LEARNING RESOURCE MATERIAL

COURSE CODE- Th 2

HYDRAULICS ENGINEERING

DEPARTMENT

OF

CIVIL ENGINEERING

GOVERNMENT POLYTECHNIC KENDRAPARA

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Chapter-I

HYDROSTATICS

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

Fluid:-

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

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

PROPERTIES OF FLUID:-

Density:-

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

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

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

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

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

Specific Weight:-

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

$$\lambda = \rho * g$$

g = local acceleration of gravity and ρ = density

Note: It is customary to use:

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

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

Relative Density (Specific Gravity):-

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

Specific volume:-

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

Viscosity:-

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

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

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

Viscosity is due to two factors-

a) Cohesion between the liquid molecules.
b) Transfer of momentum between the molecules.

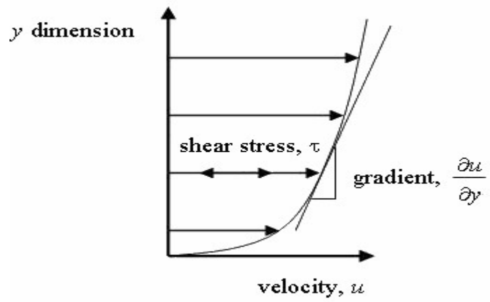


Fig. 1.1

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

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

UNIT OF VISCOSITY

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

Kinematic viscosity:-

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

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

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

In cgs system unit of kinematic viscosity is stoke.

SURFACE TENSION:-

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

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

Capillarity:-

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

Pressure and its measurement:-

INTENSITY OF PRESSURE:-

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

P=df/da

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

  Mathematically F=PA

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

  Mathematically F=∫PA

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

Pascal's law:-

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

Atmospheric Pressure:-

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

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

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

= 760 mm of Hg

=10.3 (milli bar)

Gauge pressure:-

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

The atmospheric pressure on scale is marked as zero.

Absolute pressure:-

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

Vacuum pressure:-

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

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

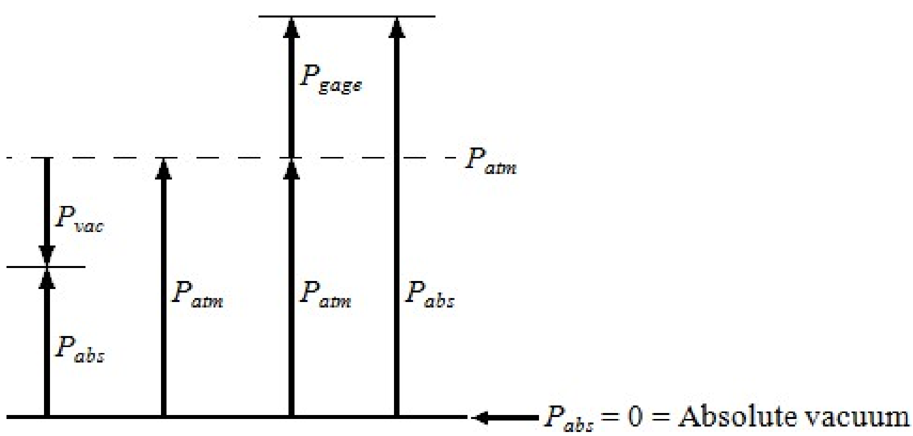


Fig. 1.2

- Equations

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

❖ Nomenclature

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

Pressure Head:-

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

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

where

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

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

Mathematically, $h = P/w$

Pressure Gauges :-

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

1. manometers
2. mechanical gauges

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

a) Simple manometers

b) Differential manometer

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

a) Diaphragm pressure gauge

b) Bourdon tube pressure gauge

c) Dead weight pressure gauge

d) Bellows pressure gauge

PRESSURE EXERTED ON IMMERSED SURFACE:-

Hydrostatic forces on surfaces:-

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

Forces on Submerged Surfaces in Static Fluids

These are the following features of statics fluids:-

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

Fluid pressure on a surface:-

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

$F = p\delta A$

TOTAL PRESSURE:-

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

Mathematically **total pressure**,

$P = p_1a_1 + p_2a_2 + p_3a_3$…………

Where,

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

The position of an immersed surface may be,

- Horizontal
- Vertical
- Inclined

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

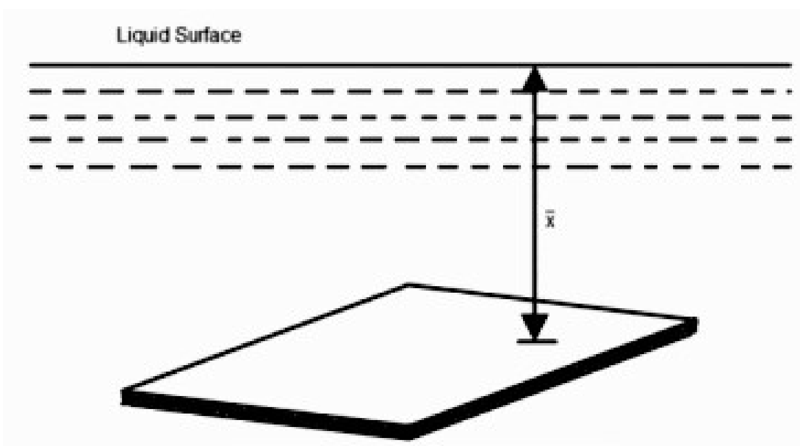


Fig. 1.3

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

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

P = Weight of the liquid above the immersed surface

= Specific weight of liquid * Volume of liquid

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

= $\omega A \chi kN$

Total Pressure On A Vertically Immersed Surface

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

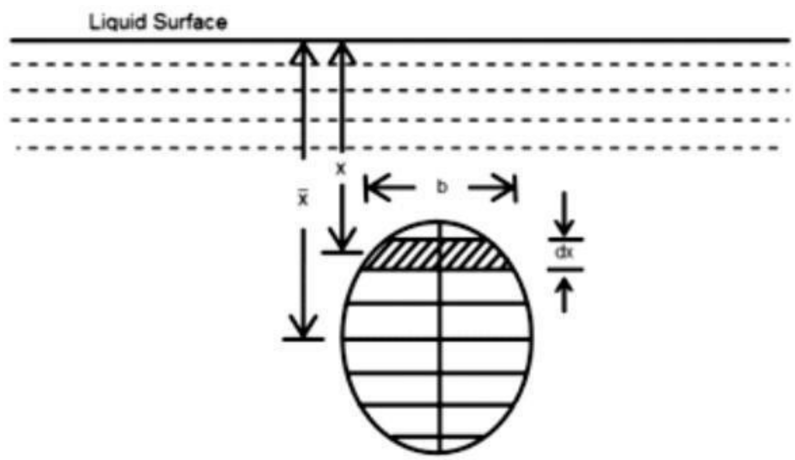


Fig. 1.4

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

Here,

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

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

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

and the area of strip =b.dx

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

Now, Total pressure on the surface,

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

$$= w\int x.bdx$$

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

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

FLOTATION AND BUOYANCY:-

Archimedes Principle:-

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

Buoyancy:-

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

Centre of pressure:-

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

Center of buoyancy:-

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

METACENTER:-

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

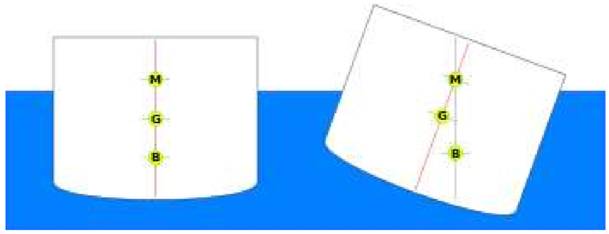


Fig. 1.5

The metacentre can be calculated using the formulae:

$$KM = KB + BM$$

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

Metacentric height:-

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

MG =BM-BG

MG=I/V-BG

Stability of a submerged body:-

Stable condition:-

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

Unstable equilibrium;-

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

Neutral equilibrium:-

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

Stability of a floating body:-

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

Chapter-II

KINEMATICS OF FLUID FLOW

Basic equation of fluid flow and their application:-

Energy of a Liquid in Motion:-
The energy, in general, may be defined as the capacity to do work. Though the energy exits in many forms, yet the following are important from the subject point of view:
1. Potential energy,
2. Kinetic energy, and
3. Pressure energy.

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

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

Pressure Energy of a Liquid Particle in Motion:-
It is the energy, possessed by a liquid particle, by virtue of its existing pressure. If a liquid particle is under a pressure of p kN/M2 (i.e., kPa), then the pressure energy of the particle-will be $\frac{p}{w.}$ mkg per kg of the liquid, where w is the specific weight of the liquid. Pressure head of the liquid under that pressure will be $\frac{p}{w.}$ metres of the liquid.

Total Energy of a Liquid Particle in Motion:-
The total energy of a liquid, in motion, is the sum of its potential energy, kinetic energy and pressure energy, Mathematically total energy,

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

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,

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

Example
Water is flowing through a tapered pipe having end diameters of 150 mm and 50 mm respectively. Find the discharge at the larger end and velocity head at the smaller end, if

the velocity of water at the larger end is 2 m/s. Solution. Given: d_1 = 150mm = 0•15 m ; d_2= 50 mm= 0•05 m and V_1 = 2•5 m/s. Discharge at the larger end We know that the cross-sectional area of the pipe at the larger end,

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

and discharge at the larger end,

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

= 44.2Jitres/s Ans.

Velocity head at the smaller end

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

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

Since the discharge through the pipe is continuous, therefore

$$a_1.v_1 = a_2.v_2$$

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

:. Velocity head at the smaller end

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

Bernoulli's Equation:-

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

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

where

Z = Potential energy,

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

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

Proof

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

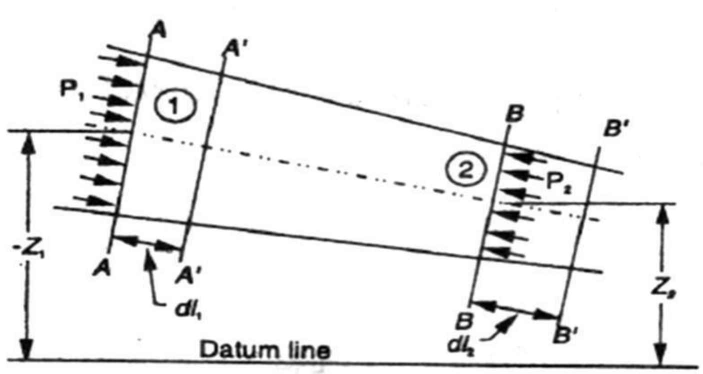


Fig. 2.1

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

Let
Z_1 = Height of AA above the datum,
P_1= Pressure at AA,
V_1 = Velocity of liquid at AA,
A_1 = Cross-sectional area of the pipe at AA, and
Z_2, P_2, V_2, A_2= Corresponding values at BB.
Let the liquid between the two sections AA and BB move to A' '' 'nd B' '' 'hrough very small lengths dl_1 and dl_2 as shown in Fig. This movement of the liquid between AA and BB is equivalent to the movement 'o' the liquid between AA and A' '' 'o BB and B' '','the remaining liquid between A' '' 'nd BB being un horoughlyLet W be the weight of the liquid between AA and A' ''.'Since the flow is continuous, therefore
$W = wa_1 dI_1 = wa_2 dL_2$
or $a_1 \times dl_1 = \frac{W}{w}$ ..I)

Similarly $a_2 dl_2 = \frac{W}{w}$

$\therefore a_1 . dL_1 = a_2 dL_2$ ..Ii)
We know that work done by pressure at AA, in moving the liquid to A' ''
' Force x Distance = $P_1 . a_1 . dL_1$
Similarly, work done by pressure at BB, in moving the liquid to B' ''’-
$P_2 a_2 dl_2$
..Iinus sign is taken as the direction of P_2 is opposite to that of P_1)
:. Total work done by the pressure
$= P_1 a1 dl_1 - P_2 a_2 dl_2$
$= P_1 a1 dl_1 - p_{2a} 1 dl_1$
…($a_1 dl_1 = a_2 dl_2$)
$= a_1 . dl_1 (P_1 - P_2) = \frac{W}{w}(P_1 - P_2)$ …($a_1 . dl_1 = \frac{W}{w}$)
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 "E“ler's' equation for steady flow of an ideal fluid along a streamline is based on the

Newton's' Second Law of Motion. The integration of the equation gives Bernoulli's' equation in the form of energy per unit weight of the flowing fluid. It is based on the 'f'llowing assumptions:

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

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

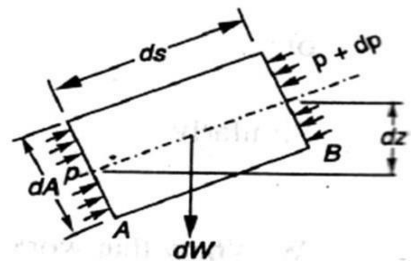


Fig. 2.2

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 = ρ g. dA . ds
From the geometry of the figure, we find that the component of the weight of the fluid element
,in the direction of flow
= - ρ g . dA . ds cosθ
= - ρ g . dA . ds($\frac{dz}{ds}$) …cosθ =$\frac{dz}{ds}$
= - ρ g. dA. dz
$\therefore$ *mass of the fluid element* = ρ .dA.Ds
,We see that the acceleration of the fluid element
$\frac{dv}{dt} = \frac{dv}{ds} \times \frac{ds}{dt} = v.\frac{dv}{ds}$
Now, as per Newton's' Second Law of Motion, we know that
Force = Mass x Acceleration
(- dp . dA) - (ρ g . dA . dz-) = ρ . dA . ds $\times \frac{dv}{ds}$
$\frac{dp}{\rho} + g.dz = v.dv$ …(dividing both side by -ρdA)
Or $\frac{dp}{\rho} + g.dz + v.dv = 0$
This is the required Euler's' equation for motion and is in the form of a differential equation. Integrating the above equation,
'

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

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

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

or in other words, $\frac{p1}{w}+Z_1+(v_1^2/2g)=\frac{p2}{w}+Z_2+(v_2^2/2g)$

which proves the Bernoulli's' equation.

Limitations of Bernoulli's' Equation:-

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

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

Example

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

Solution.'G'~en: d_1= 200 mm = 0.2 m; Z_1= 5 m; d2 = 50 mm = 0.05 m z_2 = 3 m; p = 500/

kPa = 500 kN/M2 and V_1 = 1 mls.

Let

V_2 = Velocity of flow at section 2, and

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

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

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

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

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

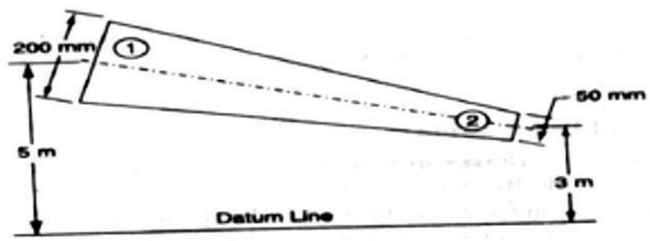


Fig. 2.3

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

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

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

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

practical Applications of Bernoulli's' Equation

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

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

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

Venturimeter

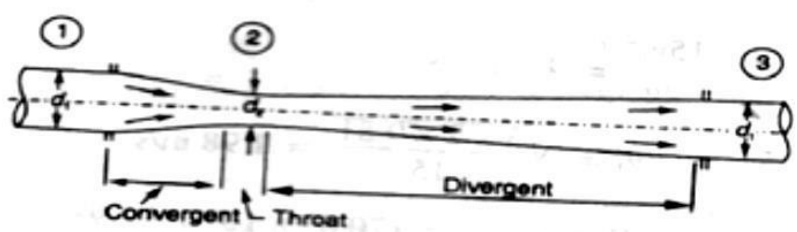


Fig. 2.4

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

(a) Convergent cone.
(b) Throat.
©Convergent cone.

(a) Convergent cone

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

(b) Throat

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

(c) Divergent cone

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

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

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

Discharge through a Venturi meter

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

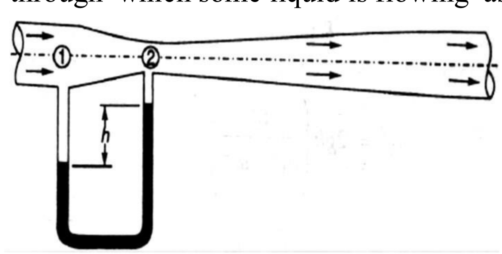


Fig. 2.5

Let

P_1 = Pressure at section 1,

V_1 = Velocity of water at section 1,

Z_1 = Datum head at section 1,

a_1 = Area of the ve horougeter at section 1, and

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

We know that the discharge through a venture meter,

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

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

Example

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

Solution. Given: d_1 = 150 mm = 0.15 m; d_2 = 100 mm = 0.1 rn; Specific gravity of oil = 0.9

h = 200 mm = O.2 m of mercury and C = 0.98.

We know that the area at inlet,

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

and the area at throat,

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

We also know that the difference of pressure head,

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

and the discharge through the ve horougeter,

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

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

Orifice Metre

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

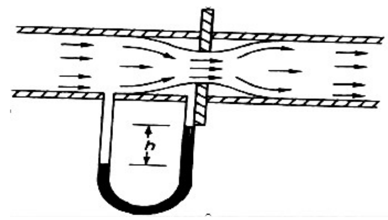


Fig. 2.6

Let

h = Reading of the mercury manometer,

P_1 = Pressure at inlet,

V_1 = Velocity of liquid at inlet,

a_1 = Area of pipe at inlet, and

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

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

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

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

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

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

$V_1 = a_2/a_1 \times v_2$ 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)

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

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

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

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

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

We know that the area of pipe,

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

and area of throat

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

We also know that the pressure difference,

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

and rate of flow,

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

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

Pitot Tube.

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

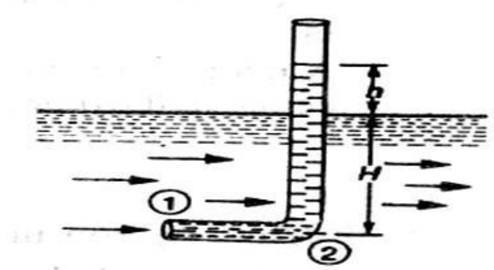


Fig. 2.7

H = Depth of tube in the liquid, and

v = Velocity of the liquid.

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

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

…..($z_1 = z_2$)

$h = v^2/2g$

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

Example .

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

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

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

We know that the velocity of water in the pipe,

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

Rate of Discharge

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

Let, a = Cross-sectional area of the pipe, and
v = Average velocity of the liquid,
:. Discharge, Q = Area × Average velocity = a.v

Notes: 1. If the area is in m^2 and velocity in m/s, then the discharge,
$Q = m^2 \times m/s = m^3/s$ = cumecs
2.member that $1m^3$ = 1000 litres.

Equation of Continuity of a Liquid Flow

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

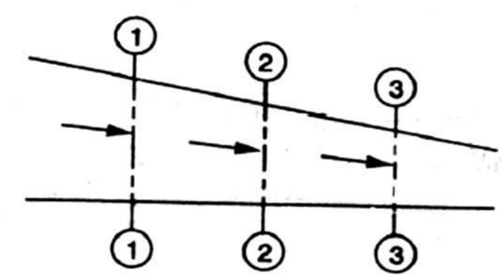


Fig. 2.8

CONTINUITY OF A LIQUID FLOW

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

Let , a_1 = Cross-sectional area of the pipe at section 1-1, and
v_1= Velocity of the liquid at section 1-1,

Similarly , a_2 , v_2= Corresponding values at section 2-2,
and a_3 , v_3 = Corresponding values at section 3-3.
We know that the total quantity of liquid passing through section 1-1,

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

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

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

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

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

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

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

Solution. Given: d_1= 100 mm = 0.1 m; V_1 = 10 m/s and d_2= 200 mm = 0.2 m.
Rate of discharge
We know that the cross-sectional area of the pipe at point 1,

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

and rate of discharge, $Q = a_1.v_1 = (7.854 \times 10^{-3}) x\ 10 = 78.54 \text{ X } 10^{-3}\ m^3/s$
= 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= $\frac{Q}{a_2}$ = $((78.54 \text{ X } 10^{-3})/(31.42 x 10^{-3}))$=2.5m/s **Ans.**

Flow over Notches:-

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

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

1. Nappe or Vein. The sheet of water flowing through a notch or over a weir is called Nappe or Vein.
2. Crest or Sill. The bottom edge of a notch or a top of a weir over which the water flows, is known as the sill or crest.

Classification Of Notches And Weirs:-

The notches are classified as :

I. According to the shape of the opening:

(a) Rectangular notch,

(b) Triangular notch,

(c) Trapezoidal notch, and

(d) Stepped notch.

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

(a) Notch with end contraction.

(b) Notch without end contraction or suppressed notch,

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

Discharge Over A Rectangular Notch Or Weir

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

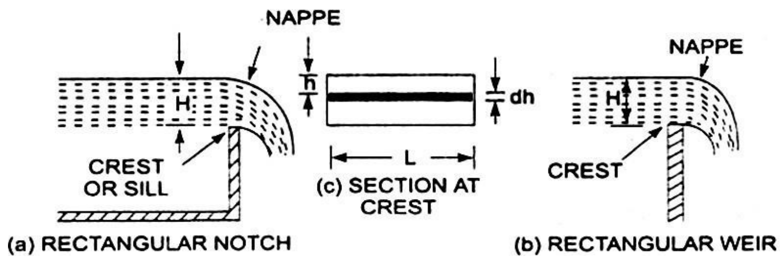


Fig. 2.9

Rectangular notch and weir:-

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

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

Problem - –

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

Solution. Given:

Length of the notch, L=2.0m

Head over notch, H = 300 m = 0.30 m

C_d=0.06

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

$=\frac{2}{3}\times 0.6\times 2.0\times\sqrt{2}\times 9.81\times[0.30]$ *1. 5* m3/s

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

Problem 2

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

Solution. Given:

Length of weir, L=6m

Depth of water, H1=1.8m

Discharge, Q = 2000 litIs = 2 m3/s

Cd/=0.6

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

The discharge over the weir is given by the equation .

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

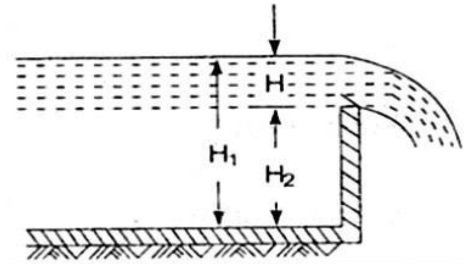


Fig. 2.10

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

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

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

$H=\left(\frac{2.0}{10.623}\right)^{2/3}=0.328$ 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 : LetH = head of water above the V- notch

θ = angle of notch

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

For a right angle V Notch ,if C_d=0.6

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

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

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

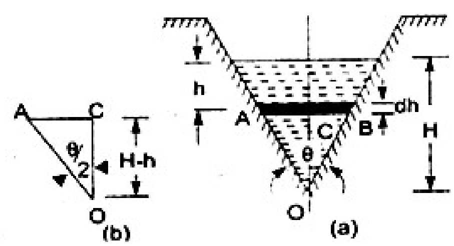


Fig. 2.11

Problem -1

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

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

Head over notch, H=0.3 m

C_d = 0.6

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

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

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

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

Problem -2

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

Solution. Given:

For rectangular weir. Length= L = 1 m

Depth of water, H = 150mm=0.15m

$C_d = 0.62$

For triangular weir.

$\theta = 90°$

$C_d = 0.59$

Let depth over triangular weir $= H_1$

The discharge over the rectangular weir IS given by equation

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

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

$= 0.10635\ m^3/s$

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

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

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

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

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

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

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

Discharge Over A Trapezoidal Notch Or Weir:-

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

Let H = Height of water over the notch

L = Length of the crest of the notch

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

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

The-discharge through rectangular portion ABCD is given by

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

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

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

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

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

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

Solution. Given:

Top width AE=1 m

Base width, CD=L=0.4 m

Head of water, H=0.20 m

For rectangular portion, C_{d1}=0.62

From ΔABC ,we have

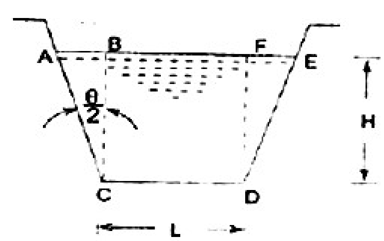


Fig. 2.12

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

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

Discharge through trapezoidal notch is given by equation

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

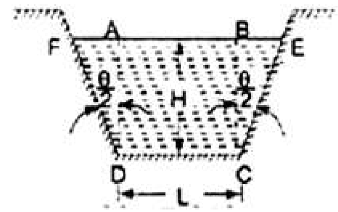
$$= \frac{2}{3} \times 0.62 \times 0.4 \times \sqrt{2 \times 9.81} \times (0.2)^{3/2} + \frac{8}{15} \times 60 \times 1 \times \sqrt{2} \times 9.81 \times (0.2)^{5/2}$$

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

Discharge Over A Stepped Notch:-

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

Consider a stepped notch as shown in Fig.



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

L_1 = Length of notch 1,

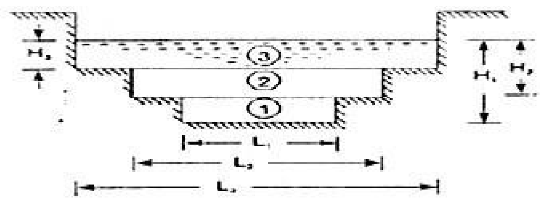
H_2, L_2 and H_3, L_3 are corresponding values for notches 2 and 3 resp

C_d=Co-efficient of discharge for all notches

Total discharge $Q=Q_1+Q_2+Q_3$

Fig. 2.12

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



Problem

Fig. 2.13

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

Solution. Given:

L_1 = 40 cm, L_2 = 80 cm,

L_3 = 120cm

H_1 = 50 + 30 + 15 = 95 crn,

H_2=80 cm, H_3=50 cm,

C_d=0.62

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

where

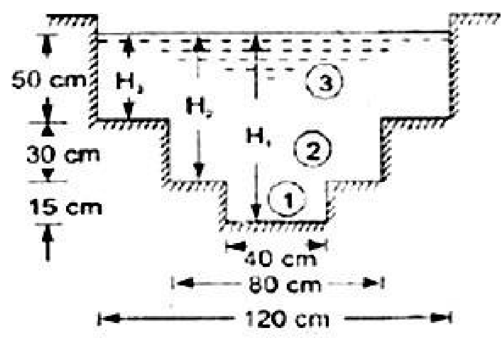


Fig. 2.14

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

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

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

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

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

$=530141 cm^3/s$

=530.144 lit/s

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

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

=776771 cm^3/s

=776.771 lit/s

∴ $Q = Q_1 + Q_2 + Q_3$

=154.067+530.144+776.771

=1460.98 lit/s Ans.

Velocity Of Approach

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

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

changed taking into consideration of velocity of approach.

