Chapter-I

HYDROSTATICS

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

Fluid:-

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

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

1.1 PROPERTIES OF FLUID:-

Density:-

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

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

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

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

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

Specific Weight:-

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

$\lambda = \rho * g$

g = local acceleration of gravity and ρ = density

Note: It is customary to use:

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

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

Relative Density (Specific Gravity):-

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

Specific volume:-

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

Viscosity:-

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

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

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

Viscosity is due to two factors-

a) Cohesion between the liquid molecules.

b) Transfer of momentum between the molecules.

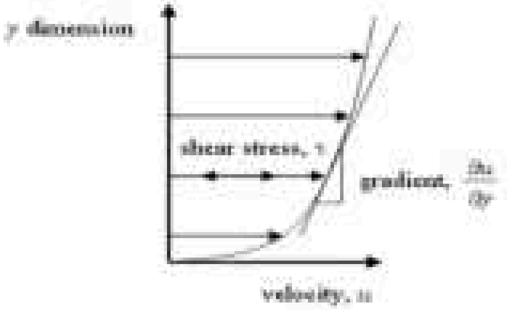


Fig. 1.1

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

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

UNIT OF VISCOSITY

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

Kinematic viscosity:-

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

Let, $\upsilon = \mu / \rho$ = 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
  - $1 N/m^2 = 1$ Pascal
  - $1 KN/m^2 = 1$ kilo Pascal
  - Kilo Pascal= 1kpa = 10^3 Pascal
  - 1 bar = 10^5 Pascal = $10^5 N/m^2$

Pascal's law:-

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

Atmospheric Pressure:-

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

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

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

= 760 mm of Hg

=10.3 (milli bar)

Gauge pressure:-

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

The atmospheric pressure on scale is marked as zero.

Absolute pressure:-

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

Vacuum pressure:-

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

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

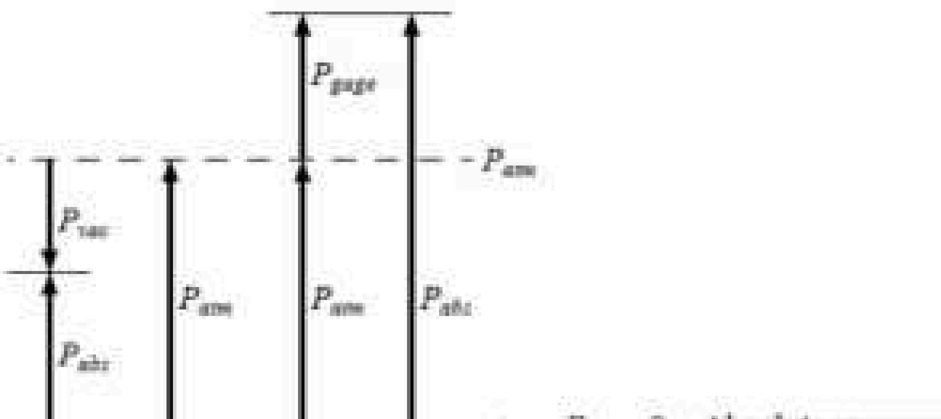


Fig. 1.2

❖ Equations

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

❖ Nomenclature

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

Pressure Head:-

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

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

where

ψ is pressure head (Length, typically in units of m);

p is fluid pressure (force per unit area, often as Pa units); and

γ is the specific weight (force per unit volume, typically N/m^3 units)

ρ is the density of the fluid (mass per unit volume, typically kg/m^3)

g is acceleration due to gravity (rate of change of velocity, given in m/s^2)

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

Mathematically, $h = P/w$

Pressure Gauges :-

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

1. manometers
2. mechanical gauges

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

a) Simple manometers

b) Differential manometer

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

a) Diaphragm pressure gauge

b) Bourdon tube pressure gauge

c) Dead weight pressure gauge

d) Bellows pressure gauge

1.3 PRESSURE EXERTED ON IMMERSED SURFACE:-

Hydrostatic forces on surfaces:-

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

Forces on Submerged Surfaces in Static Fluids

These are the following features of statics fluids:-

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

Fluid pressure on a surface:-

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

$F = p\delta A$

TOTAL PRESSURE:-

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

Mathematically **total pressure**,

$P = p_1a_1 + p_2a_2 + p_3a_3$......

Where,

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

The position of an immersed surface may be,

- Horizontal
- Vertical
- Inclined

Total Pressure On A Horizontal Immersed Surface

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

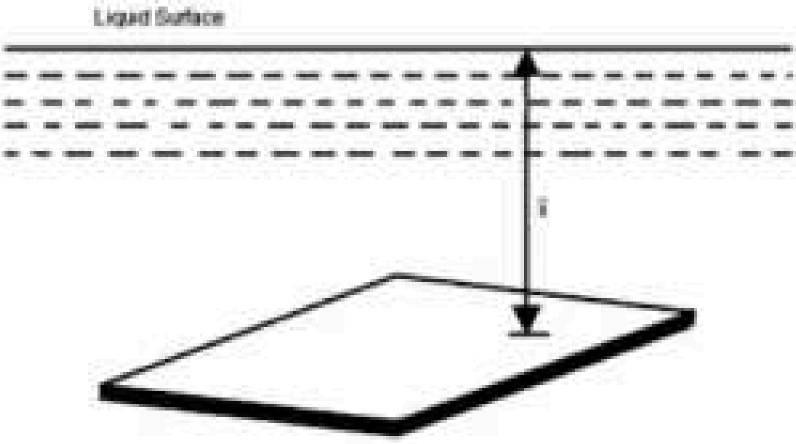


Fig. 1.3

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

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

P = Weight of the liquid above the immersed surface

= Specific weight of liquid * Volume of liquid

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

$= \omega A \bar{x} kN$

Total Pressure On A Vertically Immersed Surface

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

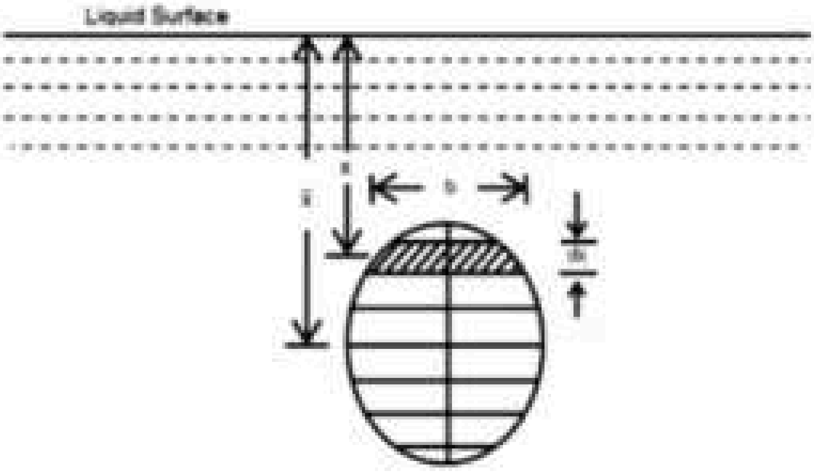


Fig. 1.4

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

Here,

- ω = Specific weight of the liquid
- A = Total area of the immersed surface
- $\bar{x}$ = 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 = ωx

and the area of strip = b.dx

$\therefore$ Pressure on the strip = Intensity of pressure * Area = ωx .bd$x$

Now, Total pressure on the surface,

$P = \int wx.bdx$.

$= w\int x.bdx$

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

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

1.4 FLOTATION AND BUOYANCY:-

Archimedes Principle:-

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

Buoyancy:-

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

Centre of pressure:-

The center of pressure is the point where the total sum of a pressure field acts on a body, causing a force to act through that point. The total force vector acting at the center of pressure is the value of the integrated pressure field. The resultant force and center of pressure location produce equivalent force and moment on the body as the original pressure field. Pressure fields occur in both static and dynamic fluid mechanics. Specification of the center of pressure, the reference point from which the center of pressure is referenced, and the associated force vector allows the moment generated about any point to be computed by a translation from the reference point to the desired new point. It is common for the center of pressure to be located on the body, but in fluid flows it is possible for the pressure field to exert a moment on the body of such magnitude that the center of pressure is located outside the body.

Center of buoyancy:-

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

METACENTER:-

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

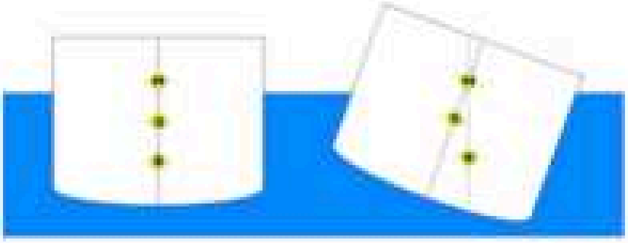


Fig. 1.5

The metacentre can be calculated using the formulae:

$$KM = KB + BM$$
$$BM = \frac{I}{V}$$

Metacentric height:-

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

MG =BM-BG

MG=I/V-BG

Stability of a submerged body:-

Stable condition:-

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

Unstable equilibrium;-

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

Neutral equilibrium:-

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

Stability of a floating body:-

- For stable condition w = f_b and the meta centre "m" is about the CG of the body.
- For unstable equilibrium w = f_b and the metacentre "m" is below CG of the body.
- In neutral equilibrium w = f_b and metacentre "m" is acting at CG of the body.

Chapter-II

KINEMATICS OF FLUID FLOW

2.1 Basic equation of fluid flow and their application:-

Energy of a Liquid in Motion:-

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

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

Potential Energy of a Liquid Particle in Motion:-

It is energy possessed by a liquid particle by virtue of its position. If a liquid particle is *Z m* above the horizontal datum (arbitrarily chosen), the potential energy of the particle will be *Z* metre-kilogram (briefly written as mkg) per kg of the liquid. The potential head of the liquid, at

point, will be *Z* metres of the liquid.

Kinetic Energy of a Liquid Particle in Motion:-

It is the energy, 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,

$Q_1 = a_1.v_1 = (17.67 x 10^{-3}) x 2.5 = 44.2 \times 10^{-3} \; m^3/s$

= 44.2litres/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 13nergy; 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-

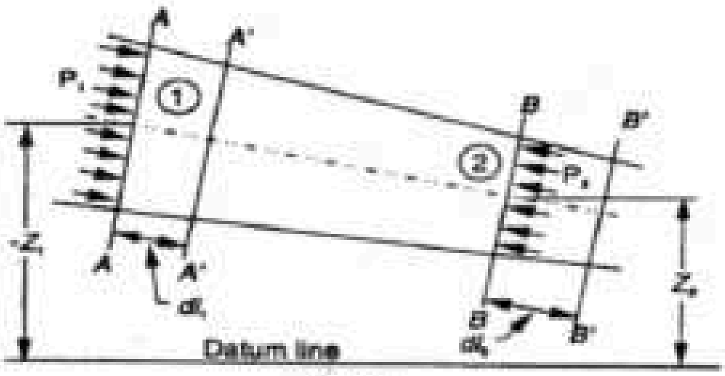


Fig. 2.1

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

$W = wa_1dl_1 = wa_2dL_2$

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

$= P_1a1dl_1 - P_2a_2dl_2$

$=P_1a1dl_1 - p_2a1dl_1$ ...($a_1dl_1=a_2dl_2$)

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

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.

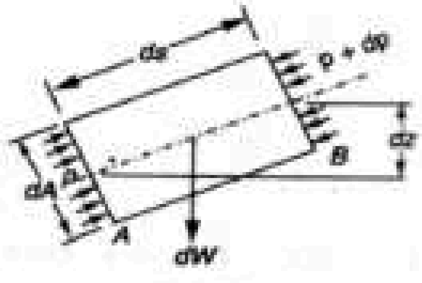


Fig. 2.2

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

$= p.\ dA - (p + dp)\ dA$

$= -dp.dA$

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

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

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

$= -\ \rho\ g.\ dA.\ dz$

$\therefore$ *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

$$(-dp \cdot dA) - (\rho\, g \cdot dA \cdot dz) = \rho \cdot dA \cdot ds \times \frac{dv}{ds}$$

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

...(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.

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

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

$P + wZ + Wv^2/2g = \text{constant}$

$\frac{p}{w} + Z + v^2/2g = \text{constant}$ (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, ifsome energy is supplied to, or, extracted from the flow, the same should also be taken into account.
3. The Bernoulli's equation has been derived, under the assumption that there is. no loss of energy of the liquid particle while flowing. But, in actual practice, -it is rarely so. In a turbulent flow, some kinetic energy is converted into heat energy. And in a viscous flow, some energy is lost due to shear forces. Thus, while using Bernoulli's equation, all such losses should be neglected.
4. If the liquid is flowing in a curved path, the energy due to centrifugal force should also be taken into account.

Example

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

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

Let

V_2 = Velocity of flow at section 2, and

P_2 = Pressure at section 2. We know that area of the pipe at section 1 $a_1 = \frac{\pi}{4} \times 0.2^2 = 31.42 \times 10^{-3} m^2$

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

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

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

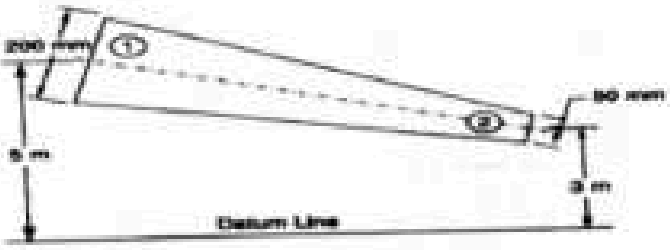


Fig. 2.3

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

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

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

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

practical Applications of Bernoulli's Equation

The Bernoulli's theorem or Bernoulli's equation is the basic equation which has the widest applications in Hydraulics and Applied Hydraulics. Since this equation is applied for the derivation

.of many formulae, therefore its clear understanding is very essential. Though the Bernoulli's equation has a number of practical applications. yet in this chapter we shall discuss its applications on the following 'hydraulic devices :

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

Venturimeter

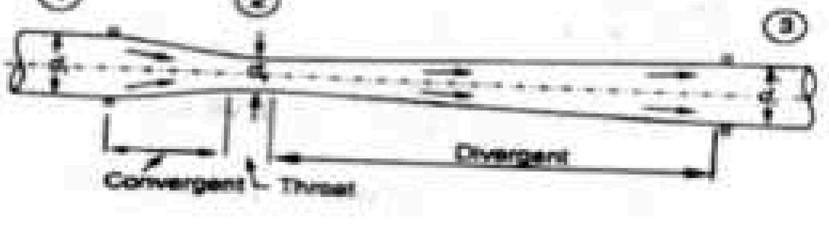


Fig. 2.4

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

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

(a) Convergent cone

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

(b) Throat

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

(c) Divergent cone

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

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

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

Discharge through a Venturi meter

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

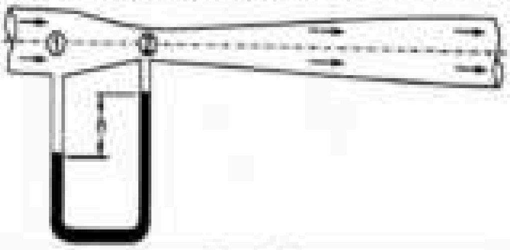


Fig. 2.5

Let

P_1 = Pressure at section 1,

V_1 = Velocity of water at section 1,

Z_1 = Datum head at section 1,

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

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

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

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

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

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

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

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

$V_1 = a_2 v_2/a_1$ $(a_1 v_1 = a_2 v_2)$

$V_1^2 = a_2^2 v_2^2/a_1^2$ ....(3)

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

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

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

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

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

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 m; Specific gravity of oil = 0.9 h = 200 mm = O.2 m of mercury and C = 0.98.

We know that the area at inlet,

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

and the area at throat,

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

We also know that the difference of pressure head,

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

and the discharge through the venturi meter,

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

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

0rifice Metre

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

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

Let

h = Reading of the mercury manometer,

P_1 = Pressure at inlet,

V_1 = Velocity of liquid at inlet,

a_1 = Area of pipe at inlet, and

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

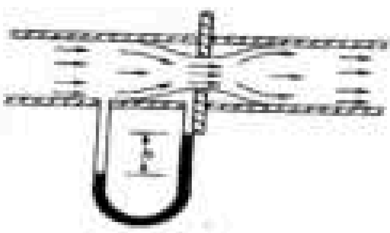


Fig. 2.6

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

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

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

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

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

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

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

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

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

We know that the discharge,

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

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

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.Ifthe 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,

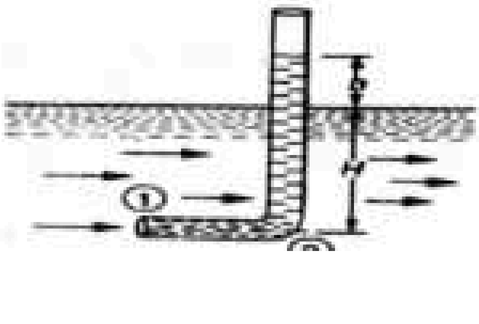


Fig. 2.7

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 pitot 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.

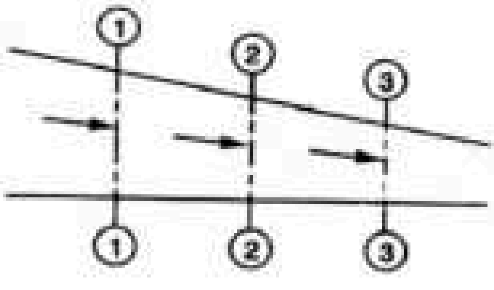


Fig. 2.8

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,

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

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

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

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

$$Q_3 = a_2 v_3 \qquad \text{.............(iii)}$$

From the law of conservation of matter, we know that the total quantity of liquid passing through the sections 1-1, 2-2 and 3-3 is the same. Therefore
$Q_1 = Q_2 = Q_3 = \ldots\ldots$ or $a_1 . v_1 = a_2 . v_2 = a_3 v_3 \ldots\ldots$ 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,

$$a_1 = \left(\frac{\pi}{4}\right) x (0.1)^2 = 7.854 x 10^{-3}\ m^2$$

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

$$a_2 = \left(\frac{\pi}{4}\right) x (0.2)^2 = 31.42 x 10^{-3}\ m^2$$

and velocity of water at point 2 , $v_2 = \frac{Q}{\mathbf{a2}} = ((78.54 \text{ X } 10^{-3})/(31.42 \text{x} 10^{-3}))$=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.

Ib) 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.

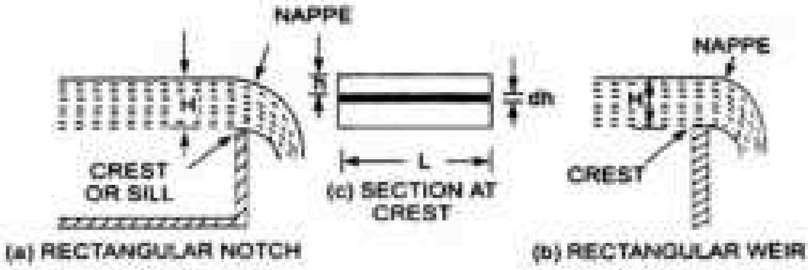


Fig. 2.9

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 litls = 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 .

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

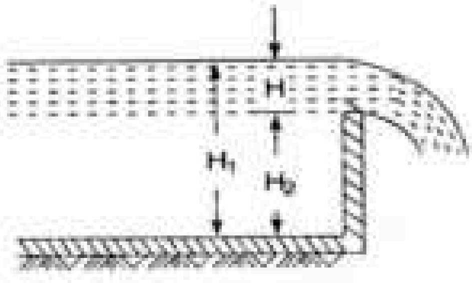


Fig. 2.10

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

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

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

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

Height of weir, H2 = H1 - H

= Depth of water on upstream side - H

= 1.8 - .328 = 1.472 m. Ans.

Discharge Over A Triangular Notch Or Weir:-

The expression for the discharge over a triangular notch or weir is the same. It is derived as :

Let H = head of water above the V- notch

θ = angle of notch

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

For a right angle V Notch ,if C_d=0.6

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

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

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

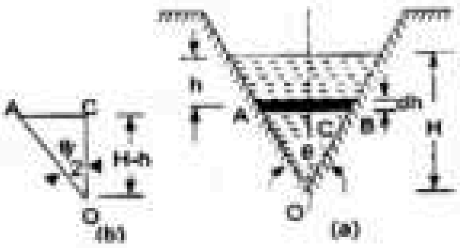


Fig. 2.11

Problem -1

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

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

Head over notch, H=0.3 m

C_d = 0.6

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

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

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

$= 0.8182 \times 0.0493 = 0.040$ 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.

$\theta = 90^o$

$C_d = 0.59$

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

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

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

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

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

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

$H_1 = (0.07631)^{0.4} = 0.3572$ 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 trapezoidal notch or weir FDCEF = $Q_1 + Q_2$

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

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

Solution. Given:

Top width AE=1 m

Base width, CD=L=0.4 m

Head of water, H=0.20 m

For rectangular portion, C_{d1}=0.62

From ΔABC ,we have

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

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

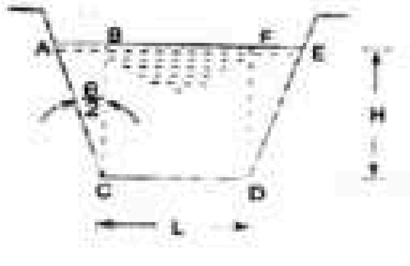


Fig. 2.12

Discharge through trapezoidal notch is given by equation

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

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

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

Discharge Over A Stepped Notch:-

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

Consider a stepped notch as shown in Fig.

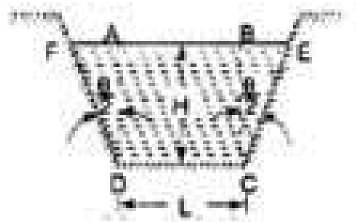


Fig. 2.12

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

L_1 = Length of notch 1 .

H_2,L_2 and H_3,L_3 are corresponding values for notches 2 and 3 res[

C_d=Co-efficient of discharge for all notches

Total discharge $Q=Q_1+Q_2+Q_3$

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

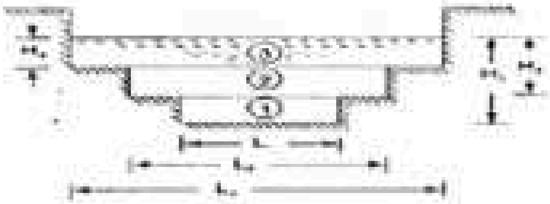


Fig. 2.13

Problem

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 cm,

H_2=80 cm,H_3=50 cm,

C_d=0.62

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

where

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

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

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

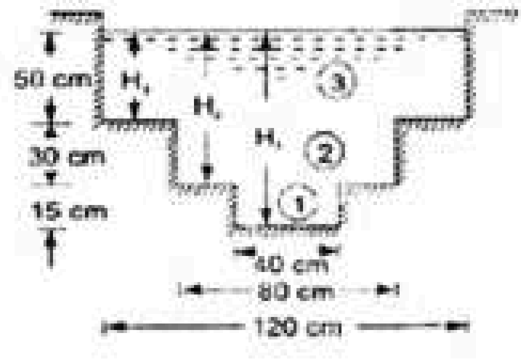


Fig. 2.14

$Q_2 = \frac{2}{3} \times C_d \times L_2 \times \sqrt{2g} [H_2^{3/2} - H_1^{3/2}]$

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

$= 530141 \text{ cm}^3/\text{s}$

$= 530.144$ lit/s

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

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

$= 776771 \text{ cm}^3/\text{s}$

$= 776.771$ lit/s

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

$= 154.067 + 530.144 + 776.771$

$= 1460.98$ 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

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

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

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

Types of Weirs :-

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

1. Narrow-crested weirs,
2. Broad-crested weirs,

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

Discharge over a Narrow-crested Weir :-

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

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

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

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

Where, Q = Discharge over the weir,

Cd = Coefficient of discharge,

L = Length of the weir, and

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

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

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

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

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

$$= 46.55 \text{ m}^3/\text{s} = 4655 \text{ lit/s}$$

Discharge over a Broad-crested Weir :-

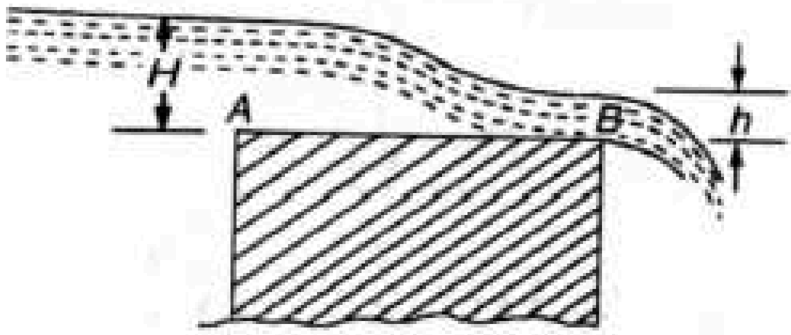


Fig. 2.15

Broad-crested weir

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

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

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

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

C_d = Coefficient of discharge, and

L = Length of the weir.

