of the upper passageway is at normal reservoir storage level. This type of siphon is also called a Saddle siphon spillway. The initial discharges of the spillway, as the reservoir level rises above normal, are similar to flow over a weir. Siphonic action takes place after the air in the bend over the crest has been exhausted. Continuous flow is maintained by the suction effect due to the gravity pull of the water in the lower leg of the siphon.

Siphon spillways comprise usually of five components, which include an inlet, an upper leg, a throat or control section, a lower leg and an outlet. Another type is hooded type of siphone spillway in which reinforced concrete hood is constructed over an ordinary overflow section of a gravity dam. The inlet of this hood is kept submerged so as to prevent entry of debris and ice. A small depriving hood is kept above the main hood and both these hoods are connected by air vent and head of the depriver hood is kept at normal pool level.