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

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

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

Discharge over a rectangular weir, with velocity of approach

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

Problem:-

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

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

velocity of approach into consideration.
Solution. Given:

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

Length of weir, L = 60 cm = 0.6 m

Head of water, H_1 = 20 cm = 0.2 m

C_d = 0.62

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

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

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

$=0.0982\ m^3/s$

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

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

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

Then discharge with velocity of approach is given by equation

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

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

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

$$Q = \frac{2}{3} \times C_d . L\sqrt{2g} \times H^{3/2}$$
$$= \frac{2}{3} \times 0.623 \times 10\sqrt{(2 \times 9.81)} \times (0.4)^{3/2}$$
$$= 46.55 \ m^2/s = 4655 \ lit/s$$

Discharge over a Broad-crested Weir :-

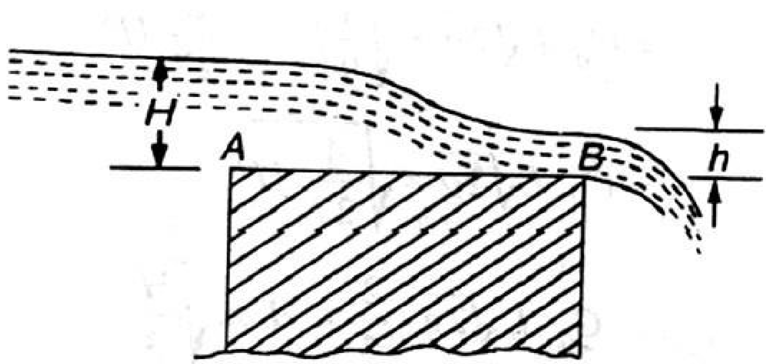


Fig. 2.15

Broad-crested weir

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

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

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

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

C_d = Coefficient of discharge, and

L = Length of the weir.

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

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

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

We know that the discharge over the weir,

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

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

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

Discharge over a Sharp-crested Weir :-

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

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

$$b<(H/2)$$

where , b = Thickness of the weir,

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

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

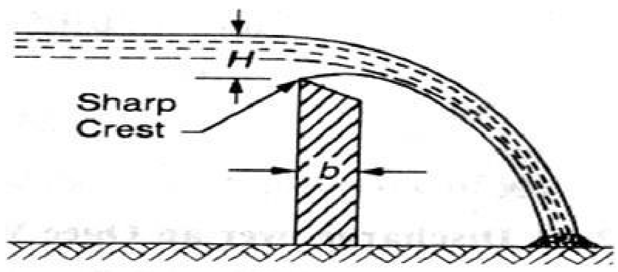


Fig. 2.16

Sharp-crested weir :-

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

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,

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

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

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

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

Where C_d = Co-efficient of discharge and

L= Length of an ogee weir

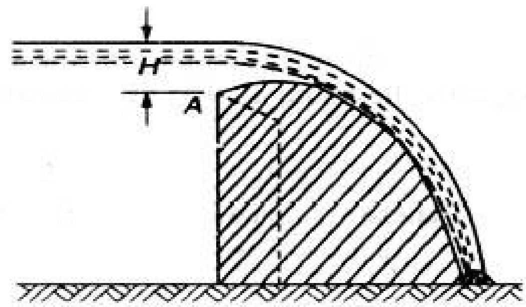


Fig. 2.17

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,

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

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

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

Discharge over a Submerged or Drowned Weir :-

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

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

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

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

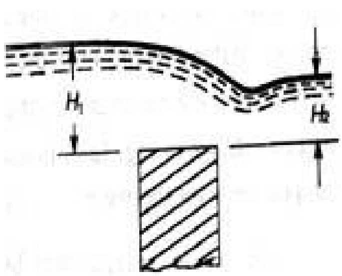


Fig. 2.18

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

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

Submerged weir :-

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

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

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

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

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

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

of water on the upstream side,

$H_1 = 1.25 - -.8 = 0.45$ m and depth of water on the downstream side,

$H_2 = 1 - 0.8 = 0.2$ m

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

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

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

$= 5.315 \text{ x } 0.125 = 0.664\ m^3/s = 664$ liters/s ... (i)

and discharge over the submerged portion of the weir,

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

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

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

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

2.3 Flow over Weirs:-

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

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

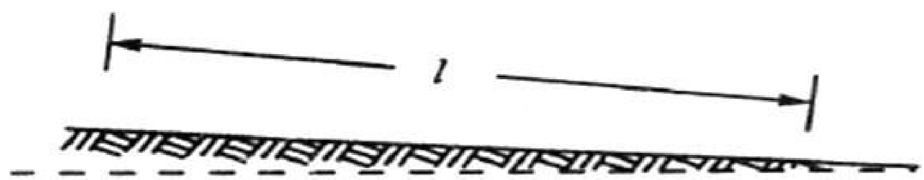


Fig. 2.19

Sloping bed of a channel :-

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

Let

I = Length of the channel,

A = Area of flow,

v = Velocity of water,

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

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

i = Uniform slope in the bed.

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

∴ Discharge Q= A X v=AC$\sqrt{}$mi

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

We know that the area of the channel,

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

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

:. Hydraulic mean depth, m = $\frac{A}{P}$ = 1 m
and the discharge through the channel,

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

Manning Formula for Discharge :-

Manning, after carrying out a series of experiments, deduced the following relation for the value of C in Chezy's' formula for discharge:
C=$\frac{1}{N}$ x $m^{1/6}$
where N is the Kutter's' constant
Now we see that the velocity,

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

where

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

Now the discharge,

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

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

Example

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

Solution.

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

We know that the area of flow,

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

and wetted perimeter,

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

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

We know that the discharge through the channel

$Q =$ Area x 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.
$A = b \times d$
Discharge $Q = A \times v = AC\sqrt{mi}$
$m = A/P$
$= d/2$

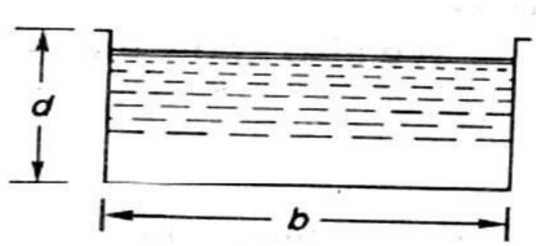


Fig. 2.20

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

Example

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

Solution. Given: $A = 8\ m2$

$i = 1/1000 = 0.001$ and $C = 55$.
Size of the channel
Let
b = Breadth of the channel, and
d = Depth of the channel.
We know that for the most economical rectangular section,
$b = 2d$
:. Area (A) $8 = b \times d = 2d \times d = 2d^2$
$= b = 2$ m

And $b = 2d = 2 \times 2 = 4$ m

Discharge through the channel

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

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

and the discharge through the channel,

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

$= 440 \times 0.0316 = 13.9\ m^3/s$, Ans.

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

A trapezoidal section is always provided in the earthen channels. The side slopes, in a channel of trapezoidal cross-section are provided, so that the soil can stand safely. Generally, the side slope in a particular soil is decided after conducting experiments on that soil. In a soft soil, flatter side slopes
should be provided whereas in a harder one, steeper side slopes may be provided.
consider a channel of trapezoidal cross- section ABCD as shown in FIg.

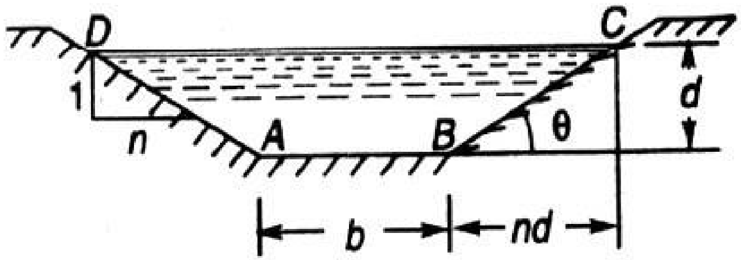


Fig. 2.21

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

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

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

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

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

m = d/2 = 0.5 m

and discharge in the channel,

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

Example .

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

Solution.

Given side slope (n)=1

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

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

and C = 50

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

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

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

or b = 0.828d

∴ Area A = d X (b + nd) = 1.828d^2

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

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

∴ $d^{5/2}$ =3/1.866 = 1.608

Or d = 1.21 m

∴ b = 0.828 d = 0.828 X 1.21 = 1 m ANS

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

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

Let r = Radius of the channel,

h = Depth of water in the channel, and

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

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

$$P = 2r\theta \quad ..I)$$

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

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

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= M$\times m^{2/3}\times i^{1/2}$ = 0.71m/s ANS

Chapter-III

PUMPS

Centrifugal Pumps:-

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

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

Main Parts Of A Centrifugal Pump:-

The followings are the main parts of a centrifugal pump:

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

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

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

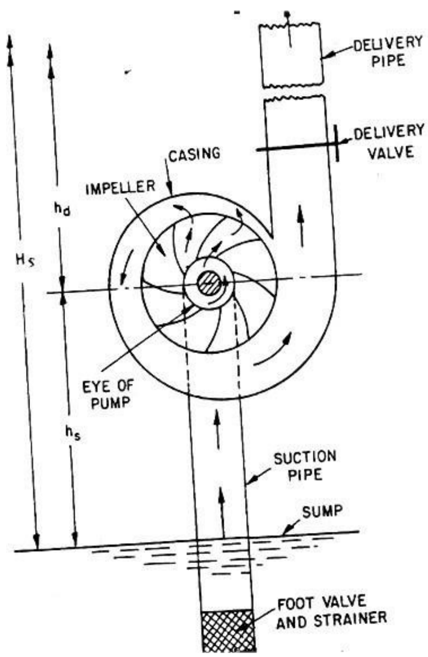
2. **Casing:** The casing of a centrifugal pump is similar to the casing of a reaction turbine. It is an air-tight passage surrounding the impeller & is designed in such a way that the kinetic energy of the water discharged at the outlet of the impeller is converted into pressure energy before the water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

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

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

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

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



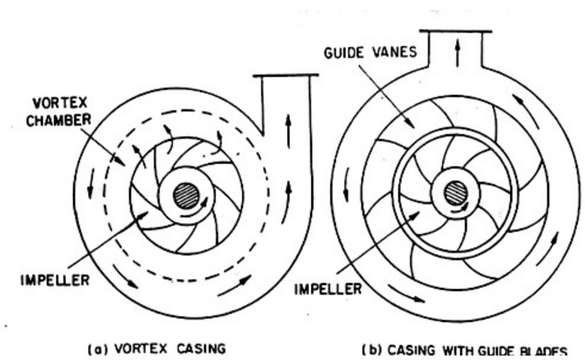
Main parts of a centrifugal pump

Fig. 3.1

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

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

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



fferent type of casing

Fig: 3.2

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

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

a. Manometric efficienciesη_{man}

b. Mechanical efficienciesη_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_{max} = Manometric head/Head imparted by impeller to water

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

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 = $\frac{\text{Work done by impeller per second}}{1000} kW$

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

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

b) Mechanical efficiencies:-

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

η_m = Power at the impeller/Power at the shaft

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

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

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

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

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

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

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

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

Speed N=1200r.p.m

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

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

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

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

Tangential velocity of impeller at inlet & outlet are,

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

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

From inlet velocity triangle,

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

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

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

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

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

Speed, N =1000r.p.m

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

Impeller diameter means the diameter of the impeller at outlet

Diameter, $D_2 = 300mm = 0.30m$

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

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

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

Now using equation

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

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

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

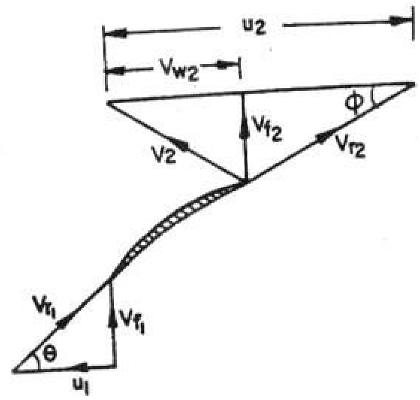


Fig. 3.3

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

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

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

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

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

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

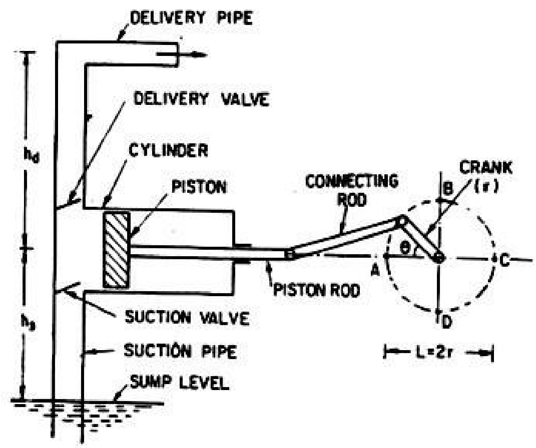
Reciprocating Pump:-

Introduction:-

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

Main parts of a reciprocating pump:-

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



Main parts of a reciprocating pump.

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

Fig. 3.4

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

Let D= dia. Of the cylinder

A= C/s area of the piston or cylinder

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

r= Radius of crank

N=r.p.m of the crank

L=Length of the stroke=2*r

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

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

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

= Area * Length of stroke = A*L

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

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

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

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

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

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

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

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

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

Substituting the value of W in equation () we get

Work done per second =

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

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

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

Classification of reciprocating pumps:

The reciprocating pumps may be classified as:

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

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

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

I. Single-acting pump

II. Double –acting pump

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

I. Single cylinder pump

II. Double cylinder pump

III. Triple cylinder pump

CHAPTER 10

DAMS

GRAVITY DAMS-EARTH DAMS

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

FORCES ACTING ON GRAVITY DAM:

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

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

Ice pressure

6. Self weight of the dam.

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

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

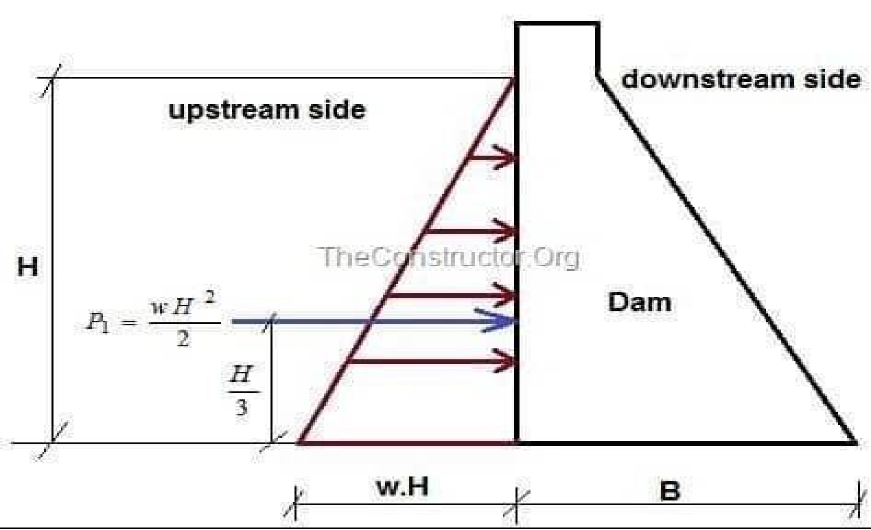
1. Water Pressure

Water pressure may be subdivided into the following two categories:

I) **External water pressure:**

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

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



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

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

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

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

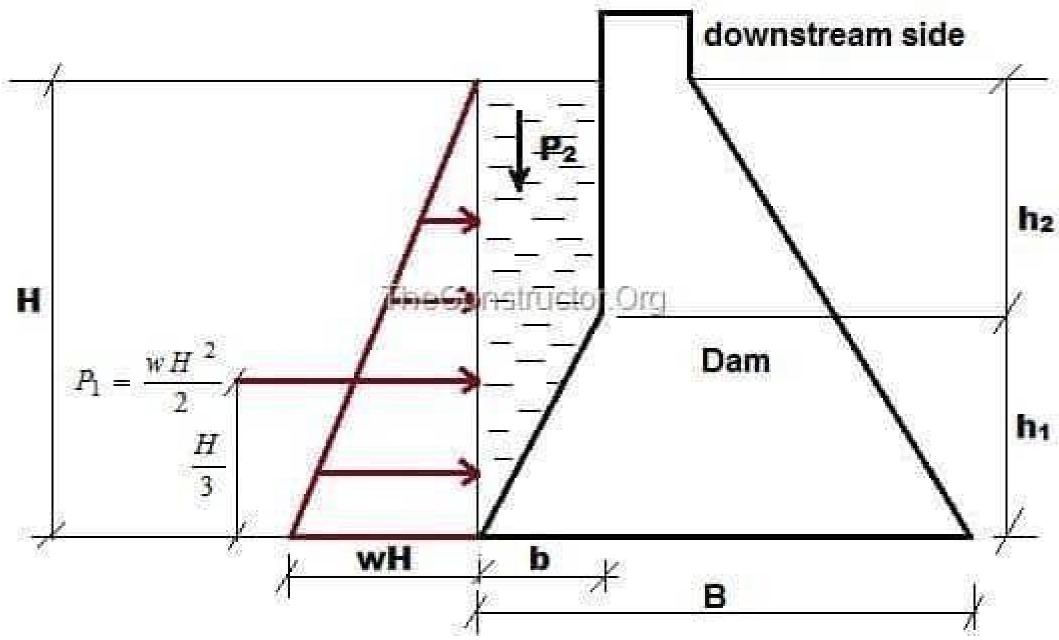


Figure 2

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

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

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

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

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

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

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

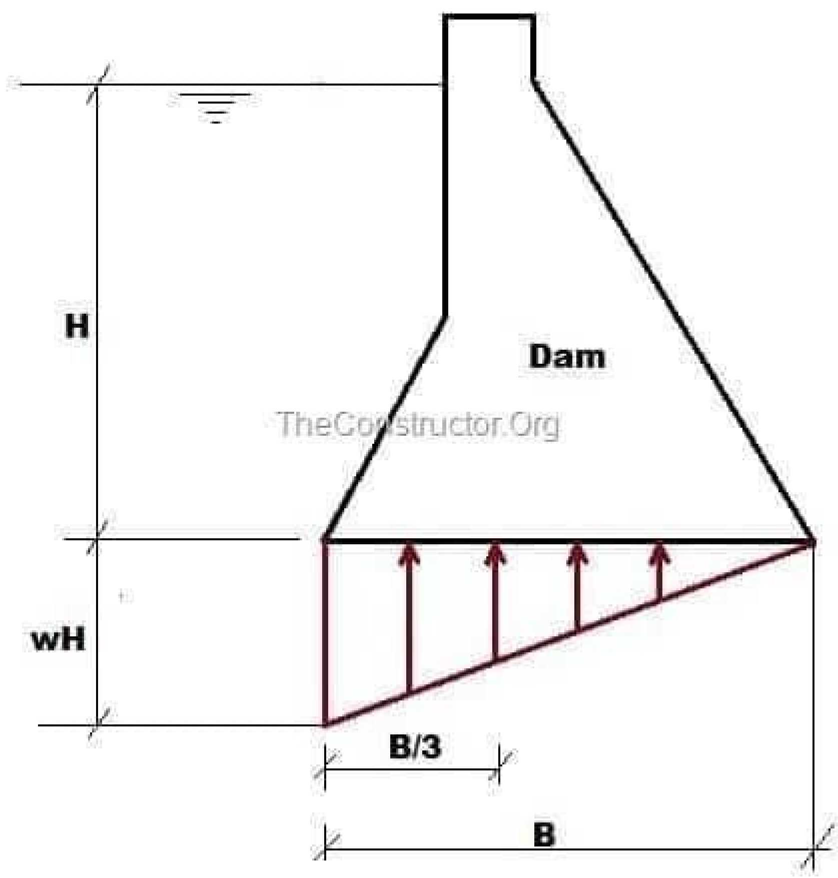


Figure 3

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

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

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

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

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

2. Earthquake Forces

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

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

3. Silt Pressure

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

base.Where,

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

h = height of silt deposited.

4. Wave Pressure

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

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

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

Where h_w is the height of water from the top of crest to bottom of trough in meters.V – wind velocity in km/hour

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

The maximum pressure intensity due to wave action may be

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

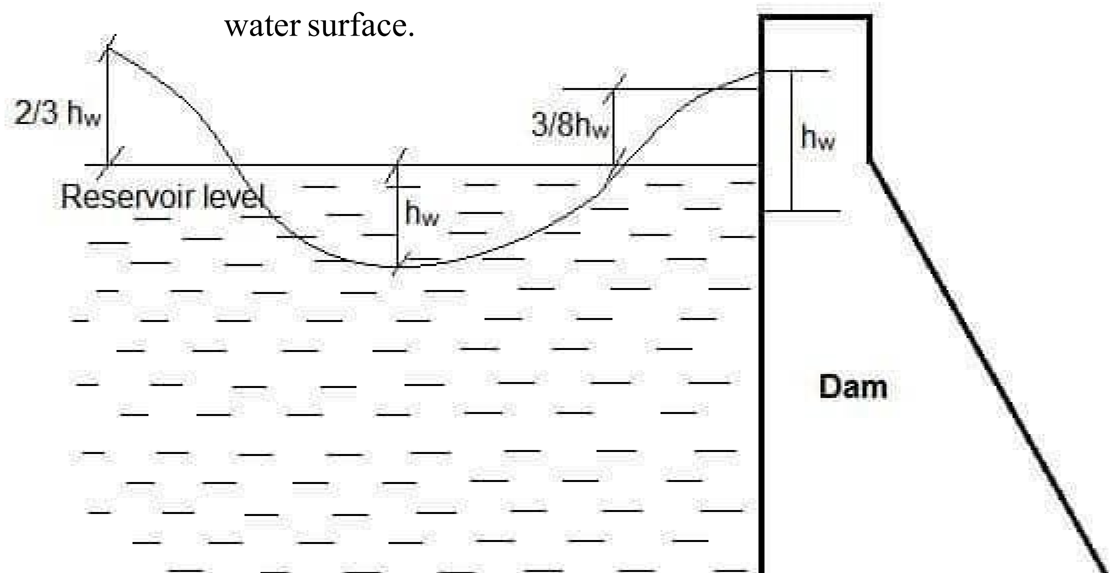


Figure 4

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

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

5. Ice Pressure

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

6. Weight of dam

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

FAILURES OF GRAVITY DAM

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

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

A gravity dam may fail in following modes:

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

Overturning Failure of Gravity Dam:

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

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

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

Factor of safety against overturning is given by

FOS = sum of overturning moments/ sum of resisting moments

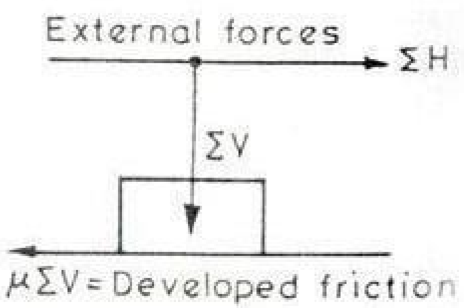


Fig:sum of external horizontal forces greater than vertical self-weight of dam (overacting, sliding occurs)

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

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

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

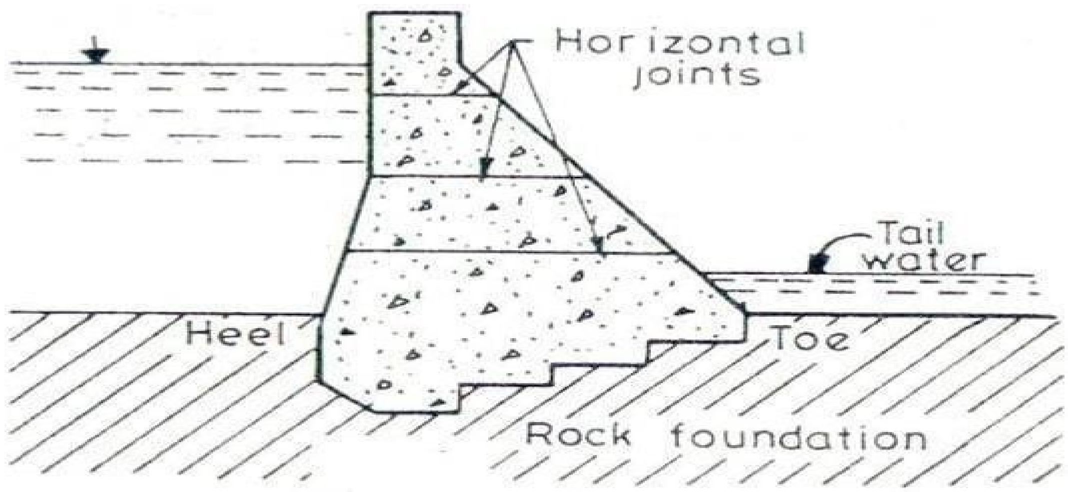
Factor of safety based on frictional resistance:

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

μ=co-efficient of friction between two surfaces

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

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



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

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

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

STABILITY ANALYSIS OF GRAVITY DAMS

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

Stability Analysis Assumptions:

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

Stability Analysis Procedure

Two dimensional analysis can be carried out analytically or graphically

Analytical Method

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

Graphical method

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

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

Profile of A Dam from Practical Considerations

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

These needs are,

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

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

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

Galleries in Gravity Dams

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

Function and types of galleries in Dams

(i) Foundation Gallery

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

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

(ii) Inspection Galleries

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

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

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

EARTH DAMS

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

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

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

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

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

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

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

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

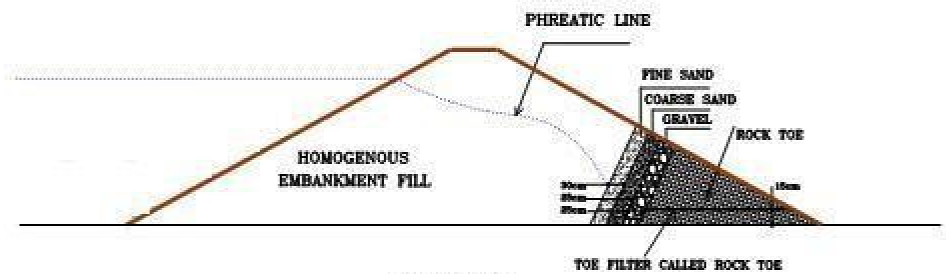
An earth dam is composed of suitable soils obtained from borrow areas or required excavation and compacted in layers by mechanical means. Following preparation of a foundation, earth from borrows areas and from required excavations is transported to the site, dumped, and spread in layersof required depth.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

Phreatic Line and Horizontal Drain In Earth fill Dams

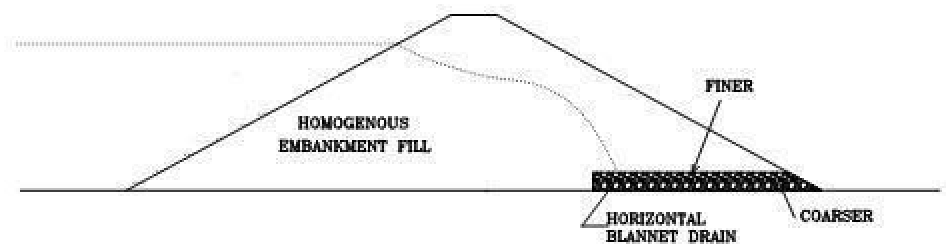
Earth dams are generally built of locally available materials in their natural state with a minimum of processing. Homogeneous earth dams are built whenever only one type of material is economically available.

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

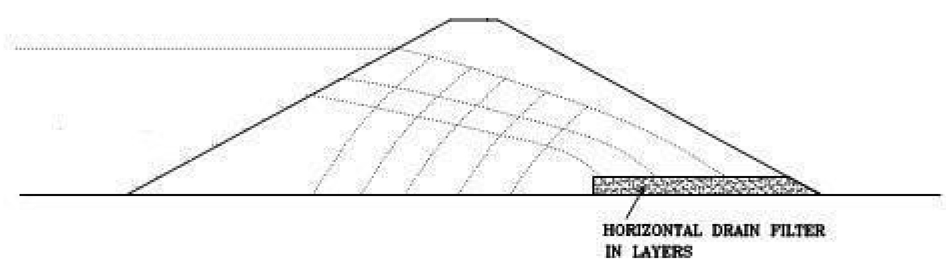
The general design procedure is to make a first estimate on the basis of experience with similar dams and then to modify the estimate as required after conducting a stability analysis except where there is a surplus of material.