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

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,

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

$= 0.6 \times 2.0 \times 0.5 \times \sqrt{2 \times 9.81(1-0.5)}$ m³/s

$= 6 \times 3.13 = 18.8$ m³/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.,

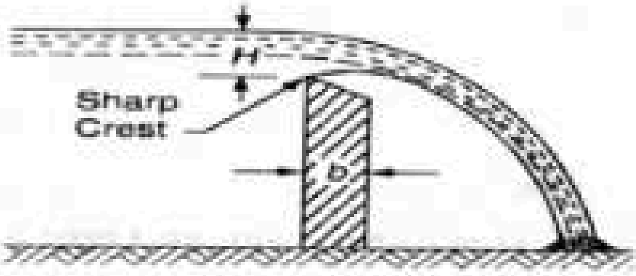


Fig. 2.16

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,

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

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

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

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

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

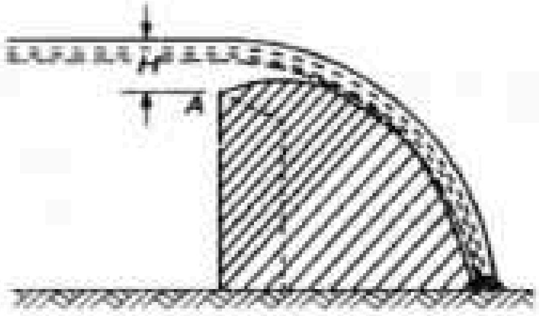


Fig. 2.17

Example

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

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

We know that the discharge over the weir,

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

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

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

Discharge over a Submerged or Drowned Weir :-

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

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

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

H_2 =height of water on the downstream side

of the weir.

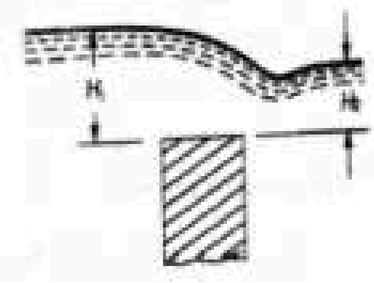


Fig. 2.18

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

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

Submerged weir :-

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

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

∴ 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

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

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

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

and discharge over the submerged portion of the weir,

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

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

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

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

2.3 Flow over Weirs:-

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

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

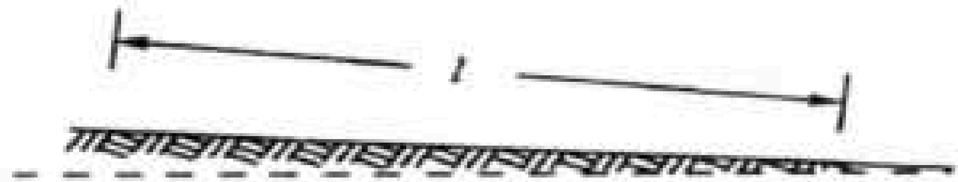


Fig. 2.19

Sloping bed of a channel :-

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

Let

l = Length of the channel,

A = Area of flow,

v = Velocity of water,

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

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

i = Uniform slope in the bed.

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

∴ Discharge $Q = A \times v = AC\sqrt{mi}$

Example.

A rectangular channel is 1. 5 metres deep and 6 metres wide. Find the discharge through channel, when it runs full. Take slope of the bed as 1 in 900 and Chezy's constant as 50.
Solution. Given: d = 1.5 m; b = 6 m; i = 1/900 and C = 50.

We know that the area of the channel,

$A = b.d = 6 \times 1.5 = 9\ m^2$

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

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

and the discharge through the channel,

$Q = AC\sqrt{mi} = 9 \times 50\sqrt{(1 \times 1/900)} = 15 m^3/s$ 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,

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

where

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

Now the discharge,

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

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

Example

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

Solution.

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

We know that the area of flow,

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

and wetted perimeter,

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

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

We know that the discharge through the channel

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

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

$=1.945 \ m^3/s$ 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 √□mi

m=A/P

=d/2

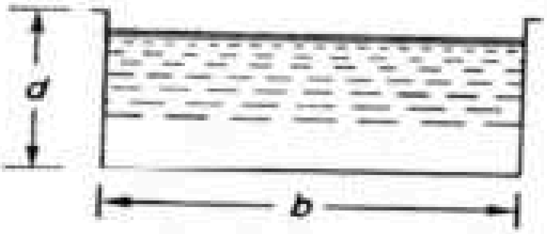


Fig. 2.20

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 √□mi= 8 x 55√□ 1 X 0.001 m^3/s

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

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

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

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

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

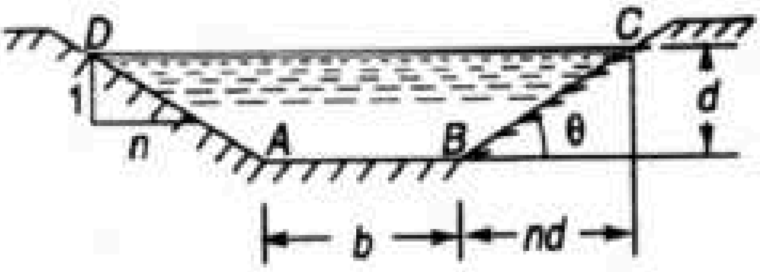


Fig. 2.21

Let

b = Breadth of the channel at the bottom,

d = Depth of the channel and

$\frac{1}{n}$=side slope (i.e., 1 vertical to n horizontal)

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

Example .

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

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

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

$m = d/2 = 0.5$ m

and discharge in the channel,

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

Example .

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

Solution.

Given side slope (n)=1

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

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

and C = 50

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

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

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

or b = 0.828d

∴ Area A = d X (b + nd) = 1.828d^2

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

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

∴ $d^{5/2}$ =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

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

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

$$P = 2r\theta \quad \text{...(i)}$$

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

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

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

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

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

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

Solution:-

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

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

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

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

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

And perimeter P = 2r θ = 1.124m

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

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

Chapter-III

PUMPS

3.1 Centrifugal Pumps:-

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

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

Main Parts Of A Centrifugal Pump:-

The followings are the main parts of a centrifugal pump:

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

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

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

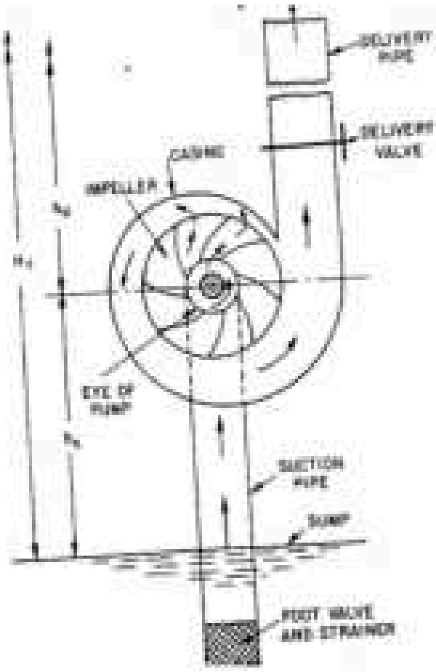
2. **Casing:** The casing of a centrifugal pump is similar to the casing of a reaction turbine. It is an air-tight passage surrounding the impeller & is designed in such a way that the kinetic energy of the water discharged at the outlet of the impeller is converted into pressure energy before the water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

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

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

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

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



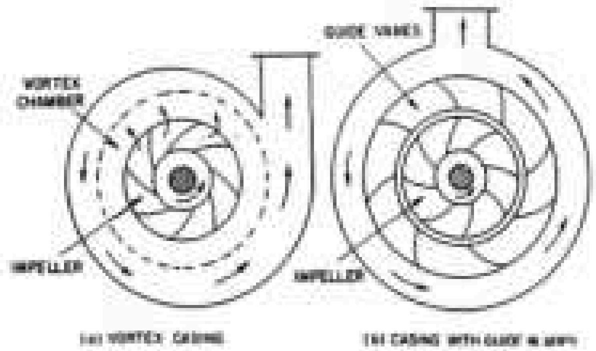
Main parts of a centrifugal pump

Fig. 3.1

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

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

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



fferent 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 η_{man}

b. Mechanical efficiencies η_m

c. Overall efficiencies η_o

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

η_{man} = Manometric head/Head imparted by impeller to water

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

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

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

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

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

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

b) **Mechanical efficiencies:-**

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

η_m = Power at the impellerPower at the shaft

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

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

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

Where S.P.= Shaft Power

c) **Overall efficiencies** η_o

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

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

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

= S.P. of the pump

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

$$= \eta_{mano} \times \eta_m \text{..................}$$

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: Internal Dia. Of impeller,=D_1=200mm=0.20m

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

Speed N=1200r.p.m

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

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

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

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

Tangential velocity of impeller at inlet & outlet are,

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

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

From inlet velocity triangle,

$$\tan\phi = \frac{V_{f1}}{u_1} = \frac{V_{f2}}{12.56}$$

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

$$V_{f2} = V_{f1} = 4.57 m/s$$

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

Solution: Net head, H_m= 14.5m

Speed, N =1000r.p.m

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

Impeller diameter means the diameter of the impeller at outlet

Diameter, $D_2 = 300mm = 0.30m$

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

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

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

Now using equation

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

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

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

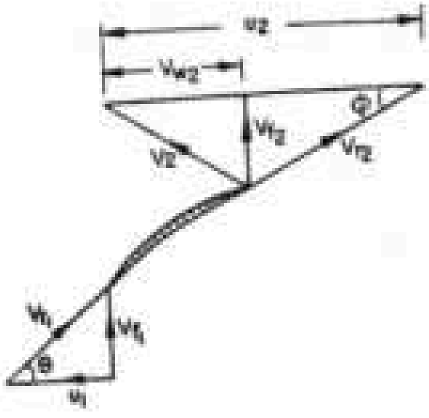


Fig. 3.3

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

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

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

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

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

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

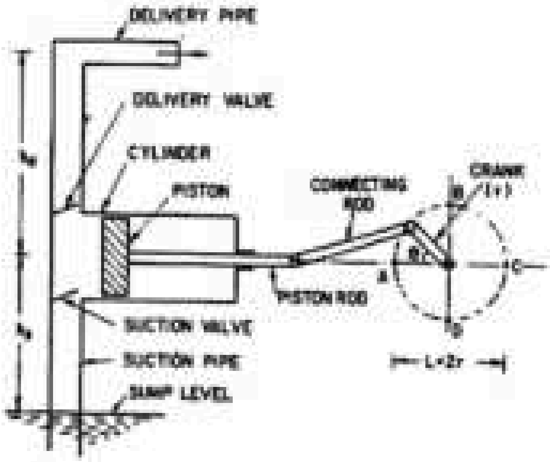
3.2 Reciprocating Pump:-

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

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

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

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

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

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

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

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

Substituting the value of W in equation () we get

Work done per second =

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

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

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

Classification of reciprocating pumps:

The reciprocating pumps may be classified as:

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

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

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

I. Single-acting pump
II. Double –acting pump

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

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

When the water is available at a higher level, it is supplied to lower level, by the more action of gravity, then it is called Flow Irrigation. But if the water is lifted up by some mechanical or manual means, such as pumps, etc. and then supplied for irrigation , then it is called Lift irrigation.

Flow irrigation can be further sub divided into:

(i) Perennial irrigation

(ii) Flood irrigation

Perennial Irrigation : In perennial system of irrigation, constant and continuous water supply is assured to the crops in accordance with requirements of the crops, throughout the crop period. In this system of irrigation, water is supplied through storage canal head works and canal distribution system.

When the water is directed into the canal by constructing a weir or a barrage across the river, it is called Direct Irrigation. Ganga canal system is an example of this type of irrigation. But if a dam is constructed across a river to store water during monsoons, so as to supply water in the off-taking channels during periods of low flow, then it is termed as storage irrigation.

Flood Irrigation : This kind of irrigation , is sometimes called 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 stored by the soil, when occasionally supplemented by natural rainfall or minor watering, brings the crop to maturity.

(2) Sub-Surface Irrigation – 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 capillarity. It may be divided into the following two types.

(a) Natural sub irrigation and

(b) Artificial sub-irrigation.

(a) Natural sub-irrigation – Leakage water from channels, etc goes underground and during passage through the sub soil, it may irrigate crops, sown on lower land , by capillarity. Sometimes , leakage causes the water table to rise, which helps in irrigation of crops by capillarity. When underground irrigation is achieved simply by natural processes, without any additional extra efforts, it is called natural sub-irrigation.

(b) Artificial sub-irrigation – When a system of open jointed drains is artificially laid below the soil, so as to supply water to the crops by capillarity, then it is known as artificial sub-irrigation. It is a very costly process and hence adopted in India on a very small scale. Sometimes irrigation water may be intentionally collected in some ditches near the fields, the percolation water may then come up to the roots through capillarity.

Sources Of Irrigation Water :

There are various ways in which the irrigation water can be applied to the fields. Their main classification is as follows :

(1) Free flooding
(2) Border flooding
(3) Check flooding
(4) Basin flooding
(5) Furrow irrigation method
(6) Sprinkler irrigation method
(7) Drip irrigation method

(1) Free flooding or Ordinary Flooding – In this method ditches are excavated in the field, and they may be either on the contour or up and down the slope. Water from these ditches, flows across the field. After water leaves the ditches , no attempt is made to control the flow by means of leaves. Since the movement of water is not restricted, it is sometimes called wild flooding. Although the initial cost of land preparation is low, labour requirements are usually high and water application efficiency is also low. Wild flooding, is most suitable for close growing crops, pastures, etc. particularly where the land is steep. Contour ditches called lateral or subsidiary ditches. Are generally spaced at about 20 to 50 metres apart, depending upon the slope, texture of soil, crops to be grown, etc. This method may be used on rolling land where borders , checks, basins and furrows are not feasible.

(2) Border flooding – In this method, the land is divided into a number of strips, separated by low leaves called borders. The land area confined in each strip is of the order of 10 to 20 metres in width and 100 to 400 metres in length. To prevent water from concentrating on either side of the border, the land should be levelled perpendicular to the flow. Water is made to flow from the supply ditch into each strip. The water flows slowly towards the lower end, and infiltrates into the soil as it advances. When the advancing water reaches the lower end of the strip, the supply of water to the strip is turned off. The supply ditch, also called irrigation stream, may either be in the form of an earthen channel or a lined channel or an underground concrete pipe having risers at intervals.

(3) Check Flooding – Check flooding is similar to ordinary flooding except that the water is controlled by surrounding the check area with low and flat levees. Levees are

generally constructed along the contours, having vertical interval of about 5 to 10 cm. These levees are connected with cross – levees at convenient places. The confined plot area varies from 0.2 to 0.8 hectare. In check flooding, the check is filled with water at a fairly high rate and allowed to stand till the water infiltrates. This method is suitable for more permeable soils as well as for less permeable soils, thus reducing the percolation losses. The water can also be held on the surface for a longer time in case of less permeable soils, for assuring adequate penetration.

(4) Basin Flooding – This method is a special type of check flooding and is adopted specially for orchard tress. One or more trees are generally placed in the basin and the surface is flooded as in check method by ditch water

(5) Furrow irrigation method – In flooding methods, discussed above, water covers the entire surface while in furrow irrigation method only one-fifth to one-half of the land surface is wetted by water. It therefore results in less evaporation, less pudding of soil and permits cultivation sooner after irrigation. Furrows are narrow field ditches, excavated between rows of plants and carry irrigation water through them. Spacing of furrows is determined by the proper spacing of plants. Furrows vary from 8 to 30 cm deep, and may be as much as 400 metres long. Excessive long furrows may result in too much percolation near the upper end. And too little water near the down slope and. Deep furrows are widely used for row crops. Small shallow furrows called corrugations are particularly suitable for relatively irregular topography and close growing crops such as meadows and small grains.

(6) Sprinkler irrigation method – In this farm water application method, water is applied to the soil in the form of a spray through a network of pipes and pumps. It is a costly process and widely used in U.S.A. It can be used for all types of soils and for widely different topographic and slopes. It can be used for many crops, because it fulfils the normal requirements of uniform distribution of water. This method possesses great potentialities for irrigation areas. This method is not only costly but requires a lot of technicalities. Special steps have to be taken for preventing entry of silt and debris, which are very harmful for the sprinkler equipments

(7) Drip irrigation method – Drip irrigation also called trickle irrigation is the latest field irrigation technique and is meant for adoption at places where there exists acute scarcity of irrigation water and other salt problems. In this methods, water is slowly and directly applied to the root zone of the plants, thereby minimising the losses by evaporation and percolation. This system involves laying of a system of head, mains,

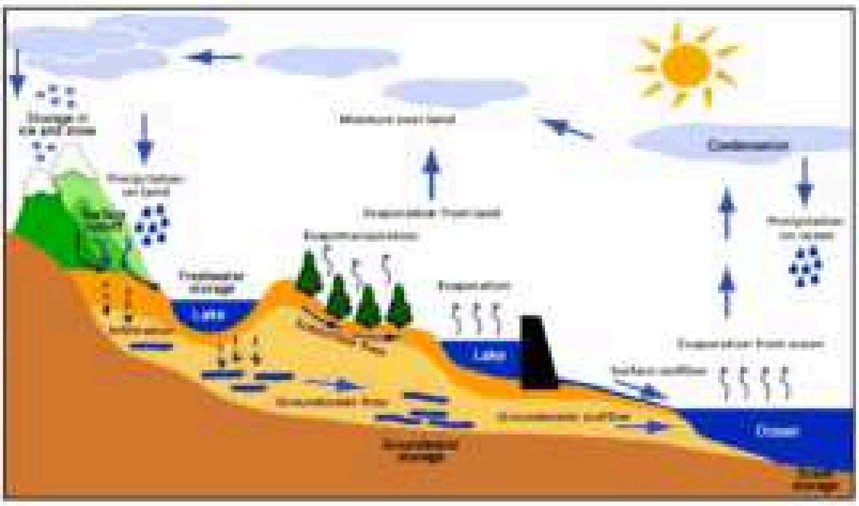
sub-mains, laterals and drop nozzles. Water oozes out of these small drip nozzles uniformly and at a berry small rate, directly in the plant roots areas. The head consists of a pump to lift water, so as to produce the desired pressure of about 2.5 atmosphere, for ensuring proper flow of water through the system. The lifted irrigation water is passed through a fertiliser tank so as to remove the suspended particles from the water to avoid clogging of drip nozzles. The mains and sub mains are the specially designed small sized pipes, made of flexible material like black PVC. These are generally buried or laid on the ground. Their sizes should be sufficient to carry the design discharge of the system.

CHAPTER -2

HYDROLOGY

Hydrological Cycle:-

The water of the universe always changes from one to other under the effect of the sun. The water from the surface source like lakes, rivers, oceans, etc. converts to vapour by evaporation due to solar heat. The vapour goes on accumulating continuously in the atmosphere. This vapour is again condensed due to sudden fall of temperature and pressure. Thus clouds are formed. These clouds again cause the precipitation (i.e. rainfall). Some of the vapour is converted to ice at the peak mountains. The ice again melts in summer and flows as rivers to meet the sea or ocean. These processes of evaporation, precipitation and melting of ice go on continuously like an endless chain and thus a balance is maintained in the atmosphere. This phenomenon is known as hydrologic cycle.



Ch-2.2 Precipitation or Rain Fall and its Measuremet:-

From the principle of hydrologic cycle we have seen that water goes on evaporation continuously from the water surface on earth (river, lake, sea, ocean, etc), by the effect of sun. The water vapour goes on collecting in the atmosphere up to a certain limit. When this limit exceeds and temperature and pressure fall to certain value, the water vapour will get condensed and thereby cloud is formed. Ultimately droplets are formed and returned to earth in the form of rain, snowfall, hail, etc. This is known as rain fall or precipitation.

Types of Precipitation or Rain Fall:-

Depending upon the various atmospheric conditions, the precipitation may be of the following types:

The hydrograph is a general representation of the discharge of river (in cumec against the time (in hr or days). The discharge plotted as ordinate -axis) and the time i plotted as abscissa (axis) (see the figur

During the dry season, there is only base flow (i.e. ground water flow) but no surfa runoff. This may be shown by a line which is approximately straight (not shown in the figu

In rainy season, at the beginning of the rainfall there is only base flow (shown by t line AB). After some period, hen the initial losses (like interception, evaporation and infiltration) are fulfilled, the surface runoff starts and hence the discharge of river goes increasing. Hence the limb of the curve rises which is called rising limb (shown by line BC This line reaches to the peak value at 'C'. Again when the rain stops, the flow in the riv decreases and the limb of the curve declines. This is known as recession limb (as sho by the line CD). This discharge at the point C indicates the maximum discha (i.e. peak discharge or flood discharge). The total area under the curve ABCDE indicates the to runoff. But this run off includes the base flow and the direct runoff. So, to get the act runoff the base flow is to be ducted by separating it from total area.

1) Cyclonic Precipitation:

This type of precipitation is caused by the difference of pressure within the air ma on the surface of the earth. If low pressure is created at some place the warm moist from the surrounding area rushes to the zone of low pressure with violent force. The wa moist air rises up with whirling motion and gets condensed at higher altitude and ultimat heavy rain fall occurs.

This may be two types.

a) Frontal Precipitatio

b) Non Frontal Precipitatic

a) Frontal Precipitation:-

When the moving warm moist air mass is obstructed by the zone of cold air mass, t warm moist air rises up (as it is lighter than cold mass) to higher altitude where it g condensed and heavy rainfall occurs. This is known as Frontal Precipitation.

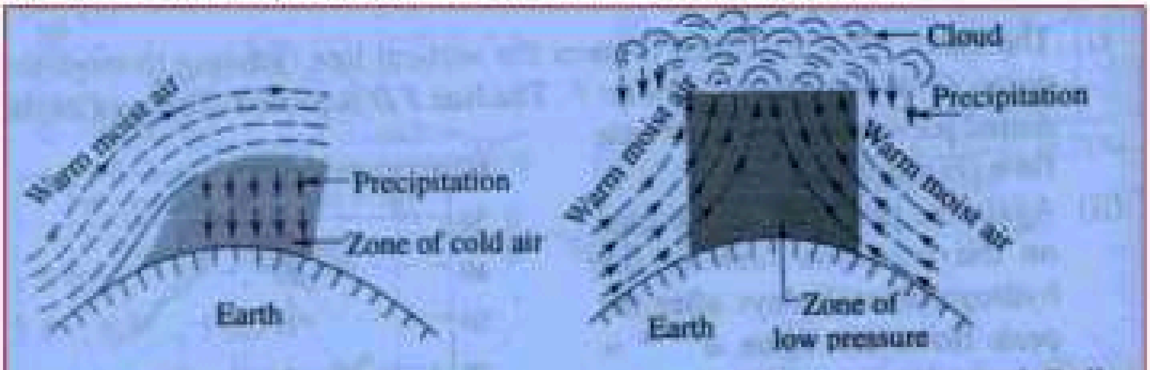


Figure: Frontal precipitation   Figure: Non frontal precipitation

b) Non Frontal Precipitation:-

When the warm moist air mass rushes to the zone of low pressure from th surrounding area, a pocket is formed and the warm moist air rises up like a chimney towa higher altitude. At higher altitude the air mass gets condensed and heavy rainfall occurs. This is known as Non Frontal Precipitati

2) Convective Precipitation:

In tropical counties when on a particular hot day the ground surface gets hea unequally, the warm air is lifted to high altitude and the cooler air takes its place with h velocity, thus, the warm moist air mass is condensed at the high altitude causing heavy r fall. This is known as convective precipitation (see from the figure

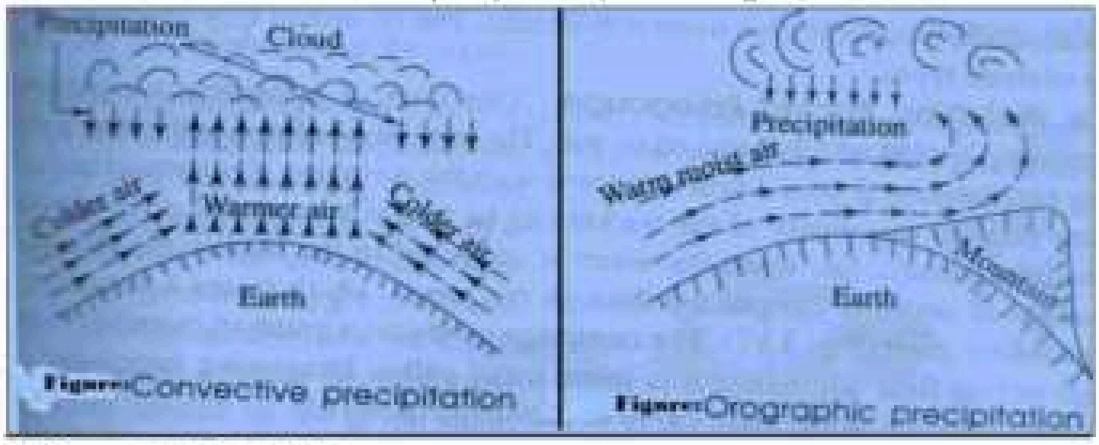


Figure: Convective precipitation

Figure: Orographic precipitation

2) Orographic Precipitation:

The moving warm moist air when obstructed by some mountain rises up to high altitude, it then gets condensed and precipitation occurs. This is known as orograp precipitation (Figure).