Rock Toe



Horizontal Fillter



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

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

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

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

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

Stability and Failure of Earth Filled Dams

Failure of earth dams may be:

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

1. Hydraulic Failure:

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

i. Overtopping of dams:

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

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

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

2. Seepage Failure:

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

Seepage failure of the dams is of the following types

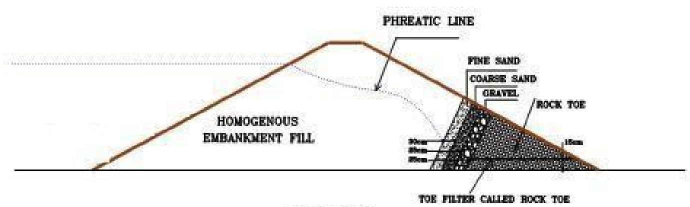
1. Piping through the dam
2. Piping through the foundation
3. Conduit Leakage

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

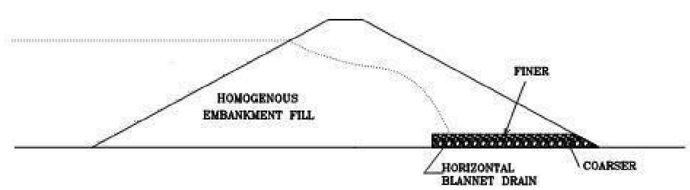
If the seepage force exceeds the weight of the material the water washes away the soil from theplate and creates a hole in the ground. This hole deepens as more and more mateial is taken away from it and extends longitudinally, making a pipe hole called "Piping in the dam".

3. Structural Failure:

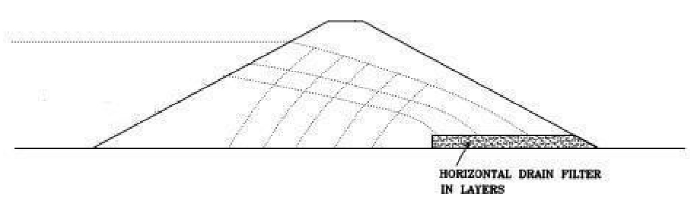
Structural failures can occur in either the embankment or the appurtenances. Structural failure of a spillway, lake drain, or other appurtenance may lead to failure of the embankment.



Rock Toe



Horizontal Fillter



Cracking, settlement, and slides are the more common signs of structural failure of embankments. Large cracks in either an appurtenance or the embankment, major settlement, and major slides will require emergency measures to ensure safety, especially if these problemsoccur suddenly. If this type of situation occurs, the lake level should be lowered, the appropriate state and local authorities notified, and professional advice sought. If theobserver is uncertain as to the seriousness of the problem, the Division of Water should be contacted immediately. The three types of failure previously described are often interrelated in a complex manner. For example, uncontrolled seepage may weaken the soil and lead to a structural failure. A structural failure may shorten the seepage path and lead to a piping failure. Surface erosion may result in structural failure.

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

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

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

UNIT – 1

CROP WATER REQUIREMENT

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

Irrigation- Definition

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

Need of the Irrigation

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

Less rainfall:

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

Non uniform rainfall:

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

Commercial crops with additional water:

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

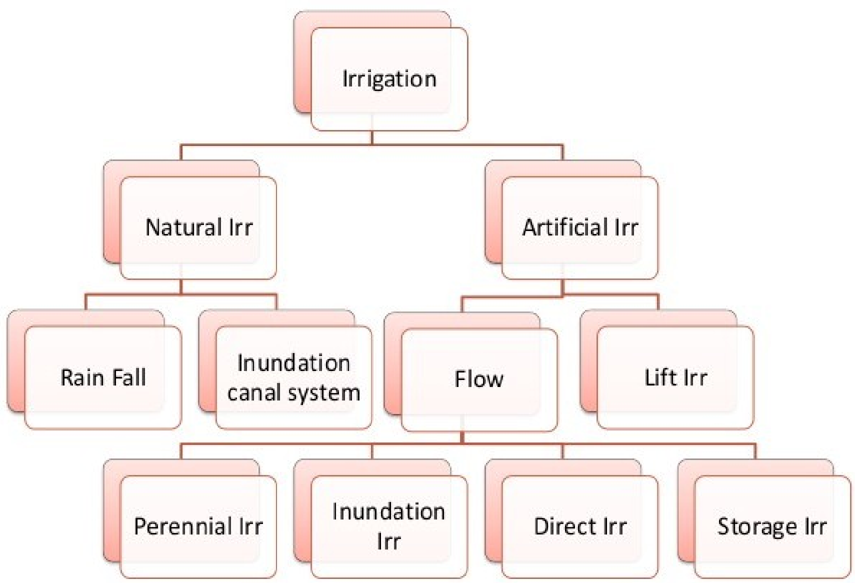
Controlled water supply:

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

Benefits of Irrigation:

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

Types of Irrigation OR Classification of Irrigation:



Natural Irrigation

- No engineering structure is constructed.

1) Rainfall Irrigation

- Rainfall is only used for raising crops.

2) Inundation canal system

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

Artificial Irrigation

- Properly designed engineering structure are constructed.

1) Flow irrigation

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

a) Perennial irrigation :

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

b) Inundation irrigation:

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

c) Direct irrigation :

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

d) Storage Irrigation:

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

2) Lift or well Irrigation:

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

Historical development of Irrigation

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

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

Present status of Irrigation:

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

Developmental Aspects of Irrigation:

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

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

Advantages of irrigation

Advantages of irrigation can be direct as well as indirect.

I. Direct Benefits

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

II. Indirect Benefits

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

Disadvantages of Irrigation

The following are the disadvantages of irrigation.

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

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

Types of Crops:

1) **Wet crops**- which lands are irrigated and than crop are cultivation

2) **Dry crops**-which do not need irrigation.

3) **Garden crops**- which need irrigation throughout the year

4) **Summer crop (Kharif)**-which are sown during the south west monsoon & harvested in autumn.

5) **Winter crops(rabi)**-which are sown in autumn & harvested in spring.

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

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

Crop

Seasons:

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

• Rabi crops are called as winter crops and kharif crops are called as summer crops.

• Kharif crops required more water than rabi crops.

• Rabi starts from 1 st oct and ends on 31 march

• In TamilNadu crops are classified as wet and dry crops.

Crops rotation:

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

Necessity for rotation

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

General rotation of crops can be summarized as:

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

Consumptive Use of Water

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

Duty :

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

Delta:

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

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

Factor affecting the duty:

1) Soil Moisture

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

2) Topography

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

3) Nature of rainfall

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

4) Nature of crop irrigated

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

5) Method of cultivation:

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

6) Season of crop

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

7) System of Irrigation

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

8) Canal Condition

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

Improving Duty

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

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

3. Water losses can be minimized by lining the canals.

4. The cultivators should be trained to use water economically without wasting.

5. The soil properties should be studied by establishing research stations in villages.

Crop Period or Base Period:

- The time period that elapses from the instant of its sowing to the instant of its harvesting is called the **crop period**.
- The time between the first watering of a crop at the time of its sowing to its last watering before harvesting is called the **base period**.

Duty and Delta of a Crop Delta:

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

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

***Solution*:**

No. of watering required = 120/10 = 12

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

Δ for rice = 120 cm

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

***Solution*:**

No. of watering required = 140/28 = 5

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

Δ for wheat = 37.5 cm

Duty:

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

Factors on which duty depends:

1. Type of crop

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

Importance of Duty

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

Measures for improving duty of water:

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

(1) Proper Ploughing:

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

(2) Methods of supplying water:

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

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

(3) Canal Lining:

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

(4) Minimum idle length of irrigation Canals:

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

(5) Quality of water:

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

(6) Crop rotation:

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

Consumptive use of crops

Definition:

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

  Mathematically,

  Consumptive Use = Evapotranspiration = Evaporation + transpiration

- It is expressed in terms of depth of water.

Factors Affecting the Consumptive Use of Water

Consumptive use of water varies with:

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

Types of Consumptive Water Use

Following are the types of consumptive use,

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

1. Optimum Consumptive Use:

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

2. Potential Consumptive Use:

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

3. Seasonal Consumptive Use:

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

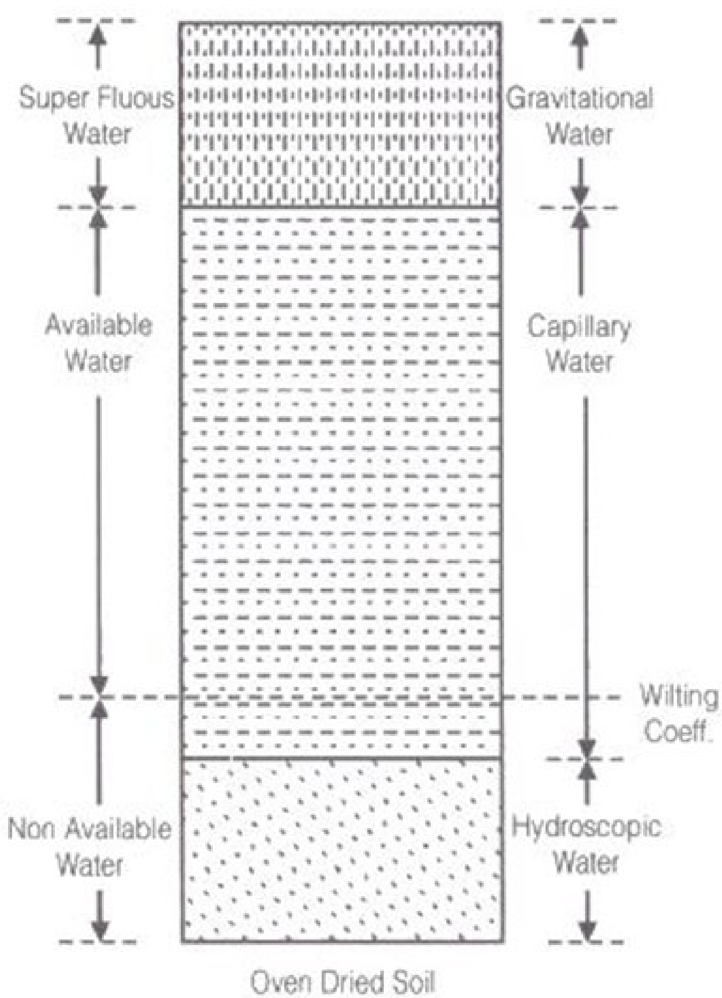
Crop Water Requirements

Soil moisture

Classes and availability of soil water

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

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



Hygroscopic water

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

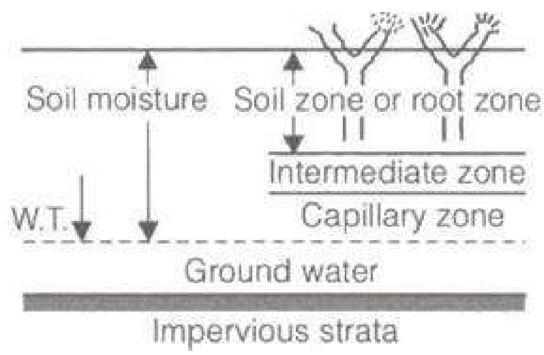
Capillary water

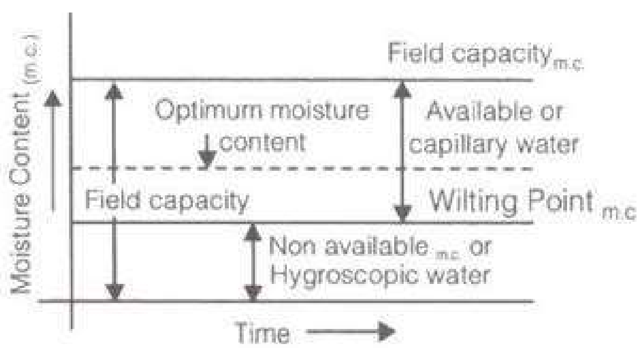
The capillary water is held within soil pores due to the surface tension forces (against gravity) which act at the liquid-vapour (or water-air) interface.

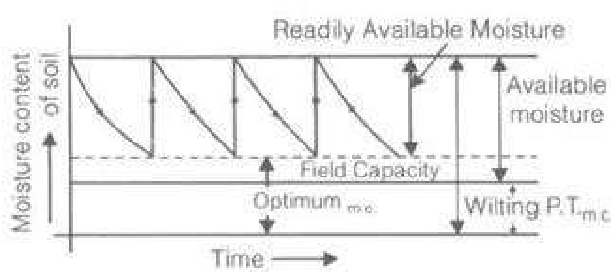
Gravitational water

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

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







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

Here FC= field capacity

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

Mo= Readily available moisture content

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

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

where, weight of water stored in soil of unit area $= \gamma_w \cdot d_w \cdot 1 \cdot$

Weight of some soil of unit area $= \gamma \cdot d \cdot l$

dw= depth of water stored in root zone.

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

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

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

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

Duty and delta

Duty:

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

Delta:

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

Relation between duty and delta

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

Where,

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

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

Where,

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

Irrigation Requirements of crops

(1) Consumptive Irrigation Requirement (CIR)

CIR = Cu- P_{eff}

Where, Cu= total consumptive use requirement

P_{eff}= Effective rainfall.

(2) Net Irrigation Requirement (NIR)

NIR = CIR + Leaching requirement

(3) Field irrigation requirement (FIR)

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

(4) Gross irrigation requirement, (GIR)

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

Methods of Determination of Evapotranspiration

To measure or estimation the consumptive use there are three main methods:

1. Direct Methods/Field Methods
2. Empirical Methods
3. Pan evaporation method

1. Direct Methods:

In this method field observations are made and physical model is used for this purpose. This includes,

i. Vapour Transfer Method/Soil Moisture Studies
ii. Field Plot Method
iii. Tanks and Lysimeter
iv. Integration Method/Summation Method
v. Irrigation Method
vi. Inflow Outflow Method

i. Vapour Transfer Method:

In this method of estimation of water consumptive use, soil moisture measurements are taken before and after each irrigation. The quantity of water extracted per day from soil is computed for each period. A curve is drawn by plotting the rate of use against time and from this curve, the seasonal use can be estimated. This method is suitable in those areas where soil is fairly uniform and ground water is deep enough so that it does not affect the fluctuations in the soil moisture within the root zone of the soil.

It is expressed in terms of volume i.e. **Acre-feet** or **Hectare-meter**

ii. Field Plot Method:

We select a representative plot of area and the accuracy depends upon the representativeness of plot (cropping intensity, exposure etc).It replicates the conditions of an actual sample field (field plot). Less seepage should be there.

Inflow + Rain + Outflow = Evapotranspiration

The drawback in this method is that lateral movement of water takes place although more representative to field condition. Also some correction has to be applied for deep percolation as it cannot be ascertained in the field.

iii. Tanks and Lysimeter:

In this method of measurement of consumptive use of water, a watertight tank of cylindrical shape having diameter 2m and depth about 3m is placed vertically on the ground. The tank is filled with sample of soil. The bottom of the tank consists of a sand layer and a pan for collecting the surplus water. The plants grown in the Lysimeter should be the same as in the surrounding field. The consumptive use of water is estimated by measuring the amount of water required for the satisfactory growth of the plants within the tanks. Consumptive use of water is given by,

$$\mathbf{Cu = Wa - Wd}$$

Where,

Cu = Consuptive use of water

Wa = Water Applied

Wd = Water drained off

Lysimeter studies are time consuming and expensive. Methods 1 and 2 are the more reliable methods as compare to this method.

iv. Integration Method:

In this method, it is necessary to know the division of total area, i.e. under irrigated crops, natural native vegetation area, water surface area and bare land area. In this method, annual consumptive use for the whole area is found in terms of volume. It is expressed in Acre feet or Hectare meter.

Mathematically,

Total Evapotranspiration = Total consumptive usex

Total Area Annual Consumptive Use = Total Evapotranspiration = A+B+C+D

Where,

A = Unit consumptive use for each cropxits area

B = Unit consumptive use of native vegetation xits area

C = Water surface evaporationxits area

D = Bare land evaporationxits area

v. Irrigation Method:

In this method, unit consumption is multiplied by some factor. The multiplication values depend upon the type of crops in certain area. This method requires an Engineer judgment as these factors are to be investigated by the Engineers of certain area.

vi. Inflow Outflow Method:

In this method annual consumptive use is found for large areas. If U is the valley consumptive use its value is given by,

$$U = (I+P) + (G_s - G_e) - R$$

Where,

U = Valley consumptive use (in acre feet or hectare meter)

I = Total inflow during a year

P = Yearly precipitation on valley floor

Gs = Ground Storage at the beginning of the year

Ge = Ground Storage at the end of the year

R = Yearly Outflow

2. Empirical Methods:

Empirical equations are given for the estimation of water requirement. These are,

a) Blaney-Criddle method
b) Lowry Johnson Method

c) Penman Equation

d) Hargreave's Method

a. Blaney-Criddle method:

- Blaney and Criddle (1950) observed that the amount of water consumptively used by crops during their growing seasons was closely correlated with mean monthly temperatures and daylight hours and the length of the growing seasons.
- The correlation coefficients are then applied to determine the ET for other areas where only climate data are available.
- Blaney-Criddle formula is one of the best known procedures for estimating Potential Evapotranspiration (PET) and is widely used.
- The popularity of the procedure is due to its simplicity and its use of readily available data.
- It requires the use of only two factors, namely, temperature which is readily available from the weather stations and information on daylight hours which is a factor based purely on the latitude of the place.
- Blaney-Criddle equation expresses the consumptive use in terms of temperature and day time hours.

If CU is monthly consumptive use, its value is given by Cu= K.f.(inches)

Where, k = crop factor to be determined for each crop; its value depends upon Certain environmental conditions

f= monthly consumptive use factor

= t×(p/100)

t = mean temperature in ^{o}F.

p = percentage of day time hours of the year, occurring during the period.

If Expressed in metric units, the above formula becomes:

$$C_u = k.\frac{p}{40}[1.8t + 32] = k.f$$

Where,

t = temperature in ^{o}C

Cu= monthly consumptive use in cm

b. Lowry Johnson Method:

The equation for this method is,

U = 0.0015 H + 0.9 (Over specified)

U = Consumptive Use

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

c. Penman Equation:

Penman(1948) proposed an equation for evaporation from open water surface, based on a combination of energy balance and sink strength which is given below with changes in certain symbols in view of the recent trends.

According to this method,

$$U = ET = AH + 0.27\ EaA - 0.27$$

ET = Evapotranspiration or consumptive use in mm Ea = Evaporation (mm/day)

H = Daily head budget at surface (mm/day)

H is a function of radiation, sunshine hours, wind speed, vapour pressure and other climatic factors.

A = Slope of saturated vapour pressure curve of air at absolute temperature in °F

d. Hargreave's Method:

- It is a very simple method.
- The pan is circular with a diameter of 1.21 m and depth of 255 mm which gives it a volume of about 0.3 m3.
- The basin is put on a 150 mm high wooden frame due to air circulation around the basin. The water level is kept about 50 mm below the rim, due to allowance of percolation and the need of water.
- The water level is measured every day, either you measure the difference between the present and the origin water level or if you have chosen to obtain the water level in the pan, you measure the amount of water you have put into the pan.

According to this method,

$$Cu = KEp$$

Where,

Cu = Consumptive Use coefficient (varies from crop to crop)

Ep = Evapotranspiration

K = Coefficient

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UNIT-2

IRRIGATION METHODS

Tank irrigation – Well irrigation – Irrigation methods: Surface and Sub-Surface and Micro Irrigation – design of drip and sprinkler irrigation – ridge and furrow irrigation-Irrigation scheduling – Water distribution system- Irrigation efficiencies.

Tank irrigation

- A tank is a reservoir for irrigation, a small lake or pool made by damming the valley of a stream to retain the monsoon rain for later use.
- It accounts for approximately 3% of the net irrigated area in India.
- Tank Irrigation is popular in the peninsular plateau area where Andhra Pradesh and Tamil Nadu are the leading states.
- Andhra Pradesh has the largest area (29%) of tank irrigation in India followed by Tamil nadu (23%).
- Tanks are known as Ery in Tamil. The temple tanks of Tamil Nadu are known as Kulam

Kinds of Tanks

- The tanks are of two kinds viz., System Tanks and Non-System Tanks.
- The canal fed tanks are known as System Tanks, which were exclusively under the management of the Public Works Department.
- The System Tanks are fed with water from rivers and run off through diversion weirs, feeder channels and surface flow.
- System Tanks are the minority of tanks that are supplied from major storage canal irrigation systems or from perennial rivers.
- The rainfed tanks are known as Non-System Tanks.
- NonSystem Tanks which command area below 40 hectares are coming under the control of Panchayat Unions.
- These Non-System Tanks have a small storage capacity.

It is practised mainly in the peninsular region due to the following reasons:

1. The undulating relief and hard rocks make it difficult to dig canals and wells
2. There is little percolation of water due to hard rock structure and ground water is not available in large quantities.
3. Most of the rivers are seasonal; there are many streams which become torrential during the rainy season – so the only way to use this water is to impound it by constructing bunds and building tanks. Also, it is easy to collect rainwater in natural or artificial pits because of impermeable rocks.
4. Scattered nature of agricultural fields

Merits

- Most of the tanks are natural and do not involve cost for their construction
- Independent source for an individual farmer or a small group of farmers
- longer life span
- can be used for fishing also

Demerits

- Depends on rain and these tanks may dry up during the dry season
- Silting of their beds
- Require large areas
- Evaporation losses
- Sometimes there might be a need to lift the water to take it to the field

Wells (and Tube Wells)

- A well is a hole dug in the ground to obtain the subsoil water. An ordinary well is about 3-5 metres deep but deeper wells up to 15 metres are also dug.
- This method of irrigation has been used in India from time immemorial. Various methods are used to lift the ground water from the well.
- Some of the widely used methods are the persian wheel, reht, charas or mot, and dhinghly (lever) etc.
- A tube well is a deeper well (generally over 15 metres deep) from which water is lifted with the help of a pumping set operated by an electric motor or a diesel engine.

Well Irrigation

- Well irrigation is gradually giving way to energized tube wells. But there are many wells still in use where electricity is not available or the farmers are too poor to afford diesel oil.
- This method of irrigation is popular in those areas where sufficient sweet ground water is available.
- It is particularly suitable in areas with permeable rock structure which allows accumulation of ground water through percolation.
- Therefore wells are seen more in areas with alluvial soil, regur soil, etc. and less seen in rocky terrain or mountainous regions.
- These areas include a large part of the great northern plains, the deltaic regions of the Mahanadi, the Godavari, the Krishna and the Cauvery, parts of the Narmada and the Tapi

valleys and the weathered layers of the Deccan trap and crystalline rocks and the sedimentary zones of the peninsula

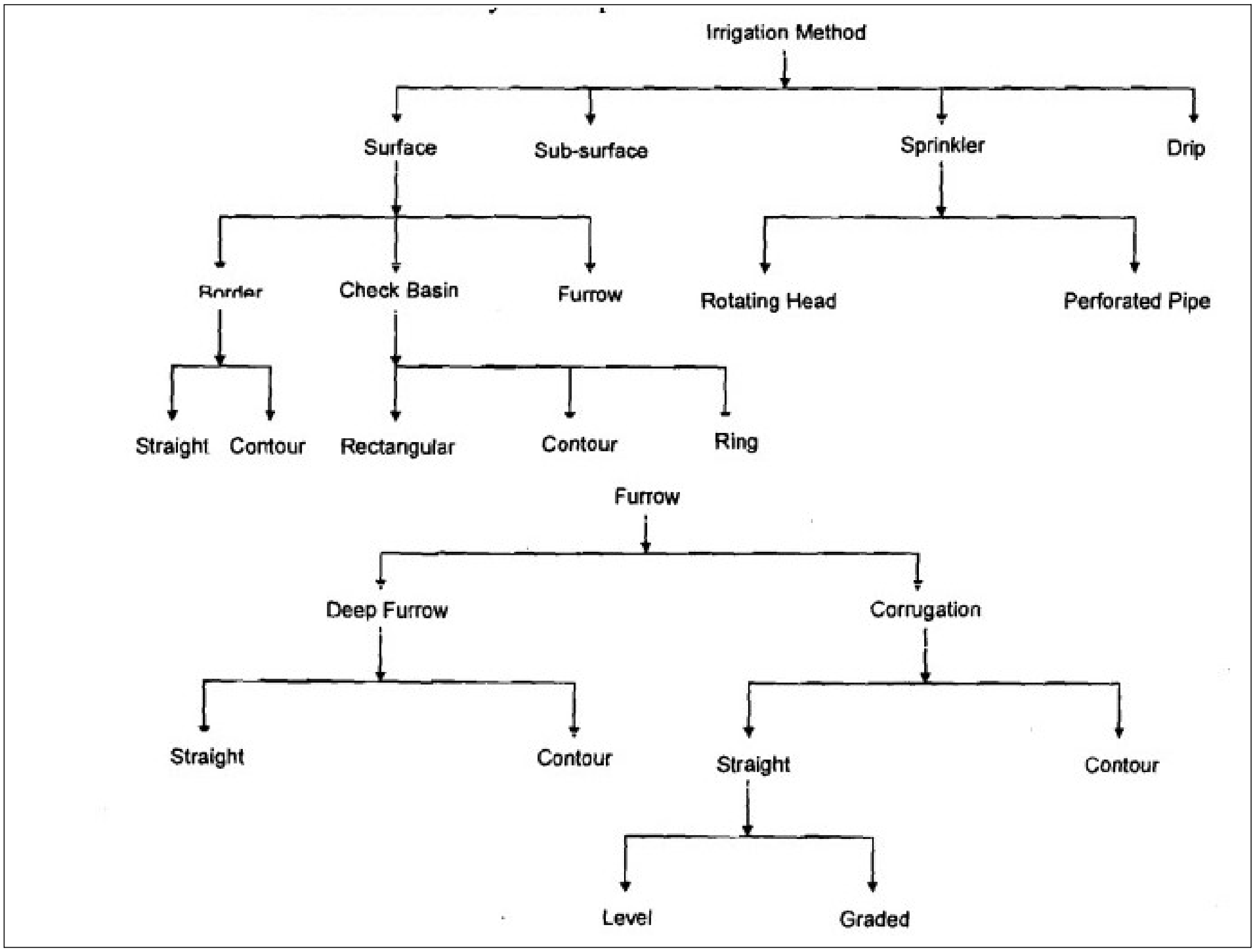
- However, the greater part of peninsular India is not suitable for well irrigation due to rocky structure, uneven surface and lack of underground water.
- Large dry tracts of Rajasthan, the adjoining parts of Punjab, Haryana and Gujarat and some parts of Up have brackish ground water which is not fit for irrigation and human consumption and hence unsuitable for well irrigation
- At present irrigation from wells and tubewells accounts for more than 60% of the net irrigated area in the country.
- UP has the largest area under well irrigation which accounts for 28% of the well irrigated area of the country. U.P., Rajasthan, Punjab, Madhya Pradesh, Gujarat, Bihar and Andhra Pradesh account for about three-fourths of the total well-irrigated area.