Hyetograph:

The graphical representation of rain fall and-off is known as hyetograph. The graph i prepared with intensity of rainfall (in cm/hr) as ordinate and time (in hrs) as abscissa. infiltration capacity curve is drawn on this graph to show the amount of infiltration lo (shown by dotted portion). The upper portion indicates the effective rainfall (shown l hatched line).

The centroid of the effective rain fall is ascertained on the graph for determination of t run-off at any specified period.

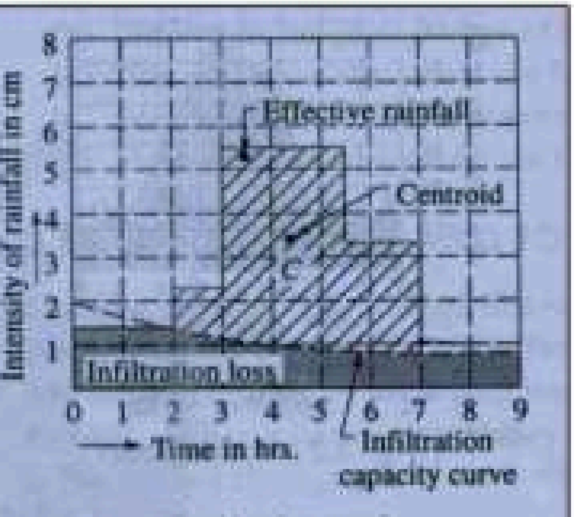


Figure-Hyetograph

2.3 Measurement of Rain Fall or Precipitation-

The instrument which is ed to measure the amount of rainfall is know Raingauge.

The principle of raingauge is that the amount of rainfall in a small area wil represent the amount of rainfall in a large area provided the metrologic characteristics of both small and largea are similar.

Types of Rainguese :

1) Non-Recoding Type Raingauge (N-Automatic)
2) Recording Type Raingauge (Automa
    (i) Weighing Bucket Raingau
    (ii) Tipping Bucket Raingau
    (iii) Float Type Raingau

1) Non-Recoding Type Raingauge

Simon's raingauge is a no-recording type raingauge which is most commonly u It consists of metal casing of diameter 127mm which set on concrete foundation A gl bottle of capacity about 100 mm of rain fall is placed within the casing. A funnel with bra rim is placed on the top of the bottle. The arrangement is shown-

The rainfall is recorded at every 24 hours. Generally the measurement is taken 8.30am every day. In case of heavy rainfall the measurement should be taken 2 to 3 tir daily so that the bottle does not overflow. To measure the amount of rainfall the glass bottl is taken off and the collected water is measured in measuring glass, and recorded in raingauge record book. When the glass bottle is taken off and it is immediately replaced with new bottle of same capacity.

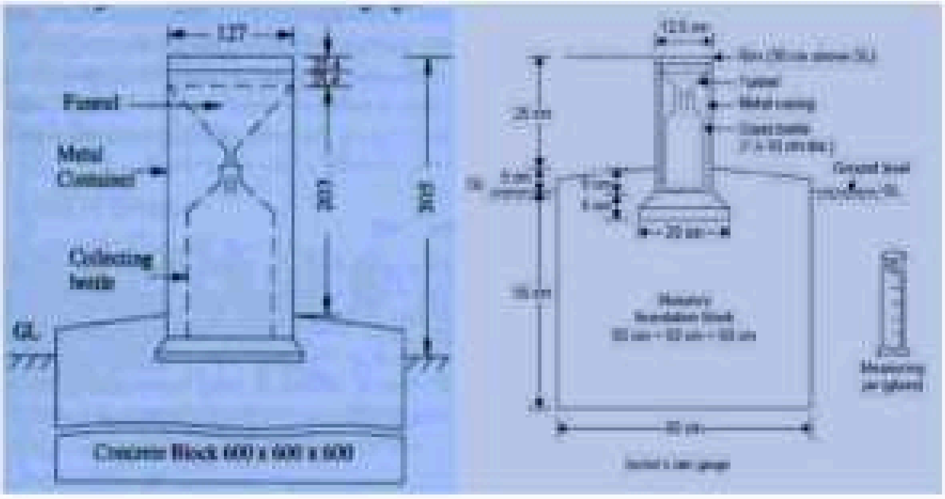


Figure: Simon's Rain Gauge

2) Recoding Type Raingauge:

In this type of raingauge, the amount of rainfall automatically recorded on a gra proper by some mechanical device (see figure). Here person is required measuring the amount of rainfall from the container in which the rain water is collected. The recording ty raingauge may be three types.

Types of Recoding Type Raingauge-

(i) Weighing Bucket Raingauge:- This type of raingauge consists of a receiv bucket which is placed on pan. The pan is again fit with some weighing mechanism. pencil arm is pivoted with the weighing mechanism such a way that the movement of bucket can be traced by a pencil on the moving recording drum. So, when the wa is collected in the bucket the increasing weight of water is transmitted through the pencil wh traces a curve on the recording drum.the rain gauge produces a grph of cumulative r-fall versus time and hence it is some time called integrating raingage. The graph is known as t mass curve of rain fall.

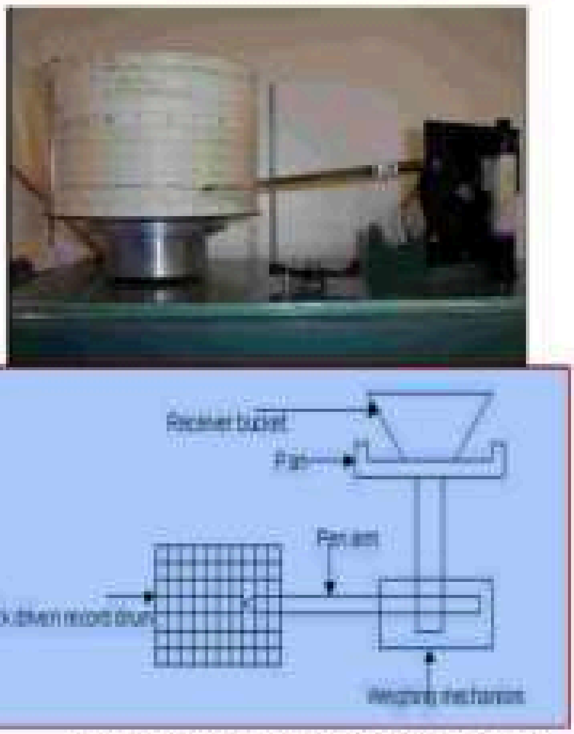


Figure :Weighing Bucket Rain Gauge

(ii) Tipping Bucket Raingauge :-It consists of a circular collector of diameter 30 in which the rain water is initially collected. The rain water then passes through a funnel fitted to the circular collector and gets collected in two compartment tipping bucket pivot below the funnel. (figure).

When 0.25 mm rain water is collected in one bucket, then it tips and discharges the water in a reservoir kept below buckets. At the same time the other bucket s comes below the funnel and the rainwater goes on collecting in it. When the requisite amount of rainwater is collected, it also tips and discharges the water the reservoir. In this way, a circular motion is generated by the buckets. This circular motion is transmitted to a pen or pen which traces a wave like curve on the sheet mounted on revolving drum. The total rainfall may be ascertained from the graph. There is an opening with stopcock at bottom of the reservoir for discharging the collected rainwater. Sometimes a measuring glass is provided to verify the results shown by the gra

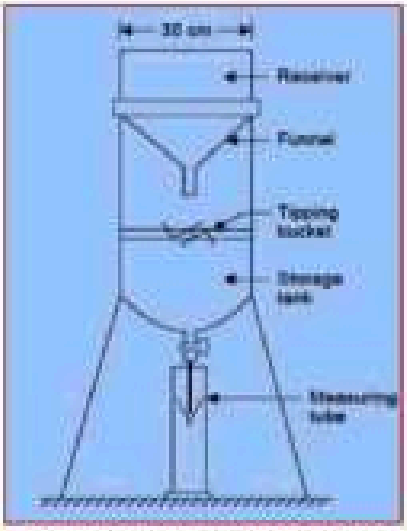


Figure :- Tipping Bucket Raingauge

(ii) Float Type Raingauge -In this type rain gauge a funnel is provided at end of rectangular container and rotating recording drum is provided at the another end. rain water enters the container through the funnel. A float is provided within the contain which rises up as the rain water gets collected. The float consists of a rod whic contains a pen arm for recoding the amount of rainfall on the graph paper dropped on recording drum. It consists of a siphon which starts functioning when the float rises to so definite height and the container goes on emptying gradually

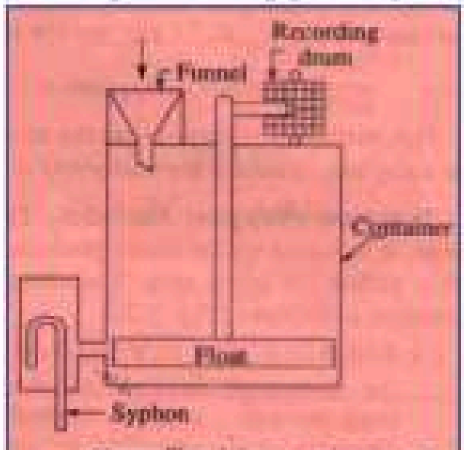


Figure:-Float type raingauge

Selection of Site for Raingauge Statio- The following points should be considere while selecting a site for rain gauge statio

a) The site should be on level ground and on open space. It should never be slopi ground.
b) The site should be such that the distance between the gauge station and the object (like tree, building etc) should be atleast twice the height of the objec
c) In hilly area, where absolutely level ground is not available, the site should be selected that the station may be well shielded from high wind.
d) The site should be easily accessible to the obs
e) The site should be well protected from cattles by fencing

Ch-2.4

Run-off:-

Runoff or Surface run-off in hydrology, the quantity of water discharged in surface streams. Runoff includes not only the waters that travel over the land surface and through channels to reach a stream but also interflow, water that infiltrates the soil surface and travels by means of gravity toward a stream channel (always above the main groundwater level) and eventually empties into the channel. Runoff also includes ground water that is discharged into a stream; stream flow that is composed entirely of groundwater is termed base flow, or fair-weather runoff, and it occurs when a stream channel intersects the water table.

The total Runoff is equal to the Total precipitation less the losses caused by evapotranspiration (loss to the atmosphere from land surfaces and plant leaves), storage (as in temporary ponds) and other such abstractions.

Catchment Area:-A hydrological catchment is defined as the area of land point (usually the sea). A hydrological catchment can vary widely in size and other characteristics such as height above sea level, slope, geology and land use and may contain different combination of freshwater bodies (surface water and ground water) coastal waters.

Estimation of Flood Discharge:-

The flood discharge may be estimated by the below mentioned methods.

(a) Dicken's Formula

$$Q= C \times A^{3/4}$$

Where Q= Discharge in cumec

A= Catchment area in sq. km

C= a constant depending upon the factors affecting discharge

** An average value of C considered as 11.5

(b) Ryve's Formula

$$Q= C \times A^{2/3}$$

Where Q= Discharge in cumec

A= Catchment area in sq. km

C= a constant

** An average value of C considered as 6.8

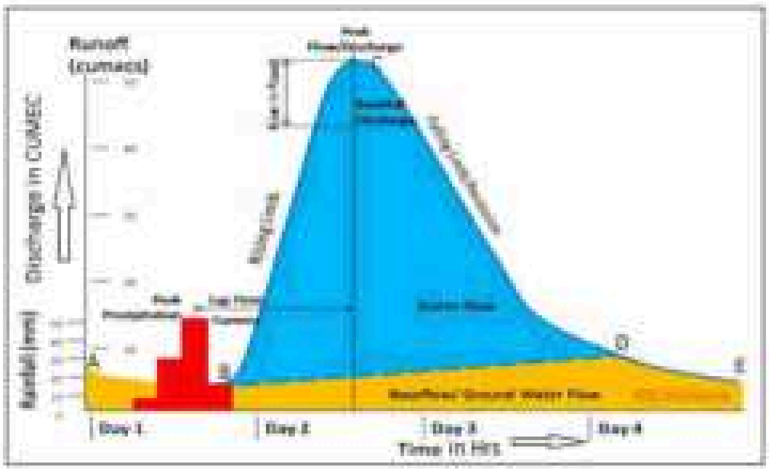
Ch-2.5

Hydrograph:-

The hydrograph is a graphical representation of discharge of river (in cumec) against the time (in hr or days). The discharge is plotted as ordinate (y-axis) and the time is plotted as abscissa (x-axis).

During the dry season, there is only base flow (ground flow) but no surface runoff. This may be shown by a line which is approximately straight (not shown in the figure).

In rainy season, at the beginning of the rainfall there is only base flow (shown by the line AB). After some period, when the initial losses (i.e. interception, evaporation and infiltration) are fulfilled, the surface runoff starts and hence the discharge of river goes increasing. Hence the limb of the curve rises whs called rising limb (shown by the li BC). This line reaches to the peak value at 'C'aAgwhen the rain stops, the flow in t river decreases and the limb of te curve decliThis limb is known as recession limb shown by the line CD). The disarge at the point C indicates the maximum disch (i.epeak discharge or flood discharge). The totaaunder the curve ABCDE indicates total runoff. But this run off includes the baswfl and the direct runoff. So, to get the ac runoff the bae flow is to be deducted by separating it fromtota



Unit Hydrograph:-

A unit hydrograph may be defined as a hydrographchis obtained from one cm effective rainfall (i.e. runoff) for unit duration. Here, effective rainfall mes the rairall excess (i.e. runoff) which directly flows to the river or streamThe unit duration is the peric during which the effective rainfall is assumed to uniformly distributed. The unit duratic may be considered as 1 hr, 2 hr, 3 hr, 4 hr, ... Etcfo example, if a hydrograph prepared for as effective rainfall of one cm lastfor 2 hrs, then it is known as 2 hr. u hydrograph, for the duration of 3 hrs it known as 3hr unit hydrograph)as sowon in the fig

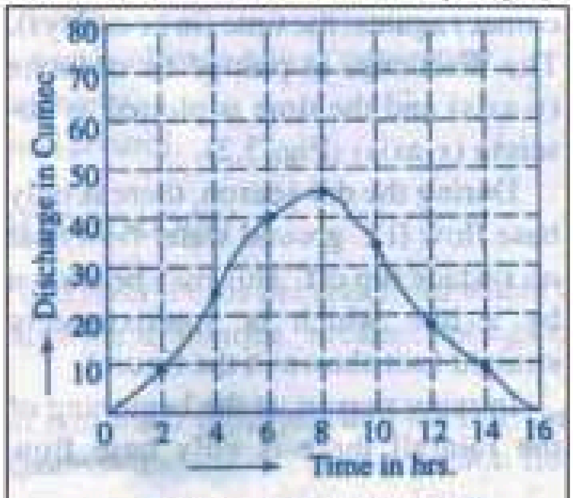


Figure:-Unit hydrograph

<u>Concept of Unit Hydrograph: -</u> The unit hydrograph theory is based on the concept that if two identical storms occur on a drainage basin with identical condition, then unit hydrograph of runoff from the two storms may be expected as same. This conception of unit hydrograph was first given by L.K. Sherman in 1932.

CHAPTER 3

CHAPTER-3

WATER REQUIREMENTS OF CROPS

3.1 CROP SEASONS:-

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

- Kharif season: -this seasons ranges from June to October. The crops are sown in the very beginning of monsoon and harvested at the end of autumn. Ex- rice, millet, jute, groundnut, etc.
- Rabi seasons: -this season ranges from October to March. These crops are sown in the very beginning of winter & harvested at the end of spring. Ex- wheat, gram, mustard, onion, etc.

3.2 DUTY:-

- The duty of water is defined as number of hectares that can be irrigated by constant supply of water at the rate of one cumec throughout the base period.
- It is expressed in hectares/cumec.
- It is denoted by "D".
- The duties of some common crops are

Crop	duty in hectares/cumec
Rice	900
Wheat	1800
Cotton	1400
Sugarcane	800

DELTA:-

- Each crop requires certain amount of water per hectare for its maturity. It is the total amount of water supplied to the crop(from 1st watering to last watering) is stored on the land without loss, then there will be a thick layer of water standing on the land. This depth of water layer is known as delta.
- It is denoted by .
- It is expressed in cm.

Kharif crop	Delta in cm

Rice	125
Maize	45
Ground nut	30
Millet	30

Rabi crop	delta in cm
Wheat	40
Mustard	45
Gram	30
Potato	75

BASE:-

- Base period is the whole period from irrigation or first watering for preparation of the ground for planting the crop to its last watering before harvesting.
- It is denoted by "B".
- It is expressed in days.

Crop	Base in days
Rice	120
Wheat	120
Maize	100
Cotton	200
Sugarcane	320

Relation between base duty and delta:-

Let ,

D =duty of water in hectares/cumec

B =base in days,

D =delta in m

From definition, one cumec of water flowing continuously for "B" days gives a depth of water D over an area D hectares. That is

1 cumec for B days gives D over D/B hectares

1 cumec for 1 days gives D over D/B hectares

1 cumec-day =D/B D hectare-meter

Again, 1 cumecdays = 1*24*60*60=86400^3

$$delta\ (\Delta) = 8.64\frac{B}{D}$$

Where,

Δ=delta in meter

B=base period in days

D=duty in hectares/cumec

Problem 1:- a channel is to be designed for irrigating 5000 hectares in kharif crop & 4000 hectares in rabi crop. The water requirement for kharif & rabi are 60 cm & cm, respectively. The kor period for kharif is 3 weeks & for rabi is 4weeks. Determine the discharge of the channel for which to be designe

Solution:-

D=8.64B/D

Discharge for kharif cro-

D=60cm=0.60m

B=3weeks=21days

Duty=(8.64*21)/0.60=302.4 hectares/cu

Area to be irrigated =5000 hecta

Required discharge of channel=5000/302.4=16.53c

Discharge for rabi crop:-

D=25cm=0.25m

B=4weeks=28 days

Duty=(8.64*28)/0.25=967.68 hectares/cu

Area tobe irrigated =4000 hecta

Required discharge of channel=4000/967.68=4.13c

Problem 2:- the command area of a channel 4000 hectares. The intensity of irrigation of a crop is 70%. The crop requires 60 cm of water in 15 days, when the effective rainfall is recorded as a 15cm during that period. Find,

a) the duty at the head of field.
b) the duty at the head of field.
c) The head discharge at the head of channel.

Assume total losses as 15%.

Solution:-

Depth of water required =60 cm

Effective rain fall =15cm

Depth of irrigation water = 60-15=45cm

Delta =45cm=0.45m, B = 15days

From relation D =(8.64*B)/D

Duty D = (8.64*15)/0.45 =288 hectares/cumec

a) So, duty at the head of field =288 ha/cumec. Due to the losses of water the duty at the head of the channel will be reduced. As losses are 15%.
b) So, the duty at the head of channel = 288*(85/100) = 244.80 hect/cumec.
c) Total area under crop =4000 * (70/100) =2800 hect

   The discharge at the head of channel = 2800/244.8 = 11.438 cumec.

3.3 GROSS COMMAND AREA:-

The whole area enclosed between an imaginary boundary line which can be included in an irrigation project for supplying water to agricultural land by the networks of canal is known as gross command area. It includes the culturable & unculturable area.

unculturable area:-

the area where the agriculture cannot be done & crops cannot be grown is known as unculturable area.

culturable area:-

the area where the agriculture can be done satisfactorily is known as culturable area

CULTURABLE COMMAND AREA:-

The total area within an irrigation project where the cultivation can be done & crops can be grown is known as culturable command area. Again c.c.a may be of two categories.

a) Culturable cultivated area:- it is the area within c.c.a where the cultivation has been actually done at present.

b) Culturable uncultivated area:- it is the area within c.c.a where the cultivation is possible but is not being cultivated at present due to some reasons.

INTENSITY OF IRRIGATION:-

- The total culturable command area may not be cultivated at the same time in a year due to various reasons. Some area may remain vacant every year. Again various crops may be cultivated in the culuturable command area. So, The intensity of irrigation may be defined as a ratio cultivated land for a particular crop to the total culturable command area
- It is expressed as a percentage of c.c.a.

FIELD CAPACITY:-

The field capacity is defined as the amount of maximum moisture that can be held by the soil against gravity.

PERMANENT WILTING POINT:-

It is defined as the amount of moisture held by soil which cannot be extracted by the roots for transpiration. At this point the wilting of the plant occurs. It is also expressed in percentage.

FREQUENCY OF IRRIGATION

The irrigation water is applied to the field to raise the moisture content of the soil up to its field 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 the immediately replaced by irrigation and it should be raised to the field capacity. The frequencyrrigation should be worked out in advance so that it can be applied in proper interva

a. The frequency of irrigation may be ascertainedthe following expressions,

Dw=——

where, Dw = Depth of water to be applied in eachtering;

d = Depth of root zone

Ws= Unit wt. of soil;

Ww = Unit wt. of water;

Fc = Field capacity;

M0 = Optimum moisture content.

b. fw =—

Where. fw = Frequency of watering;

Dw = Depth of water to be applied in each watering;

Cu = Daily consumptive use of water.

CHAPTER 4

FLOW IRRIGATION

DEFINATION:-

The irrigation system in which the water flows under gravity from the source to agricultural land is known as flow irrigation

PERENNIAL IRRIGATION :-

- In this irrigation water is available throught the year. Hydraulic structures are necessary across the river for raising the water level i.e
- Large area can be included under this system
- Negligible silting takes place in the canal bed

TYPES OF CANAL:-

1. BASED ON THE PURPOSE:-

Based on the purpose of service, the canal is 4 types

a) Irrigation canal:- the canal which is constructed to carry water from the source to the agricultural land for the purpose of irrigation is known as irrigation canal.
b) Navigation canal:-the canal which is constructed for the purpose of inland navigation is known as navigation canal.
c) Power canal:-the canal which is constructed to supply water with very high force to the hydroelectric power station for the purpose of revolving turbine to generate electric power is known as power canal.
d) Feeder canal:-the canal which is constructed to feed another canal or river for the purpose of irrigation or navigation is known as feeder canal

2. BASED ON THE NATURE OF SUPPLY:-

Based on the nature of supply the canal are 2 types

a) Inundation canal:- the canal which is excavated from the banks of the inundation river to carry the water to agricultural land in rainy season only is known as inundation canal.
b) Perennial canal:-the canal which can supply water to the agricultural land throughout the year is known as perennial canal.

3. BASED ON DISCHARGE:-

According to discharge capacity the canals are

a) Main canal:-the large canal which is taken directly from the diversion head work or from the storage to supply water to the network of small canal is known as main canal.
b) Branch canal:-the branch canals are taken from either side of main canal at suitable points so that the whole command area is covered by the network
c) Distributory channel:- these channels are taken from the branch canal to supply water to different sectors.
d) Field channel:- these channels are taken from the outlets of distributory channel by the cultivators to supply water to their own land

4. BASED ON ALLIGNMENT:-

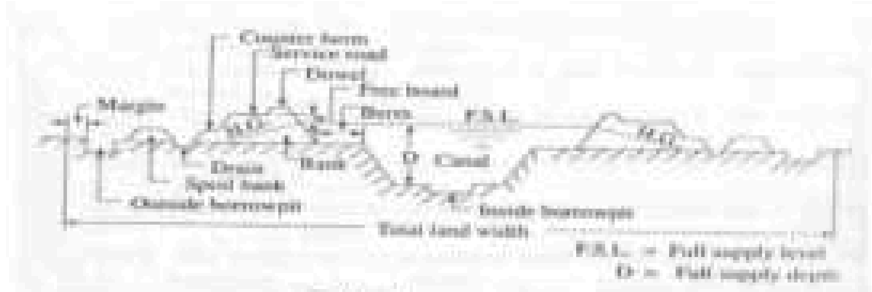
Depending upon the alignment the canal are following types

a) Ridge or water shed canal:- the canal which is aligned along the ridge line is known as ridge canal.
b) Contour canal:- the canal which is aligned approximately parallel to the contour lines is known as contour canal.
c) Side slope canal:-the canal which is aligned approximately at right angle to the contour lines is known as side slope canal.