Merits of well irrigation

- Simplest
- Cheapest
- Well is an independent source of irrigation and can be used as and when the necessity arises. Canal irrigation, on the other hand, is controlled by other agencies and cannot be used at will.
- Some ground water salts are useful for crops
- Does not lead to salinization and flooding problems
- There is a limit to the extent of canal irrigation beyond the tail end of the canal while a well can be dug at any convenient place.

Demerits

- Only limited area can be irrigated.
- Normally, a well can irrigate 1 to 8 hectares of land.
- Not suitable for dry regions
- Overuse may lead to lowering of water table



SURFACE IRRIGATION:

- Surface irrigation is defined as the group of application techniques where water is applied and distributed over the soil surface by gravity.
- It is by far the most common form of irrigation throughout the world and has been practiced in many areas virtually unchanged for thousands of years.

Surface irrigation:

There are four variations under this method viz.

1. Flooding,
2. Bed or border method (Saras and flat beds
3. Basin method (ring and basin) and
4. Furrow method (rides and furrows, broad ridges or raised beds)

Flooding:

- It consist of opening a water channel in a plot or field so that water can flow freely in all directions and cover the surface of the land in a continuous sheet.

- It is the most inefficient method of irrigation as only about 20 percent of the water is actually used by plants. The rest being lost as a runoff, seepage and evaporation.
- Water distribution is very uneven and crop growth is not uniform. It is suitable for uneven land where the cost of leveling is high and where a cheap and abundant supply of water is available.
- It is unsuitable for crops that are sensitive to water logging the method suitable where broadcast crops, particularly pastures, alfalfa, peas and small grains are produced.

Adaptations:

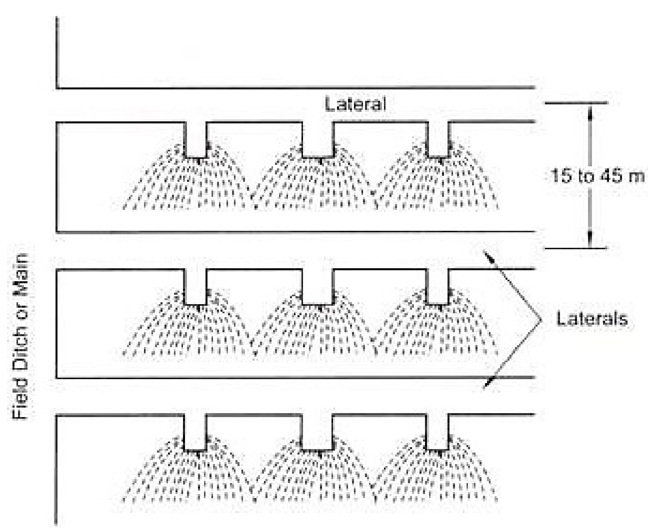
1. An abundant supply of water
2. Close growing crops
3. Soils that do not erode easily
4. Soils that is permeable
5. Irregular topography
6. Areas where water is cheap.

Advantages:

1. Can be used on shallow soils
2. Can be employed where expense of leveling is great
3. Installation and operation costs are low
4. System is not damaged by livestock and does not interfere with use of farm implements.

Disadvantages:

1. Excessive loss of water by run of and deep percolation
2. Excessive soil erosion on step land.
3. Fertilizer and FYM are eroded from the soil.



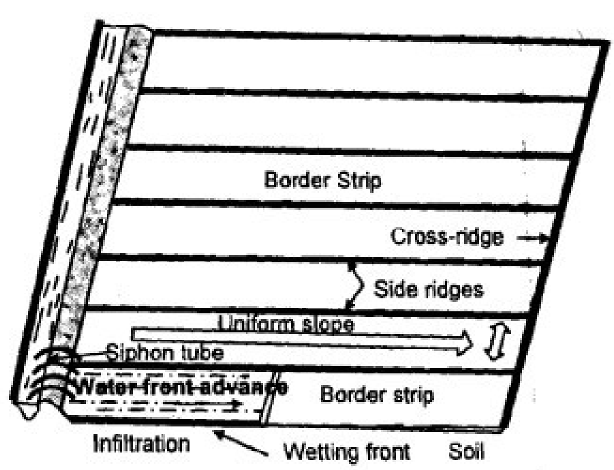
Bed or border method (Sara and Flat beds or check basin):

- In this method the field is leveled and divided into small beds surrounded by bunds of 15 to 30 cm high. Small irrigation channels are provided between two adjacent rows of beds.
- The length of the bed varies from 30 meters for loamy soils to 90 meters for clayey soils.
- The width is so adjusted as to permit the water to flow evenly and wet the land uniformly.
- For high value crops, the beds may be still smaller especially where water is costly and not very abundant.
- This method is adaptable to most soil textures except sandy soils and is suitable for high value crops. It requires leveled land.
- It is more efficient in the use of water and ensures its uniform application. It is suitable for crops plant in lines or sown by broadcast. Through the initial cost is high requires less labour and low maintenance cost.
- This may also be called a sort of sara method followed locally in Maharashtra but the saras to be formed in this method are much longer than broader.

Types of Border Irrigation

Two types of borders are formed :

- Straight Border
- These border are formed along the general slope of the field. These are preferred when fields can be levelled or be given a gentle slope economically.



Contour Border

- These are formed across the general slope of the field and are preferred when land slope exceeds the safe limits.

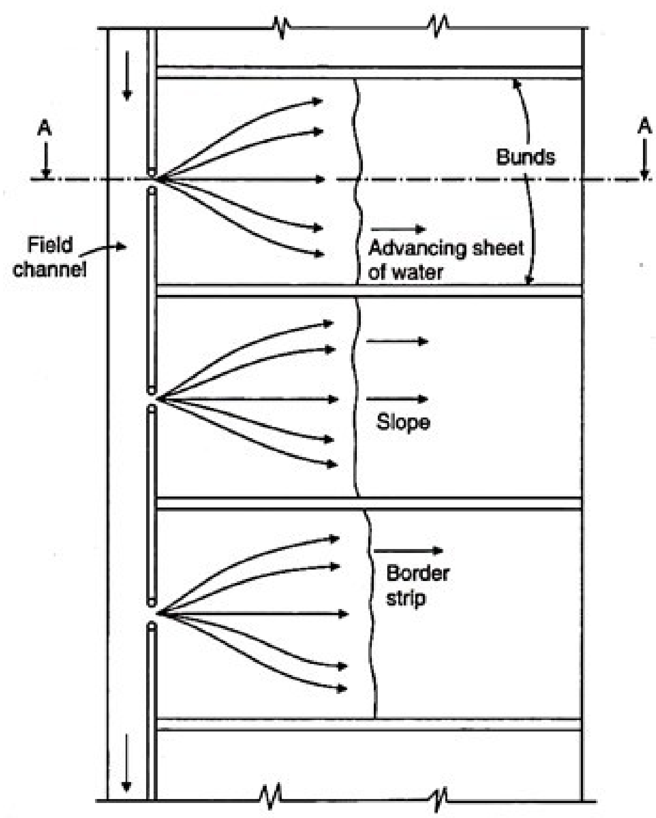
- As fields are undulating and require a lot of earth work to level, economical levelling is not possible. Design criteria for both are not different.

Adaptations:

1. A large supply of water
2. Most soil textures including sandy Loam, loams and clays
3. Soil at least 90 cm deep
4. Suitable for close growing crops.

Advantages:

1. Fairly large supply of water is needed.
2. Land must be leveled
3. Suited only to soils that do not readily disperse.
4. Drainage must be provided



Basin irrigation:

- This method is suitable for orchids and other high value crops where the size of the plot to be irrigated is very small.

- The basin may be square, rectangular or circular shape. A variation in this method viz. ring and basin is commonly used for irrigating fruit trees.
- A small bund of 15 to 22 cm high is formed around the stump of the tree at a distance of about 30 to 60 cm to keep soil dry.
- The height of the outer bund varies depending upon the depth of water proposed to retain. Basin irrigation also requires leveled land and not suitable for all types of soil. It is also efficient in the use of water but its initial cost is high.
- There are many variations in its use, but all involve dividing the field into smaller unit areas so that each has a nearly level surface. Bunds or ridges are constructed around the areas forming basins within which the irrigation water can be controlled. Check basin types may be rectangular, contour and ring basin.

Types of Check Basins

Based on Size and Shape

The size of check basins may vary from one meters square, used for growing vegetables and other intensive cultivation, to as large as one or two hectares or more, used for growing rice under wet land conditions. While the following points need to be considered :

Rectangular

The basins are rectangular in shape when the land can be graded economically into nearly level fields.

Contour

- The ridges follow the contours of the land surface and the contour ridges are connected by cross ridges at intervals when there is rolling topography.
- The vertical interval between contour ridges usually varies fkom 6 to 12 cm in case of upland irrigated crops like wheat and 15 to 30 cm in case of low land irrigated crops like rice.

Adaptations:

1. Most soil texture
2. High value crops
3. Smooth topography.
4. High water value/ha

Advantages:

1. Varying supply of water

2. No water loss by run off

3. Rapid irrigation possible

4. No loss of fertilizers and organic manures

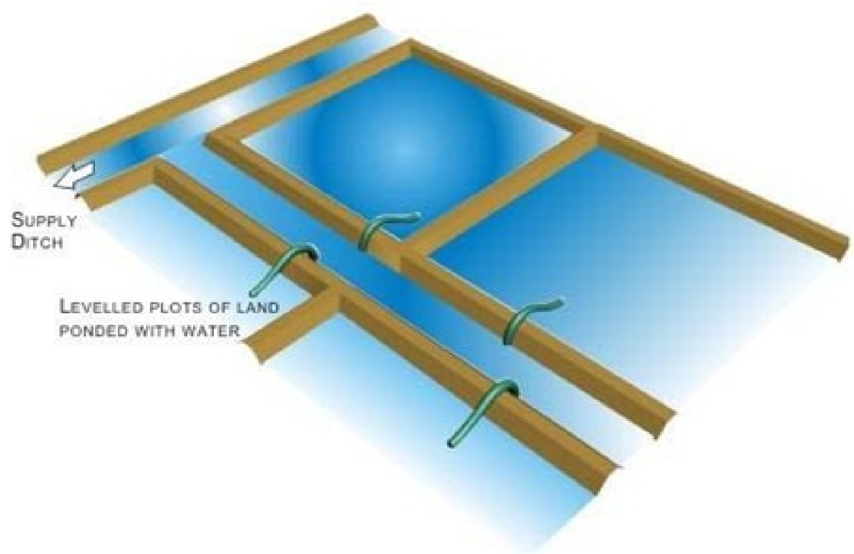
5. Satisfactory

Disadvantages:

1. If land is not leveled initial cost may be high

2. Suitable mainly for orchids, rice, jute, etc.

3. Except rice, not suitable for soils that disperse easily and readily from a crust.



Furrow Method

- In this method, irrigation water is useful for row crops. Narrow channels are dug at regular intervals. Water from the main supply is allowed to enter these small channels or furrows.

- Water from the furrows infiltrates into soil and spread laterally to saturate the root zone of the crops.

- It is suitable for row crops like potatoes, sugarcane, tobacco, maize, groundnut, cotton, jowar, etc.

- Row crops such as potatoes, cotton, sugarcane, vegetable etc. can be irrigated by furrow method. Water is allowed to flow in furrow opened in crop rows.

- It is suitable for sloppy lands where the furrows are made along contours. The length of furrow is determined mostly by soil permeability.

- It varies from 3 to 6 meters. In sandy and clay loams, the length is shorter than in clay and clay loams. Water does not come in contact with the plant stems.
- There is a great economy in use of water. Some times, even in furrow irrigation the field is divided into beds having alternate rides and furrows. On slopes of 1 to 3 percent, furrow irrigation with straight furrows is quite successful.
- But on steeper slopes contour furrows, not only check erosion but ensure uniform water penetration.

Irrigation furrows may be classified into two general types based on their alignment. They are :

(a) straight furrows, and

(b) contour furrows.

Straight Furrows

- They are best suited to sites where the land slope does not exceed 0.75 per cent. In areas of intense rainfall, however, the furrow grade should not exceed 0.5 per cent so as to minimise the erosion hazard.
- The range in furrow slopes for efficient irrigation in different soil types are the same as those recommended for borders.

Contour Furrows

- Contour furrows carry water across a slopping field rather than the slope. Contour furrows are curved to fitthe topography of the land.
- Contour furrow method can be successfully used in nearly all irrigable soils. The limitations of straight furrow are overcome by contouring to include slopping lands. Light soils can be irrigated successfully across slopes up to 5 per cent.

Adaptations:

1. Medium and fine textured soils.
2. Variable water supply
3. Farms with only small amount of equipment.

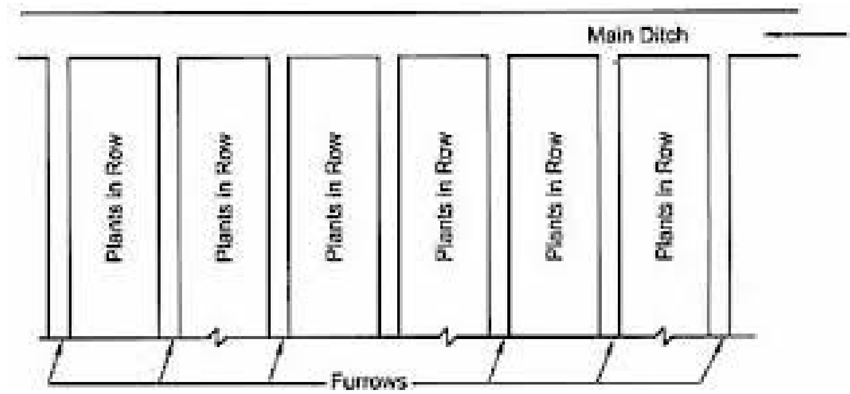
Advantages:

1. High water efficiency
2. Can be used in any row crop
3. Relatively easy in stall
4. Not expensive to maintain

5. Adapted to most soils.

Disadvantages:

1. Requirement of skilled labour is more
2. A hazard to operation of machinery
3. Drainage must be provided.



Contour farming

- Contour farming involves ploughing, planting and weeding along the contour, i.e, across the slope rather than up and down.
- Contour lines are lines that run across a (hill) slope such that the line stays at the same height and does not run uphill or downhill.
- As contour lines travel across a hillside, they will be close together on the steeper parts of the hill and further apart on the gentle parts of the slope.
- Experiments show that contour farming alone can reduce soil erosion by as much as 50% on moderate slopes.
- However, for slopes steeper than 10%, other measures should be combined with contour farming to enhance its effectiveness.

Benefits :

1. Contouring can reduce soil erosion by as much as 50% from up and down hill farming
2. By reducing sediment and run off and increasing water infiltration
3. Contouring promotes better water quality
4. It gives 10-15% additional yield.

Criteria for Surface Irrigation Method Selection

- The deciding factors for the suitability of any surface irrigation method are natural conditions (slope, soil type), type of crop, required depth of application, level of technology, previous experiences with irrigation, required labour input.
- Moreover the irrigation system for a field must be compatible with the existing farming operations, such as land preparation, cultivation, and harvesting practices.
- The following outline lists a number of factors of the environment which will have a bearing on the evaluation of irrigation system alternates and the selection of a particular system.
- Not all points will be equally significant in each case, but the outline can serve as a useful checklist to prevent overlooking important factors.

Physical Factors

- Crops and cultural practices are of prime importance while selecting an irrigation system.
- Hence, proper knowledge of agronomic practices and irrigation intervals is necessary for proper use of irrigation water and to increase water use efficiency.
- The following physical factors need to be given due consideration.

Crop Parameters

- Tolerance of the crop to soil salinity during development and maturation.
- Magnitude and temporal distribution of water necessary for maximum production.
- Economic value of crop.

Soils Parameters

- Texture and structure;infiltration rate and erosion potential;salinity and internal drainage, bearing strength.
- Sandy soils have a low water storage capacity and a high infiltration rate. Under these circumstances, sprinkler or drip irrigation are more suitable than surface irrigation. Clay soils with low infiltration rates are ideally suited to surface irrigation.
- High intake characteristicrequire higher flow rate to achieve the same uniformity and efficiency.
- Crusting of soil and its effects on infiltration
- Reclamation and salt leaching- basin irrigation
- Spatial variability

Field Topography

- Uniform, mild slopes facilitate surface irrigation.

- Location and relative elevation of water source – water diversion, pumping
- Acreage in each field
- Location of roads, natural gas lines, electricity lines, water lines and other obstructions.
- Shape of field – non rectangular shapes are more difficult to design for
- Field slope – steepness & regularity
- Furrow&borders 2-6% maximum

Climate and Weather Conditions

- Under very windy conditions, drip or surface irrigation methods are preferred.
- Scalding (the disruption of oxygen-carbon dioxide exchange between the atmosphere and the root)& the effect of water temperature on the crop at different stages of growth -risk in basin irrigation.
- Irrigation with cold water early in the spring can delay growth, whereas in the hot periods of the summer, it can cool the environment— both of which can be beneficial or detrimental in somecases.

Water Supply

The following parameters are important:

1. Source and delivery schedule
2. Water quantity available and its reliability
3. Water quality
4. Water table in case of ground water source.
5. Availability and Reliability of Electricity
6. Availability and reliability of energy for pumping of water is of muchimportance.

Economic Considerations

The following points need to be considered while selecting irrigation alternatives.

1. Capital investment required and recurring cost.
2. Credit availability and interest rate.
3. Life of irrigation system, efficiency and cost economics.

Social Considerations

- The education and skill of common farmers and labours available for handling the irrigation system
- Social understanding of handling of cooperative activities and sharing of water resources
- Legal and political considerations, local cooperation and support, availability and skill of labour and level of automatic control

Suitability and Limitations of Surface Irrigation Methods

- Some form of surface irrigation is adaptable to almost any vegetable crop. Basin and border strip irrigation have been successfully used on a wide variety of crops.
- Furrow irrigation is less well adapted to field crops if cultural practices require travel across the furrows. However, it is widely used in vegetables like potato.
- Basin and border strip irrigations flood the soil surface, and will cause some soils to form a crust, which may inhibit the sprouting of seeds.
- Surface irrigation systems perform better when soils are uniform, since the soil controls the intake of water. For basin irrigation, basin size should be appropriate for soil texture and infiltration rate.
- Basin lengths should be limited to 100 m on very coarse textured soils, but may reach 400 m on other soils. Furrow irrigation is possible with all types of soils, but extremely high or low intake rate soils require excessive labor or capital cost adjustments that are seldom economical.
- A major cost in surface irrigation is that of land grading or leveling. The cost is directly related to the volume of earth that must be moved, the area to be finished, and the length and size of farm canals.

MICRO IRRIGATION METHOD

- Micro irrigation methods are precision irrigation methods of irrigation with very high irrigation water efficiency.
- In many parts of the country there is decline of irrigation water and conventional methods are having low water use efficiency.
- To surmount the problem, micro irrigation methods h8s recently been introduced in Indian agriculture.
- These methods save a substantial amount of water and helps increasing crop productivity particularly valuable cash crops like vegetables.

- The research results have confirmed a substantial saving of water ranging between 40 to 80% and there are reports of two times yield increase for different crops crops by using micro irrigation.

Two main micro irrigation systems are :

Advantages of Micro Irrigation

(a) Water saving, possibility of using saline water.

(b) Efficient and economic use of fertilizers.

(c) Easy installation, flexibility in operation.

(d) Suitable to all types of land terrain also suitable to waste lands.

(e) Enhanced plant growth and yield and uniform and better quality of produce.

(f) Less weed growth.

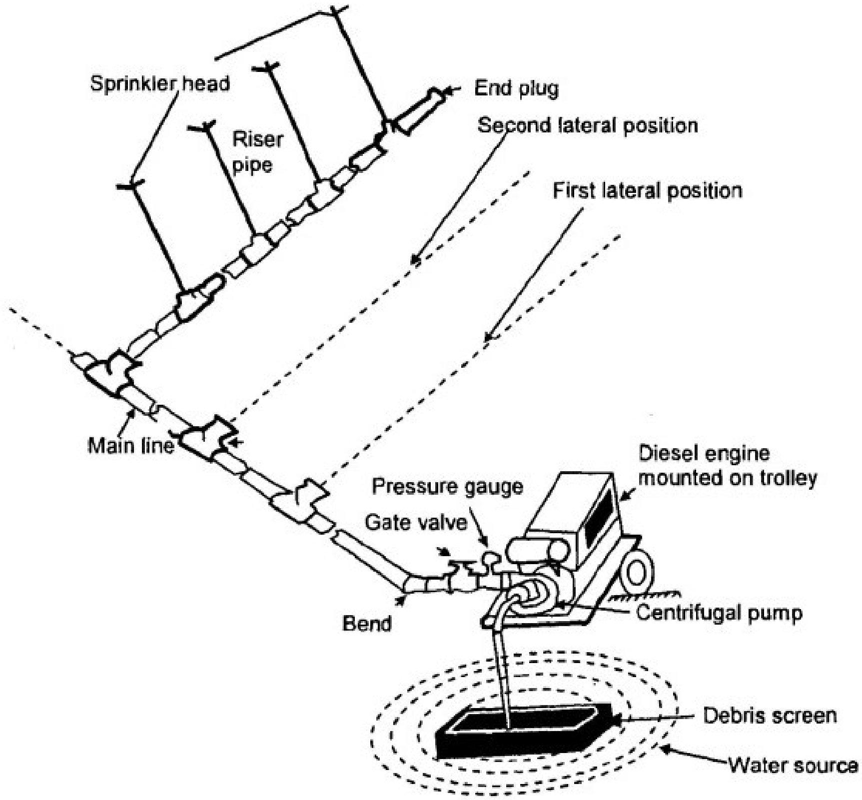
(g) Labour saving.

(h) No soil erosion, saves land as no bunds, etc. are required.

(i) Minimum diseases and pest infestation.

SPRINKLER IRRIGATION

- In sprinkler irrigation, water is delivered through a pressurized pipe network to sprinklers nozzles or jets which spray the water into the air.
- To fall to the soil in an artificial "rain". The basic components of any sprinkler systems are : a water source. a pump to pressurize the water.
- A pipe network to distribute the water throughout the field. sprinklers to spray the water over the ground, and valves to control the flow of water.
- The sprinklers when properly spaced give a relatively uniform application of water over the irrigated area.



Components of

Sprinkler irrigation System

- Sprinkler systems are usually fthere are some exceptions) designed to apply water at a lower rate than the soil infiltration rate so that the amount of water infiltrated at any point depends upon the application rate and time of application but not the soil infiltration rate.

General Classification of Sprinkler Systems

Sprinkler systems are classified into the following two major types on the basis of the arrangement for spraying irrigation water.

(a) Rotating head or revolving sprinkler system.

(b) Perforated pipe system.

Components of Sprinkler Irrigation System

Sprinkler system usually consists of the following components :

(a) A pump unit

(b) Tubings-main/sub-mains and laterals

(c) Couplers

(d) Sprinker head

(e) Other accessories such as valves, bends, plugs and risers.

Suitability and Limitations

With regards to crops, soils, and topography nearly all crops can be irrigated with some type of sprinkler system though the characteristics of the crop especially the height, must be considered in system selection.

Sprinklers are sometimes used to germinate seed and establish ground cover for crops like lettuce alfalfa and sod.

The light frequent applications that are desirable for this purpose are easily achieved with some sprinkler systems.

Sprinklers are applicable to soils that are too shallow to permit surface shaping or too variable for efficient surface irrigation.

In general, sprinklers can be used on any topography that can be formed. Land leveling is not normally required.

With regards to labour and energy considerations, it has been observed that labour requirements vary depending on the degree of automation and mechanization of the equipment used.

Hand-move systems require the least degree of skill, but the greatest amount of labor.

Advantages of Sprinkler Irrigation

The followings are the advantages of sprinkler irrigation :

(a) Elimination of the channels for conveyance, therefore no conveyance loss.

(b) Suitable to all types of soil except heavy clay, suitable for irrigating crops where the plant population per unit area is very high. It is most suitable for oil seeds and other cereal and vegetable crops.

(c) Water saving, closer control of water application convenient for giving light and frequent irrigation and higher water application efficiency.

(d) Increase in yield.

(e) Mobility of system.

(f) May also be used for undulating area, saves land as no bunds etc. are required, areas located at a higher elevation than the source can be irrigated.

(g) Influences greater conducive micro-climate.

(h) Possibility of using soluble fertilizers and chemicals.

(i) Less problem of clogging of sprinkler nozzles due to sediment laden water

Capacity of Sprinkler System

The capacity of the sprinkler system may be calculated by the formula :

$$Q = 2780 \times \frac{A \times d}{F \times H \times E}$$

Where,

Q = Discharge capacity of the pump, liter/second,

A = Area to be irrigated, hectares,

d = Net depth of water application, cm,

F = Number of days allowed for the completion of one irrigation,

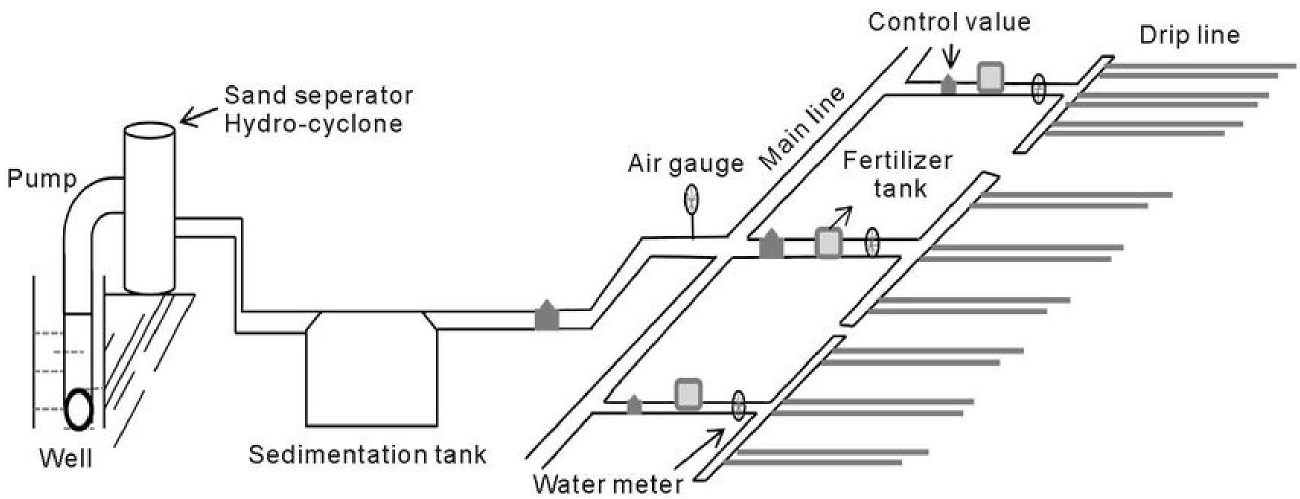
H = Number of actual operation hours per day, and

E = Water Application Efficiency in %

DRIP IRFUGATION

- Drip irrigation, also known as trickle irrigation or microirrigation is an irrigation method which minimizes the use of water and fertilizer by allowing water to drip slowly to the roots of plants, either onto the soil surface or directly onto the root zone, through a network of valves, pipes, tubing, and emitters.
- It is becoming popular for row crop irrigation. This system is used in place of water scarcity as it minimizes conventional losses such as deep percolation, evaporation and run-off or recycled water is used for irrigation.
- Small diameter plastic pipes fitted with emitters or drippers at selected spacing to deliver the required quantity of water are used. Drip irrigation may also use devices called micro-spray heads, which spray water in a small area, instead of dripping emitters.
- Subsurface drip irrigation (SDI) uses permanently or temporarily buried drip per line or drip tape located at or below the plant roots.
- Pump and valves may be manually or automatically operated by a controller Drip irrigation is the slow, frequent application of water to the soil though emitters placed along a water delivery line.
- The term drip irrigation is general, and includes several more specific methods. Drip irrigation applies the water through small emitters to the soil surface, usually at or near the plant to be irrigated.