CANAL SECTION

TERMS RELATED TO CANAL SECTION :

1. Canal bank
2. Berm
3. Hydraulic gradient
4. Counter berm
5. Free board
6. Side slope
7. Service road or inspection road
8. Dowel or Dowla
9. Borrow pit
10. Spoil bank
11. Land width



canal section

CANAL BANK:

The canal bank is necessary to retain water in the canal to the full supply level. According to different site conditions the bank of the canals are two types..

(a) Canal bank in cutting :

The banks are constructed on both side of the canal to provide only a inspection load. The side slope will be 1.5:1 or 2:1 according to the nature of soil.

(b) Canal bank in full banking:

Both canal banks are constructed above the ground level. The height of the bank will be high and the section will be large due to hydraulic gradient.

BERM:

The distance between toe of the bank and the top edge of the cutting is called berm.

The berm is provided for following reasons:

(a) To protect the bank from erosion.

(b) To provide a space for widening the canal section in future if necessary.

(c) To protect the bank from sliding down towards the bed section.

(d) The silt deposition on the berm makes an impervious lining.

(e) If necessary borrow pit can be excavated on the berm.

The width of the berm varies from D to 2D, where D is the full supply level

HYDRAULIC GRADIENT :

When water is retained by the canal bank the seepage occurs through the body of the bank. Due to the resistance of soil, the saturation line forms a sloping line which may pass through countryside of the bank. The sloping line is known as hydraulic gradient.

SOIL	H.G
Clayey soil	1:4
Sandy soil	1:6
Alluvial soil	1:5

COUNTER BERM:

When the water is retain by a canal bank the hydraulic gradient line passes through the body of the bank. The gradient should not intersect the outer side of the bank. It should pass through the base and a minimum cover of 0.5m should always be maintained. It may occur that the hydraulic gradient line intersect the outside of the bank in that case a projection is provided on the bank to obtain minimum cover. This projection is known as counter berm.

FREE BOARD:

It is the distance between the full supply level and top of the bank. The amount of free board varies from 0.6 m to 0.75 m.

It is provided for the following reasons:

(a) To keep a sufficient margin so that the canal water does not overtop the bank.

(b) To keep the saturation gradient much below the top of the bank

SIDE SLOPE:

The side slope of the canal bank and canal section depends upon the angle of repose of the soil existing on side. So to determine the side slope of different sections the soil sample should be collected from the site and should be tested in the soil testing laboratory.

Permissible side slope for some soil

Types of soil	Side slope in cutting	Side slope in banking
Clayey soi	1:1	1 ½:1
Alluvial soil	1:1	2:1
Sandy loam	1 ½:1	2:1
Sandy soil	2:1	3:1

SERVICE ROAD:

The road is provided on the top of the canal bank for inspection and maintenance work is known as service road or inspection road. For main canal the service road are provided on both side of the bank but for branch canal the road is provided on the bank only. The width of the service road for main canal varies from 3-4 m. But finally these roads serve the purpose of communication between different villages.

DOWEL:

The protective small embankment which is provided on the canal side of the service road for safety of the vehicles playing on it is known as dowel or dowla. The top width is generally 0.5m and the height above the road level is about 0.5m.

SPOIL BANK:

When the canal is constructed in pooling the excavated earth may not be sufficient for forming the bank. In such case the extra earth is deposited in the form of small bank which is known as spoil bank. The spoil banks are provided on one side or both side of the canal bank depending upon the quantity of extra earth and available space. The spoil bank are not continuous sufficient space are left between the adjacent spoil banks for proper drainage.

BORROWPIT:

When the canal is constructed in partially cutting and partial banking, the excavated earth may not be sufficient for forming the required bank. In such case, the extra earth required for the construction of banks is taken from some pits which are known as borrow pits. The borrow pit may be inside or outside of the canal. The maximum depth should be 1m. The excavation is done in a no of borrow pit living a gap between them. The gap is generally half of the length of each borrow pit.

LAND WIDTH:

The total land width required for construction of canal depends upon the nature of site condition. Such as fully in cutting of fully in banking or partially in cutting and partially in banking. To determine the total width the following dimension should be added.

(a) Top width of the canal.

(b) Twice the berm width.

(c) Twice the bottom width of banks.

(d) A margin of one meter from the heel of the bank both sides.

(e) Width of the external borrow pit if any

(f) A margin of 0.5m from the outer edge of borrow pit both sides, if external borrow pit becomes necessary.

BALANCING DEPTH:

If the quantity of excavated earth can be fully utilized when making the bank on both side then that canal section is known as economical section. The depth of cutting for the idle condition is known as balancing depth.

SKETCHES OF DIFFERENT CANAL CROSS-SECTION:-

Canal in full cutting:-

Fig. 6.2 Canal in full cutting

canal in partially cutting & partially banking:-

Fig. 6.3 Canal in partial cutting and partial banking

canal in fully bankin:-

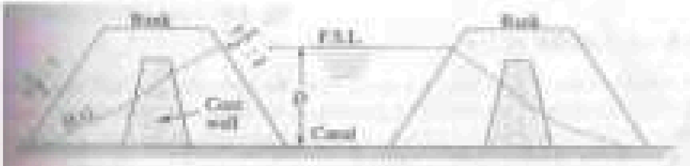


Fig. 6.4 Canal in full banking

CANAL LINING

OBJECTS OF CANAL LINING :

(a) To control seepage.

(b) To prevent water logging.

(c) To increase the capacity of canal.

(d) To increase the command area.

(e) To protect the canal from damage by flood.

(f) To control the growth of weeds.

ADVANTAGES OF CANAL LINING :

(a) It reduces the loss of water due to seepage and hence the duty is enhanced.

(b) It controls the water logging and hence the bad effects of water logging are eliminated.

(c) It provides smooth surface and hence the velocity of flow can be increased.

(d) Due to increased velocity the discharge capacity of canal is also increased.

(e) Due to increased velocity, the evaporation loss is also reduced.

(f) It eliminates the effect of scouring in the canal bed.

(g) The increased velocity eliminates the possibility of silting in the canal bed.

(h) It controls the growth of weeds along the canal sides and bed.

(i) It provides the stable section of the canal.

(j) It reduces the requirement of land width for the canal because smaller section of the canal can produce greater discharge.

(k) It prevents the sub soil salt to come in contact with the canal water.

(l) It reduces the maintenance cost of canals.

DISADVANTAGES:

(a) The initial cost of canal lining is very high

(b) It take too much time to complete the project work

(c) It involves much difficulties for repairing of damaged section of line

(d) It becomes difficult if the outlet are required to be shifted or new outlet are requred to be provided

TYPES OF LINING:

The following are the types of lining according to their site condition

(1) Cement concrete lining

(2) Pre-cast concrete lining

(3) Cement mortar lining

(4) Lime concrete lining

(5) Brick lining

(6) Boulder lining

(7) Shot crete lining

(8) Asphalt lining

(9) Bentonite and clay lining

(10) Soil-cement lining

1. CEMENT CONCRETE LINING :

This type of lining is recommended for canal in full banking. It is widely accepted as best impervious lining. The velocity of flow may be kept above 2.3 m/sec. It can eliminate completely growth of weeds.

Following are the steps for cement concrete lining.....

(A) Preparation of sub-grade :

The subgrade is prepared by ramming the surface properly with a layer of sand(about 15cm) . then a slurry of cement and sand (1:3) is spread uniformly over the prepared bed.

(B) Laying of concrete :

The cement concrete of grade M15 is spread uniformly according to desire thickness (100-150mm). after laying the concrete is tapped until the slurry comes on the top . then the curing is done for two weeks..

2. PRE-CAST CONCRETE LINING :

The lining is recommended for the canal in full banking. It consist of pre cast concrete slabs of size (60cm*60cm*5cm) which are set along the canal bank and bed with cement mortar (1:6). A network of 6mm dia rod is provided in the slab with spacing 10cm centre of centre. Expansion joints are also provided. The slabs are set in the following sequence,

(a) The sub grade is prepared by properly ramming the soil with a layer of sand.

(b) The slabs are stacked as per estimate along the course of the canal.

(c) The curing is done for a week.

3. CEMENT MORTAR LINING

This type of lining is recommended for the canal fully in cutting where hard soil or clayey soil is available. The thickness of cement mortar(1:4) is generally 2.5cm. This lining is impervious, but is not durable. The curing should be done properly.

4. LIME CONCRETE LINING

When hydraulic lime, surkhi and brick ballast are available in plenty along the course of the canal or in the vicinity of the irrigation project, then the lining of the canal may be made by the lime concrete of proportion 1:1:6. The thickness of concrete varies from 150mm to 225mm and the curing should be done for longer period. This lining is less durable then the cement concrete lining.

5. BRICK LINING

This lining is prepared by double layer brick flat soling laid with cement mortar (1:6) over the compacted sub grade. The first class bricks should be recommended for the work. The curing should be done properly. The lining is preferred for following reasons

(a) The lining is economical

(b) Work can be done very quickly

(c) Repair work can be done easily

(d) Brick can be manufactured from the excavated earth at the site

DISADVANTAGES

(a) It is completely impervious

(b) It has low resistance against erosion

(c) It is not so much durable

6. BOULDER LINING

In hilly areas where the boulders are available in plenty, this type of lining is generally recommended. The boulders are laid in single or double layer maintaining the slope of the banks and the level of the canal. The lining is very durable and impervious. But the transporting cost of material is very high. So ,it cannot be recommended for all cases.

7. SHOT CRETE LINING

In this system, the cement mortar is directly applied on the sub grade by equipment known as cement gun. The mortar is called as shot crete and the lining is known as shot crete lining. The process is known as guniting, as a gun is used for laying the mortar. The line is done in two ways

(A) By dry mix :

A mixture of cement and moist sand is prepared and loaded in the cement gun. Then it is forced through the nozzle of the gun with the help of compressed air. The mortar spreads over the sub grade in a thickness which varies from 2.5 cm to 5 cm.

(B) By wet mix :

The mixture of cement ,sand and water is prepared according to the approved consistency. The mixture is loaded in the gun and forced on the sub-grade.... This type of lining is very costly and not durable.

8. ASPHALT LINING

This lining is prepared by spraying asphalt at a very high temperature (about150') on the subgrade to a thickness from 3mm to 6mm. the hot asphalt when becomes cold forms a water proof membrane over the sub grade. This membrane is covered with a layer of earth and gravel. The lining is very cheap

9. BENTONITE AND CLAY LINING

A mixture of benotine and clay are mixed to gather to form a sticky mass. The mass is spread over the sub-grade to form an impervious membrane which is effective in controlling the seepage of water, but it cannot control the growth of weeds. This lining is recommended for small channel

10. SOIL-CEMENT LINING

This lining is prepared with a mixture of soil and cement. The usual quantity of cement is10 percent of the weight of soil. The soil and cement are thoroughly mixed to get an uniform texture. The mixture is laid on the surface of the sub-grade and it is made thoroughly compact. The lining is efficient to control the seepage of water, but it cannot control the growth of weeds. So this is recommended for small channels only.

SELECTION OF TYPES OF LINING :

(A) Imperviousness
(B) Smoothness
(C) Durability
(D) Economy
(E) Sitecondition
(F) Life of project

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CHAPTER- 6

DIVERSION HEAD WORKS AND REGULATORY STRUCTURES

Introduction

Any hydraulic structure which supplies water to the off-taking canal is called headwork. Head work may be divided into two classes such as

(a) Diversion Head Work and (b) Storage Head W

(a) Diversion Head Work : A diversion head work is that which divert t required supply into the canal from the ri

(b) Storage Head Work A storage head work is the construction of a dam across the river. It stores water during the period of exc supplies in the river releases it when demand overtakes available sup

Necessity of Diversion Head Work

(1) The necessity of diversion head works in the irrigation projects is to divert the riv water into the canal and a constant and continuous water supply is ensured in the canal even during the periods of low fl

(2) It controls the silt entry into the car

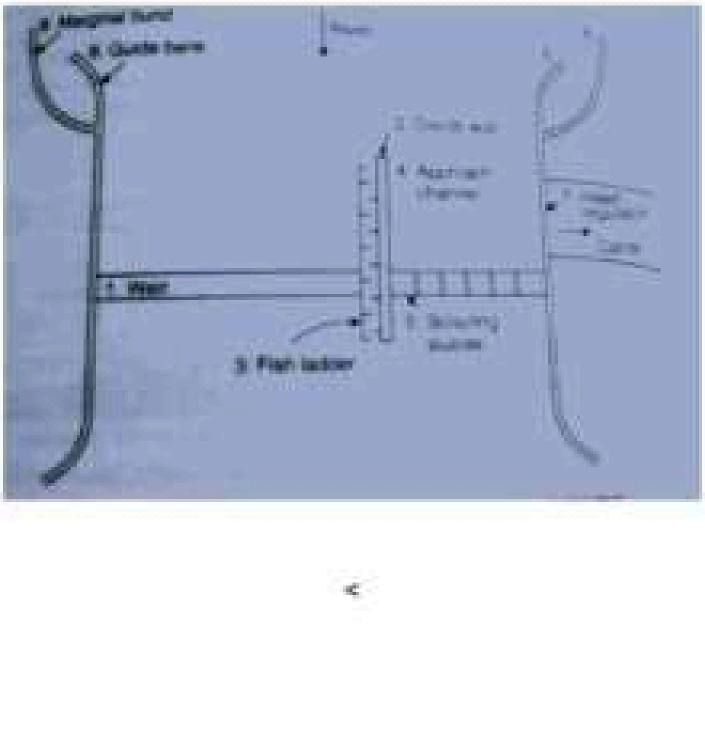
(3) It raises the water level in the river so that the commanded area can be incres

(4) It reduces fluctuations in the level of supply in the river

(5) It regulates the intake of water into the can

(6) It stores water for tiding over small periods of short supplies

General layout and different part of a Diversion Head Work



(Fig.1Layout of a diversion head work)

Divide Wall : A divide wall is constructed parallel to the direction of flow of river to separate the weir section and the under sluices section to avoid cross flows. If there are under sluices at both the sided, there are 2 divide walls.

Scouring sluices :Provide adjacent to the canal head regulator should be able to pass fair weather flow for which the crest shutters on the weir proper need not be dropped.

Fish Ladder : A passage provided adjacent to the divide wall on weir side for the fish to travel from up stream to down stream and vice versa. Fish migrate upstream or down stream in search of food or to rich their spreading places

Guide Banks : Guide Banks are provided on either side of the diversion head work for a smooth approach and to prevent the river from outflanking.

Marginal bunds : Marginal bunds are provided on either side of river up stream of diversion head works to protect the land and property which is likely to be submerged during ponding of water in floods.

Head regulators : A canal head regulator is provided at the head of the canal off taking from the diversion headwork. It regulates the supply of water into the canal, controls the entry silt into the canal and prevents the entry of river floods into canal.

A diversion head work is further divided in to two parts:

(a) Weir :

The weir is a solid obstruction put across the river to raise its water level and divert the water in to the canal.

Here the water level is raised up to the required height and the surplus water is allowed to flow over the weir.

Generally it is constructed across an inundation river. Weirs are commonly used to alter the flow of rivers to prevent flooding, measure discharge and help render rivers navigable.

If a weir also stores water for tiding over small periods of short supplies, it is called a storage weir.

The main difference between a storage weir and a dam is only in height and th duration for which the storage is stored. A dam reserves the supply for comparatively longer duratic

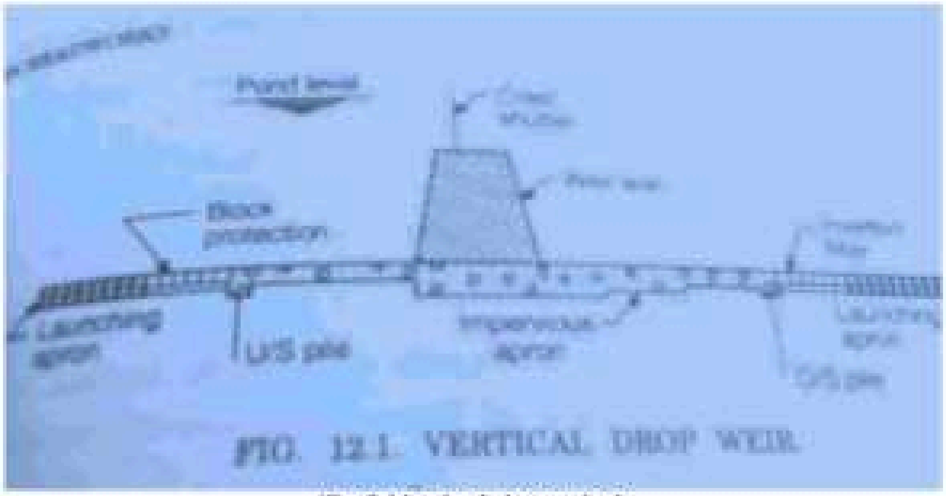


FIG. 12.1. VERTICAL DROP WEIR

(fig.2 Vertical drop wier)

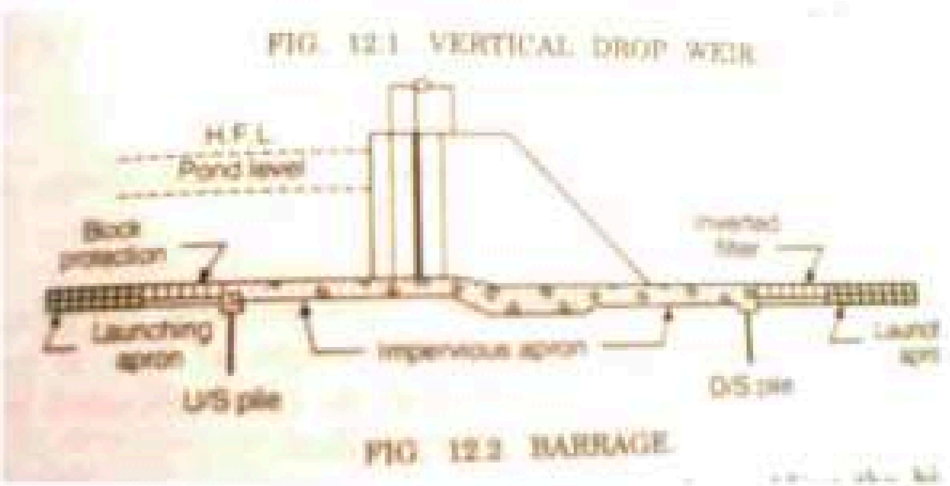
(b)Barrage :

The function of barrage is similar to that of a weir but the heading up of water is effected by the gates alo

It consists of a number of large gates that can be opened or closed to control t amount of water passing through the structure and regulate and stabilise riv water.

No solid obstruction is put across the river. The crest level in the barrage is kept a low level. During the floods the gates are raised to clear off the high flood leve enabling the high flood to pass downstream with minimum afflux. When the floc recedes, the gates are lowered and the flow is obstructed, thus raising the water le to the upstream of the barra

Due to this, there is less silting and better control over the level



(fig.3 barrage)

Differences between Weir and Barrage :

Barrage	Weir
Low set crest	High set crest
Gated over entire length	Shutters in part length
Gates are of greater height	Shutters are of smaller height
Gates are raised clear off the high floods	Shutters are dropped to pass floods
Perfect control on river flow	No control of river in low floods
Longer construction period	Shorter construction period
Costly structure	Relatively cheaper structure
Silt removal is done through under sluices	No means for silt disposal

Functions of Regulatory structures :

A regulatory structure is provided at the head of off- taking canal and serves the following functions.

(1) It regulates the supply of water entering the canal

(2) It controls the entry of silt in the canal.

(3) It prevents the river floods from entering the canal

Head Regulators and Cross regulators :

Head regulators and cross regulators regulate supplies of the off-taking channel and parent channel respectively. The distributary head regulator is provided at the head of the distributary and controls the supply entering distributary. A cross regulator is provided on the main canal at the down stream of the off-take to head up the water level and to enable the off-taking channel to draw the required supply.

Functions of distributary head regulators :

(1) These regulate orcontrol the supplies to the off-taking canal.

(2) To control silt entry into the off-take canal.

(3) To serve as a meter for measuring discharge.

Functions of cross regulators :

(1) To effectively control the entire canal irrigation system.

(2) When the water level in the main channel is low, it helps is heading up water on the up stream and to feed the off-take channels to full demand in rotation.

Falls :

Irrigation canals are constructed with some permissible bed slopes so that there is no silting or scouring in the canal bed. But it is not always possible to run the canal at the desired bed slope throughout the alignment due to the fluctuating nature of the country slope.

Generally the slope of the natural ground surface is not uniform through the alignment. Sometimes the ground surface may be steep and sometimes it may be very irregular with abrupt change of grade.

In this case " a vertical drop is constructed across the canal to lower down its water level and destroy the surplus energy liberated from the falling water which may otherwise scour the bed and banks of the canal. This is done to avoid unnecessary huge earth work in filling. Such vertical drops are known as Canal falls or Falls simply."

Types of Falls :

The followings are the different types of canal fall

(I) Ogee fall : In this type of falls, an ogee curve (a combination of convex and concave curve) is provided for carrying the canal water from higher level to lower level. This fall is recommended when the natural ground surface suddenly changes to a steeper slope along the alignment of canal. The fall consists of a concrete vertical wall and concrete bed.

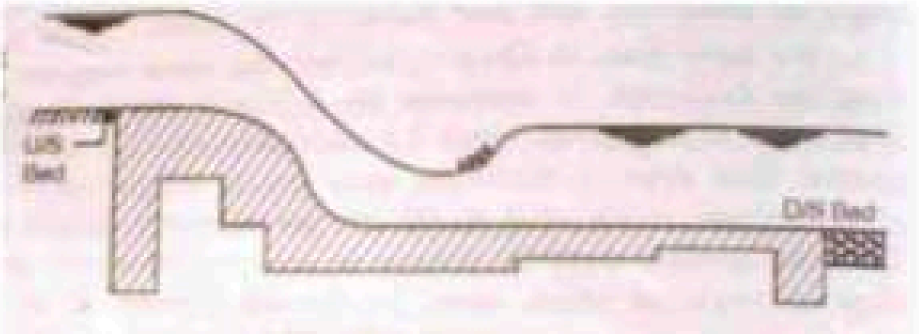


FIG. 18.1 OGEE FALL.

(fig.4 Ogee fall)

(II) Rapid fall : The Rapid fall is suitable when the slope of the natural ground surface is even and long. It consists of a long sloping glacis with longitudinal slope which varies 1 in vertical to 10 – 20 in horizontal. Curtain walls are provided on the upstream and downstream side of sloping glacis. The sloping bed is provided with rubble masonry. The masonry face is finishes with rich cement mortar (1:2).

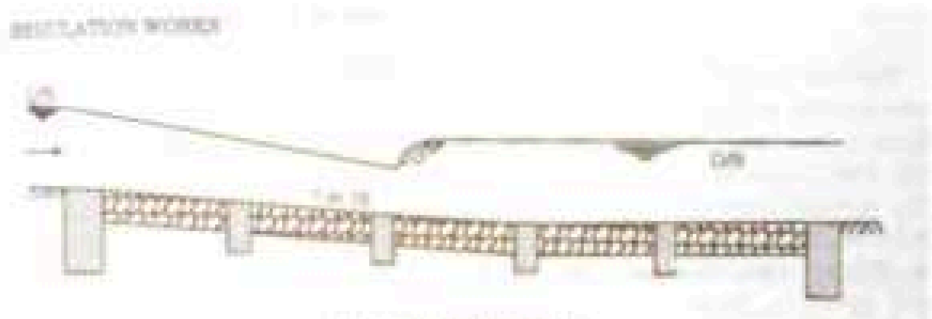
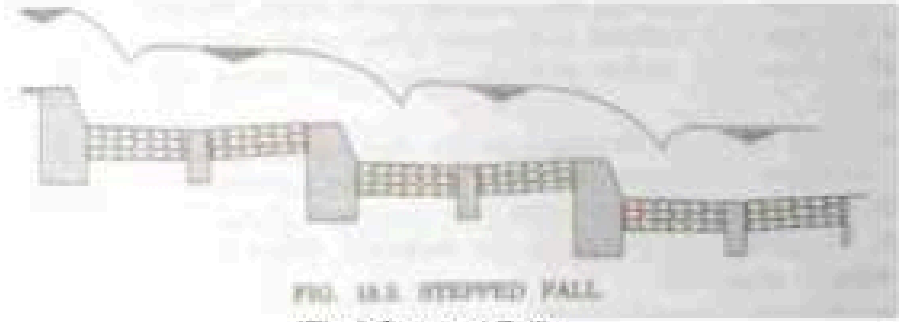


FIG. 18.2 RAPID FALL.

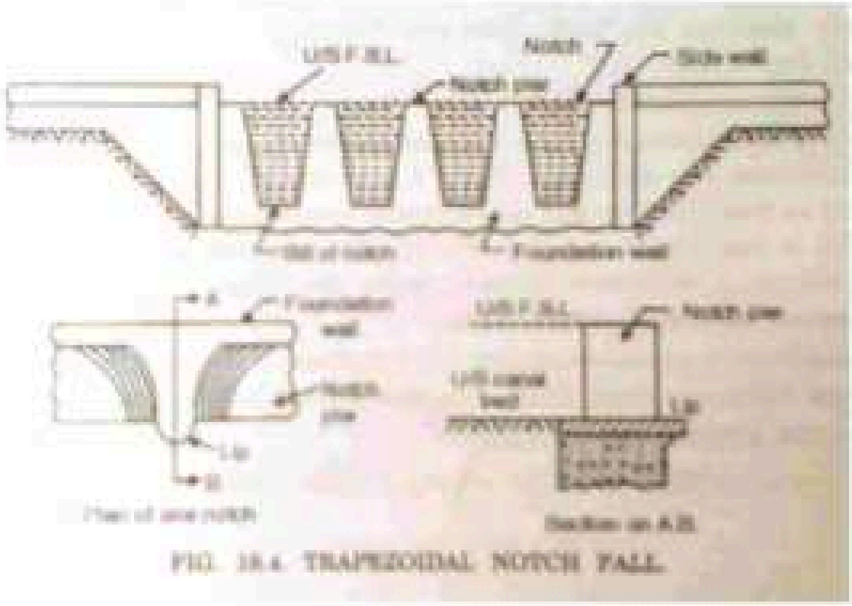
(Fig.5 Rapid Fall)

(III) Stepped fall : Stepped fall consists of a series of vertical drop in the form of steps. This fall is suitable in places where the sloping ground is very long and requires long glacis to connect the higher bed level with lower bed level. This fall is practically a modification of the rapid fall. The sloping glacis is divided into a number of drops so that the following water may not cause any dmage to the canal bed. Brick walls are provided at each of the drops.



(Fig.6 Stepped Fall)

(IV) Trapezoidal North Fall : In this type of fall a body wall is constructed across the canal. The body wall consists of several trapezoidal notches between the side piers and the intermediate pier or piers. The sills of notches are kept at the upstream bed level of the canal.



(Fig.7 Trapezoidal notch fall)

(V) Glacis type fall :The glacis type fall utilised the standing wave phenomenon for dissipation of energy. The glacis fall may be straight glacis or parabolic glacis type.

Energy dissipaters :

The water flowing over the spillway acquires a lot of kinetic energy by the time it reaches near the toe of the spillway. The arrangement is made to dissipate the huge kinetic energy and the velocity of water is reduced on downstream side near the toe of the dam. This arrangement is known as energy dissipaters.

Canal Outlets :-

An outlet is a small structure which admits water from the distributing channel to a water course or field channel. Thus, an outlet is a sort of head regulator for the field channel delivering water to the irrigation fields.

Types of Outlets :-

Outlets may be classified as 3 types :

(1) Non modular outlet

(2) Semi- module or Flexible Module

(3) Rigid module.

1) Non modular outlet :-A non modular outlet is the one in which the discharge depends upon the difference in level between the water levels in the distributing channel and water course. The discharge through an outlet varies in wide limits with the fluctuations of the water levels in the distributing and field channels. The common examples under this category are : Submerged pipe outlet, masonry sluice and Orifices.

2) Semi module or Flexible module :-A flexible outlet or Semi module outlets the one in which the discharge is affected by the fluctuations in the water level of the discharging channel while the fluctuations in the water levels of the field channel do not have any effect on its discharge.

Example:- Pipe Outlet, Kennedy's gauge outlet, Pipe open flume outlet etc.

3) Rigid Module :- A rigid module is the one which maintains constant discharge, within limits, irrespective of the fluctuations in water levels in the distributing channel or field channel. Example : Gibb's Rigid module.

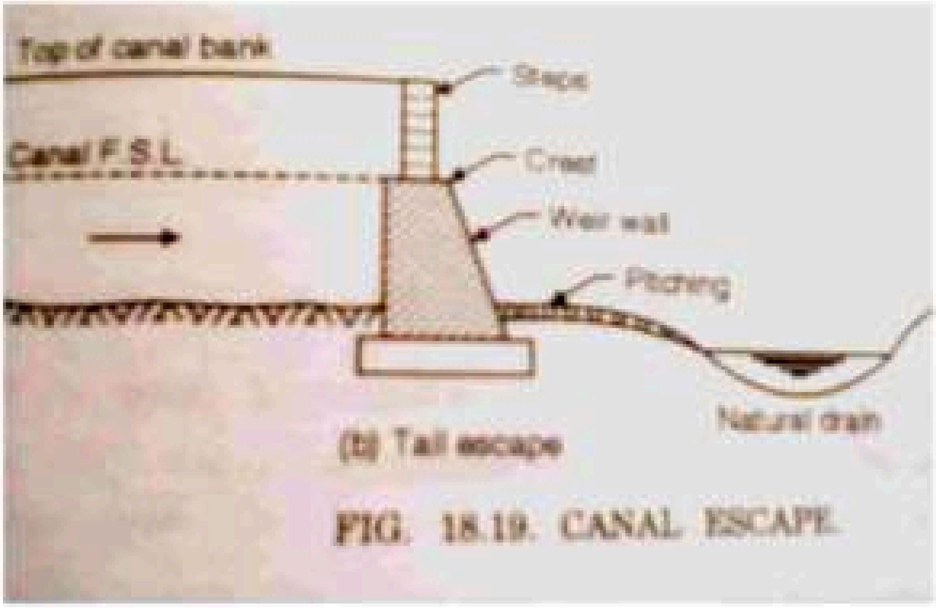
CANAL ESCAPES :-

Canal escapes is defined as an channel meant for removal of surplus or excess water from the canal into nearby drainage. The functions served by canal escape are :

(i) Safety valve to protect the canal against possible damage by flooding.

(ii) Emptying of the canal reach, below the escapes for weed removal, repairs and maintenance.

(iii) Periodical flushing off the silt prone head of canal through the escape.

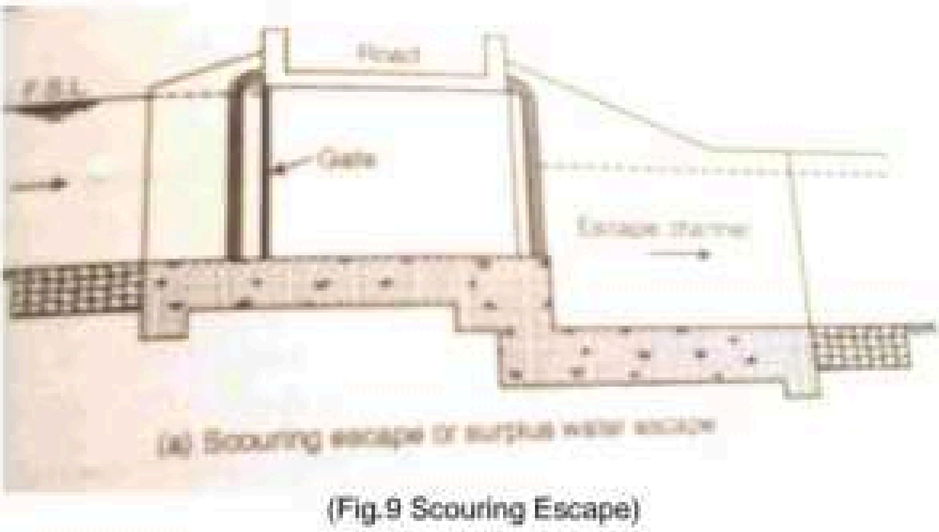


(Fig.8 Canal escape)

Depending upon the purpose, there can be 3 types of escapes such as :

a) Canal scouring escapes
b) Surplus escapes
c) Tail escapes

Canal scouring escapes :The scouring escape is constructed for the purpose of scouring off excess silt from time to time. Escapes are also constructed to dispose off excess supplies of the parent channel. Excess supplies in the canal take place either during heavy rains or due to the closure of canal outlet by the farmers. In that case, the escapes save the down stream section of the canal from overflow of banks.



(Fig.9 Scouring Escape)

Surplus escape :-A canal surplus escape may be weir type, with crest of weir wall at F.S.L. of parent bed level.

Tail escape :-A tail escape is required F.S.L. at tail end. The structure is weir type with its crest level at the required F.S.L. of canal at tail end.

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A Cross drainage works is a structure carrying the discharge from a natural stream across a Canal intercepting the stream.

OR

Canal comes across obstruction like rivers, natural drains, and other canals. The various types of structure that ate built to carry the canal water across the above mentioned obstructions or vice versa are called cross drainage works.

There are many different factors involved in selection of a specific type of cross drainage works and in selection of a suitable site for cross drainage works. It is generally very costly item & should be avoided by

- Diverting one stream into another.
- Changing the alignment of the canal, so that it crosses below the junction of two streams.

NECESSITY OF CROSS DRAINAGE WORKS

The following factors justify the necessity of cross drainage works.

(a) At the crossing point, the water of the canal and the drainage get intermixed, So for the smooth running of the canal with its design discharge, the cross drainage works are required.

(b) The site condition of the crossing point may be such that without any suitable structure, the water of the canal and drainage cannot be diverted to their natural directions. So, the cross drainage works must be provided to maintain their natural direction of flow.

(c) The water shed canals do not cross natural drainages. But in actual orientation of the canal network, this ideal condition may not be available and the obstacles like natural drainages may be present across the canal. So, cross- drainage works must be provided for running the irrigation system.

TYPES OF CROSS- DRAINAGE WORKS

The drainage water intercepting the canal can be disposed of in either of the following ways.

(1) By passing the canal over the drainage. The structures that fall under this type are:-

- (a) An Aqueduct.
- (b) Syphon aqueduct.

(2) By Passing the canal below the drainage. The structures that fall under this type are :

- (a) Super passage
- (b) Canal Syphon or syphon super passage.

(3) By passing the drain through the canal, so that canal water and drainage water are allowed to intermingle with each other.

Following are the structures under this type of cross drainage works:

- (a) Level crossing.
- (b) Inlet and outlet.

PROPER SITE FOR DAINAGE CROSSING

The site selected for the cross. Drainage works should have the following main characteristics.

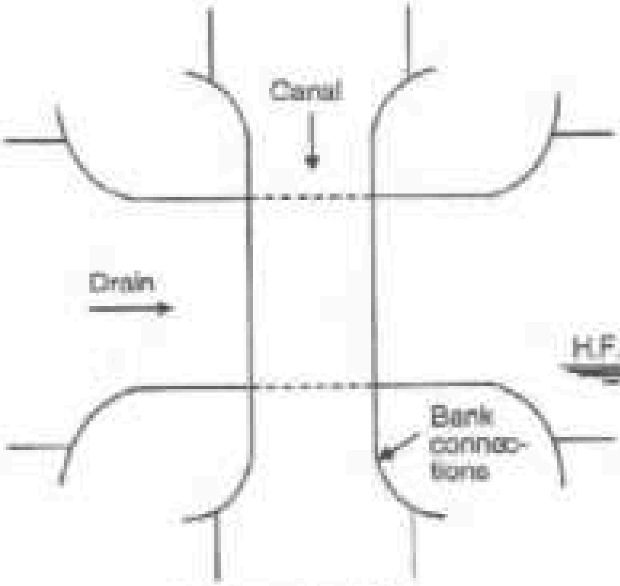
(1) It should be such that it requires minimum disturbance regarding the approach and tail reaches of the drainage channel.

(2) Suitable foundation soil should be available at reasonable depth.

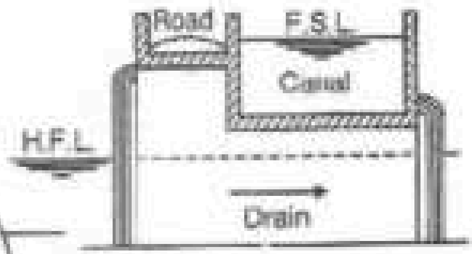
(3) Sufficient headway is available for the super structure of the aqueduct over the H.F.L. of the natural stream.

(4) Suitable existing topography, geological and hydraulic conditions for cross drainage works at reasonable costs.

<u>AQUEDUCT</u>



(a) Plan of crossing



(b) Aqueduct

The hydraulic structure in which the irrigation canal is taken over the drainage is known an aqueduct.

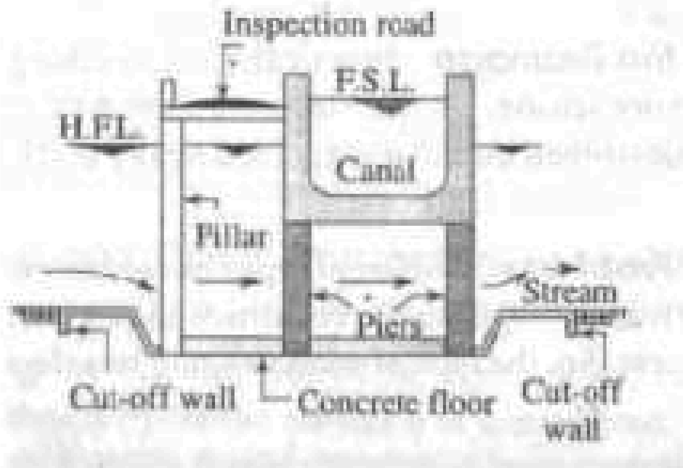
OR

When the HFL of the drain is sufficiently low the bottom under gravity, such type structure is known as aqueduct. In this type of work the canal water is taken across drainage in a trough supported an piers. An aqueduct is just like a bridge except that inste of carrying a road or a railway, it carries a canal on its top. The advantage of such arrangement is that the canal, running perennially above the ground and is open inspection.

An aqueduct is provided when sufficient level difference is available between t canal and natural drainage, and canal bed level is sufficiently higher than the torrent lev

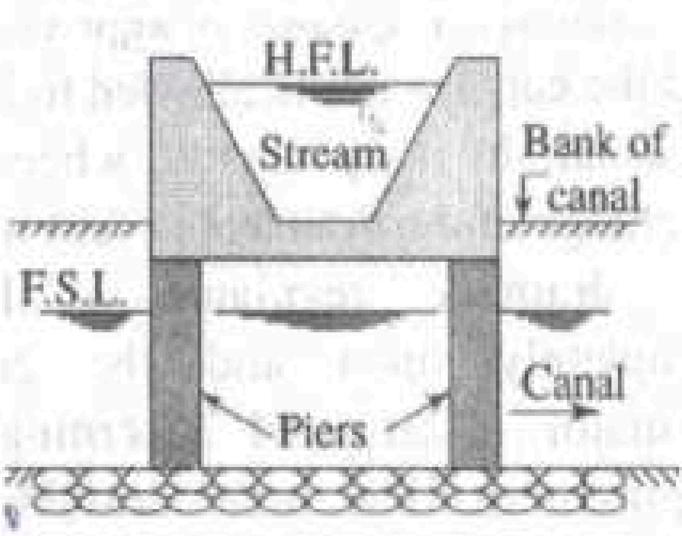
Generally the canal is in the shape of a rectangular trough and sometimes may trapezoidal section, which is constructed with R.C.C. The section of the trough is designe per the full supply level of the canal. The height section of piers are designed accordin the highest flood level and velocity of flow of the drainage. The piers constructed may R.C.C., stone masonry or brick masonry. According the availabilit of soil, the depth & type of foundation provided. The bed & banks of drainage is protected by boulder pitchi

SYPHON AQUEDUCT



In case of the syphon aqueduct, the HFL of the drain is much higher above the ca bed, and water runs under syphonic action through the aqueduct barrels tunnels. Here th sloping apron provided on both sides of the crossing. The apron may be constructed cement concert or stone pitching. The section of the drainage below the canal trough construed with P.C.C. in the form of tunnel or barrels. This Tunnel acts as a syphon. -off walls are provided on both sides of the apron to prevent scouring diring heavy flood. Al boulder pitching should be provided on the u/s and d/s of the cı-off wall.

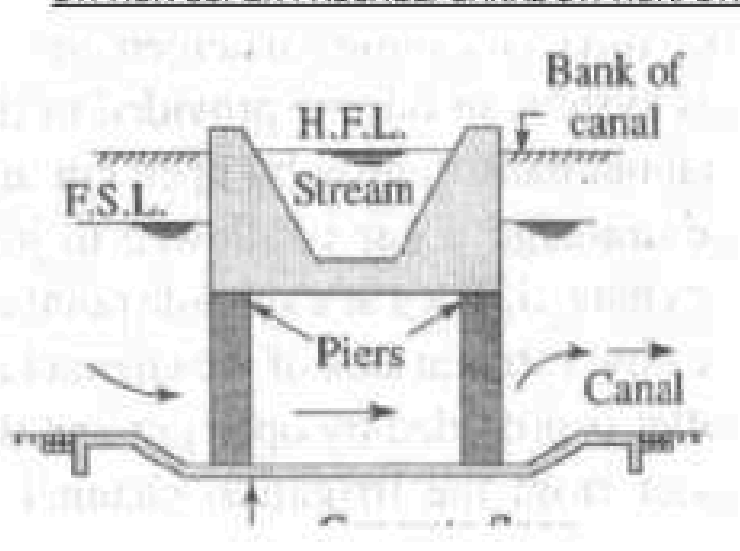
SUPER PASSAGE



The hydraulic structure in which the drainage is passing over the irrigation canal known as super passage. It is reverse of an aqueduct. A super passage is similar to aqueduct, except in this case the c is over the canal.

The FSL of the canal is lower than the underside of the trough carrying drainaı water. Thus the canal water runs under the gravity. The drainage is taken through rectangular or trapezoidal trough of channel which constructed on the deck supported t piers. The section of the drainage trough depends on the high flood discharge. A free boarc about 1.5 m. should be provided for safety. The trough should be constructed of R.C.C. ˈ bed & banks of the canal below the drainage th should be protected by boulder pitch or lining with concrete slabs.

SYPHON SUPER PASSAGE/ CANAL SYPHON/ SYPHOI

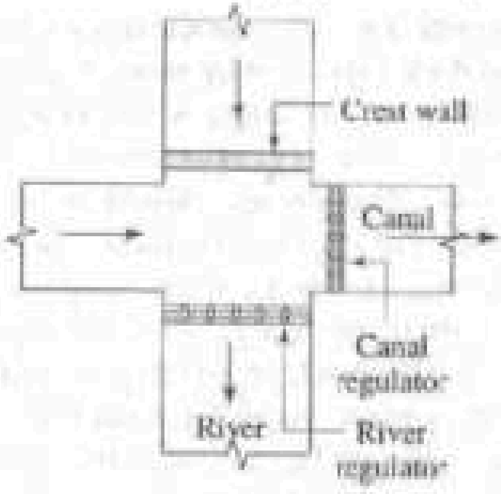


If the FSL of the canal is sufficiently above the bed level of the drainage trough, so that canal flows under syphonic action ur the trough, the structure is known as canal syphon a siphon super passage.

This structure is reverse of an syphon aqueduct. Section of the trough is design according to high flood discharge. The canal bed lowered and a ramp is provided at entry & exit, so that the trouble of silting is minimized. The sloping apron may constructed with stone pitching or concert slabs. The section of the canal below the trough constructed with cement concrete in the form of barrel which acts as siph. Cut-off walls are provided on u/s & d/s of the sloping apron to minimize scouring affect during high floo

LEVEL CROSSING

In this type of cross drainage work, the canal water and drain water are allowed intermingle with each other. A level cross is generally provided when a large canal huge drainage (Such as a stream or a river) approach each other practically at the same le



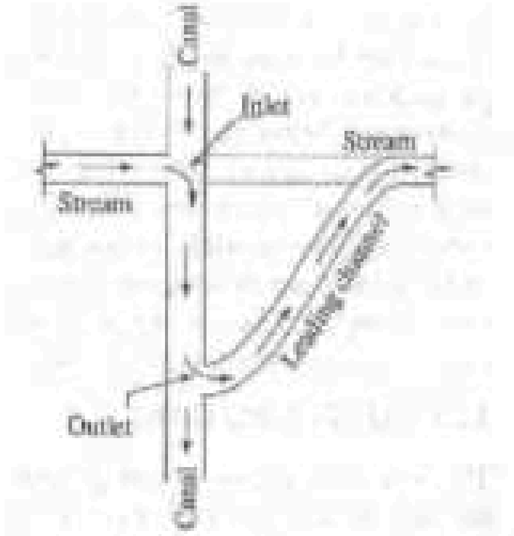
In this type of work, the drainage water is passed into the canal & then taken out the opposite bank. The work consists

I. Construction of crest, with its top at the FSL of the canal, at the u/s juncti with the canal.
II. Provision of the head regulator across the drainage at its d/s junction with th canal.
III. A cross regulator across the cl at its d/s junction with the drainag

A regulator at the end of the incoming canal is also sometimes require

When the drainage does not carry any water, its regulator is closed while the cro regulator of the canal is kept open so that the canal flows without any interruption. Durin the floods, however, the drainage regulator is opened so that the flood discharge, a spilling over the crest & mixing with the canal water passes through it to the downstrear the drainage.

INLET & OUTLET



In Case of crossing of a small irrigation channel with a small drainage, no hydrau structure is constructed, because, the discharge of the drainage and the channel practically low and these can be easily tackled by easy system like inlet and outle arrangement. In this system an inlet is provided in the channel bank simply by open cut the drainage water is allowed to join the channel. Then at a suitable point on the downstre side of the channel an outlet is provided by open cut and the water from the irrigation cha is allowed to flow through a leading channel towards the original course of the drainage. the points of inlet the bed and banks of the drainage are protected by stone pitching. The and banks of the irrigation channel between inlet and outlet points should also be protect by stone pitching.

According to the relative bed levels of the canal and the river or drainage, the typ of cross drainage works are generally selected. However, in actual field such idea conditions may not be available and the choice of the work then depends on following poir-

(a) The crossing should be at right angles to each c

(b) Availability of funds.

(c) Suitability of soil for embankment

(d) Position of water table and availability of dewatering equipments

(e) Well defined c/s of the canal or river or drainage should be available.

(1) Availability of suitable foundation:-

For the construction of cross drainage works suitable foundation is required. By boring test, if suitable foundation is not available, then the type cross drainage works should be selected according to site condition.

(2) Economical Consideration:-

The cost of construction of cross drainage works should be justified with respect to the project cost and overall benefits of the project. So, the type of works should be selected considering the economical point of view.

(3) Discharge of the Drainage:-

Practically, the discharge of the drainage is very uncertain in rainy season. So, the structure should be carefully selected so that it may not be destroyed due to unexpected heavy discharge of the river or drainage

(4) Construction Problems:-

Different types of constructional problems may arise at the site such as sub-soil water, construction materials, communication, availability of land, etc. So, the type of works should be selected according to the site condition.

CHAPTER-8

DAM

8.1 INTRODUCTION

A dam is a hydraulic structure of fairly impervious material built across a river to create a reservoir on its upstream side for impounding water for various purposes. It is suitable in hilly region where a deep gorge section is available for the strange reservoir. These purposes may be Irrigation, Hydro-power, Water-supply, Flood Control, Navigation, Fishing and Recreation. Dams may be built to meet the one of the above purposes or they may be constructed fulfilling more than one. As such, it can be classified as: Single-purpose and Multipurpose Dam.

The dam is meant for serving multipurpose functions such as,

(a) Irrigation,(b)Hydroelectric power generation,(c)Flood control, (d)Water supply, (e) Fishery, (f) Recreation.

Weir and Barrage are also impervious barriers across the river. Which are suitable in plain terrain but not in hilly region. The purpose of weir is only to raise the water level to some desired height and the purpose of barrage is to adjust the water level at different levels when required. These hydraulic structures are suitable for irrigation only.

DIFFEREENT PARTS & TEREMELOGY OF DAMS:-

Crest: The top of the dam structure. These may in some cases be used for providing a roadway or walkway over the dam.

Parapet walls: Low Protective walls on either side of the roadway or walkway on the crest.

Heel: Portion of structure in contact with ground or river-bed at upstream side.

Toe: Portion of structure in contact with ground or river-bed at downstream side.