- Subsurface irrigation is the application of water below the soil surface. Emitter discharge rates for drip and subsurface irrigation are generally less than 12 liters per hour.



Components of Drip Irrigation System (Listed in Order from Water Source)

(a) Pump or pressurised water source.

(b) Water Filter(s) - Filtration Systems : Sand Separator, Cyclone, Screen Filter, Media Filters.

(c) Fertigation Systems (Venturi injector).

(d) Backwash Controller.

(e) Main Line (larger diameter Pipe and Pipe Fittings).

(f) Hand-operated, electronic, or hydraulic Contvl Valves and Safety Valves.

(g) Smaller diameter polytube (often referred to as "laterals").

(h) Poly fittings and Accessories (to make connections).

(i) Emitting Devices at plants (Example : Emitter or Drippers, micro spray heads, inline drippers, trickle rings).

Suitabi1ity:and Limitation

(a) From stand point of crops, soil, and topography, drip irrigation is best suited for tree, vine, and row crops. A lot of research work has been conducted to establish the suitability of drip irrigation for different vegetable crops. Drip irrigation has been found suitable both for field vegetable crops and also under covered cultivation practices.

(b) With respect to water quantity and quality, drip irrigation uses a slower rate of water application over a longer period of time than other irrigation methods. The most economical design would have water flowing into the farm area throughout most of the day, every day, during peak use periods. If water is not available on a continuous basis, on-farm water storage may be necessary.

(c) Though a form of pressurized irrigation, drip is a low pressure, low flow rate method. These conditions require small flow channel openings in the emission devices, which are prone to plugging.

(d) High efficiencies are USP of drip irrigation system. Properly designed and maintained drip systems are capable of high efficiencies. Design efficiencies should be on the order of 90 to 95%.

(e) Labour and energy considerations are very important consideration in drip irrigation system. Due to their low flow characteristics, drip irrigation systems usually have few sub-units, and are designed for long irrigation times.

(f) Drip irrigation systems generally use less energy than other forms of pressurized irrigation systems. The emission devices usually operate at pressures ranging from 5 to 25 PSI. Additional pressure is required to compensate for pressure losses through the control head (filters and control valves) and the pipe network.

(g) Economic factors need special attention in case drip irrigation system as initial cost and operational cost is reasonably high. Drip systems costs can vary greatly. Depending on crop (plant. and therefore. emitter and hose spacings) and type of hose employed (permanent or "disposable" thin-walled tubing).

Advantages

The advantages of drip irrigation are :

1. Minimised fertilizerlnutrient loss due to localized application and reduced leaching, allows safe use of recycled water.
2. High water distribution efficiency. Moisture within the root zone can be maintained at field capacity.
3. Leveling of the field not necessary. Soil type plays less important role in frequency of irrigation, minimised soil erosion.
4. Highly uniform distribution of water, i.e. controlled by output of each nozzle.
5. Lower labour cost.
6. Early maturity and good harvest.
7. Foliage remains dry thus reducing the risk of disease.

Performance Indicator	Conventional Irrigation Methods	Drip Irrigation
Water saving	Waste lot of water. Losses occur due to percolation, runoff and evaporation	40-70% of water can be saved over conventional irrigation methods. Runoff and deep percolation losses are nil or

		negligible.
Water use efficiency	30-50%, because losses are very high	80-95%
Saving in labour	Labour engaged per irrigation is higher than drip	Labour required only for operation and periodic maintenance of the system
Weed infestation	Weed infestation is very high	Less wetting of soil, weed infestation is very less or almost nil.
Use of saline water	Concentration of salts increases and adversely affects the plant growth. Saline water cannot be used for irrigation	Frequent irrigation keeps the salt concentration within root zone below harmful level
Diseases and pest problems	High	Relatively less because of less atmospheric humidity
Suitability in different soil Type	Deep percolation is more in light soil and with limited soil depths. Runoff loss is more in heavy soils	Suitable for all soil types as flow rate can be controlled
Water control	Inadequate	Very precise and easy
Efficiency of fertilizer use	Efficiency is low because of heavy losses due to leaching and runoff	Very high due to reduced loss of nutrients through leaching and runoff water
Soil erosion	Soil erosion is high because of large stream sizes used for irrigation.	Partial wetting of soil surface and slow application rates eliminate any possibility of soil erosion
Increase in crop yield	Non-uniformity in available moisture reducing the crop yield	Frequent watering eliminates moisture stress and yield can be increased up to 15- 150% as compared to conventional methods of irrigation.

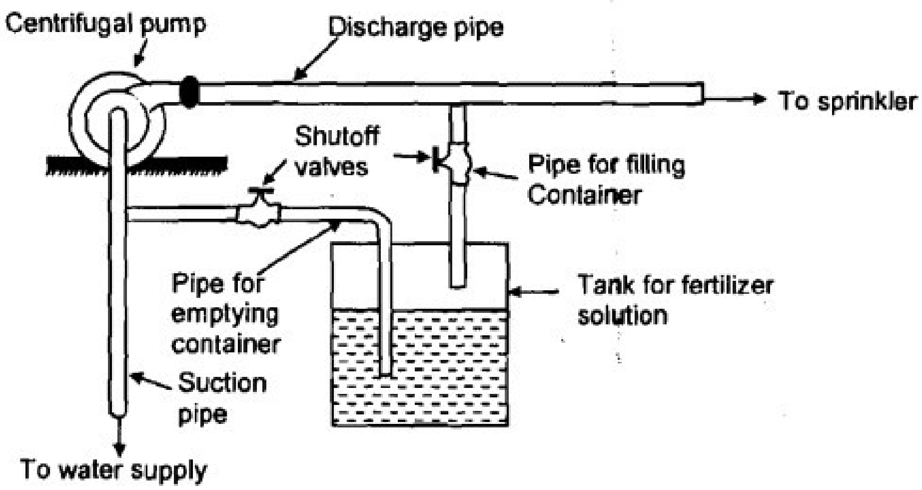
Extent of Water Saving and Increase in Yield with Drip Irrigation Systems

Crops	Water Saving (%)	Increase in Yield (%)
Sugarcane	50	99
Tomato	42	60
Watermelon	66	19
Cucumber	56	45
Chili	68	28
Cauliflower	68	70
Okra	37	33
Ground nut	40	152
Mulberry	22	23
Banana	45	52
Grapes	48	23
Sweet lime	61	50
Pomegranate	45	45

Source : INCID 1994 Drip irrigation in India, New Delhi.

FERTIGATION

- Fertigation is the process of application of water soluble solid fertilizer or liquid fertilizers through drip irrigation system.
- Through fertigation nutrients are applied directly into the wetted volume of soil immediately below the emitter where root activity is concentrated.
- Fertigation is practiced only in drip irrigation system. However, fertilizer solution can be added with sprinkler irrigation system also.



Components of Fertigation

The main component of a fertigation is drip irrigation system. The main components are :

(a) Venturi pump (injector)

(b) Fertilizer tank with flow bypass

(c) Pressure bypass tank

(d) Injection pump.

Advantages of Fertigation

1. The fertilizer solution is distributed evenly in the irrigation network with the same uniformity as the irrigation water.
2. The availability of nutrients including micro-nutrients is high, therefore the efficiency is very good.
3. The fertilizer system can also be used for other activities such as incorporating acid to flush the drip system.
4. It eliminates the work of spreading fertilizer. Manual spreading of fertilizer causes soil compaction and may damage the growing crop.

5. Fertilizer placement is exactly to the root zone of plant and can be uniformly applied through drip irrigation system.
6. All types of nutrients can be given simultaneously.
7. Lower doses of fertilizer could be applied daily or weekly (i.e. a large number of split application) to avoid leaching and fixation in soil.
8. Some liquid fertilizers are free of sodium and chloride salts, so these are not harmful to soil.
9. (i) Optimum production in light soil is possible.
10. Spraying with liquid fertilizer is possible.
11. Liquid fertilizers are immediately available to plants.
12. Fertilizer use efficiency can be increased by 25 to 30% over the tradition method of fertilizer application.
13. It decreases labour and energy cost.
14. The quality and quantity of crop production can be improved

Limitations

- The fertigation system also has some limitations. The main one is the danger of poisoning people who drink the irrigation water particularly laborers those work on the farm.
- It is therefore necessary to warn the people in the field about drinking water separately and put up warning signs. The reverse flow of water mixed with fertilizer must be prevented.

Toxicity and Contamination

Care must be taken whenever fertilizer solution is introduced into a water supply system.

Fertilizer Suitability

Slowly water-soluble fertilizer such as super phosphate or calcium ammonium phosphate is not suitable. This method is suitable for liquid fertilizers or those that are readily soluble in water.

Corrosion

The metallic parts of the equipment are highly prone to corrosion. Sensitive parts of the equipment must be made out of corrosion resistant materials and extra care should be taken when filling the tanks.

Keywords

Border Irrigation: It uses land formed into strips which are located across the narrow dimension, but sloping along the long dimensions.

Check Basin Irrigation : In this irrigation system, water is applied to a completely level or dead level area enclosed by dikes or boarders.

Furrow Irrigation : Furrows are sloping channels formed in the soil. Infiltration occurs laterally and vertically through the wetted perimeter of the furrow and plants get water in its root zone.

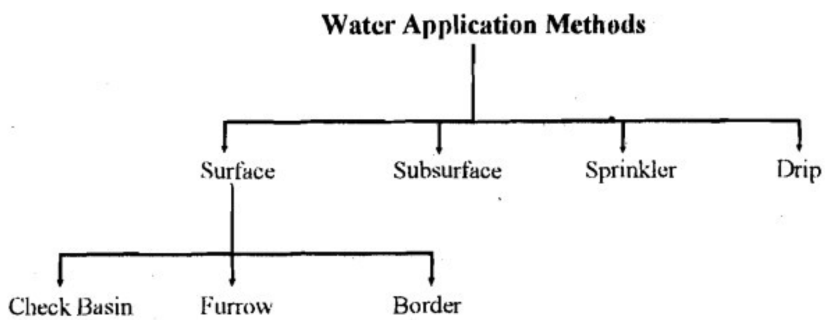
Sprinkler Irrigation : In this system of irrigation, water is delivered through a pressurised pipe network to sprinklers nozzle or jets which spray water into the air.

Drip Irrigation : It minimises the use of water and fertilizer by allowing water to drip slowly to the roots of plants.

Fertigation : It is the process of application of water soluble solid fertilizer or liquid fertilizer through drip irrigation system.

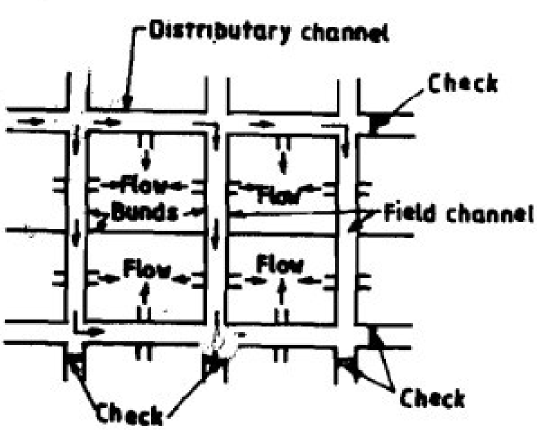
Water distribution system

Irrigation water inay be applied to crops either by flooding the field. by applying water beneath the soil surface, by spraying it under pressure. or by applying it in drops. Selection of the suitable method, from among these methods, depends on topography. soil condition, land preparation, type of crop and its value. available water supply and other factors

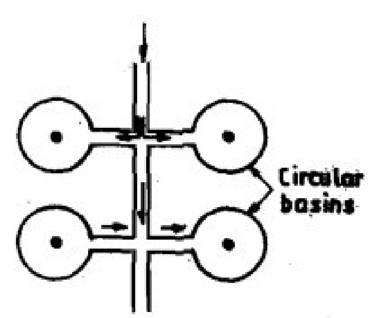


CHECK BASIN IRRIGATION

Check basin irrigation or simply basin irrigation is the simplest available mode of irrigation and commonly practised in India end other countries. The principle underlying this system involves dividing tile field or fanil into smaller unit areas such that each has a nearly level surface.



(a) Check basins

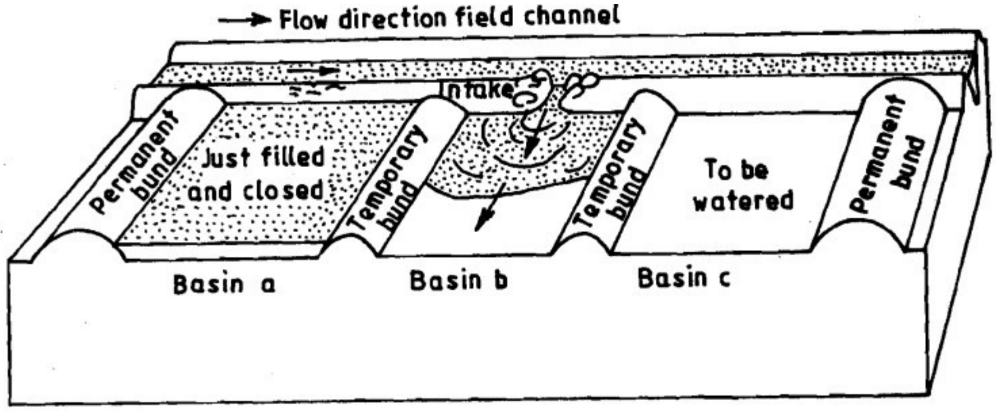


(b) Ring or circular basins

Methods to Apply Irrigation Water to check Basins

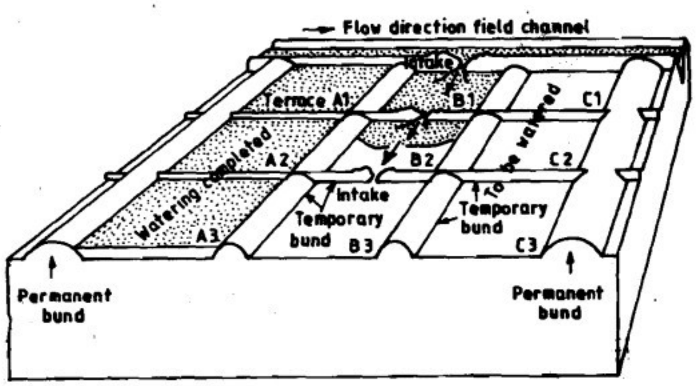
There are two methods to supply imgation water to check basins, namely, direct method, and cascade inetliod.

In the direct method. irrigation water is led dircctly from thc field channel into the basins through siphons. or bund breaks, basin A is irrigated first and then basin B and so on. This method can bc used for most crop types. and is also suitable for nlost type of soil.



Direct Method

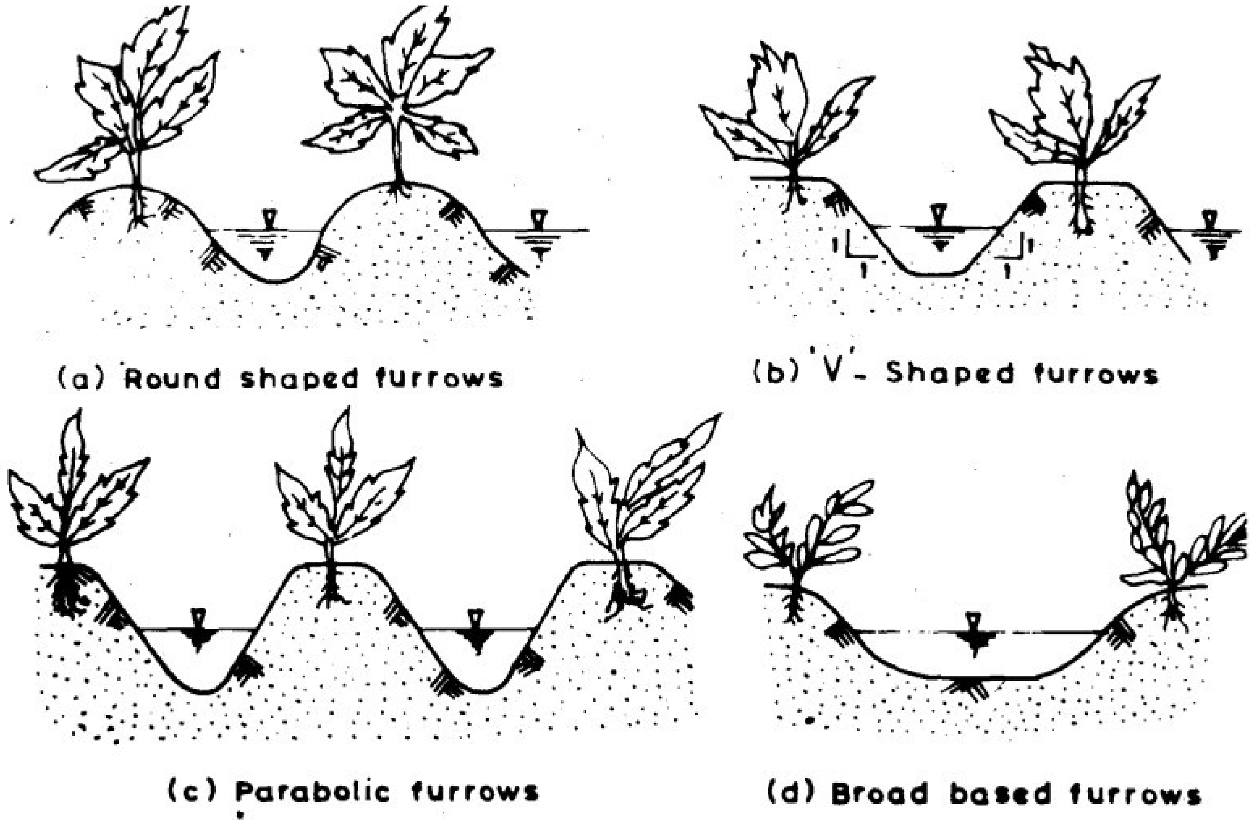
The other method. namely, the cascade method is suitable for sloping land where terraces are used. In this method, the irrigation water is supplied to the highest terrace, and then allowed to flow to a lower terrace and so on. In Figure water is supplied to tlie tcrrace A1 until tlie lowest terrace A3 is fillcd. Tlie supply to A1 is tlien closed and irrigation water is diverted to terrace B1 until B I, B2 and B3 are filled, and so on.



Cascade Method

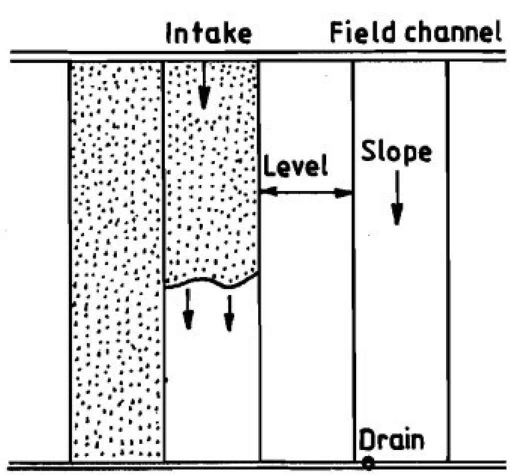
FUIIROW IRRIGATION

Furrows arc small, parallel channels, made lo c a m water for irrigating the crops. The crops are usually grown on thc ridges between the furrows.



BORDER IRRIGATION

Borders are usually long,uniformly gradcd strips of land. separated by earth bunds In contrast to basin irrigation thesc bunds arc not to contain water for ponding but to guide its flow down the field.



CHOICE OF METHOD OF IRRIGATION

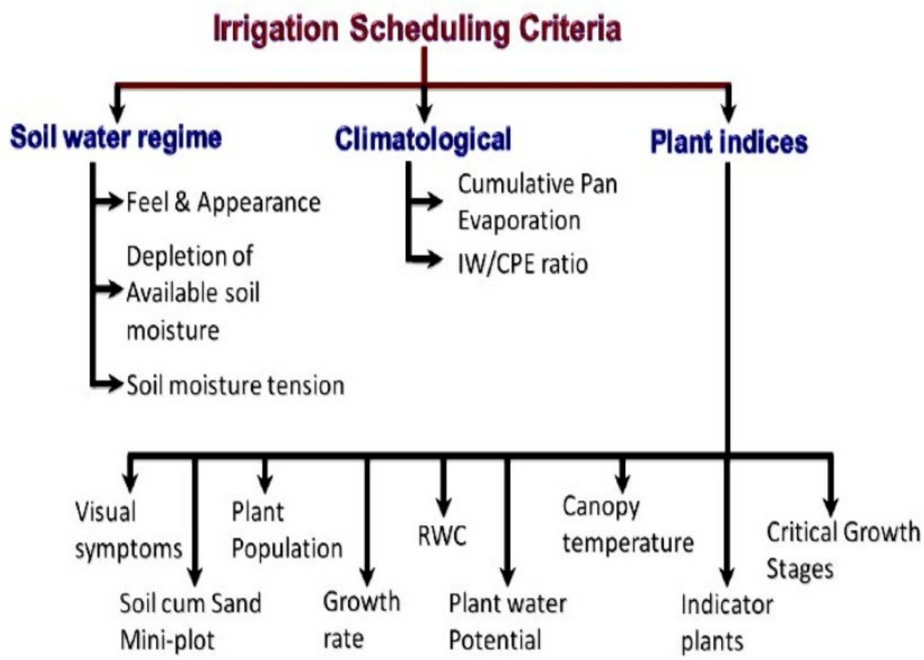
1. Natural conditions (slope & soil type).
2. Type of crop,
3. Level of technology that is available,
4. Previous experience with the practice of irrigation and
5. Required labour inputs.

Irrigation scheduling

Irrigation scheduling is the process used by irrigation system managers to determine the correct frequency and duration of watering.

Advantages of Irrigation Scheduling

1. It enables the farmer to schedule water rotation among the various fields to minimize crop water stress and maximize yields.
2. It reduces the farmer's cost of water and labour
3. It lowers fertilizer costs by holding surface runoff
4. It increases net returns by increasing crop yields and crop quality.
5. It minimizes water-logging problems
6. It assists in controlling root zone salinity problems
7. It results in additional returns by using the "saved" water to irrigate non-cash crops



Soil water regime approach

- In this approach the available soil water held between field capacity and permanent wilting point in the effective crop root zone
- Alternatively soil moisture tension, the force with which the water is held around the soil particles is also sometimes used as a guide for timing irrigations.

Feel and appearance of soil

- This is one of the oldest and simple methods of determining the soil moisture content.
- It is done by visual observation and feel of the soil by hand.
- The accuracy of judgement improves with experience.

Table 18.1. Guidelines for judging soil moisture by feel & appearance of soil

Available soil moisture range	Coarse texture (loamy sand)	Moderately coarse (sandy loamy)	Medium texture (loamy and silt loamy)	Fine texture (clay loamy and silty clay loamy)
Above field capacity	Free water appears when soil is bounded in hand	Free water is released with kneading	Free water can be squeezed out	Puddles; free water forms on surface.
At Field capacity (100%)	On squeezing no free water appears on soil, but wet outline of ball is left on hand	Same as for coarse textured soils at field capacity	Same as for coarse textured soils at field capacity	Same as for coarse textured soils at field capacity
75% to 100%	Tends to stick together slightly, may form a very weak ball under pressure	Forms weak ball that breaks easily, does not slick	Forms a ball, very pliable, sticks readily if relatively high in clay	Easily ribbons out between fingers; has a slick feeling
50% to 75%	Appears to be dry, does not form a ball under pressure	Forms a ball under pressure but seldom holds together	Forms a ball under pressure; somewhat plastic, slicks slightly under pressure	Forms a ball; ribbons out between thumb and forefinger
25% to 50%	As above, but ball is formed by squeezing very firmly	Appears to be dry, do not form a ball unless squeezed very firmly	Somewhat crumbly but holds together with pressure	Somewhat pliable, forms a ball under pressure
0 to 25%	Dry, loose & single grained, flows through fingers.	Dry and loose, flows though fingers	Powdery dry, sometimes slightly crusted, but breaks down easily into powder.	Hard, baked and cracked, has loose crumbs on surface in some places

Depletion of the available soil moisture (DASM)

- In this method the permissible depletion level of available soil moisture in the effective crop root zone depth is commonly taken as an index.
- In general, for many crops scheduling irrigation's at 20 –25% DASM in the soil profile was found to be optimum at moisture sensitive stages.
- While at other stages irrigations scheduled at 50% DASM were found optimum.

Soil moisture tension

- Soil moisture tension a physical property of film water in soil, as monitored by tensiometers at a specified depth in the crop root zone could also be used as an index for scheduling irrigations to field crops.
- Tensiometers are installed in pairs, one in the maximum rooting depth and the other below this zone.
- Whenever critical soil moisture tension is reached the irrigation is commenced.

- While the lower one (tensiometer) is used to terminate the irrigations based on the suction readings in the below soil profile zone.
- It is generally used for irrigating orchards and vegetables in coarse textured soils because most of the available soil moisture is held at lower tensions.

Climatological Approach

The potential rate of water loss from a crop is primarily a function of evaporative demand of the atmosphere In this method the water loss expressed in terms of either potential evapotranspiration (PET) or cumulative pan evaporation (CPE)

Different climatological approaches are described below:

Potential evapotranspiration (PET)

Penmen (1948) introduced the concept of PET

It is defined as "the amount of water transpired in a unit time by short green crop of uniform height, completely covering the ground and never short of water".

PET can be estimated by several techniques viz.,

1. Lysimetric methods
2. Energy balance
3. Aerodynamic approach
4. Combination of energy balance and empirical formulae etc.

Plant Indices Approach

Visual plant symptoms

- In this method the visual signs of plants are used as an index for scheduling irritations.
- For instance, plant wilting, drooping, curling and rolling of leaves in maize is used as indicators for scheduling irrigation
- Change in foliage colour and leaf angle is used to time irrigations in beans.
- Water stress in some crops leads to appearance of carotenoid (yellow and orange colour) and anthocyanin pigments
- Shortening of internodes in sugarcane and cotton; retardation of stem elongation in grapes;
- Leaf abscission and lack of new growth and redness in terminal growth points of almond

Soil-cum-sand mini-plot technique or profile modification technique

- Commonly used for scheduling irrigations to crops.
- The principle involved in this technique is to reduce artificially the available water holding capacity of soil profile (i.e., effective root zone depth) in the mini-plot by mixing sand with it.
- When this is done plants growing on the sand mixed plot show wilting symptoms earlier than in the rest of the field.
- An area of 1.0 x 1.0m is selected in the field and a pit of 1.0m depth is excavated.