Spillway: It is the arrangement made (kind of passage) near the top of structure for the passage of surplus/ excessive water from the reservoir.

Abutments: The valley slopes on either side of the dam to which the left & right end of dam are fixed to.

Gallery: Level or gently sloping tunnel like passage (small room like space) at transverse or longitudinal within the dam with drain floor for seepage water. These are generally provided for having space for drilling grout holes and drainage holes. These may also be used to accommodate the instrumentation for studying the performance of dam.

Sluice way: Opening in the structure near the base, provided to clear the silt accumulation in the reservoir.

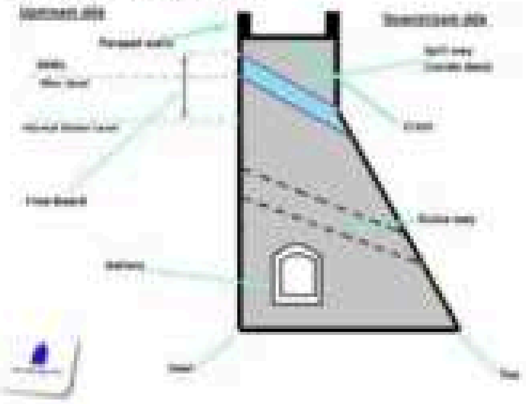


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Illustration of dam- parts in a typical cross section (click the image to view it clearly)

Free board: The space between the highest level of water in the reservoir and the top of the structure.

Dead Storage level: Level of permanent storage below which the water will not be withdrawn.

Diversion Tunnel: Tunnel constructed to divert or change the direction of water to bypass the dam construction site. The hydraulic structures are built while the river flows through the diversion tunnel.

SELECTION OF SITE FOR DAM

While selecting the site for a dam, the following points should be considered:

Good rocky foundation should be available at the dam site. The nature of the foundation soil should be examined by suitable method of soil exploration.

The river valley should be narrow and well defined so that the length of the dam may be short as far as possible.

Site should be in deep gorge section of the valley so that large capacity storage can be formed with minimum surface area and minimum length of dam.

Valuable property and valuable land should not be submerged due to the construction of dam.

- The proposed river or its tributaries should not carry large quantity of sediment. If unavoidable, the sources of sediment should be located and necessary measures should be recommended to the arrest sediment.

The site should be easily accessible by road or railway for the transport of construction materials, equipment's, etc.

The construction materials should be available in the vicinity of the dam site.

Sufficient space should be available near the site for the construction of labour colony, godowns and staff quarters for the persons associated with the constructional activities.

The basin should be free from cracks, fissures, etc to avoid percolation loss. It is done by physical verification and other observation. If unavoidable, the area should be located and necessary measures should be recommended to make the area leak-proof.

From the rainfall records in the catchment area empirical formulae the maximum discharge of the river should be ascertained whether the required quantity of water shall be available or not.

INVESTIGATION WORKS FOR DAM SITE

The following investigation works should be done before final selection of dam site and the preparation of the project report.

1. Preliminary Survey: the preliminary survey involves the following steps.

   (a) Reconnaissance survey the reconnaissance survey should be conducted for the dam site and surrounding area to gather information regarding the natural features of the area, nature of dam site, location of labour colony and staff quarters, stack yard, dodowns, etc. the nature of land and the localities in the basin area should also be recorded. And index map should be prepared.

   (b) Topographical survey: A topographical map is to be prepared for the proposed project area by traverse surveying. The traverse survey may be conducted any suitable method depending on the nature of the area.

   (c) Contour Survey: A contour map should be prepared for the basin area to determine the capacity of the reservoir.

   (d) Contour Survey: longitudinal leveling and cross-sectional leveling should be done at the dam site at least one km upstream and downstream of the proposed centerline of the dam. This is done to select the best suitable dam site.

2. Geological Survey:the geological survey involves the following steps

   a. Soil Survey: To work the nature of the foundation at the dam site. Soil exploration should be done by suitable method. The sub-soil formation should be thoroughly studied to determine the type of foundation for the dam.

   b. Study of formation in basin area: Soil exploration should be done at different spot in the basin area to ascertain the nature of sub-soil. This is done to calculate the probable percolation loss.

   c. Study of source of Sediment:The sources of sediments carried by the river or its tributaries should be studied and located, slips or areas of loose soil with mica particles are found, then the stabilization of those areas should be done.

3. Hydrological Survey: It involves the following steps:

   a. Gauge and discharge site:The gauge and discharge stations should be established near the dam site to record the discharge of the river throughout the year.

   b. Site analysis: In rainy season the river carries heavy silt or sediment. The analysis of the silt should be carried out through the season for some specific period to determine grade of silt. This done to ascertain the possible sedimentation in the reservoir and thus suitable methods can be employed to reduce the sedimentation.

4. Communication Survey: The route survey for the possible communication to the dam site the nearest highway or railway station should be done. It involves the preparation of longitudinal section and cross-section along the proposed alignment. It is done to estimate the cost of construction of this connecting road or railway lie. The possible route for telephone communication and electrical connections should also be located.

5. Construction Materials Survey: The availability of construction materials like stone, sand, etc. should be located in the topographical map of the concerned district or state. The possible route for carrying these materials should also be located in the map.

6. Compensation Report: A detailed report should be prepared for the compensation which is likely to be paid by the government during the implementation of the project. This will include the dam site, area of labour colony and staff quarters, area required for stockyards and godowns, valuable lands and properties that may be submerged by the reservoir, etc.

7. Project Report: The project involves the following steps:

   a. Design and estimate of dam and other allied structures

   b. Detailed drawings of dam section with foundation and other buildings or structures.

   c. Detailed estimate for the road or railway communications

   d. Comprehensive report for compensation.

   e. The project is forwarded to the higher authority with recommendation for approval.

   f. The project is forward to the higher authority with recommendation for approval.

CLASSIFICATION OF DAMS:

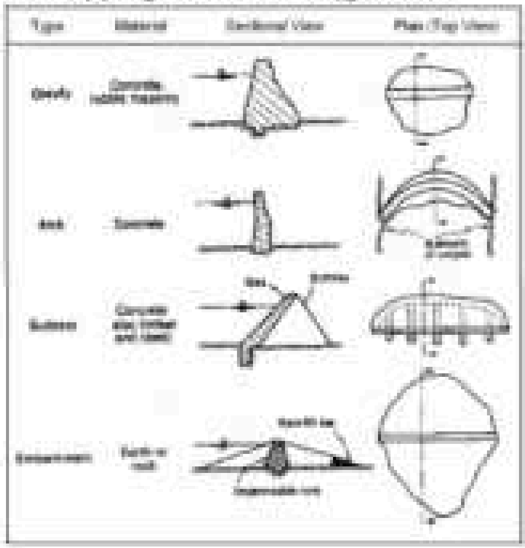
Dams can be classified in number of ways. But most usual ways of classification i.e. types of dams are mentioned below:

<u>Based on the functions of dams, it can be classified as follows:</u>

1. Storage dams: They are constructed to store water during the rainy season when there is a large flow in the river. Many small dams impound the spring runoff for later use in dry summers. Storage dams may also provide a water supply, or improved habitat for fish and wildlife. They may store water for hydroelectric power generation, irrigation or for a flood control project. Storage dams are the most common type of dams and in general the dam means a storage dam unless qualified otherwise.
2. Diversion dams: A diversion dam is constructed for the purpose of diverting water of the river into an off-taking canal (or a conduit). They provide sufficient pressure for pushing water into ditches, canals, or other conveyance systems. Such shorter dams are used for irrigation, and for diversion from a stream to a distant storage reservoir. It is usually of low height and has a small storage reservoir on its upstream. The diversion dam is a sort of storage weir which also diverts water and has a small storage. Sometimes, the terms weirs and diversion dams are used synonymously.
3. Detention dams: Detention dams are constructed for flood control. A detention dam retards the flow in the river on its downstream during floods by storing some flood water. Thus the effect of sudden floods is reduced to some extent. The water retained in the reservoir is later released gradually at a controlled rate according to the carrying capacity of the channel downstream of the detention dam. Thus the area downstream of the dam is protected against flood.
4. Debris dams: A debris dam is constructed to retain debris such as sand, gravel, and drift wood flowing in the river with water. The water after passing over a debris dam is relatively clear.
5. Coffer dams: It is an enclosure constructed around the construction site to exclude water so that the construction can be done in dry. A coffer dam is thus a temporary dam constructed for facilitating construction. These structure are usually constructed on the upstream of the main dam to divert water into a diversion tunnel (or channel) during the construction of the dam. When the flow in the river during construction of hydraulic structures is not much, the site is usually enclosed by the coffer dam and pumped dry. Sometimes a coffer dam on the downstream of the dam is also required.

Based on structure and design, dams can be classified as follows

1. Gravity Dams: A gravity dam is a massive sized dam fabricated of concrete or stone masonry. They are designed to hold back large volumes of water. By using concrete, the weight of the dam is actually able to resist the horizontal thrust of water pushing against it. This is why it is called a gravity dam. Gravity essentially holds the dam down to the ground, stopping water from toppling over

   a. 

   b. Types of dam

      i. Gravity dams are well suited for blocking rivers in wide valleys or narrow gorge ways. Since gravity dams must rely on their own weight to hold back water, it is necessary that they are built on a solid foundation of bedrock.
      Examples of Gravity dam: Grand Coulee Dam (USA), Nagarjuna Sagar (India) and Itaipu Dam (It lies Between Brazil and Paraguay) is the largest in the world

2. Earth Dams: An earth dam is made of earth (or soil) built up by compacting successive layers of earth, using the most impervious materials to form a core and placing more permeable substances on the upstream and downstream sides. A facing of crushed stone prevents erosion by wind or rain, and an ample spillway, usually of concrete, protects against catastrophic washout should the water overtop the dam. Earth dam resists the forces exerted upon it mainly due to shear strength of the soil. Although the weight of the structure also helps in resisting the forces, the structural behavior of an earth dam is entirely different from that of a gravity dam. The earth dams are usually built in wide valleys having flat slopes at flanks (abutments). The foundation requirements are less stringent than those of gravity dams, and hence they can be built at the sites where the foundations are less strong. They can be built on all types of foundations. However, the height of the dam will depend upon the strength of the foundation material.
   Examples of earthfill dam: Rogunsky dam (Russia) and New Cornelia Dam (U

3. Rockfill Dams: A rockfill dam is built of rock fragments and boulders of large size. An impervious membrane is placed on the rockfill on the upstream side to reduce the seepage through the dam. The membrane is usually made of cement concrete or asphaltic concrete.

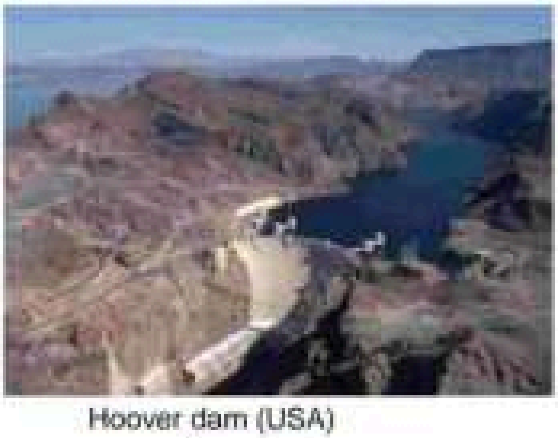
a. 

b. Mohale Dam, Lesotho, Africa

In early rockfill dams, steel and timber membranes were also used, but now they are obsolete. A dry rubble cushion is placed between the rockfill and the membrane for the distribution of water load and for providing a support to the membrane. Sometimes, the rockfill dams have an impervious earth core in the middle to check the seepage instead of an impervious upstream membrane. The earth core is placed against a dumped rockfill. It is necessary to provide adequate filters between the earth core and the rockfill on the upstream and downstream sides of the core so that the soil particles are not carried by water and piping does not occur. The side slopes of rockfill are usually kept equal to the angle of repose of rock, which is usually taken as 1.4:1 (or 1.3:1). Rockfill dams require foundation stronger than those for earth dams.

Examples of rockfill dam: Mica Dam (Canada) and Chesen Dam (Mexico).

4. Arch Dams: An arch dam is curved in plan, with its convexity towards the upstream side. They transfer the water pressure and other forces mainly to the abutments by arch action.



Hoover dam (USA)

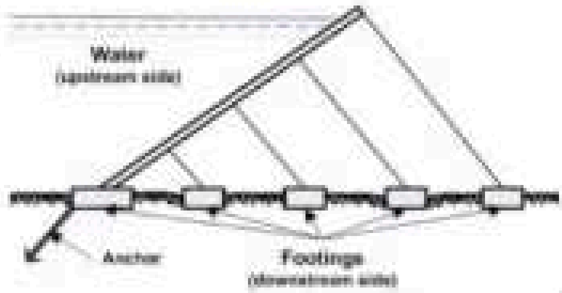
An arch dam is quite suitable for narrow canyons with strong flanks which are capable of resisting the thrust produced by the arch action. The section of an arch dam is approximately triangular like a gravity dam but the section is comparatively thinner. The arch dam may have a single curvature or double curvature in the vertical plane. Generally, the arch dams of double curvature are more economical and are used in practice. Examples of Arch dam: Hoover Dam (USA) and Idukki dam (India).

5. Buttress Dams Buttress dams are of three types : (i) Deck type, (ii) Multiple-arch type, and (iii) Massive-head type. A deck type buttress dam consists of a sloping deck supported by buttresses. Buttresses are triangular concrete walls which transmit the water pressure from the deck slab to the foundation. Buttresses are compression members. Buttresses are typically spaced across the dam site every 6 to 30 metre, depending upon the size and design of the dam. Buttress dams are sometimes called hollow dams because the buttresses do not form a solid wall stretching across a river valley.The deck is usually a reinforced concrete slab supported between the buttresses, which are usually equally spaced

In a multiple-arch type buttress dam the deck slab is replaced by horizontal arches supported by buttresses. The arches are usually of small span and made of concrete. In a massive-head type buttress dam, there is no deck slab. Instead of the deck, the upstream edges of the buttresses are flared to form massive heads which span the distance between the buttresses. The buttress dams require less concrete than gravity dams. But they are not necessarily cheaper than the gravity dams because of extra cost of form work, reinforcement and more skilled labour. The foundation requirements of a buttress are usually less stringent than those in gravity dam. Examples of Buttress type: Bartlett dam (USA) and The Daniel-Johnson Dam (Canada).

6. Steel Dams Dams: A steel dam consists of a steel framework with a steel skin plate on its upstream face. Steel dams are generally of two types: (i) Direct-strutted, and (ii) Cantilever type . In direct strutted steel dams, the water pressure is transmitted directly to the foundation through inclined struts. In a cantilever type steel dam, there is a bent supporting the upper part of the deck, which is formed into a cantilever truss. This arrangement introduces a tensile force in the girder which can be taken care

of by anchoring it into the foundation at the upstream toe. Hovey suggested t tension at the upstream toe may be reduced by varying the slopes of the lower str in the bent.



However, it would require heavier sections struts. Another alternative to redu tension is to frame together the entire bent rigidly so that the moment due to t weight of the water on the lower part of the deck is utilized to offset the momen induced in the cantilever. This arrangement would, however, require bracing and tl will increase the cost. These are quite costly and subjected to corrosion. The dams are almost obsolete. Steel dams are sometimes used as temporary coffer dams during the construction of the permanent one. Steel coffer dams are supplemen with timber or earthfill on the inner side to make them water tight. The area betw the coffer dams is dewatered so that the construction may be done in dry for the permanent dam.

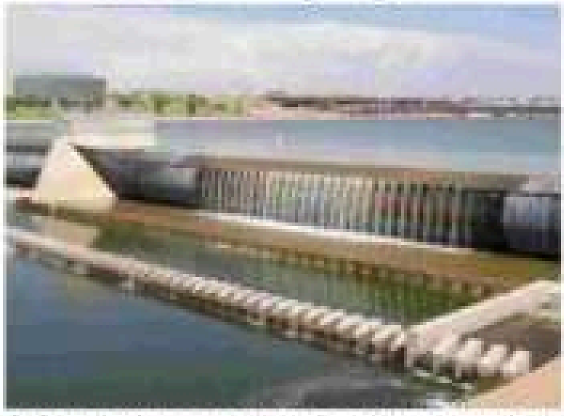
Examples of Steel type: Redridge Steel Dam (USA) and Ashfork-Bainbridge Stee Dam (USA).

7. Timber Dams: Main load-carrying structural elements of timber dam are made wood, primarily coniferous varieties such as pine and fir. Timber dam are made for small heads (2-4 m or, rarely, -8 m) and usually have sluices; according to the type of the apron they are divided into pile, crib, -crib, and buttressed dar

Timber dam

The openings of timber dams are restricted by abutments; where the sluice is very long it is divided into several openings by intermediate supports: piers, buttresses, and posts. The openings are covered by wooden shields, usually several in a row one above the other. Simple hoists—permanent or mobile winches—are used to raise and lower the shields.

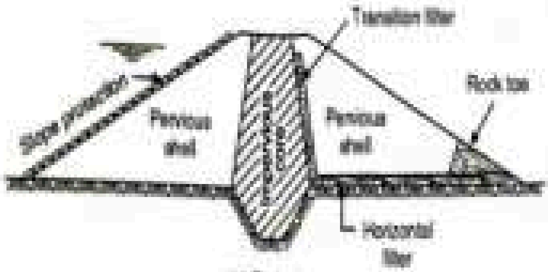
8. Rubber Dams: A symbol of sophistication and simple and efficient design, this most recent type of dam uses huge cylindrical shells made of special synthetic rubber and inflated by either compressed air or pressurized water. Rubber dams offer ease of construction, operation and decommissioning in tight schedules.

   a. 

   b. These can be deflated when pressure is released and, even the crest level can be controlled to some extent. Surplus water also simply overflow the inflated shell. They need extreme care in design and erection and are limited to small projects.

<u>EARTHEN DAM</u>

Earthen dams are constructed purely by earth work in trapezoidal section. These are most economical and suitable for weak foundation. Earthen dams are classified as follows:-



(a) Earth dam

Based on Method of Construction

Rolled fill Dam:

In this method, the dam is constructed in successive layers of earth by mechanical compaction. The selected soil is transported from borrow pits and laid on the dam section, to layers of about 45 cm. These layers are thoroughly compacted by rollers of recommended weight and type. When the compaction of one layer is fully achieved, the next layer is laid and compacted in the usual way. The designed dam section hence is completed layer by layer.

Hydraulic Fill Dam:

In this method, the dam section is constructed with the help of water. Sufficient water is poured in the borrow pit and by pugging thoroughly, slurry is formed. This slurry is transported to the dam by a pipe line and discharged near the upstream and downstream faces of the dam. The coarse material gets deposited near the face and the finer material move towards the centre and gets deposit there. Thus the dam section is formed with faces of coarse materials and central core is of impervious materials like clay and silt. In this case compaction is not necessary.

Semi-Hydraulic Fill Dam:

In this method the selected earth is transported from the borrowpit and dumped within the section of the dam, as done in the case of rolled fill dam. While dumping no water is used. But, after dumping the water jet is forced on the dumped earth. Due to the action of water the finer materials move towards the centre of the dam and an impervious core is formed with fine materials like clay. The outside body is formed by coarse material, In this case also compaction is not necessary.

Homogeneous Type Dam:

This type of dam is constructed purely with earth trapezoidal section having the side slopes according to the angle of repose of the soil. The top width and height deponents on the depth of water to be retained and gradient of the seepage line. The phreatic line (top level of seepage line) should pass well within the body of the dam. This type of dam is completely pervious. The upstream face of the dam is protected by stone pitching. Now-a- days, the earth dam is modified by providing horizontal drainage blanket or rock toe.

Zoned Type Dam:

This type of dam consists of several materials. The impervious core is made of puddle clay and the outer previous shell is constructed of the mixture of earth, sand, gravel, etc. the core is trapezoidal in section and its thickness depends on the seepage characteristics of the soil mixture on the upstream side. The core is extended below the both sides of the impervious core to control the seepage. The transition filter is made of gravel and coarse sand. The upstream face of the dam is protected by stone pitching.

Diaphragm Type Dam

In this type of dam, a thin impervious core of diaphragm is provided which may consist of puddle clay or cement concrete or bituminous concrete. The upstream and downstream body of the dam is constructed with pervious shell which consists of the mixture of soil, sand, gravel, etc. the thickness of the core is generally less than 3m. A blanket of stones is provided on the toe of the dam for the drainage of the seepage water without damaging the base of the dam. The upstream face is protected by stone pitching. The side slope of the dam should be decided according to the angle of repose of the soil mixture.

CAUSES OF FAILURE OF EARTHEN DAM

The failure of the earthen dam may be caused due to the reasons.

1. Hydraulic Failure : this type of failure may be caused by:

a. Overtopping: If the actual flood discharge is much much more than the estimated flood discharge or the fee board is insufficient or there is settlement of the dam or the capacity of spill way is insufficient, then it results in the overtopping of the dam. During the overtopping the crest of the dam may be washed out and the dam may collapse.

b. Erosion: If the stone of the upstream side is insufficient, then the upstream face may be damaged by erosion due to wave action. The downstream side also may be damaged by tail water, rainwater, etc. The toe of the dam may also get damaged by the water flowing through the spill ways.

c. Seepage Failure : this type of failure may be caused by

d. Piping of undermining: Due to the continuous seepage flow through the body of the dam and through the sub-soil below the dam, the downstream side gets eroded or wshed out and a hollow pipe like groove is formed which extends gradually towards the upstream through the base of the dam. This phenomenon is known as piping or undermining. This effect weakens the dam and ultimately causes failure of the dam.

e. Sloughing: the crumbling of the tow of the dam is known as sloughing. When the reservoir runs full, for a longer time, the downstream base of the dam remains saturated. Due to the force of the seepage water, the toe of the dam goes on crumbling gradually. Ultimately the base of the dam collapses.

f. Structural Failure : This type failure may be caused by

g. Sliding of the side slopes : Sometimes, it is found that the side slope of the dam slides down to form some steeper slope. The dam goes depressing gradually and then overtopping occurs which leads to the failure of the dam.

h. Damage by burrowing animals: due to earthquake cracks may develop on the body of the dam and the dam may eventually Collapse

SOLID GRAVITY DAM

The solid gravity dam may be constructed with rubble masonry or concrete. The rubble masonry is done according to the shape of dam with rich cement mortar. The upstream and downstream face are finished with rich cement mortar. Non-a-days, concrete gravity dams are preferred, because they can be easily constructed by laying concrete, layer by layer with construction joints. But good rocky foundation must be available to bear the enormous weight of the dam. The distance between the heel and toe is considered as the base width. It depends on the height of the dam. Again, the height depends on the nature of foundation. If good rocky foundation is available, the height may be above 200 m. If hard foundation is available, the height of the dam should be limited to about 20m. The upstream and downstream base of dam is made sloping. The horizontal trace (or line) passing through the upstream top edge is known as axis of the dam or the base line. The layout of the dam is done corresponding to this base line. Drainage gallery is provided at the base of the dam. Spill ways are provided at the full reservoir level to allow the surplus water to flow to the downstream. The solid gravity dam resists all the forces acting on it by its self-weight.

FORCES ACTING ON GRAVITY DAM.

1. Weight of the dam.
2. Water pressure

3. Uplift pressure.
4. Pressure due to earthquake
5. Ice pressure
6. Wave pressure
7. Silt Pressure
8. Wind Pressure

<u>Following are the modes of failure of a Gravity Dam</u>

1.Overturning
2.Sliding
3. Compression or Crushing
4. Tension

SPILLWAYS :

Spillways are structures constructed to provide safe releas flood waters from a dam to a downstream are, normally the river which the dam has been constructed.

Every reservoir has a certain capacity to store water the reservoir is full and flood waters enter the same, the reservoir level go up and may eventually result in over-topping of the dam. To avoid this situation the flood has to be passed to the downstream and this is done by providing a spillway which draws water from the top of the reservoir. A spillway can be a part of the dam or separate from it.

Spillways can be controlled or uncontrolled. A controlled spillway is provided with gates which can be raised or lowered. Controlled spillways have certain advantages as will be clear from the discussion follows. When a reservoir is full, its water level will be the same as the crest level of the spillway.

This is the normal reservoir level. If a flood enters the reservoir at this time, the water level will start going up and simultaneously water will start flowing out through the spillway. The rise in water level in the reservoir will continue for some time and so will the discharge over the spillway. A reaching a maximum, the reservoir level will come down and eventually come back to the normal reservoir level.