- About 5% of sand by volume is added to the dug up soil and mixed well.
- The pit is then filled back with the mixture and while filling up every 15 cm layer is well compacted, so that the soil in the pit retains the original bulk density as that of surrounding soil.
- Crop is sown normally and is allowed to grow as usual with the rest of the field.
- As and when the plants in the mini-plot show wilting symptoms it is taken as a warning of impending water need and cropped field is irrigated.

Plant population

- Increase in plant population by 1.5 to 2.0 times that of optimum
- This happens because when more plants are there per unit area, the available water within that zone is depleted rapidly as compared to other area
- This result in drooping or wilting of plants earlier, which can be taken as an indication of water deficits and accordingly irrigations are scheduled to crops.

Rate of growth

- Growth of a plant is dependent on turgor, which in turn is dependent on a favourable soil water balance.
- So fluctuations in the water balance are reflected by parallel fluctuations in the growth rate of expanding organs.
- Stem elongation is markedly reduced when the available soil moisture level approaches the critical level, but accelerates again after irrigation.

Canopy temperatureIndicator plants

- In wheat, scheduling irrigations on the basis of wilting symptoms in maize and sunflower gave the highest grain yields.

Critical growth stages

- The crop plants in their life cycle pass through various phases of growth, some of which are critical for water supply.
- The most critical stage of crop growth is the one at which a high degree of water stress would cause maximum loss in yield.

Irrigation Efficiencies

• Efficiency is the ratio of the water output to the water input, and is usually expressed as percentage.

• Input minus output is nothing but losses, and hence, if Losses are more, output is less and, therefore, efficiency is less. Hence, efficiency is inversely proportional to the losses.

• Water is lost in irrigation during various processes and, therefore, there are different kinds of irrigation efficiencies, as given below

- Efficiency of Water-conveyance

- Efficiency of Water Application
- Efficiency of Water Use
- Efficiency of water storage
- Water Distribution Efficiency

Efficiency of Water-conveyance (η_c)

• It is the ratio of the water delivered into the fields from the outlet point of the channel, to the water entering into the channel at its starting point. It may be represented by η_c. It takes the conveyance or transit losses into consideration.

$$\boxed{\eta_c = (W_f/W_r) \times 100}$$

Where

- η_c= Water conveyance efficiency,
- W_f = Water delivered to the irrigated plot at field supply Channel,
- W_r = Water diverted from the source (river or reservoir)

Efficiency of Water Application (η_a)

• It is ratio of water stored into the root zone of the crop to the quantity of water delivered at the field (Farm).

$$\boxed{\eta_a = W_s/W_f \times 100}$$

Where,

- η_a = Water application efficiency,
- W_s = Water stored at the root zone during the irrigation
- W_f= Water delivered to the farm.

Efficiency of Water Use (η_u)

- It is the ratio of the water beneficially used including leaching water, to the Quantity of water delivered. It may be represented by η_u

$$\boxed{\eta_u = (Wu/Wd) \times 100}$$

Where,

- η_u = Water use efficiency,
- Wu = Beneficial use of water or consumptive.
- Wa = Water delivered to the field.

Efficiency of water storage: (η_s)

- The concept of water storage efficiency gives an insight to how completely the required water has been stored in the root zone during irrigation.

$$\boxed{\eta_s = (Ws/Wn) X\ 100}$$

Where,

- η_s= Water storage efficiency,
- Ws = water stored in the root zone during irrigation.
- Wn = Water need in the root zone prior to irrigation.

Water Distribution Efficiency (η_d)

Water distribution efficiency evaluates the degree to which water is uniformly distributed throughout the root zone. Uneven distribution has many undesirable results. The more uniformly the water is distributed , the better will be crop response.

$$\boxed{\eta_d = 100\ (1-y/d)}$$

Where,

- η_d= Water distribution efficiency,
- y= avg numerical deviation in depth of water stored from avg depth stored in the root zone during irrigation
- d = Avg depth of water stored during irrigation..

Consumptive use Efficiency (η_{cu})

It is the ratio of consumptive use of water to the water depleted from the root zone.

$$\boxed{\eta_{cu} = (W_{cu}/W_d) X\ 100}$$

Where,

- η_{cu}= Consumptive use efficiency,
- W_{cu}= Nominal consumptive use of water
- W_d = Net amount of water depleted from the root zone soil.

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UNIT -3

DIVERSION AND IMPOUNDING STRUCTURES

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

Impounding structure

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

Diversion headwork.

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

The purposes of diversion headwork.

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

Weir

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

The component parts of diversion headwork

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

Dam

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

Types of Dams

Based on Materials of Construction

- Rigid.
- Non-Rigid.

Based on Structural Behaviour

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

Based on Functions

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

Based on Hydraulic Behaviour

- Over Flow Dam.
- Non Over Flow Dam.

General Types

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

Gravity dam

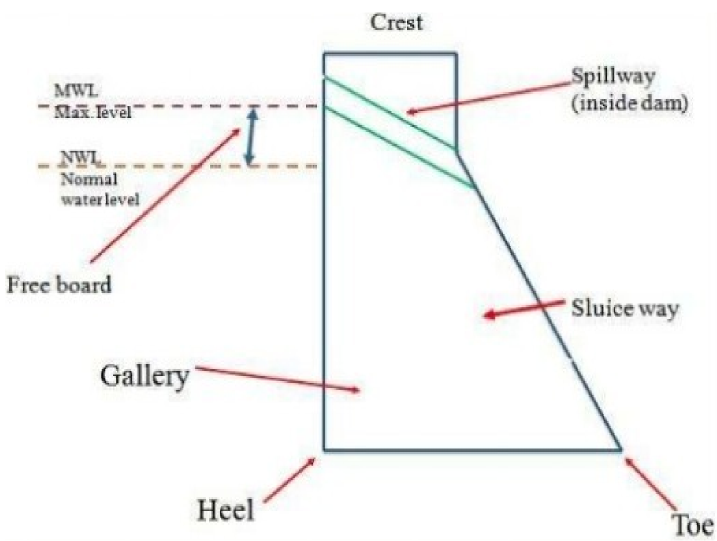
- A gravity dam is a structure so proportioned that its own weight resists the forces exerted upon it. It requires little maintenance and it is most commonly used.
- A Gravity dam has been defined as a "structure which is designed in such a way that its own weight resist the external forces".
- This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

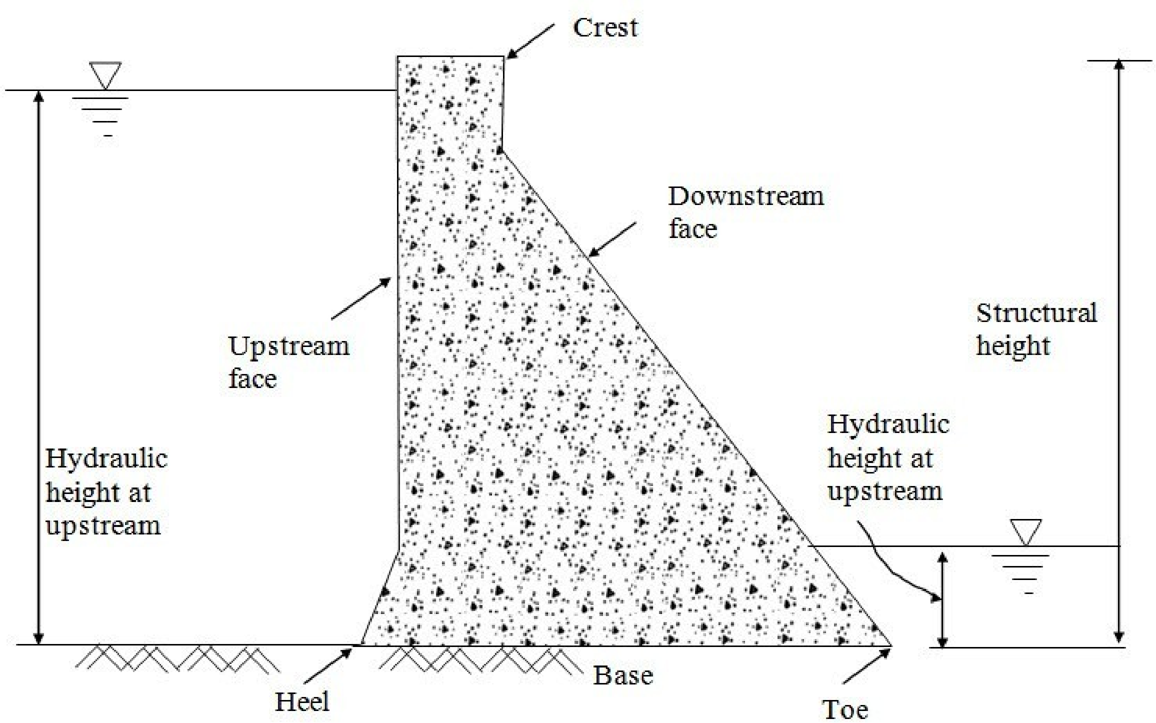
of the dam or the "Axis of The Dam" When suitable conditions are available such dams can be constructed up to great heights.

The different components of a solid gravity dam are

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

Typical cross section of gravity Dam:





Heel: contact with the ground on the upstream side

Toe: contact on the downstream side

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

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

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

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

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

Forces Acting on Gravity Dam

The Various external forces acting on Gravity dam may be:

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

Self weight of the Dam

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

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

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

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

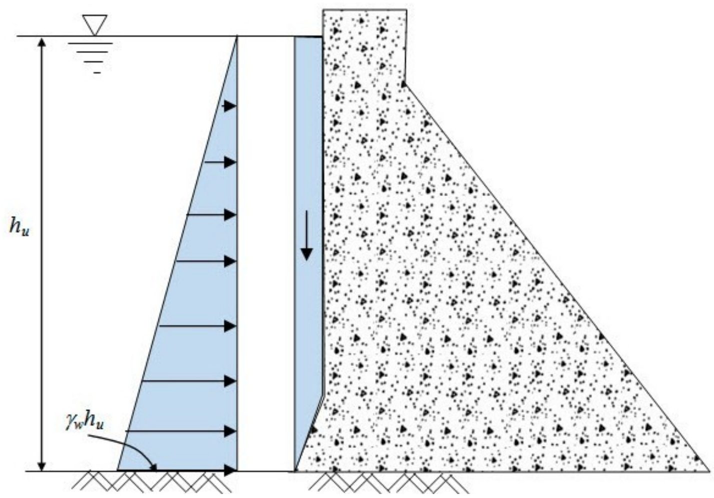
Where γ_m is the specific weight of the dam's material.

Water pressure

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

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

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



Uplift water pressure

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

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

Wave Pressure

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

The magnitude of waves depend upon –

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

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

Where 'h_w' is the wave height.

WIND PRESSURE :

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

SEISMIC FORCES :

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

ELEMENTRY PROFILE

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

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

- where, σ_c=allowable compressive stress

Practical Profile

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

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

For elementary profile –

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

Low Gravity Dam

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

High Gravity Dam

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

Failure of Gravity Dam

Failure of gravity dams are caused due to,

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

Precautions against Failure

- To prevent overturning, the resultant of all forces acting on the dam should remain within the middle-third of the base width of the dam.
- In the dam, the sliding should be fully resisted when the condition for no sliding exists in the dam section.

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

Temperature Control

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

To control the temperature the following steps may be taken

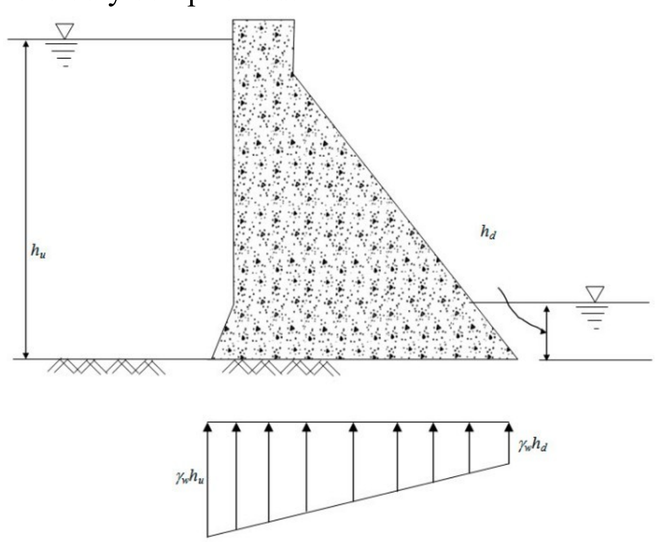
1. Low heat cement may be used in concrete.
2. The water & coarse aggregates should be cooled down to 5°C by suitable means before mixing.
3. During laying the height of concrete blocks should not be more than 1.5 m. It helps radiate heat to the atmosphere more quickly.
4. The water is cooled by crushed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

Advantages

1. Gravity dams are more suitable in narrow valleys.
2. Maintenance cost is lower
3. Failure of these dams is not very sudden.
4. Gravity dams may be built to any height.
5. Loss of water by seepage in gravity dams is less

Disadvantages

1. Initial cost for construction of gravity dams is very higher.
2. Gravity dams of greater height can only be constructed on sound rock foundations.
3. Require skill labour for construction.
4. Design of gravity dams is very complicated.



General Requirement for Stability

A gravity dam may fail in the following modes,

- Overturning
- Sliding
- Compression
- Tension

Therefore, the requirements for stability are,

- The dam should be safe against overturning.
- The dam should be safe against sliding.
- The induced stresses (either tension or compression) in the dam or in the foundation should not exceed the permissible value.

DESIGN OF GRAVITY DAM

Example 19.2. *Fig. 19.20 (a) shows the section of a gravity dam built of concrete. Examine the stability of this section at the base.*

The earthquake forces may be taken as equivalent to 0.1 g for horizontal forces and 0.05 g for vertical forces. The uplift may be taken as equal to the hydrostatic pressure at the either ends and is considered to act over 60% of the area of the section.

A tail water depth of 6 m is assumed to be present when the reservoir is full and there is no tail water when the reservoir is empty.

Also indicate the values of various kinds of stresses that are developed at heel and toe. Assume the unit wt. of concrete as 24 kN/m^3 ; and unit wt. of water = 10 kN/m^3.

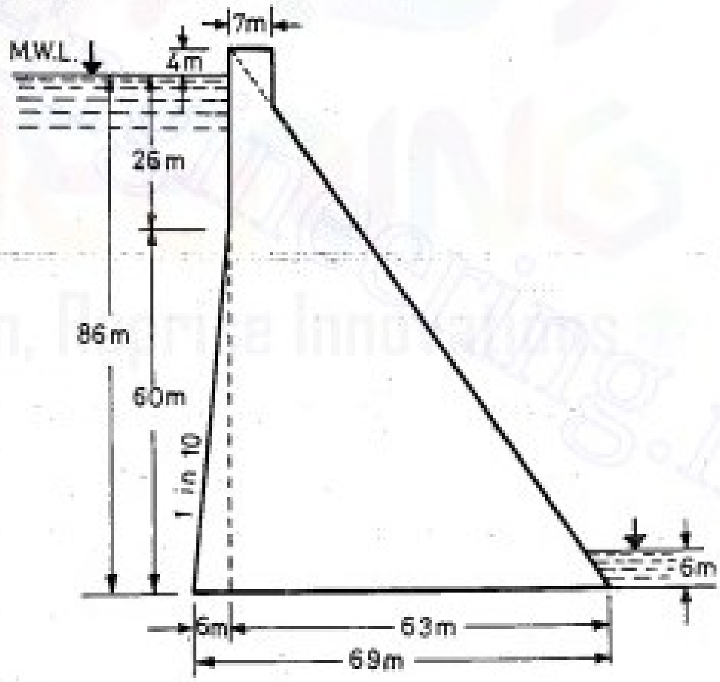


Fig. 19.20 (*a*)

Solution. The stability analysis shall be carried out for both the cases, *i.e.* (1) Reservoir Empty, and (2) Reservoir Full.

Case (I) Reservoir Empty. Consider 1 m length of the dam.

When the reservoir is empty, the various forces are worked out in Table 19.2 (*a*) with reference to Fig. 19.20 (*b*). Horizontal earthquake forces acting towards upstream are considered. Stability is examined for two sub-cases, *i.e.* (*a*) When vertical earthquake forces are additive to the weight of the dam ; (*b*). When vertical earthquake forces are subtractive to the dam weight.

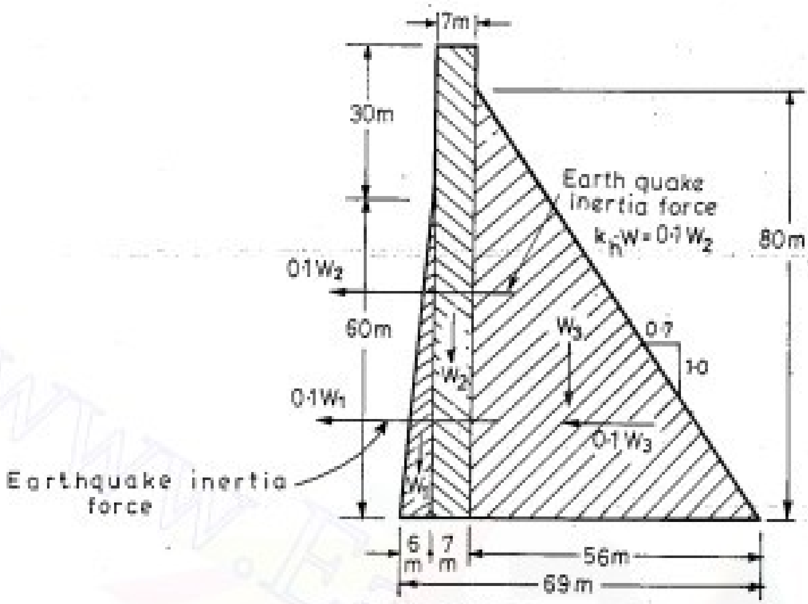


Fig. 19.20 (*b*). Reservoir empty case.

Table 19.2 (*a*)

Name of the force	*Designation if any*	*Magnitude of force in kN.*		*Lever arm m*	*Moments about the toe anti-clockwise (+ve) in kN.m.*
		Vertical	*Horizontal*		
Downward wt. of dam	W_1	(+) $\frac{1}{2} \times 6 \times 60 \times 24 = 4{,}320$		65.0	(+) 2,80,400
	W_2	(+) $7 \times 90 \times 24 = 15{,}110$		59.5	(+) 8,99,000
	W_3	(+) $\frac{1}{2} \times 56 \times 80 \times 24 = 53700$		37.33	(+) 20,00,000
		$\Sigma V_1 = 73{,}130$			$\Sigma M_1 =$ (+) 31,79,400
Horizontal earthquake forces	P_{w_1}		$0.1\ W_1 = 0.1 \times 4320 = 432$	20.0	(+) 8640
	P_{w_2}		$0.1\ W_2 = 0.1 \times 1{,}511 = 1511$	45.0	(+) 68000
	P_{w_3}		$0.1\ W_3 = 0.1 \times 5{,}370 = 5370$	26.67	(+) 1,43,200
			$\Sigma H = 7313$		$\Sigma M_2 = 2{,}19{,}840$
Vertical earthquake forces		$\Sigma V_2 = 0.05 \times \Sigma V_1 = 0.05 \times 73130 = 3{,}657$			$\Sigma M_2 = 0.05 \times \Sigma M_1 = 0.05 \times 31{,}79{,}400 = 1{,}58{,}970$

Case (I). (*a*) *Reservoir empty and vertical earthquake forces are acting downward.*

From table 19.2 (*a*), we have $\Sigma M = \Sigma M_1 + \Sigma M_2 + \Sigma M_3$

$$= 31{,}79{,}400 + 2{,}19{,}840 + 1{,}58{,}970 = \mathbf{35{,}58{,}210\ kN \cdot m}$$

Also, $\Sigma V = \Sigma V_1 + \Sigma V_2 = 73{,}130 + 3{,}657 = 76{,}787$ kN

$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{35{,}58{,}210}{76{,}787} = 47.3 \text{ m}$$

$$e = \frac{B}{2} - \bar{x} = \frac{69}{2} - 46.3 = 34.5 - 46.3 = -\mathbf{11.8\ m} > \frac{B}{6}, \textit{ i.e. } 11.5 \text{ m.}$$

Resultant acts near the heel and slight tension will develop at toe.

$$p_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

$$\therefore \quad p_{max/min} = \frac{76{,}787}{69}\left[1 \pm \frac{6 \times 11.8}{69}\right] = 1114\ [1 \pm 1.026]$$

p_v at heel $= 1114 \times 2.026 = 2260$ kN/m^2; which is ≤ 3000 (safe)

p_v at toe $= 1114 \times (-0.026) = -29$ kN/m^2 ; which is < 420 (*safe*)

Average vertical stress

$$= \frac{\Sigma V}{B} = \frac{76787}{69} = 1114 \text{ kN/m}^2 \text{ ; which is} < 3000 \text{ (safe)}$$

Principal stress at toe,

$$\sigma = p_v \sec^2 \alpha \text{ ; } (\tan \alpha = 0.7)$$

$$= -29\ (1 + 0.49) = -29 \times 1.49 = -43 \text{ kN/m}^2 \text{ ; which is} < 420 \text{ (safe)}$$

Principle stress at heel

$$\sigma_1 = p_{v \cdot (heel)} \sec^2 \phi \qquad \text{where } \tan \phi = 0.1$$

$$\text{or} \quad \sec^2 \phi = 1 + \tan^2 \phi = 1 + 0.01 = 1.01.$$

or $\sigma_1 = 2260 \times 1.01 = \mathbf{2280\ kN/m^2}$; which is < 3000 (safe).

Shear stress at toe

$$\tau_{0(toe)} = p_{v(toe)} \tan \alpha$$

$$= -29 \times 0.7 = -20.3 \text{ kN/m}^2 \text{ ; which is} < 420 \text{ (safe)}$$

Shear stress at heel

$$\tau_{0(heel)} = p_{v \cdot (heel)} \tan \phi$$

$$= 2260 \times 0.1 = 226 \text{ kN/m}^2 \text{ ; which is} < 3000 \text{ (safe).}$$

Case I. (*b*) *Reservoir empty and vertical earthquake forces are acting upward.*

Then $\Sigma V = \Sigma V_1 - \Sigma V_3$

$$= 73{,}130 - 3657 = 69473 \text{ kN}$$

$$\Sigma M = \Sigma M_1 + \Sigma M_2 - \Sigma M_3$$

$$= 31{,}79{,}400 + 2{,}19{,}840 - 1{,}58{,}970 = 32{,}40{,}270 \text{ kN} \cdot \text{m.}$$

$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{32{,}40{,}270}{69473} = \mathbf{46.7\ m.}$$

$$e = \frac{B}{2} - \bar{x} = 34.5 - 46.7 = (-)\ 12.2 \text{ m} < \frac{B}{6}$$

[– ve sign shows that resultant lies near the heel and, therefore, tension will develop at toe.]

Average vertical stress

$$= \frac{\Sigma V}{B} = \frac{69{,}473}{69} = 1004 \text{ kN/m}^2$$

$$p_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

$$= \frac{69473}{69}\left[1 \pm \frac{6 \times 12.2}{69}\right] = 1004\,[1 \pm 1.06]$$

$$p_v \text{ at heel} = 1004 \times 2.06 = 2070 \text{ kN/m}^2 < 3000 \text{ (safe)}$$

$$p_v \text{ at toe} = (-)\,1004 \times 0.06 = -\,60.3 \text{ kN/m}^2 < 420 \text{ (safe)}$$

Principal stress at toe

$$= \sigma = p_{v(toe)} \sec^2 \alpha$$

$$= -\,60.3\,(1 + 0.49) = -\,60.3 \times 1.49 = 90 \text{ kN/m}^2$$

Shear stress at toe

$$= \tau_0 = p_{v(toe)} \tan \alpha = -\,60.3 \times 0.7$$

$$= \mathbf{-\,42.21\ kN/m^2}\text{ ; which is} < 420 \text{ (safe)}$$

stresses at heel remain critical in this 1st case.

Case II. When the reservoir is full

Horizontal earthquake moving towards the reservoir causing upstream acceleration, and thus producing horizontal forces towards downstream is considered, as it is the worst case for this condition. Similarly, a vertical earthquake moving downward and thus, producing forces upward, *i.e.* subtractive to the weight of the dam is considered.

The uplift coefficient C is taken as equal to 0.6, as given in the equation, and thus uplift pressure diagram as shown in Fig. 19.20 (c), is developed.

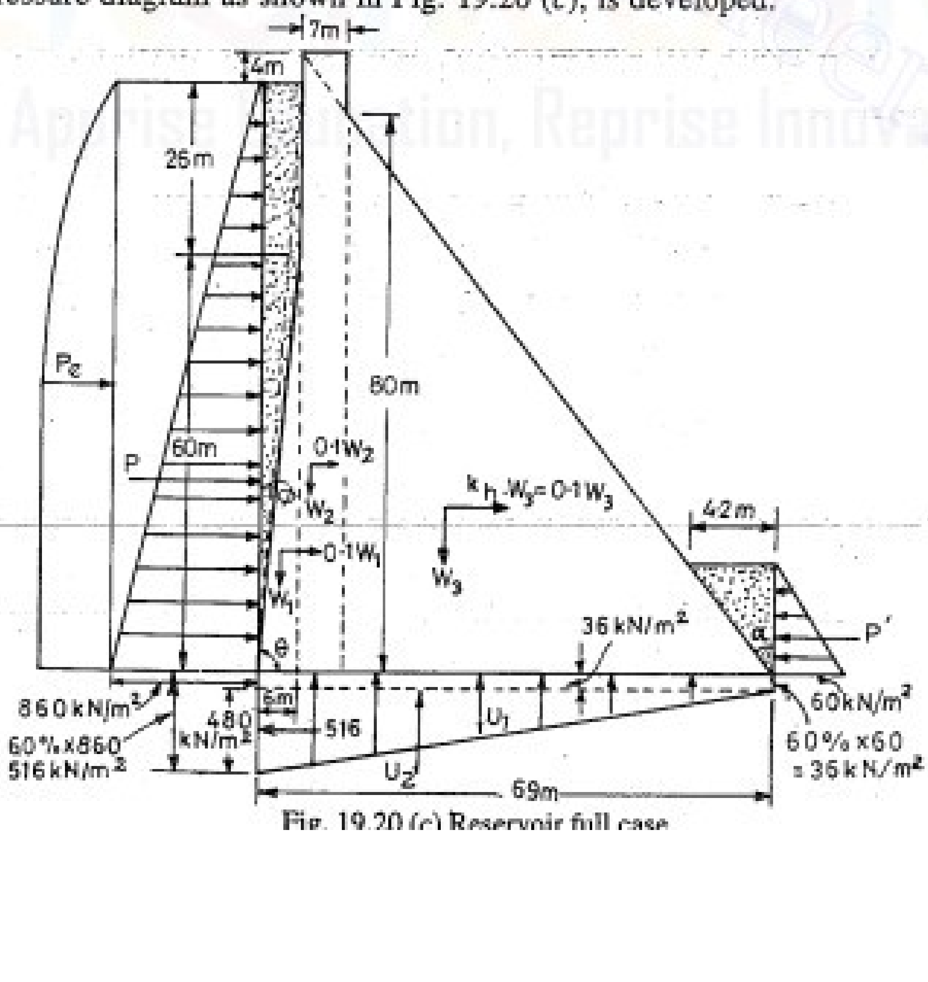


Fig. 19.20 (c) Reservoir full case

The various forces acting in this case are :

(*i*) Hydrostatic pressures P and P'.

(*ii*) Hydrodynamic pressure P_e. (P_e' is neglected as it is very small and neglection is on conservative side.)

(*iii*) Uplift forces U_1 and U_2

(*iv*) Weight of the dam, W_1, W_2 and W_3.

(*v*) Horizontal inertial earthquake forces acting towards downstream, equal to 0.1 W_1, 0.1 W_2 and 0.1 W_3 at *c.gs.* of these weights W_1, W_2 and W_3 respectively.

(*vi*) A vertical force equal to 0.05 W or $(0.05\ \Sigma V_1)$ acting upward.

Calculation of P_e

P_e and the moment due to this hydrodynamic force is calculated, and then all the forces and their moments are tabulated in Table 19.2 (*b*).

Calculation of P_e from Zanger's formulas

$$P_e = 0.726\, p_e\, H \qquad \text{...(19.3)}$$

$$\text{where } p_e = C_m \cdot K_h \cdot \gamma_w \cdot H \qquad \text{...(19.4)}$$

$$\text{and} \quad C_m \doteq 0.735 \frac{\theta}{90°}$$

Since the u/s inclined face is extended for more than half the depth, the overall slope up to the whole height may be taken.

Fig. 19.20 (*d*)

$$\therefore \qquad \tan\theta = \frac{86}{6} = 14.33$$

$$\theta = 81.9°$$

$$\therefore \qquad C_m = 0.735 \times \frac{81.9°}{90°} = 0.668.$$

$$p_e = 0.668 \times 0.1 \times 10 \times 86 = 57.5$$

$$P_e = 0.726 \times 57.5 \times 86 = \mathbf{3580\ kN.}$$

$$M_e = 0.412 \cdot P_e \cdot H = 0.412 \times 3580 \times 86 = \mathbf{1{,}26{,}500\ kN.m.}$$

Case 2 (*a*) Reservoir full with all forces including uplift

$$\Sigma M = [31{,}79{,}400 + 2{,}23{,}380 - 8{,}47{,}500 - 1{,}58{,}970 - 10{,}59{,}730 - 1{,}26{,}500 - 2{,}19{,}840]$$

$$= \mathbf{34{,}02{,}780 - 24{,}12{,}540 = 9{,}90{,}240\ kN/m.}$$

$$\Sigma V = 73130 + 3486 - 19030 - 3657 = 53929\ \text{kN}$$

$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{9{,}90{,}240}{53{,}929} = 18.36\ \text{m}$$

$$e = \frac{B}{2} - \bar{x} = 34.5 - 18.36 = 16.14 > \frac{B}{6}$$

The resultant is nearer the toe and tension is developed at the heel.

Average vertical stress

$$= \frac{\Sigma V}{B} = \frac{53929}{69} = 782\ \text{kN/m}^2.$$

$$p_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

Table 19.2 (b)

Name of force	Designation if any	Magnitude of force in kN — Vertical forces Downward = +ve Upward = −ve	Magnitude of force in kN — Horizontal forces Towards Upstream = +ve Towards Downstream = −ve	Lever arm in m	Moments about toe in kN. Anticlock wise (+ve) and clockwise (− ve) in kN.m
(1)	(2)	(3)	(4)	(5)	(6)
Weight of Dam	W_1	(+) $\frac{1}{2} \times 6 \times 60 \times 1 \times 24 = 4320$		65.0	(+) 2,80,400
	W_2	(+) $7 \times 90 \times 1 \times 24 = 15,110$		59.5	(+) 8,99,000
	W_3	(+) $\frac{1}{2} \times 56 \times 80 \times 1 \times 24 = 53,700$		37.33	(+) 20,00,000
		$\Sigma V_1 = (+)\ 73,130$			$\Sigma M_1 = 31,79,400$
Weight of water supported on u/s slope	—	(+) $26 \times 6 \times 1 \times 10 = 1560$		66.0	(+) 1,02,800
	—	(+) $\frac{1}{2} \times 60 \times 6 \times 1 \times 10 = 1800$		67.0	(+) 1,10,400
water on d/s slope.	—	(+) $\frac{1}{2} \times 6 \times 4.2 \times 1 \times 10 = 126$		1.4	(+) 180
		$\Sigma V_2 = (+)\ 3486$			$\Sigma M_2 = (+)\ 2,23,380$
Uplift forces	U_1	(−) $69 \times 3.6 \times 10 = 2,480$		34.5	(−) 85,500
	U_2	(−) $\frac{1}{2} \times 69 \times 48 \times 10 = 16,550$		46.0	(−) 7,62,000
		$\Sigma V_3 = (-)\ 19,030$			$\Sigma M_3 = (-)\ 8,47,500$
Upward vertical earthquake forces 0.05 W		$\Sigma V_4 = (-)\ 0.05 \cdot \Sigma V_1$ $= (-)\ 0.05 \times 73,130$ $= (-)\ 3,657$			$= (-)\ 0.05 \cdot \Sigma M_1$ $= (-)\ 0.05 \times 31,79,400$ $\Sigma M_4 = (-)\ 1,58,970$
Horizontal hydrostatic pressure	P		(−) $\frac{1}{2} \times 10 \times 86 \times 86 \times 1$ $= (-)\ 36,980$	28.67	(−) 10,60,090
	P'		(+) $\frac{1}{2} \times 10 \times 6 \times 6 \times 1 = (+)\ 180$	2.0	(−) 360
			$\Sigma H_1 = (-)\ 36,800$		$\Sigma M_5 = (-)\ 10,59,730$
Horizontal hydro-dynamic pressure	P_e		Calculated separately earlier : $= (-)\ 3,580$ $\Sigma H_2 = (-)\ 3,580$		$\Sigma M_e = (-)\ 1,26,500$ (calculated separately earlier
Horizontal inertia forces due to earthquake	P_{W_1}		(−) $0.1\ W_1 = (-)\ 432$	20.0	(−) 8,640
	P_{W_2}		(−) $0.1\ W_2 = (-)\ 1,511$	45.0	(−) 68,000
	P_{W_3}		(−) $0.1\ W_3 = (-)\ 5,370$	26.67	(−) 1,43,200
			$\Sigma H_3 = (-)\ 7,313$		$\Sigma M_7 = (-)\ 2,19,840$

$$\Sigma H = \Sigma H_1 + \Sigma H_2 + \Sigma H_3 = (-)36,800 - 3580 - 7313 = (-)\ 47,693$$

$$= \frac{53929}{69}\left[1 \pm \frac{6 \times 18.32}{69}\right] = 782\,[1 \pm 1.595]$$

p_v (at toe) $= 782 \times 2.595 = \mathbf{2030\ kN/m^2}$; which is < 3000 kN/m² (∴ Safe)

p_v (at heel) $= -782 \times 0.405$

$= \mathbf{-316.7\ kN/m^2}$; which is < 420 kN/m² (∴ Safe)

Since the tensile stress developed is less than the safe allowable value, the dam is safe even when examined with seismic forces, under reservoir full condition.

Principal stress at toe

$$= \sigma = p_v \cdot \sec^2\alpha - p' \tan^2\alpha \quad \textit{i.e. } \text{Eq. (19.17)}$$

where $\tan\alpha = 0.7$, $p' = 60$ kN/m² ; $p_v = 2030$ kN/m²

$$\sigma = 2030\,(1 + \tan^2\alpha) - p' \tan^2\alpha$$
$$= 2030\,(1 + 0.49) - 60 \times 0.49 = 2030 \times 1.49 - 29$$
$$= 3025 - 29 = \mathbf{2996\ kN/m^2 : which\ is < 3000} \quad \text{(just Safe)}$$

Principal stress at heel is

$$\sigma_1 = p_{v(heel)} \sec^2\phi - (p + p_e)\tan^2\phi \quad \textit{i.e. } \text{Eq. (19.19)}$$

where ϕ is the angle which the upstream face makes with the vertical

$\tan\phi = 0.1$

$$\therefore \quad \sigma_1 = -316.7\,[1 + (0.1)^2] - (860 + 57.5)\,(0.1)^2$$
$$= -316.7 \times 1.01 - 917.5 \times 0.01 = -319.9 - 9.2$$
$$= \mathbf{-329.1\ kN/m^2} \text{ ; which is} < 420 \text{ kN/m}^2 \quad \text{(Hence, safe)}$$

Shear stress at toe

$$\tau_{0(toe)} = (p_{v(toe)} - p')\tan\alpha = (2030 - 60)\,0.7$$
$$= 1970 \times 0.7 = \mathbf{1379\ kN/m^2.}$$

Shear Stress at heel

$$\tau_{0(heel)} = -\,[p_{v(heel)} - (p + p_e)]\tan\phi$$
$$= -\,[-329.1 - (860 + 57.5)]\,0.1$$
$$= -\,[-329.1 - 917.5]\,0.1 = +1246.6 \times 0.1 = \mathbf{124.7\ kN/m^2}$$

Factor of safety against overturning

$$= \frac{\Sigma M\,(+)}{\Sigma M\,(-)} = \frac{34{,}02{,}780}{24{,}12{,}540} = 1.41 \text{ ; which is} < 1.5 \quad \text{(Hence, Unsafe)}$$

Factor of safety against sliding

$$= \frac{\mu \cdot \Sigma V}{\Sigma H}$$

where $\mu = 0.7$

$\Sigma V = 53{,}929$

$\Sigma H = \Sigma H_1 + \Sigma H_2 + \Sigma H_3$

$= -36800 - 3580 - 7313 = -47{,}693$ kN

$$\textit{Sliding factor} = \frac{0.7 \times 53929}{47693} = 0.79, \text{ which is} < 1 \qquad \text{(Hence, Unsafe)}$$

Shear friction factor

$$\text{S.F.F.} = \frac{\mu \cdot \Sigma V + B \cdot q}{\Sigma H}$$
$$= \frac{0.7 \times 53929 + 69 \times 1400}{47693}$$
$$= 2.81 \text{ ; which is less than 3} \qquad \text{(Hence, slightly unsafe)}$$

Case 2 (*b*). Reservoir full, without uplift

Sometimes, values of stresses at toe and heel are worked out when there is no uplift, s the vertical downward forces are maximum in this case. For this case, we shall ılculate ΣM and ΣV by ignoring the corresponding values of ΣV_3 and ΣM_3 caused by plift.

$$\therefore \quad \Sigma M = \Sigma M_1 + \Sigma M_2 + \Sigma M_4 + \Sigma M_5 + \Sigma M_6 + \Sigma M_7$$
$$= 31{,}79{,}400 + 2{,}23{,}380 - 1{,}58{,}970 - 10{,}59{,}730 - 1{,}26{,}500 - 2{,}19{,}840$$
$$= 34{,}02{,}780 - 15{,}65{,}040 = 18{,}37{,}740$$
$$\Sigma V = \Sigma V_1 + \Sigma V_2 + \Sigma V_4 = 73130 + 3486 - 3657 = 72{,}959 \text{ kN}$$
$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{18{,}37{,}740}{72{,}959} = 25.19 \text{ m}$$
$$e = \frac{B}{2} - \bar{x} = 34.5 - 25.9 = 9.31 \text{ m} > \frac{B}{6} \quad \textit{i.e.} \quad \frac{69}{6} = 11.5 \text{ m}$$

Resultant is nearer the toe and no tension is developed any where.

$$p_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$
$$= \frac{72{,}959}{69}\left[1 \pm \frac{6 \times 9.31}{69}\right] = 1057\,[1 \pm 0.81]$$

p_v at toe $= 1057 \times 1.81 =$ **1913 kN/m²** < 3000 (∴ Safe)

p_v at heel $= 1057 \times 0.19 =$ **201 kN/m²** < 3000 (∴ Safe)

Principal stress at toe $= \sigma = p_v \cdot \sec^2 \alpha - p' \tan^2 \alpha$ ...(19.17)

$$p' = 60, \tan \alpha = 0.7$$

$$\therefore \quad \sigma = 1913\,(1 + 0.49) - 60 \times 0.49 = 1913 \times 1.49 - 29 = 2821 \text{ kN/m}^2 < 3000$$

(Hence, Unsafe)

Principal stress at heel

$$\sigma_1 = p_{v(heel)} \sec^2 \phi - (p + p_e) \tan^2 \phi \qquad ...(19.19)$$

where $\tan \phi = 0.1$

$$\therefore \quad \sigma_1 = 201(1 + 0.01) - (860 + 57.5) \times 0.01$$
$$= 203 - 9 = 194 \text{ kN/m}^2 < 420 \quad \text{(Safe)}$$

Shear stress at toe

$$\tau_0 = (p_v - p') \tan \alpha \qquad \textit{i.e. } \text{Eq. (19.20)}$$
$$= (1913 - 60)\, 0.7$$
$$= 1853 \times 0.7 = 1297 \text{ kN/m}^2 < 1400 \qquad (\therefore \text{ safe})$$

Note. Shear friction factor, etc. are not worked out here as they were more critical in the 1st case, *i.e.* in 'Reservoir full with uplift' case.

Conclusion. The dam is unsafe only in sliding and S.F.F., for which shear key etc. can be provided.

Example 19.3. *Examine the stability of the dam section given in the previous example, if there are no seismic forces acting on the dam. Also state the magnitude of maximum compressive stress and maximum shear stress that may develop under any conditions of loading in the dam and also state whether tension is developed anywhere or not.*

Solution. The figures calculated earlier in Table 19.2 (*a*) and (*b*) shall be used here.

Case I. When the reservoir is empty

$$\Sigma V = \Sigma V_1 \text{ from Table 10.2 } (a) = 73130$$
$$\Sigma M = \Sigma M_1 \text{ from Table 19.2 } (b) = 3179400$$

$$\therefore \quad \bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{3179400}{73130} = 43.4 \text{ m}$$

$$e = \frac{B}{2} - \bar{x} = 34.5 - 43.4 = -8.9 \text{ m}$$

–ve sign means that the resultant is towards left side, *i.e.* nearer to the heel, and since $e < \frac{B}{6}$, no tension is develoned

$$p_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$
$$= \frac{73130}{69}\left[1 \pm \frac{6 \times 8.9}{69}\right]$$
$$= 1060\,[1 \pm 0.774]$$

$$p_v \text{ at heel} = 1060(1 + 0.774) = 1060 \times 1.774 = \mathbf{1880\ kN/m^2}$$
$$p_v \text{ at toe} = 1060(1 - 0.774) = 1060 \times 0.226 = \mathbf{239\ kN/m^2}$$

Average vertical stress

$$= \frac{\Sigma V}{B} = \frac{73130}{69} = \mathbf{1060\ kN/m^2}$$

Principal stress at toe

$$\sigma = p_{v(toe)} \sec^2 \alpha$$
$$= 239(1 + 0.49) = 239 \times 1.49 = \mathbf{357\ kN/m^2}$$

Principal stress at heel,

$$\sigma = p_{v(heel)} \sec^2 \phi$$
$$\text{where } \tan \phi = 0.1$$
$$= 1880(1 + 0.01) = 1880 \times 1.01 = \mathbf{1896\ kN/m^2}$$

Shear stress at toe

$$\tau_0 = p_{v(toe)} \tan \alpha$$
$$= 239 \times 0.7 = 167.3 \text{ kN/m}^2$$

$$= 734\,(1 + 0.01) - 860 \times 0.01$$
$$= 742 - 9 = \mathbf{733\ kN/m^2}$$

Shear stress at toe

$$\tau_0 = [p_{v(toe)} - p'] \tan \alpha$$
$$= (1490 - 60)\,0.7 = 1430 \times 0.7 = 1001\ \text{kN/m}^2$$

Shear stress at heel

$$= -\,[p_{v(heel)} - p] \tan \phi$$
$$= -\,[734 - 860] \times 0.7 = 126 \times 0.7 = \mathbf{88.2\ kN/m^2}$$

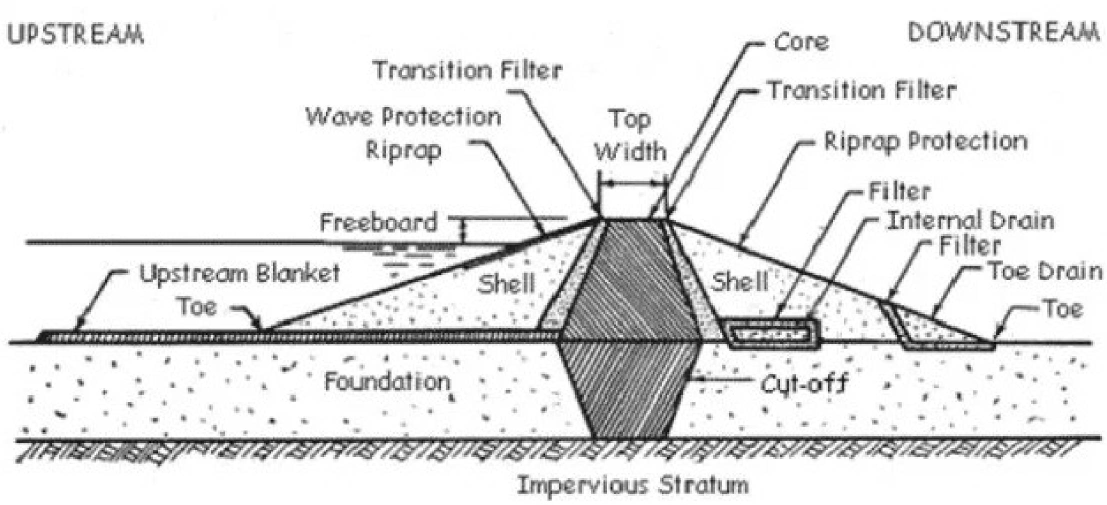
Conclusions. We find that the dam is safe throughout except that the S.F.F. is equal to 3.72, while generally it should be between 4 to 5. The dam thus remains slightly unsafe in S.F.F. even when the seismic forces are not considered.

The results of stability analysis are given below :

The maximum shear stress developed in dam	$= 1001\ \text{kN/m}^2$.	**Ans.**
Maximum compressive stress developed in dam	$= 2191\ \text{kN/m}^2$	
No tension is developed anywhere.		
Factor of safety against sliding	$= 1.10$	
S.F.F.	$= 3.72$	
Factor of safety against overturning	$= 1.78$	

EARTHEN DAMS

- An earthen embankment is a raised confining structure made from compacted soil.
- The purpose of an earthen embankment is to confine and divert the storm water runoff. It can also be used for increasing infiltration, detention and retention facilities.
- Earthen embankments are generally trapezoidal in shape and most simple and economic in nature. They are mainly built with clay, sand and gravel, hence they are also known as earth fill dams or earthen dams.
- They are constructed where the foundation or the underlying material or rocks are weak to support the masonry dam or where the suitable competent rocks are at greater depth.
- They are relatively smaller in height and broader at the base.



Components of an Earthen Dam

1. **Shell, Upstream Fill, Downstream Fill or Shoulder:** These components of the earthen dam are constructed with pervious or semi-pervious materials upstream or downstream of the core. The upstream fill is called the upstream shell and the downstream portion is the downstream shell.
2. **Upstream Blanket:** It is a layer of impervious material laid on the upstream side of an earthen dam where the substratum is pervious, to reduce seepage and increase the path of flow. The blanket decreases both the seepage flow and excess pressure on the downstream side of the dam. A natural blanket is a cover of naturally occurring soil material of low permeability.
3. **Drainage Filter:** It is a blanket of pervious material constructed at the foundation to the downstream side of an earthen dam, to permit the discharge of seepage and minimize the possibility of piping failure.
4. **Cutoff Wall or Cutoff:** It is a wall, collar or other structure intended to reduce percolation of water through porous strata. It is provided in or on the foundations.
5. **Riprap:** Broken stones or rock pieces are placed on the slopes of embankment particularly the upstream side for protecting the slope against the action of water, mainly wave action and erosion.
6. **Core Wall, Membrane or Core:** It is a centrally provided fairly impervious wall in the dam. It checks the flow of water through the dam section. It may be of compacted puddled clay, masonry, or concrete built inside the dam.
7. **Toe Drain:** It is a drain constructed at the downstream slope of an earthen dam to collect and drain away the seepage water collected by the drain filters.
8. **Transition Filter:** It is a component of an earthen dam section which is provided with core and consists of an intermediate grade of material placed between the core and the shells to serve as a filter and prevent lateral movement of fine material from the core.

Advantages

1. Design procedures are straightforward and easy.
2. Local natural materials are used.
3. Comparatively small establishment and equipment are required.
4. Earth fill dams resist settlement and movement better than more rigid structures and can be more suitable for areas where earth movements are common.

Disadvantages

1. An earthen embankment is easily damaged or destroyed by water flowing on, over or against it. Thus, a spillway and adequate upstream protection are essential for any earthen dam.
2. Designing and constructing adequate spillways is usually the most technically difficult part of any dam building work. Any site with a poor quality spillway should not be used.
3. If it is not adequately compacted during construction, the dam will have weak structure prone to seepage.

4. Earthen dams require continual maintenance to prevent erosion, tree growth, subsidence, animal and insect damage and seepage.

Types of Earthen Dam

1. Based on the method of construction:

(a) Rolled Fill Earthen Dams:

- In this type of dams, successive layers of moistened or damp soils are placed one above the other.
- Each layer not exceeding 20 cm in thickness is properly consolidated at optimum moisture content maintained by sprinkling water.
- It is compacted by a mechanical roller and only then the next layer is laid.

(b) Hydraulic Fill Earthen Dam:

In this type of dams, the construction, excavation and transportation of the earth are done by hydraulic methods.

Outer edges of the embankments are kept slightly higher than the middle portion of each layer.

During construction, a mixture of excavated materials in slurry condition is pumped and discharged at the edges.

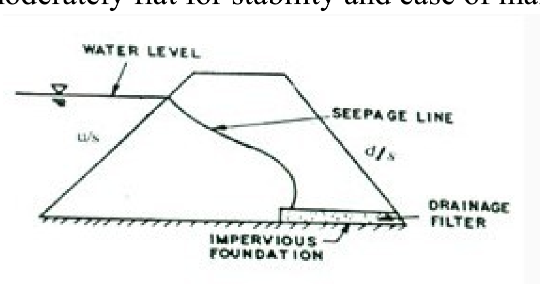
This slurry of excavated materials and water consists of coarse and fine materials. When it is discharged near the outer edges, the coarser materials settle first at the edges, while the finer materials move to the middle and settle there.

Fine particles are deposited in the central portion to form a water tight central core. In this method, compaction is not required.

2. Based on the mechanical characteristics of earth materials used in making the section of dam:

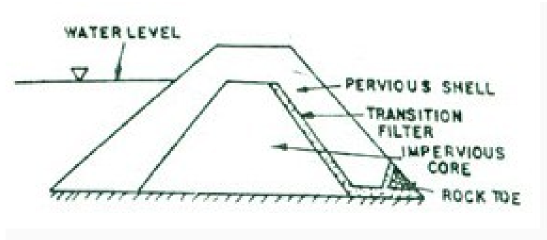
(a) Homogeneous Earthen Dams:

- It is composed of one kind of material (excluding slope protection).
- The material used must be sufficiently impervious to provide an adequate water barrier, and the slopes must be moderately flat for stability and ease of maintenance



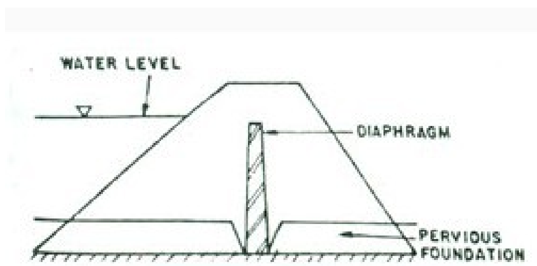
(b) Zoned Earthen Dams:

- It contains a central impervious core, surrounded by zones of more pervious material, called shells.
- These pervious zones or shells support and protect the impervious core.



(c) Diaphragm Earthen Dam:

- This type of dam is a modified form of homogenous dam which is constructed with pervious materials, with a thin impervious diaphragm in the central part to prevent seepage of water.
- The thin impervious diaphragm may be made of impervious clayey soil, cement concrete or masonry or any impervious material.
- The diaphragm can be constructed in the central portion or on the upstream face of the dam.
- The main difference in zoned and diaphragm type of dams depends on the thickness of the impervious core or diaphragm. The thickness of the diaphragm is not more than 10 m.



Design Criteria

Following main design criteria may be laid down for the safety of an earth dam:

1. To prevent hydraulic failures the dam must be so designed that erosion of the embankment is prevented. For this purpose, the following steps should be followed:
   a) Spillway capacity is sufficient to pass the peak flow.
   b) Overtopping by wave action at maximum water level is prevented.
   c) The original height of structure is sufficient to maintain the minimum safe freeboard after settlement has occurred.
   d) Erosion of the embankment due to wave action and surface runoff does not occur.
   e) The crest should be wide enough to withstand wave action and earthquake shock.
2. To prevent the failures due to seepage:
   a) Quantity of seepage water through the dam section and foundation should be limited.
   b) The seepage line should be well within the downstream face of the dam to prevent sloughing.
   c) Seepage water through the dam or foundation should not remove any particle or in other words cause piping.

d) There should not be any leakage of water from the upstream to the downstream face. Such leakage may occur through conduits, at joints between earth and concrete sections or through holes made by aquatic animals.

3. To prevent structural failures:

- The upstream and downstream slopes of the embankment should be stable under all loading conditions to which they may be subjected including earthquake.
- The foundation shear stresses should be within the permissible limits of shear strength of the material.

Design of Earthen Dam

The preliminary design of earthen dam is done on the basis of past experiences. For designing purpose several parameters, given below should be considered.

1. Top Width
2. Free Board
3. Settlement Allowance
4. Casing or Outer Shell
5. Cut-off Trench
6. Downstream Drainage System

1. Top Width:

- Minimum top width (W) should be such that it can enhance the practicability and protect it against the wave action and earth wave shocks.
- Sometimes it is also used for transportation purposes.
- It depends upon the height of the earthen dam and can be calculated as follows:

$$W = \frac{H}{5} + 3 \quad \text{(for very low dam)}$$

$$W = 0.55\sqrt{H} + 0.2H \quad (H \le 30)$$

$$W = 1.65\sqrt[3]{H + 1.5} \quad (H \ge 30)$$

where H = the height of the dam (m), for Indian conditions it should not be less than 6 m.

2. Free board:

- It is the vertical distance between the top of the dam and the full supply level of the reservoir or the added height.
- It acts as a safety measure for the dam against high flow condition that is waves and runoff from storms greater than the design frequency from overtopping the embankment.
- The Recommended values of free board for different heights of earthen dams, given by U.S.B.R., are given in Table.

Recommended Values of Free Board given by U.S.B.R.

Nature of spillway	Height of dam	Free board
Free	Any	Minimum 2 m and maximum 3 m over the maximum flood level
Controlled	< 60 m	2.5 m above the top of the gate
Controlled	> 60 m	3 m above the top of the gate

If fetch length or exposure is given then the free board can also be calculated by Hawksley's formula: $h_w = 0.014\, D_m^{0.5}$ where, = wave height (m); Dm = fetch or exposure (m).

2. Settlement Allowance:

- It is the result of the settlement of the fill and foundation material resulting in the decrease of dam storage.
- It depends upon the type of fill material and the method and speed of construction.
- It varies from 10% of design height for hand compacted to 5% for machine compacted earthfill.