The top of the dam will have to be higher than the maximum reservoir level corresponding to the design flood for the spillway while the effective storage available is only up to the normal reservoir level. The storage available between the maximum reservoir level and the normal reservoir level is called the surcharge storage and is only a temporary storage in uncontrolled spillways. Thus for a given height of the dam, part of the storage - the surcharge storage is not being utilized. In a controlled spillway, water can be stored even above the spillway crest level by keeping the gates closed. The gates can be opened when a flood has to be passed.

Parameters considered in Designing Spillways

Thus controlled spillways allow more storage for the same height of the dam. Many parameters need consideration in designing a spillway. These include:

1. The inflow design flood hydro-graph
2. The type of spillway to be provided and its capacity
3. The hydraulic and structural design of various components and
4. The energy dissipation downstream of the spillway.

The topography, hydrology, hydraulics, geology and economic considerations all have a bearing on these decisions. For a given inflow flood hydro graph, the maximum rise in the reservoir level depends on the discharge characteristics of the spillway crest and its size and can be obtained by flood routing. Trials with different sizes can then help in getting the optimum combination.

Types of Spillways - Classification of Spillways

There are different types of spillways that can be provided depending on the suitability of site and other parameters. Generally spillway consists of a control structure, a conveyance channel and a terminal structure, but the former two may be combined in the same for certain types. The more common types are briefly described below.

Ogee Spillway

The Ogee spillway is generally provided in rigid dams and forms a part of the main dam itself if sufficient length is available. The crest of the spillway is shaped to conform to the lower nappe of a water sheet flowing over an aerated sharp crested weir.

Chute (Trough) Spillway

In this type of spillway, the water, after flowing over a short crest or other kind of control structure, is carried by an open channel (called the "chute" or "trough") to the downstream side of the river. The control structure is generally normal to the conveyance channel. The channel is constructed in excavation with stable side slopes and invariably lined. The flow through the channel is super-critical. The spillway can be provided close to the dam or at a suitable saddle away from the dam where site conditions permit.

Side Channel Spillway

Side channel spillways are located just upstream of and to the side of the dam. The water after flowing over a crest enters a side channel which is nearly parallel to the crest. This is then carried by a chute to the downstream side. Sometimes a tunnel may be used instead of a chute.

Shaft (Morning Glory or Glory hole) Spillway

This type of spillway utilizes a crest circular in plan, the flow over which is carried by a vertical or sloping tunnel on to a horizontal tunnel nearly at the stream bed level and eventually to the downstream side. The diversion tunnel constructed during the dam construction can be used as the horizontal conduit in many cases.

Siphon Spillway

As the name indicates, this spillway works on the principle of a siphon. A hood provided over a conventional spillway forms a conduit. With the rise in reservoir level water starts flowing over the crest as in an "ogee" spillway. The flowing water however, entrains air and once all the air in the crest area is removed, siphon action starts. Under this condition, the discharge takes place at a much larger head. The spillway thus has a larger discharging capacity. The inlet end of the hood is generally kept below the reservoir level to prevent floating debris from entering the conduit. This may cause the reservoir to be drawn down below normal level before the siphon action breaks and therefore arrangement for de-priming the siphon at the normal reservoir level is provided.

CHAPTER-9

GROUND WATER AND ITS DEVELOPMENT

9.1 Occurrence of Ground Water:

The rainfall that percolates below the ground surface, passes through the voids of the rocks, and joins the watertable. These voids are generally inter-connected, permitting the movement of the ground water. But in some rocks they may be isolated, and thus, preventing the movement of water between the interstices. Hence it is evident that the mode of occurrence of ground water depends largely upon type of formation, and hence upon the geology of the area.

In fact, all the materials of variable porosity (interstices) near the upper portion of the earth's crust can be considered as a potential storage place for ground water, and hence might be called as the ground water reservoir. Volume of water contained in the ground water reservoir in any localized area, i.e. the water storage capacity of the ground water is dependent upon (i) the porosity of the rocks; (ii) the rate at which water is added to it by infiltration, transpiration, seepage to surface sources, and withdrawn by man.

Ground Water Yield(Quantity of Ground water):

The interstices present in the given formation get filled up with water during the process of ground-water replenishment. If all these voids are completely filled with water, then it is known as saturated formation. The water contained in these voids is drained by digging wells under the action of 'gravity-drainage' explained later. When these saturated formations are drained under the action of 'gravity drainage', it is found that the volume of water so drained is less than the volume of the void space as indicated by its porosity. This is because of the fact, that the entire water contained in these voids cannot be drained out by mere force of gravity. Some of the water is being retained by these interstices due to their molecular attraction. The water so retained is known as pellicular water.

Specific Yield:

The volume of ground-water extracted by gravity drainage from a saturated water bearing material is known as the yield, and when it is expressed as ratio of the volume of the total material drained, then it is known as specific yield.

$$\therefore \text{Sp yield} = \frac{\text{volume of water obtained by gravity drainage}}{\text{Total volume of the material drained or dewatered}} \times 100$$

1.4 Specific retention or Field Capacity:

On the other hand, the quantity of water retained by the material against the pull of gravity is termed as specific retention or field capacity, and this is also expressed as percentage of the total volume of the material drained.

$$\text{Specific retention or field capacity} = \frac{\text{volume of water held against gravity drainage}}{\text{Total volume of the material drained}} \times 100$$

It is evident that the sum of the specific yield and specific retention is equal to its porosity.

1.5 Specific Retention of different Kinds of Formations:

As has been said earlier, the specific retention is the amount of the water held between the grains due to molecular attraction. This film water is thus held by molecular adhesion on the walls of the interstices. Therefore, the amount of this water will depend upon the total interstitial surface in the rock. If the total interstitial surface is more, the specific retention will be more and vice versa.

Now, if the effective size of the grains decreases, the surface area between the interstices will increase, leading to, more specific retention and less specific yield.

It, therefore, follows that, in fine soils like clay, the specific retention would be more, and hence, such soils would result in very small specific yields.

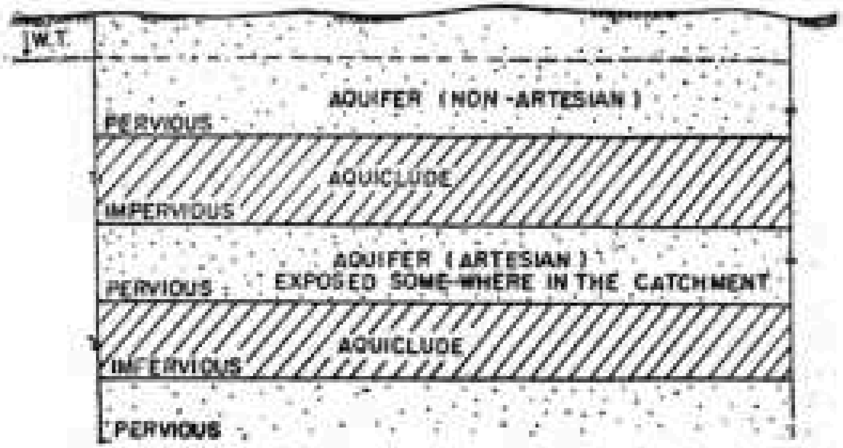
The reverse is also true when the grain size increases; the interstitial surface area reduces, and , therefore, sp. Retention reduces and hence sp. Yield increases. It, therefore, follow that in large particle soils like coarse gravels, the specific retention would be small and it would result in large specific yields.

This conclusion is very important from practical standpoint, because it follows from this that a water bearing formation of coarse gravel would supply large quantities of water to wells, whereas, clay formations although saturated and of high porosity, would be of little ably upon the type of neighboring formations.

Aquifers and Their Types:

A permeable stratum or a geological formation of permeable material, which capable of yielding appreciable quantities of ground water under gravity known as aquife The term 'appreciable quantity' is relative, depending upon the availability of the grou water. In the regions, where grou-water is available with great difficult, even f-grained materials containing very less quantities of water may be classified as Principal aquif

When an aquifer is obtained by a confined bed of pervious material, then th confined bed of overburden is called as aquiclude shown in Fig. 9



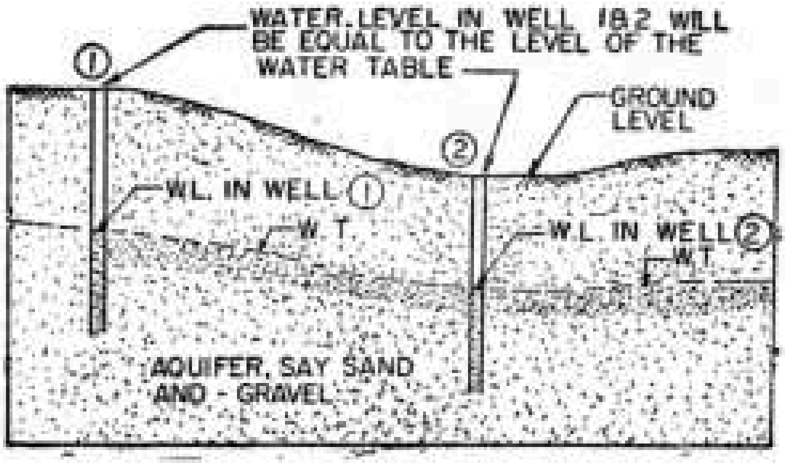
The ... depends many factors, some of which, such as well diameter, inherent in the well itself. But other things being equal, the permeability and thickness of the aquifer are the most important.

Aquifer very in depth, lateral extend, and thickness but in general, all aquifer fall in one of the two categories, i.e.,

1. Unconfined or Non-artesian aquifers; a
2. Confined or Artesian aquif

Unconfined Aquifer or Non-artesian aquifers:

The top most water bearing stratum having no confined impermeable over burd (i.e., aquiclude) lying over it, is known as an unconfined aquifer or non-artesian aquife (Refer Fig. 9.2)

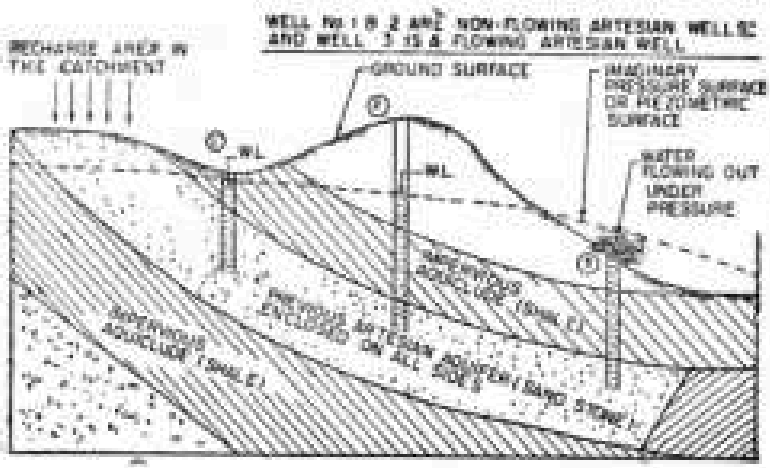


The ordinary gravity wells of 2 to 5 m diameter, iwhare costructed to gap watı from the top most water bearing strata, i.e., frthe unconfined aquifers, are known unconfined or nonartesian wells. The water levels in these wells bel equal to the level ı the watertable. Such well are, therefore, alsown as wells or gravity wells.

Confined Aquifers or Artesian Aquifers:

When an aquifer is confined on its upper and undsurface, by impervious roc formations (i.e., aquicludes), and is also broadinclined so as to expose the aqu somewhere to thecatchments area at a higher level for the creation sufficient dydraulic head, it is called a confined aquifer or an artesian aquifer. A well excavated through such aquifer, yields water than often flows out automatically, under the hydrostatic prure, and may thus, even rise or gush out of surface for asonable height. However, where ground profile is high , the water may remain well below the ground level. The former ty of artesian wells, where after is gushing out automatically, are caed flowing wells

The level to which water will rise in an artesian well is determined by the highe point on the aquifer from where it is fed fr



The rains falling in the catchments (i.e., by dargle). However, the water will not rise to this full height, because the friction of water moving through the aquifer uses up some of the energy.

The question whether it will be a flowing artesian well or a non-flowing artesian well depends upon the topography of the area, and is not the inherent property of the artesian aquifer. In fact, if the pressure surface lies above the ground surface, the well will be a flowing artesian well, whereas, if the pressure surface is below the ground surface, the well will be artesian but non-flowing, and will require a pump to bring water to the surface, as shown in Fig.9.3. Such non-flowing artesian wells is sometimes called as sub-artesian wells.

Perched Aquifers:

Perched aquifer is a special case which is sometimes found to occur within an unconfined aquifer.

If within the zone of saturation, an impervious deep below a pervious deposit is found to support a body of saturated materials, this body of saturated materials which is a kind of aquifer is known as perched aquifer. The top surface of the water held in the perched aquifer is known as perched water table. It is shown in Fig. 9.4.

Wells

A water well is a hole usually vertical, excavated in the eared for brining ground water to the surface. The wells may be classify into two types:

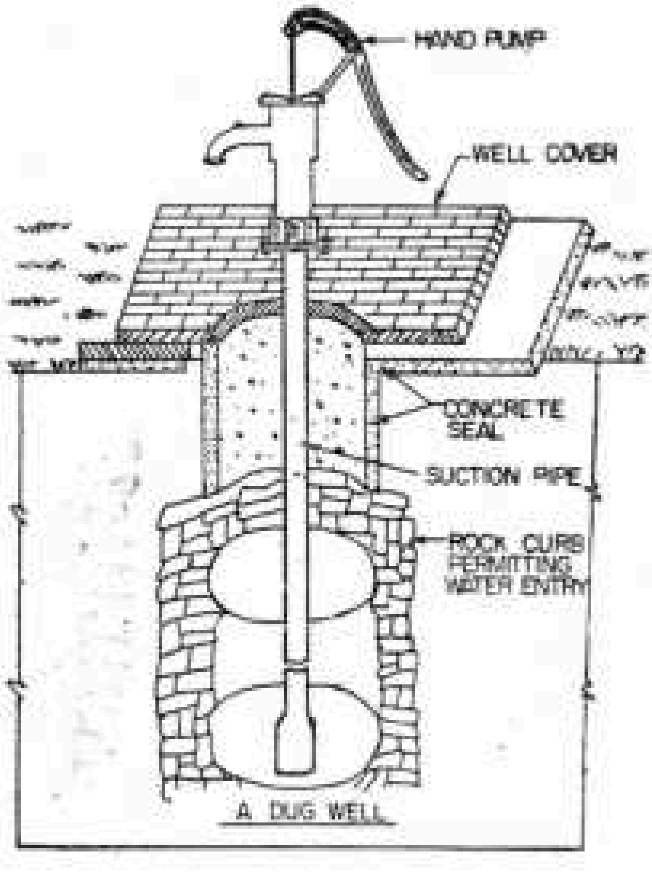
1. Open wells; and
2. Tube wells.

Open Wells or Dug Wells.

Smaller amount of ground water has been utilized from ancient times by open wells. Open wells are generally open masonry wells having comparatively bigger diameters, and are suitable for low discharges of the order of 18 cubic meters per hour (i.e. about 0.005 cumecs). The diameter of than 20 m in depth. The walls of open well may be built of precast concrete rings or in brick or stone masonry, the thickness generally varies from 0.05 to 0.75 m, according to the depth of the well.

The yield of an open well is limited because such wells can be excavated only to limited depth where the ground water storage is limited.

Moreover, in such a well the water, the water can withdrawn only at the critic velocity for the soil. Higher velocities cannot be permitted as that may lead to disturbanc soil grains and consequent subsidence of well grin the hollow so formed. The lim placed



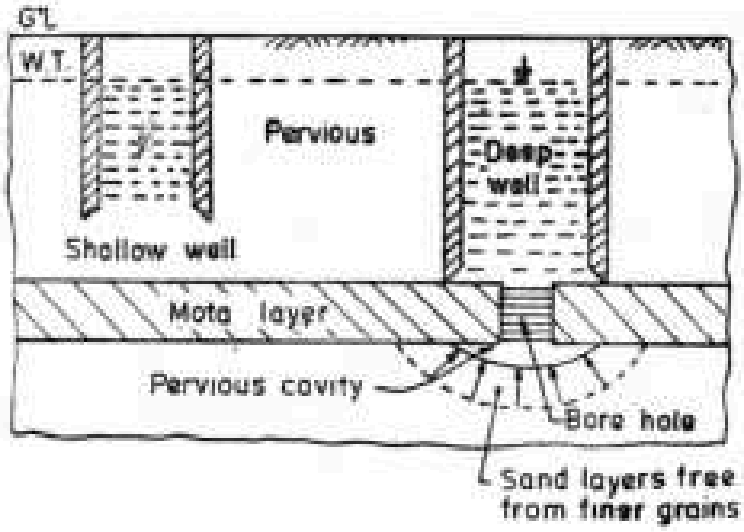
On velocity , therefore, also limits the maximum possible saf dis-charge of an open we

One of the recent methods used to improve the yield of an open well is to put in a 8 10 cm diameter bore hole in the centre of the well so as to tap the additional water from aquifer or from fissures in the rock. If lay or kankar layer is available at a smaller depth as to support the open masonry well, a bore hole can be made in its centre so as to reach sand strata. Such an arrangement will not only give structural support to the open well will also considerably increase its yield. Depending upon the availability of such a provisior the open wells may be classified into the following two types

(a) Shallow wells; and

(b) Deep wells.

Shallow wells are those which rest in a pervious stratum and draw their supers from the surrounding materials. On the other hand, a deep well is one which rests on an impervic 'mota' layer and draws its supply from the pervious formation lying below the mota laye through a bore hole made into the 'mota' layer, as shown in Fi; 4016. The term "Mota laye also sometimes known as "Matbarwa" or "Magasan" refers to a layer of clay, cement sand,



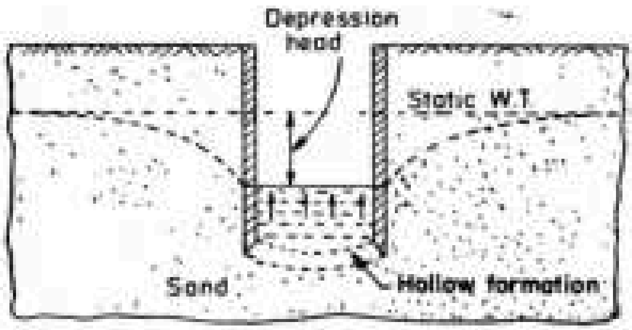
kankar or other hard materials, which are often found lying a few metres below th watertable in the subsoil. The names a not applied to layers of hard material lying ab the watertable. The main advantage of such a mota layer lies in giving structural support the open well resting on its surface. It is useful for unlined and partly lined wells, and indispensable for a heavy masonry well which would not remain stable under steady us without such a support. The mota layer is generally found throughout the in-Gangtic plain These mote layer may either be continuous or may be localized, and are generally found different thicknesses and depths at different pl

The nomenclature of shallow and deep wells is purely technical and has nothing to with the actual depth of the well. A "shallow well" might be having more depth than a "de well".

Since a shallow well draws water from the topmost water bearing stratum, its water is liable to be contaminated by the rain water percolating in the vicinity and may take with minerals or organic matters such as decomposing animals and plants, etc. The water in a d well, on the other hand, is not liable to get such impurities and infections. Secondly, t pervious formations below the mota layer generally obtain greater discharge and gre supplies can be obtained from a deep well as compared to those from a shallow well.

Water is generally drawn from dug or open wells by means of a bucket and a ro However, due to the possible surface contamination of water in an uncovered well and a the individual buckets adding contamination to water, such open wells have been covered in many parts of India and fitted with hand pumps (Fig. 15).

2.1 Cavity Formation in Wells. Consider a well from which no water is be withdrawn. The water level in such a well will obviously be the same as is the st watertable outside the well. Now, if a discharge is withdrawn from the well at a constar rate, the level in the well will go down and stabilise at a lower level than that of the outs watertable. The head difference between these two levels is called depression head (I 4.17). Under the influence of this head difference, water enters the well from outside so a fill



the gap created by withdrawn water. As the water from the surrounding soil travels towa the well, there is a gradual loss of head, and water surface drops towards the well. Since same discharge is passing through reducing soil areas as it approaches the well, there gradual increase in flow velocity towards the well. Now according to Darcy's law, th velocity can gradually increase only if the hydraulic gradient gets gradually increased. He the water surface will fall gently in the beginning and will fall more and more rapidly as approaches the well. The surface of water-table surrounding the well, therefore, takes up a curved shape and is called Cone of Depression. At certain distance from the well, there is no appreciable depression of watertable. This distance from, the central line of the well is called radius of influence of the well.

The velocity of percolating water into the well depends upon the depression head. If more amount of water is withdrawn from the well and thereby increasing the depression head, higher flow velocities will prevail in the vicinity of the well. Thus, at a certain rate of withdrawal, it is very much possible that the flow velocity may exceed the critical velocity for the soil, thereby causing the soil particles to lift up. As more and more sand particles are lifted, a hollow is created in the bottom of the well resulting in increased effective area, so that ultimately, the velocity falls below the critical value and then no further sand goes out of the well.

As pointed out earlier, the formation of such hollow beneath the wells is dangerous in shallow wells, because there is always a danger of subsidence of the well lining. The maximum rate of withdrawal from such wells is, therefore, limited.

In case of deep well resting on mota layer, the cavity or hollow formation below the bore hole (Fig. 4'16) is not dangerous, because the well lining remains supported on the mota layer. Hence, a hollow, much larger in area than the cross-sectional area of the well, may safely form in deep wells, and thereby giving higher yields. In a shallow well of an equivalent yield, the well area will have to be increased equal to the area of the cavity under the deep well, which would make it costlier.

2.2 Construction of Open Wells. From the construction point of view, the open wells may be classified into the following three types :

Type I. Wells with an impervious lining, such as masonry lining, and generally resting on a mota layer.

Type II. Wells with a pervious lining, such as dry brick or stone lining, and fed through the pores in the lining.

Type III. No lining at all, i.e., a Kachha well.

Type I. Wells with impervious lining. They provide the most durable and useful type of wells for obtaining water supplies. For constructing such a well, a pit is first of all excavated, generally by hand tools, up to the soft moist soil. Masonry lining is then built up on a kerb upto a few metres above the ground level. A "kerb" is a circular ring of R.C.C., timber or steel having a cutting edge at the bottom and a top, wide enough to support the thickness of well lining called "steining". The kerb is then descended into the pit by loading the masonry by sand bags, etc. As excavation proceeds below the kerb, the masonry sinks down. As the masonry sinks down, it is further built up on top. To ensure vertical sinking, plum bobs are suspended around the well steinning, and if the well starts tilting, it may be corrected by adjusting the loads or by removing the soil from below the kerb which may be causing the tilt. The well lining (steinning) is generally reinforced with vertical steel bars.

After the well has gone up to the watertable, further excavation and sinking may be done either by continuously removing the water through pumps, etc., or the excavation may be carried out from top by Jhams. A Jham is a dredging bucket which is tied to a rope and worked up and down over a pulley. When the Jham is thrown into the well, its jaws strike the bottom of the well, dislodging some of the soil materials. As the Jham is pulled up, the soil cuttings get retained but the water oozes out. This sinking is continued till the mota layer is reached. A smaller diameter bore hole is then made through the mota layer in the centre of the well, which is generally protected by a timber lining.

Sometimes, when mota layer is not available, shallow wells may be sunk as described above upto a required depth, and partly filled with gravel or broken ballast. This will function as a filter through which water will percolate into the well but the sand particles will be prevented from rising up.

In a pucca well, lined with an impervious lining on its sides, the flow is not radial. The water enters only from the bottom and the flow becomes spherical when once the cavity has been formed at the bottom.

Type II. Wells with pervious lining. In this type of wells, dry brick or stone lining is used on the sides of the well. No mortar or binding material is used. The water, thus enters from the sides, through the pores in the lining. The flow is, therefore, radial. Such wells are generally plugged at the bottom by means of concrete. If the bottom is not plugged, the flow pattern will be a combination of radial flow and a spherical flow. Such wells are generally suitable in strata as of gravel or coarse sand. The pervious lining may have to be sorrounded by gravel, etc., when such a well is constructed in finer soils, so as to prevent the entry of sand into the well along with the seeping water.

Type III. Kachha wells. These are temporary wells of very shallow depths, and are generally constructed by cultivators for irrigation supplies to their fields. Such wells can be constructed in hard soils, where the well walls can stand vertically without any support. They can, therefore, be constructed only where the water table is very near to the ground. Though they are very cheap and useful, yet they collapse after some time, and may sometimes prove to be dangerous.

2.3 Yield of an Open Well.The yield of an open well can be determined with the help of theoretical methods, with practical methods by carrying out a practical test and then calculating it from the observations. This third method is useful for calculating the yields of open wells as well as that of tube-wells penetrating through confined aquifers.

(1) Theoretical method If a well is penetrated through the aquifer, water will rush into it with a velocity V. If A is the area of the aquifer opening into the well, then

Q=AV

where V=vK, where v is the actual flow velocity and V is the velocity with which water rushes into the well and is constant.

Q=K.A.v

where K is a constant depending upon the soil and known as permeability constant.

In the above equation, the velocity of ground water flow (v) can be found by using Slichter's or Hazen's formula or by actual measurements by chemical or electrical methods.

A, the area of the aquifer, and can be found by knowing the •diameter of the well and the depth of porous strata.

K, the constant can be found by studying the sample of the soil in the laboratory.

Knowing v, A and K, the discharge can be easily calculated.

2:3 Tube-wells.

The discharge from an open well is generally limited to 3 to 6 litres/sec. Mechanical pumping of small discharges available in open wells is not econofnical. To obtain large discharge mechanically, tubewell, which is a long pipe or a tube, is bored or drilled deep into the ground, intercepting one or more water bearing stratum. The discharge of an open well is smaller, because : (0 open wells can tap only the topmost or at the most the next lower water bearing stratum. O'O water from open wells can be drawn only at velocity equal to or smaller than the critical velocity for the soil, so as to avoid the danger of well subsidence. But in the tube wells, larger discharges can be obtained by getting a larger velocity as well as a larger cross-sectional area of the water bearing stratum. Since, we have an enormous storage of ground water in India, the tubewells provide excellent means for providing water supplies, although they are generally used for irrigation.

2.4 Tube-Wells in Alluvial Soil. Most of our land, especially the entire area from Himalayas to Vindhya mountains (such as the Indo-Gangetic plain), coastal areas, Narmada valley, etc., consist of deep alluvial soils. The subsoil water slowly penetrates and is stored in the porous sand and gravel beds which are extensively found in India, except that in the desert areas. Tube-wells can be easily installed in such soils and are very useful for irrigation. It is in this context that the tube-wells are assuming greater and greater importance for tapping out ground water resources, especially in alluviums.

Deep tube-wells are generally constructed by Government and are called State tube-wells The depth of such wells, generally vary from 50 to 500 m, and may yield as high as 200 litres/sec. The general average yield from the deep State tube-wells is of the order of 40 to 45 litr. s/sec. A 300 m deep tube-well has been constructed at Allahabad (UP.) at the edge of river Gauges, and is yielding at about 140 litres/s. Trie diameter of the hole is 0'6 m upto 60 m depth, and then 0'56 m below 60 m. The diameter of the strainer is 0'25 m, and drawdown is 10 m. Tnere are about 20,000 deep tube wells in our country, and every year about 1,500 such wells are being added.

Besides deep tube-wells, shallow tube-wells are also constructed by cultivators. Their depths generally vary from 20 to 40 m or so, and yield as high as 15 litres/sec, if located at proper places. Each well iirigates about 8 hectares. There are about 10 lakh shallow tube-wells in India, and every year about 1'5 lakh such wells are being added

2.5 Tube-wells in Hard Rocky Soils. It is very difficult to construct a tube-well irrigation system in rocky areas. Therefore, in rocky areas, tube-wells or open wells are resorted to, only when, there are no other alternate sources of water. Hence, in rocky areas, only isolated holes of 10 to 15 cm diameter may sometimes be drilled using down the hole rigs. Only wells in alluviums have been treated here.

2.6 Various Types of Tube-wells. Tube-wells are generally of the following types :

(1) Strainer wells ;

(2) Cavity wells ;

(3) Slotted wells ; and

(4) Perforated pipe wells.

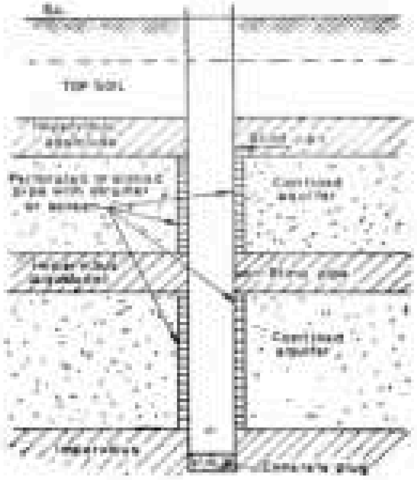
Out of these four types, the first type, i.e., the strainer wells are the most important and widely used in India, while the last type, i.e., perforated pipe wells are not of much importance, as they have not been used in India to any appreciable extent. The first three important types of tube-wells are described below :

(1) Strainer Type Tube-wells. As pointed out earlier, a strainer well is the most important of all the types of tube-wells, and has been extensively and widely used in our country. So much so that whenever we refer to a 'tube-well', we generally mean a strainer

type of a'iubewell'. All the state tub-wells constructed in U.P., from where the techniqu tube-well construction got started in 1931, are exclusively of this type

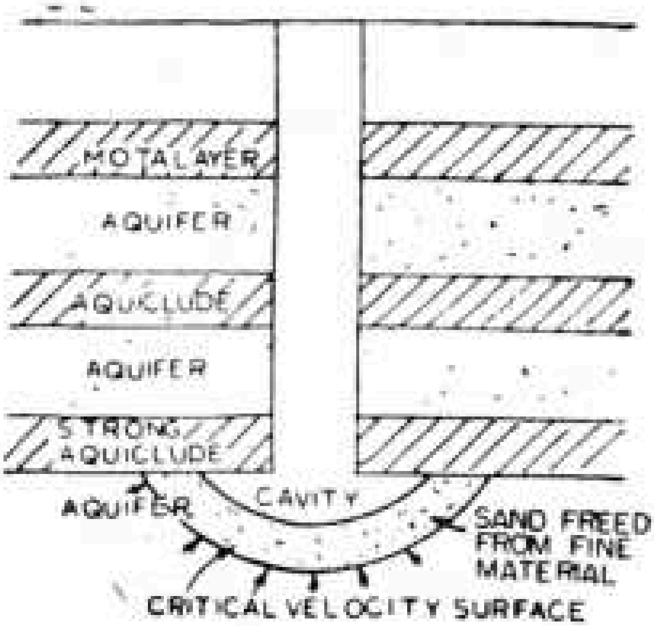
In this type of a well, a strainer or a screen is placed against the water bearing strat The strainer is generally constructed of a wire screen wrapped round a slotted or perfed pipe with a small annular space between the two. The screen prevents sand partic from entering the tubewell. The water, therefore, enters the well pipe through the fine mes (i.e., the screen) and the sand particles of size larger than the ze of the mesh, are kept aw from entering the pipe. This reduces the danger of sand removal and hence, larger fl velocities can be permitted. Moreover, the strainer penetrates into a number of water bea strata, and thus" does not depend only or or two strata for the well supplies. The slot pipe is made to have the cross sectional area of its openings equal to that in the screen so that no change of velocity occurs between the two. The annular space between the p and the screen is required, otherwise the wires of the screen part of the opening of th pipe.

The strainer type of tub-well is generally unsuitable for very fine sandy strata, because in that case, the size of the screen opening will have to be considerably reduc which may result in chocking of the strainer, and the screen openings are kept bigger, well will start discharging sand.



The boring for such a well is generally carried out by a "casing pipe of about 5 to cm larger than the diameter of the well pipe. Thus, for a 15 cm diameter well, a bore hole o 20 to 25 cm diameter shall be drilled. After boring the hole, the well pipe assembly which partly in ordinary plain pipe (called blind pipe) and partly of strainer pipe, is lowered into bore hole. The lengths of blind pipe and strainer pipe are so adjusted that the blind pipe r against the aquicludes, while the strainer rests against the aquifers, as shown in Fig 4'19. bottom, a short blind pipe is provided, so as to permit settlement of any sand particles, passed through the strainer. The well is generally plugged at bottom by cement concr Abyssinia tubewell is a special type of strainer well, in which the diameter of the well pipe kept equal to 3'8 cm (I^T) and the strainer is provided only for a length of about 1 to 1.5m (i.e., 4 to 5 feet).

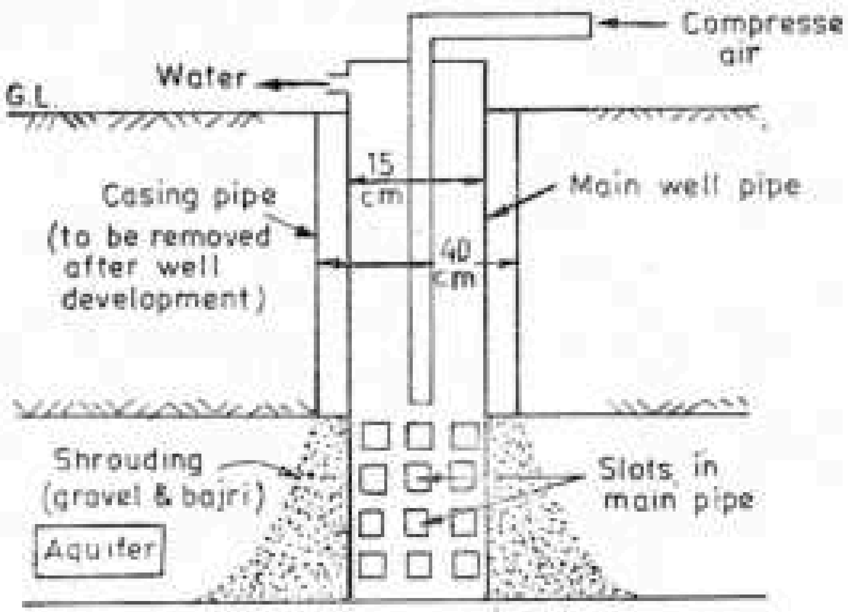
(2) Cavity Type Tube-wells. They are those which do not utilise strainers and draw their supplies from the bottom, and not from the sides. Since the water is drawn from bottom, only one particular aquifer can be tapped. The principle behind a cav-type tubewell is essentially similar to that of a deep open well, with the only difference th whereas an open deep well taps the first aquifer below the mota lay a cavity tub-well need not do SO;, and may even tap the lower strata as shown in Fig. 4'2



A cavity type tubewell essentially consists of a pipe bored through the soil and resting on the bottom of a strong clay layer. A cavity is formed at the bottom and the wa from the aquifer enters the well pipe through this cavity, as shown in Fig. 4'20. In the init stages of pumping, fine sand comes out with water and, consequently, a hollow or a cavity formed. As the spherical area of the cavity increases outwards, the radial critical velocity decreases for the same discharge, thus redut Ythe velocity and consequently stoppi the entry of sand. Hence, the flow in the beginning sandy, but becomes clear with passage of time.

The essential difference in the flow pattern of strainer well and a cavity well is th whereas in a strainer well, the flow is radial, flow in a cavity well is spherical. Also, in strainer well, the area of flow is increased by increasing the length of strainer pipe, while in cavity well, the area of flow is increased by enlarging the size of the cavity. The cav formed with a certain discharge enlarges in size if increased discharge is pumped

(3) Slotted Type Tubi-wells. If sufficient depth of water bearing strata is available even at deep depths of 75 to 100 m, so as to obtain the required discharge fror strainer well, and if a suitable strong clay roof is not available for a cavity well, a slotted w is adopted, provided at least one good stratum having sufficient amount of water is available A slotted well essentially consists of a slotted wrought iron pipe, penetrating a high pervious confined aquifer (Fig. 4.21). The size of slots may be 25 mmX3mm at 10 to 12 n spacing In order to prevent the entry of fine s



particles into the pipe, the pipe is surrounded by a mixture of gravel and bajri. This mixture called shrouding and is poured from the top into annular space between the strainer the casing pipe before withdrawing the casing pipe. The tube-well is developed by pumping water with an air compressor or with a bigger capacity pumping set. In the process of developing such a tube-well, water is drawn at a high rate, causing high flow velocities and consequent removal of appreciable quantities of fine sand. Shrouding is continuously fed through the annular space, so as to fill up the space of removed sand particles. The process is continued till the sand-free water is obtained.

The diameter of the bore hole or casing pipe is generally kept more than that of a strainer type of tube-well. For example, a casing pipe of about 40 cm diameter is required for a well pipe of 15 cm diameter.

The essential difference between a strainer well and a slotted well are :

(i) A strainer well uses a 'strainer' for preventing sand entry in the water, whereas a slotted well uses a gravel 'shrouding' for this purpose.

(ii) A strainer well can tap one or more strata, whereas a slotted well can tap only one stratum.

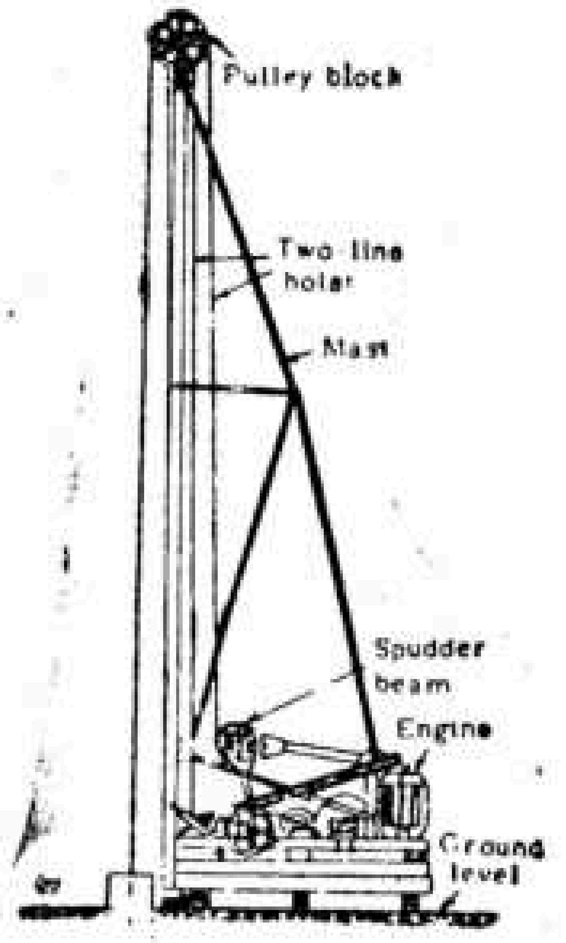
2.6 Methods for Drilling Tube-wells. Deep and high capacity wells are constructed by drilling. Various different techniques are employed in drilling the well hole. Different techniques have comparative advantages and disadvantages over each other, depending upon the type of formation to be drilled. Therefore, each well should be treated as an individual project, and one particular method adopted, depending upon its suitability. Some of the drilling methods, common'y used, are described below

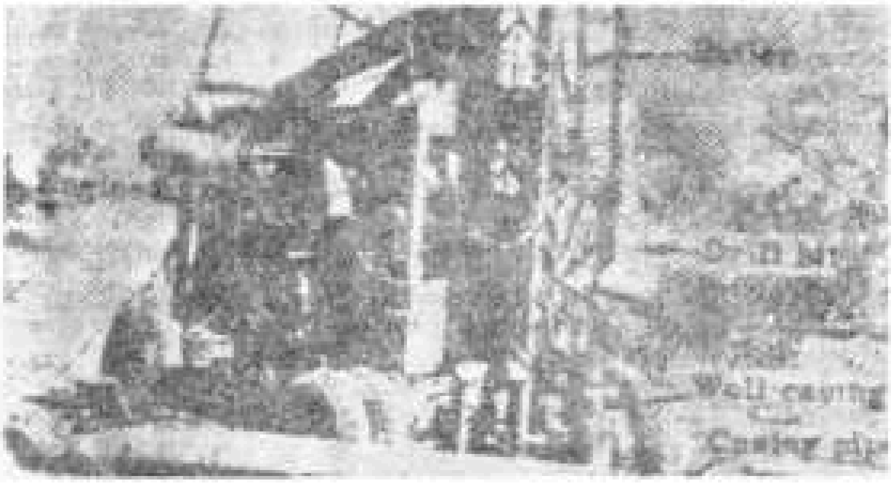
(I) Standard method or Cable tool method.This method of drilling the well hole is also known as percussion drilling ; because in this method the well hole is made by percussion, (i.e., by hammering and cutting). This method is very useful for cutting consolidated rocks from soft clay to hardest rocks, and is generally unsuitable in loose formations, such as unconsolidated sand and gravel or for quick sand. This method becomes ineffective in loose materials, because the loose material slumps and caves around the drilling bit. The drill bit has a chisel sharp edge, which breaks the rock by impact when alternately lifted and-dropped. This drilling bit is connected at the lower end of the entire' falling and rising arrangement' known as String of tools. [Refer Fig. 2.(a) and (b)]. From top to bottom, the string of tools consists of a rope socket, a set of jars, a drill stem, and the drilling bit.

Tools are made of steel and are joined with tapered box and pin screw joints. The entire assembly weighs several tonnes. The most important tool of the entire assembly is the drilling bit (or drill) as it does the actual rock cutting. The drill stem is the long steel bar which adds weight and length to the drill, so that it can cut rapidly and vertically.

The set of jars have no ect effect on the drilling. They only loosen the tools when they stick in the hole. A rope or a cable is fastened at the upper end to the rope socket and a dead man (or a heavy weight) at the lower

The entire assembly of tools is suspended from assembly of a mast and a walk beam, etc. This assembly, known as drilling rig unit, is generally mointed on a truck, so to make it easily portable. The mast should be sufficiently high, so as to allow the longest tools to be hoisted.





As the drilling proceeds, the tools make 40 to 60 strokes per minute, from a height of 0'4 t m. Water is sometimes added in the hole, so as to form a paste with the cuttings, th reducing friction on the falling bit. After the bit has cut 1 to 2 m through a formation, th string of tools is lifted out, and the hole is cleaned and cleared of the cuttings by means bailer. The process is known as bailing out the hole.

A bailer essentially consists of a pipe with a valve at the bottom and a ring at the top. When lowered into the well, the valve permits the cuttings to enter the bailer but preve them from escaping the bailer. After it is filled with cuttings, it is lifted up to the surface a emptied.

In unconsolidated formations, casing should be driven down and maintained near bottom of the hole to avoid caving. Casing is driven down by means of drive clamps faste to the drill stem. The up and down motion of the tools striking the top of the casing, protected by a drive head, sinks the casing. On bottom of the casing, a drive shoe fastened to protect the casing, as it is being driven.

(2) Hydraulic rotary or Direct rotary method. This is the fastest method of drillin and is especially useful in unconsolidated formations. The method involves a continuousl rotating hollow bit, through which a mixture of clay and water or mud is forced. The cuttings are carried up in the hole by the rising mud. No casing is required during drilling, because the mud itself makes a lining on the walls of the hole, which prevents cavi

The drill bit is connected to a hollow steel rod (drill stem), which, in turn, i connected at the top to a square rod, known as Kelley [Refer Fig. 4'23 (a). The drill is rotated by a rotating table which fits closely around the Kelley, and allows the drill rod to slide down, as the h progresses.

The drilling rig, such as shown Fig. 4'23 (b), consists of a mast, a rotating table pump for forcing the mud, a hoist and the engine. The mud, after emerges out of the hole, carried to & tank where the cuttings settle out; the mud can be repumped into the h

After the drilling is completed, the casing is lowered into he hole. The clay, deposit in the well-walls during mud pumping, is removed by washing with water. Wate containing some chemicals like sodium hexametaphosphate is forced through the drill rc and the washings come out through the perforations the casing When the washing at c level is completed, the bit is raised and the process repeate

(3) Reverse Rotary method or Jetting metho. A modification of the hydraulic rota method is known as Reverse Rotary method. This is gaining popularity day b day. It is quite useful for making large wells (diameter up to 1'2 m app.) in unconsolidated formation This method is also known as jetting method. The tool consists of a hollow drill, a drill pipe and water swivel. In this method, the cuttings removed by water through a suction p called drill pipe. The equipment consists of a mast or a derrick, a centrifugal pump, a necessary water and power cas

The hole is driven by pumping water under pressure through the drill bit, while it is churned up and down.

The walls of the hole are supported by hydrostatic pressure acting against a film fine grained material deposited on the walls by the drilling water. Cuttings are removed water ; and after the mixture (water + cuttings) comes out to the surface, it is passed throug a settling tank (Refer Fig. 4'24).

The sand settles out here, but the fine grained clays are recalculated, so as to h in stabilising the walls. Casing and cleaning of the walls, etc. is the same as in the hydrau rotary method.

Comparison of Cable Tool and Hydraulic Rotary methods.

Advantages of Cable Tool Method are given below :

1. A more accurate sample of the formation can be obtained.
2. Lesser amount of water is required during drilling operations.
3. Cable tool rig is lighter and easy to transport.
4. Very useful for consolidated rocks and less useful for loose formations.
5. For shallow-wells, in unconsolidated materials, it comes out to be cheaper.

Advantages of Hydraulic Rotary Method are given below :

1. Can be used for larger holes up to 1'5 m diameter
2. Can be best used for drilling test holes, because the hole can be abandoned with minimum cost.
3. Rotary drilled hole can be gravel packed, which increases its specific capacity, and keeps the fine particles away, thus causing less trouble.
4. Casing is to be driven only after the hole complete, and hence, can be set at any desired depth.
5. It is the fastest method of drilling and especially useful in unconsolidated formations.
6. It can handle alternate hard and soft formations with ease and the danger of accidents is lesser. In quick sands, clays, etc., cable tool method is likely to give troubles, as there is a danger of freezing.

2.7 Completion of a Tube-well During drilling the well hole by any of the above methods, care should be taken to see that the hole remains straight and vertical. A common specification allows a deviation of 15 cm from the vertical in a length of 30 m. After the bore hole has been constructed or drilled, the well must be completed, so as to provide free entrance of clear water into the well.

Casings and Screens. In consolidated formations, water enters the well hole directly, and no casing is provided, because the surrounding is quite stable. But in unconsolidated formations, a casing is necessary which supports outside material and helps in freely admitting water into the well.

For the entry of water, the casing should either contain perforations or its lower part be replaced by a screen or a strainer. Perforations can be made in the field or at home. Horizontal louvered openings are generally preferred and their size is kept between De0 to D70 of the surrounding soil. Generally, a separate screen forms the lowest part of the casing. They are made from various corrosion-resistant metals. Plastic screens are also forming their way in the market. Wellscreens are very useful in sandy formations, so that water containing only a limited quantity of sand (below given size) may nter the well, and th bigger particles are screened and kept away from ering the well. However, the mesh s of the screen is generally decided by the manufactor a given project, depending up the grain size distribution of the aquif

Gravel Packing. Many a times, a layer of gravel surrouging the screen casing, provided, so as to increase the effective well diam and to keep the fine materials ou the well. Such a well will have a greater specific capacity than the one of the samiameter not surrounded by gravel. The thickness of the gravay vary with the type of formatic and method of drilling. However, a minimum of 15 cm thickness is generally used. A sect of the gravel packed well is shown in Fig. 4'

2.8 Factors Affecting the Selection of a Particular Type of Pump

The various factors which must be thoroughly considered while selecting a particu type of a pump for a particular project a

(i) Capacity of pump

(ii) Importance of water supply sche

(iii) Initial cost of pumping arrangem

(iv) Maintenance cost

(v) Space requirements for locating the pupm

(vi) Number of units required

(vii) Total lift of water required

(viii) Quantity of water to be pumped

Truly speaking, reciprocating pumps are also outdated these days, and for all ordinary conditions of pumping, the centrifugal types of dynamic pumps are frequently used these days, as they provide satisfactory and economical service. However, pumps other than those of centrifugal types may be used under extreme conditions. The choice between various types of pumps is guided by the following considerations:

For very small discharge, the rotary pumps may prove to be equally satisfactory as the centrifugal pumps, and less costly if the water to be pumped is free of sediment.

A centrifugal pump may pose operational problems when constant discharge is needed at variable heads (because it requires variable speed driving in that case) whereas, the discharge through a reciprocating pump depends only on the speed of the pump. Hence, under such circumstances, where water is to be pumped against very high but variable heads with a higher suction lift, reciprocating pumps may be useful. However, they can be used only when the water to be pumped is free of sediments and ample finances are available for installing the costly reciprocating pumps.

Centrifugal pumps are especially useful for pumping waste water and water containing solids, although they are equally suitable for pumping treated waters. Even among the centrifugal pumps, a choice is sometimes made between the horizontal shaft centrifugal pump and the vertical spindle bore hole pump and submergible pump. The horizontal pump is the cheapest and is used under a wide range of pumping conditions, but vertical spindle and submergible pumps may be preferred for handling large quantities of water under low heads and for wells and bore holes.

Air lift pumps may prove to be cheaper and, thus may be selected when water is required to be pumped simultaneously from a number of wells; because in that case, a common compressor unit can be used to feed all the pumps. However, their efficiency is generally low.

The hydraulic ram and jet pumps may also be used under special circumstances, as pointed out earlier.