3. Casing or Outer Shell:

- Its main function is to provide stability and protection to the core.
- Depending upon the upstream and downstream slopes, a recommendation for the casing and outer shell slopes for different types of soils given by Terzaghi is presented in Table

Recommended Slopes of Earthen Dam (Sources: S.K. Garg, 2008)

Sl. No.	Types of material	u/s slope	d/s slope
1.	Homogenous well graded material	$2\frac{1}{2}:1$	$2:1$
2.	Homogenous coarse silt	$3:1$	$2\frac{1}{2}:1$
3.	Homogenous silty clay or clay a) Height less than15 m b) Height more than 15 m	 $2\frac{1}{2}:1$ $3:1$	 $2:1$ $2\frac{1}{2}:1$
4.	Sand or sand and gravel with clay core	$3:1$	$2\frac{1}{2}:1$
5.	Sand or sand and gravel with R.C. core wall	$2\frac{1}{2}:1$	$2:1$

Cutoff Trench:

- It is provided to reduce the seepage through the foundation and also to reduce the piping in the dam.

- It should be aligned in a way that its central line should be within the upstream face of the impervious core.
- Its depth should be more than 1 m. Bottom width of cutoff trench (B) is calculated as:

  $B=h-d$ where h = reservoir head above the ground surface (m); and d = depth of cutoff trench below the ground surface (m).

4. Downstream Drainage System:

- It is performed by providing the filter material in the earthen dam which is more pervious than the rest of the fill material.
- It reduces the pore water pressure thus adding stability to the dam.
- Three types of drains used for this purpose are:
  1. Toe Drains
  2. Horizontal Blanket
  3. Chimney Drains.

Causes of Failure

1.Hydraulic Failures 40%

2. Seepage Failures 30%

3. Structural Failures 30%

a) Piping

b) Sloughing

a) Overtopping

b) Wave Erosion

c) Toe Erosion

d) Gullying

a) Upstream slope failure due to sudden drawdown

b) Failure by excessive pore pressure

c) Downstream slope failure by sliding

d) Failure due to settlement of foundation

e) Failure by sliding of foundation

f) Failure by spreading

i. Overtopping:

- The dam is overtopped when the volume of incoming flow into the reservoir is more than the actual storage capacity of the reservoir, or the capacity of spillway is not sufficient.
- Sometimes, the faulty operation of spillway also leads to the overtopping problem.
- Similarly, insufficient free board or settlement of foundation as well as embankment also cause the overtopping problem in earthen dam.

ii. Wave Erosion:

- Wave action removes the soil particles from the unprotected part of upstream face of the clam, continuously.
- This is one of the effective factors to cause the hydraulic type failure in earthen dam.

Toe erosion in the earth fill dam, mainly occurs due to following reasons:

1. **Erosion caused by the tail water**; and

- (ii) Erosion due to cross-currents produced by the storage water, spillway bucket or from the outlet, create the problem of hydraulic failure.
- This type of failure can be overcome by providing a thick layer of stone riprap on the downstream face upto the height of tail water level.

iv. Gullying:

- Development of gully in earthen dam is the result of heavy down pour. Such type of failures can be eliminated by providing a proper size of berm, turf or good drainage system towards down-stream side of the dam.

2. Seepage Type Failures:

Failure of earthen dam due to seepage phenomena may be due to following two reasons:

i. Piping; and

ii. Sloughing.

i. Piping:

The continuous flow of seepage water through the body as well as foundation of the dam is the main reason of piping. It causes catastrophic failures in the earth fill dams.

The flow of seepage water through the body of earth dam develop following four effects:

a) The flow of seepage water generates an erosive force, which tends to dislodge the soil particles from the dam section. The dislodged particles are migrated into the voids of the filter materials, down-stream side; and thus clogged them, as result the drainage system gets failed.

b) The seepage flow develops differential pore pressure which tends to lift up the soil mass, causing boiling effect in the dam.

c) Piping is also the result of internal erosion of the soil mass due to seepage flow through the earth dam.

d) The pore pressure developed in the soil reduces the soil strength, which makes the soil mass weak, as result there is failure of dam due to shear force.

Sometimes, the leakage from earthen dam also generates the piping type failure. Furthermore, it is also observed that, the piping type failure is most prominent in those dams, which are poorly constructed. Generally, this is due to poor compaction surrounding the concrete outlets or other parts of the structure etc.

ii. Sloughing:

- Failure of earthen dam due to sloughing is closely related to the water level in the reservoir.
- In full reservoir condition the downstream toe of the dam becomes fully saturated, which is failure by producing a small slump or miniature slide.
- Under miniature slide the saturated steep face of the dam is dislodged.
- This process is continued till the remaining portion of the dam is being very weak to withstand against pore water pressure.

3. Structural Failures:

i. Structural failure mainly caused by the following reasons:

ii. Upstream and downstream slope failures due to formation of excessive pore pressure.

iii. Upstream failure due to sudden drawdown in the reservoir water level.

iv. Downstream failure at the time of full reservoir.

v. Foundation slide.

vi. Failure of dam due to earthquake.

vii. Failure of dam due to unprotected side slope.

viii. Failure due to damage caused by burrowing animals.

ix. Failure due to damage caused by water soluble materials.

i. Upstream and Downstream Slope Failure due to Pore Pressure:

- Development of pore pressure in the body of earthen dam, is mainly due to poor compressibility of the soil.
- This occurrence is more susceptible, when dam is constructed with relatively impervious compressible soils, in which drainage of seepage water is extremely low, which causes the development of pore pressure in the soil.
- The compressibility of soil is related to the permeability.
- It has been observed that, when permeability of soil is less than 10–6 cm/s, then there is no substantial drop in pore pressure in the central part of the dam by the end of construction.
- A pore pressure equal to 140% of total weight of soil develops a very crucial situation regarding dam stability. In this condition the slope of dam is likely to failed.

ii. Failure of Upstream Slope due to Sudden Draw down in the Reservoir Water level:

- Failure of upstream slope due to sudden draw down in reservoir water level is a critical condition.
- During this stage, the hydrostatic pressure acting along the upstream slope is suddenly removed, as result the face of the dam gets slide.
- In this failure the upstream side slope did not get complete failure, because when slide takes place due to sudden draw down in reservoir water level, the pore pressure acting along the

sliding surface is reduced to a large extent. In this way, the tendency to continue the process of sloughing and sliding of upstream face of the dam, is checked.

iii. Downstream Slope Slide during Full Reservoir Condition:

- When the reservoir is in full condition, then there happens maximum percolation/seepage loss through the dam section.
- This results into reduction of stability of the dam, which causes the downstream slope gets collapse.
- In this case, the failure of downstream slope generally takes place in-following two types of slide:

(a) Deep Slide:

- Deep slide generally takes place in the clay foundations.
- In deep slide the magnitude of free board given to the dam is reduced due to extending of upstream face beyond its edge of the crest.
- In this type of slide the pore pressure does not decrease, and the unstable vertical face tends to slough or slide again and again, until to breach the entire dam.

(b) Shallow Slide:

- The shallow slide extends in the dam section not more than 2 m in the direction normal to the slope.

iv. Failure due to Foundation Slide:

- This type of failure of earthen dam generally takes place, when foundation is constructed, using fine silt or soft soil materials.
- Sometimes, when soft and weak clayey soil exists under foundation, then dam also tends to get slide.
- Similarly, excess water pressure in confined sand and silt is also developed in the foundation, which causes the failure of dam due to creation of unbalanced condition.

v. Failure of Dam due to Earthquake:

It generally takes place due to following reasons:

1. Earthquake develops cracks in the body of dam; and thus leading to flow of water, which ultimately causes to failure the dam.
2. It compresses the foundation and embankment, both, thereby the total free board provided to the dam gets reduce and thus, increasing the chances of overtopping problem.
3. It shakes the bottom of the reservoir, as result there develop wave action, which causes the problem of failure of dam due to overtopping and wave erosion.
4. It generates an additional force on the face of embankment that can lead to develop shear slide of dam slope.

5. Earthquake is also responsible for sliding the top of dam, which may cause overtopping; and thus damaging the structure.

vi. Failure of Earthen Dam due to Slope Protection:

- Generally, slopes are protected by rip-rap or revetment using a layer of gravel or filter blanket.
- When a heavy storm occurs, then water wave beats the dam slope repeatedly above the reservoir level.
- This action of wave produces the following two effects:
- The wave enters the voids of the rip-rap and washout the filter layer from the dam face. This causes the embankment to get expose to the wave action; and
- If rip-rap is not done by heavy rocks, then there is greater chance of their removal by the forces generated from water waves.

vii. Failure due to Damage Caused by Burrowing:

- Burrowing develops piping type failure in earthen dam. Generally, the animals like muskats burrow the embankment section, either to make shelter for their living or to make a direct passage for running from one end to another.
- If several muskats involved together to make the hole, then their holes may extremely weaken the dam section.

viii. Failure due to Water Soluble Materials:

- Based on several observations on this aspect of failure of earthen dams, it has been found that the leaching of natural water soluble materials such as zypsum etc. from the dam tends to create water leakage problem through the dam section.
- In this condition, the foundation also gets settle down, and thus creates the problem of overtopping and ultimately the dam reaches to the point of its failure.

DESIGN OF EARTHEN DAM

Example 20.4. *An earthen dam made of homogeneous material has the following data :*

Level of the top of the dam	= *200.00 m*
Level of deepest river bed	= *178.0*
H.F.L. of reservoir	= *197.5 m*
Width of top of dam	= *4.5 m*
Upstream slope	= *3 : 1*
Downstream slope	= *2 : 1*
Length of the horizontal filter from d/s toe, inwards	= *25 m*
Cohesion of soil of dam	= *24 kN/ m*^{2}
Cohesion of soil of foundation	= *54 kN/ m*^{2}
Angle of internal friction of soil in the dam	= *25°*
Angle of internal friction of soil in the foundation	= *12°*
Dry weight of the soil in the dam	= *18 kN/m*^{3}
Submerged weight of the soil in the dam	= *12 kN/m*^{3}
Dry unit weight of the foundation soil	= *18.3 kN/m*^{3}
Coefficient of permeability of soil in the dam	= 5×10^{-6} *m/sec.*

The foundation soil consists of 8 m thick layer of clay, having negligible coefficient of permeability. Check the stability of the dam and its foundations.

Solution.

(1) **Overall stability of the dam section as a whole**

We will consider 1 m length of the dam. The section of the dam and the phreatic line is first of all drawn, as given in example 20.3 and shown in Fig. 20.29 (*a*). The dam section, etc. is generally drawn on a graph sheet so as to facilitate in measuring the areas above and below the seepage line, if planimeter is not available.

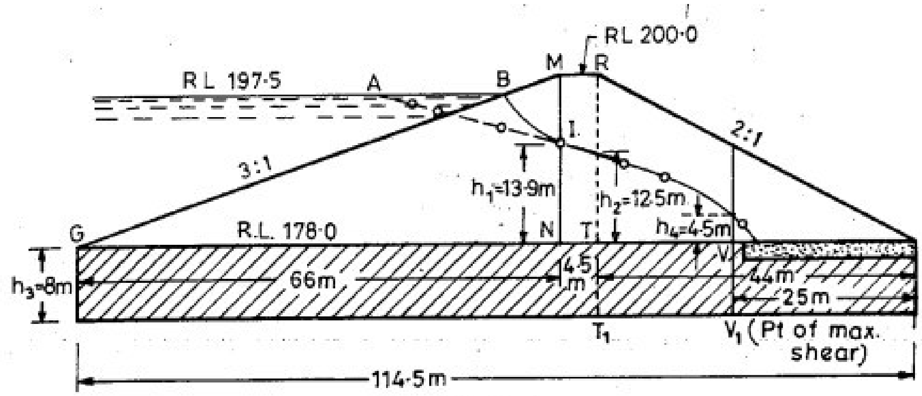


Fig. 20.29 (*a*)

The total area of dam section $= (114.5 + 4.5)\,\dfrac{22}{2} = 1{,}409$ sq. m

The area above the seepage line is measured and is approximately found to be $380\ \text{m}^2$. (In the absence of a planimeter, graph can be used).

$\therefore$ Area below the seepage line $= 1{,}409 - 380 = 1{,}029$ sq. m

Now

Weight of the dry portion of the dam section

$$= (380\ \text{m}^2 \times 1\ \text{m} \times 18\ \text{kN/m}^3 = 6830\ \text{kN}.$$

Weight of the submerged portion of the dam section

$$= 1029\ \text{m}^2 \times 1\ \text{m} \times 12\ \text{kN/m}^3 = 12{,}350\ \text{kN}$$

Total weight of dam (called average weight)

$$= 6{,}830 + 12{,}350 = 19{,}180\ \text{kN}$$

Shear resistance of the dam at the base

$$= C + W \tan \phi$$

where C = Total cohesive strength of the soil at the base

$= c \times B \times 1 = (24 \times 114.5 \times 1)$ kN

B = Total base width = 114.5 m

$W \tan \phi = 19{,}180 \tan 25°$

$\therefore$ Shear resistance at base,

$$R = 24 \times 114.5 \times 1 + 19180 \tan 25° = 11690 \text{ kN}$$

Horizontal force = Horizontal pressure of water.

$$= P = \tfrac{1}{2} \gamma_w h^2 = \tfrac{1}{2} \cdot 9.81\,(19.5)^2 = 1865 \text{ kN}$$

Factor of safety against failure due to horizontal shear at base

$$= \frac{11690}{1865} = 6.27 > 1.3 \ (\therefore \text{ Safe})$$

(2) **Stability of the u/s slope portion of dam (Under sudden drawdown)** *horizontal shear along the base under the u/s slope of dam*

Draw a vertical through the u/s extremity of the top width of dam [*i.e.* point M, Fig. 20.29 (*a*)] so as to cut the base of the dam at point N. This vertical MN cuts the seepage line at a point, the height of which is measured as $h_1 = 13.6$ m above the base of the dam.

Horizontal force (P_u) acting on the ΔGMN is given by equation (20.27) as :

$$P_u = \left[\frac{\gamma_1 h^2}{2} \tan^2\left(45 - \frac{\phi}{2}\right) + \gamma_w \cdot \frac{h_1^{\,2}}{2}\right]$$

where γ_1 = the weighted density at the centre of triangular shoulder upstream (ΔGMN) and is given by equation (20.28) as :

$$\gamma_1 = \frac{\gamma_{sub} \cdot h_1 + \gamma_{dry}\,(h - h_1)}{h}$$

$$= \frac{12 \times 13.9 + 18\,(22.0 - 13.9)}{22.0}$$

$$= 14.7 \text{ kN/m}^3$$

$$\therefore \quad P_u = \frac{14.7 \times (22.0)^2}{2} \tan^2\left(45° - \frac{25°}{2}\right) + 9.81 \times \frac{(13.9)^2}{2} = 2391 \text{ kN}$$

Shear resistance R_u of the u/s slope portion of dam developed at the base GN is given by equation (20.29) as :

$$R_u = C + W \tan \phi$$

$= c\,(B_u \times 1) + (\gamma_{sub} \tfrac{1}{2} B_u\, h) \tan \phi$; neglecting the small dry soil area BMI, as it is very small and this neglection is on a safer side.

$B_u = 66$ m

$$\therefore \quad R_u = 24 \times 66 + (12 \cdot \tfrac{1}{2} \cdot 66 \cdot 22.0) \tan 25°$$

$$= 1584 + 4062 = 5646 \text{ kN}$$

Factor of safety against horizontal shear along base under u/s slope

$$= \frac{R_u}{P_u} = \frac{5646}{2391} = 2.36 > 2.0 \qquad (\therefore \text{ safe})$$

Horizontal shear stress induced in the u/s slope portion of dam at base.

$$\tau_{av} = \frac{P_u}{B_u \times 1} = \frac{2391}{66} \text{ kN/m}^2 = 36.23 \text{ kN/m}^2$$

$$\tau_{max} = \text{Maximum shear}$$

$$= 1.4\,\tau_{av} = 1.4 \times 36.23 = 50.72 \text{ kN/m}^2$$

The maximum shear is developed at a point $0.6\,B_u$

$$= 0.6 \times 66 = 39.6 \text{ m away from point } G$$

The unit shear resistance developed at this point

$$\tau_f = c + 0.6\,\gamma_{sub} \tan \phi$$

$$= 24 + 0.6 \times 22.0 \times 12 \tan 25° = 97.9 \text{ kN/m}^2$$

$$\therefore \qquad \text{F.S.} = \frac{\tau_f}{\tau_{max}} = \frac{97.9}{50.72} = 1.93 > 1 \qquad (\therefore \text{ safe}).$$

(3) **Stability of d/s portion of dam.** *Horizontal shear along base under the d/s slope of dam.*

Draw a vertical through the d/s extremity of the top width of dam (*i.e.* point R) to cut the base at point T [Fig. 20.29 (*a*)]. Let this vertical cut the seepage line in a point, the height of which from the base is measured as $h_2 = 12.5\,m$.

Horizontal force P_d acting on the portion of downstream dam (RTS) during steady seepage is given by equation (20.35) as :

$$P_d = \left[\frac{\gamma_2 h^2}{2} \tan^2\left(45° - \frac{\phi}{2}\right) + \gamma_w \frac{h_2^2}{2}\right]$$

where γ_2 is the weighted density at the centre of the triangular shoulder RTS and given by equation (20.36) as :

$$\gamma_2 = \frac{\gamma_{sub}\, h_2 + \gamma_{dry}\,(h - h_2)}{h}$$

$$= \frac{12 \times 12.5 + 18 \times (22.0 - 12.5)}{22.0}$$

$$= 14.6 \text{ kN/m}^3$$

$$P_d = \frac{14.6\,(22)^2}{2} \tan^2\left(45° - \frac{25°}{2}\right) + 9.81 \cdot \frac{(12.5)^2}{2} = 2200 \text{ kN}$$

Shear resistance R_d of the d/s slope portion of dam developed at base TS is given as :

$$R_d = C + W \tan \phi$$

The area A_1 of the dry soil within the ΔRTS above the seepage line ≈ 300 sq. m (from graph or planimeter).

The total area of the $\Delta RTS = \frac{1}{2} \times 44 \times 22 = 484\ \text{m}^2$

$\therefore$ Area of submerged soil

$$A_2 = 484 - 300 = 184 \text{ sq. m}$$

$$R_d = cB_d + \left[\gamma_{dry} A_1 + \gamma_{sub} A_2\right] \tan \phi$$

$$= 24 \times 44.0 + [18 \times 300 + 12 \times 184] \tan 25° = 4604 \text{ kN.}$$

F.S. against horizontal shear along base under d/s slope

$$= \frac{R_d}{P_d} = \frac{4604}{2200} = 2.09 > 2 \ (\therefore \text{ Safe})$$

Average shear induced at base

$$= \frac{P_d}{B_d} = \frac{2200}{44} = 50 \text{ kN/m}^2$$

Maximum shear stress induced

$$\tau_{max} = 1.4 \times 50 = 70 \text{ kN/m}^2$$

The maximum shear stress is developed at a point $0.6\ B_d$

$$= 0.6 \times 44 = 26.4 \text{ m away from toe}$$

This unit shear resistance developed at this point

$$\tau_f = c + 0.6h\ \gamma_{sub} \tan \phi$$

(assuming the entire height as submerged as it will give safer results)

$$= 24 + 0.6 \times 22 \times 12 \tan 25° = 97.9 \text{ kN/m}^2$$

$\therefore$ F.S. $= \dfrac{\tau_f}{\tau_{max}} = \dfrac{97.9}{70} = 1.40 > 1$ ($\therefore$ Safe)

(4) **Stability of the foundation soil**

Average compressive stress on foundation soil

$$= \frac{\text{Weight of dam}}{\text{Base area on which it acts}}$$

Since the compressive stress is maximum when the entire dam soil is dry, therefore, we will first calculate the dry weight of the dam.

Area of section of dam

$$= 1{,}409 \text{ sq. m (calculated earlier)}$$

Dry weight of dam section

$$= 18 \times 1{,}409 = 25{,}362 \text{ kN}$$

Average compressive stress at base

$$= \frac{25362}{114.5} = 221.5 \text{ kN/m}^2$$

Shear stress induced at base

The total horizontal shear force (P) under the d/s slope of the dam (which is the worst case, *i.e.* the steepest slope) is given by equation (20.39) as :

$$P = \gamma_{eq}\left[\frac{(h+h_3)^2 - {h_3}^2}{2}\right]\left[\tan^2\left(45° - \frac{\phi_1}{2}\right)\right]$$

where γ_{eq} is the equivalent weight of dry soil in foundation and dam

$$\gamma_{eq} = \frac{18h + 18.3\, h_3}{h + h_3}$$

[$\because$ Unit wt. of foundation soil of thickness $h_3 = 18.3\ \text{kN/m}^3$]

where $h = 22$ m

$h_3 = 8$ m.

$$\therefore \quad \gamma_{eq} = \frac{18 \times 22 + 18.3 \times 8}{22 + 8} = 18.1\ \text{kN/m}^3$$

ϕ_1 is given by equation (20.41) as :

$$\gamma_{eq}\,(h + h_3) \tan\phi_1 = c_f + \gamma_{eq}\,(h + h_3) \tan\phi_f$$

or $\quad 18.1\,(22 + 8) \tan\phi_1 = 54 + 18.1\,(22 + 8) \tan 12°$

or $\quad \tan\phi_1 = 0.312$

or $\quad \phi_1 = 17.3°$

$$\therefore \quad P = 18.1\left[\frac{(22+8)^2 - (8)^2}{2}\right][\tan^2(45° - 8.65°)]$$

$$= \frac{18.1}{2}[900 - 64]\,[(0.737)^2] = 4100\ \text{kN}$$

Average shear stress induced at base of d/s slope

$$\tau_{av} = \frac{4100}{44} = 93.2\ \text{kN/m}^2$$

Maximum shear stress induced at $0.6 \times 44 = 26.4$ m away from the d/s toe inwards at point V_1 is given by

$$= \tau_{max} = 1.4 \times 93.2 = 130.4\ \text{kN/m}^2$$

Shear resistance of the foundation soil below the d/s slope portion of dam

Unit shear resistance τ_{f_1} below the toe at point S_1

$$= [c_f + \gamma_f \times h_3 \tan\phi_f]$$

$$= 54 + 18.3 \times 8 \times \tan 12°$$

$$\therefore \quad = 85.1\ \text{kN/m}^2$$

Unit shear resistance τ_{f_2} at point T_1

$$= c_f + \gamma_3\,(h + h_3) \tan\phi_f$$

where
$$\gamma_3 = \frac{\gamma_{sub\,for\,dam} \times h_2 + \gamma_{dry\,for\,dam} \times (h - h_2) + \gamma_f\, h_3}{h + h_3}$$

$$= \frac{12 \times 12.5 + 18 \times 9.5 + 18.3 \times 8}{30} = 15.6\ \text{kN/m}^3$$

$$\therefore \quad \tau_{f_2} = 54 + 15.6\,(22 + 8) \times \tan 12° = 153.5\ \text{kN/m}^2$$

The average unit shear resistance developed at foundation level in a length equal to $T_1S_1 = 44$ m, is given by

$$\tau_f = \frac{\tau_{f_1} + \tau_{f_2}}{2} = \frac{85.1 + 153.5}{2} = 119.3 \text{ kN/m}^2$$

Over all F.S. against shear

$$= \frac{\tau_f}{\tau_{av}} = \frac{119.3}{93.2} = 1.28 < 1.5 \qquad \textbf{(Hence, unsafe)}$$

The foundation soil is thus weaker to carry the load and hence the d/s slope will have to be flattened.

Shear resistance at the point of maximum shear, *i.e.* at point V_1 is given as :

$$(\tau_f)_{max} = c_f + (0.6h + h_3)\, \gamma_4 \tan \phi_f$$

$$\gamma_4 = \frac{12 \times 4.5 + 18\,(0.6 \times 22 - 4.5) + 18.3 \times 8}{0.6 \times 22 + 8} = 16.8 \text{ kN/m}^3$$

$$(\tau_f)_{max} = 54 + [0.6 \times 22 + 8]\ 16.8 \tan 12° = 129.8 \text{ kN/m}^2$$

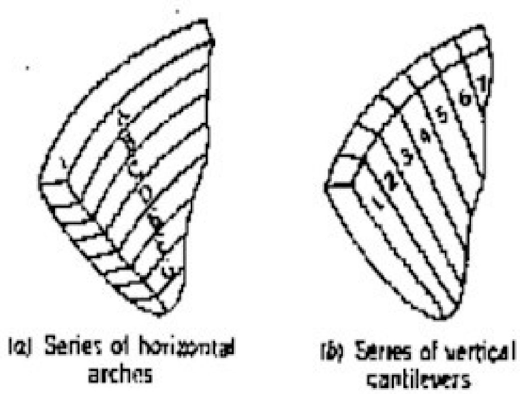
$$\text{F.S.} = \frac{[\tau_f]_{max}}{\tau_{max}} = \frac{129.8}{130.4} = 0.995 < 1.0 \qquad \text{(Hence, Unsafe)}$$

The foundation shear and F.S. can also be calculated below the u/s portion of dam soil, in the same manner as has been done for d/s slope portion, if required.

DESIGN OF ARCH DAM

- Dam, Barrage and a weir structure is a natural or man-made obstruction or barrier build across a river in-between the two ends of the river to raise the water, called head of the structure as well produce a massive storage.
- A barrage is built to divert this river water into the nearby link canals by regulating head for irrigation purpose.
- A dam is built to utilize the head for power generation and for storage.
- Dams are classified in different categories based on the type of material used; shape of the core structure; purpose of the project etc.
- Arch dam, as the name implies, is a curved obstruction from the upstream side singly spanned that mainly carries the load of the impounded water through arch action as well as cantilever action.
- Arch dams through arch action transfers portion of the load of the water thrust horizontally to the side abutments and the other portion of that load is transferred to the dam foundation vertically by cantilever action.
- In the arch action, hydrostatic pressure / force of water press against the face of the arch which in return compresses and strengthens the matrix of the arch dam structure.
- Arch Dams throughout the world are mostly made of concrete (either conventional concrete or roller compacted concrete); however, in the past some are also made with rubble and stone masonry.
- Let us now discuss in detail about the distribution of the load by an arch dam, which is briefly explained above;

- Let us consider an arch dam made up of two connected components one is series of arches and other is series of vertical cantilevers; as shown:-



Series of horizontal arches and vertical cantilevers

- The load caused due to thrust of impounded water in the arch dam is transferred to the abutments rested on solid / stable rocky side walls of the valley / canyon.
- Thus the load on the cantilever wall is reduced in arch dam as compared to that of the gravity dam. It is one of the major benefit due to which arch dam is considered economic.

Situations When Arch Dam is a must to use

1. Arch dam is proved most economical and efficient when the width of the canyon or valley to be spanned on the river is least.
2. As a major share of the impounding water thrust is taken by abutment walls resting on the sides of the canyon, thus these must be stable, strong and firm.
3. Arch dam can be used most economically on a terrain where width of the valley is less than 6 times of its height or in other words B/h ratio is less than 6.
4. If the area is remote such that the naturally available material are not enough to provide sufficient supply of concrete or earth-fill arch dam should be used as it needs minimum amount of construction concrete.
5. The slope of the adjoining hills for the abutment should be steep i.e. more than 45 degrees.
6. During the design of the arch dam it is considered that the stresses generated are upto that of allowable stresses of the concrete.

Advantages of Arch Dam – Few but long lasting

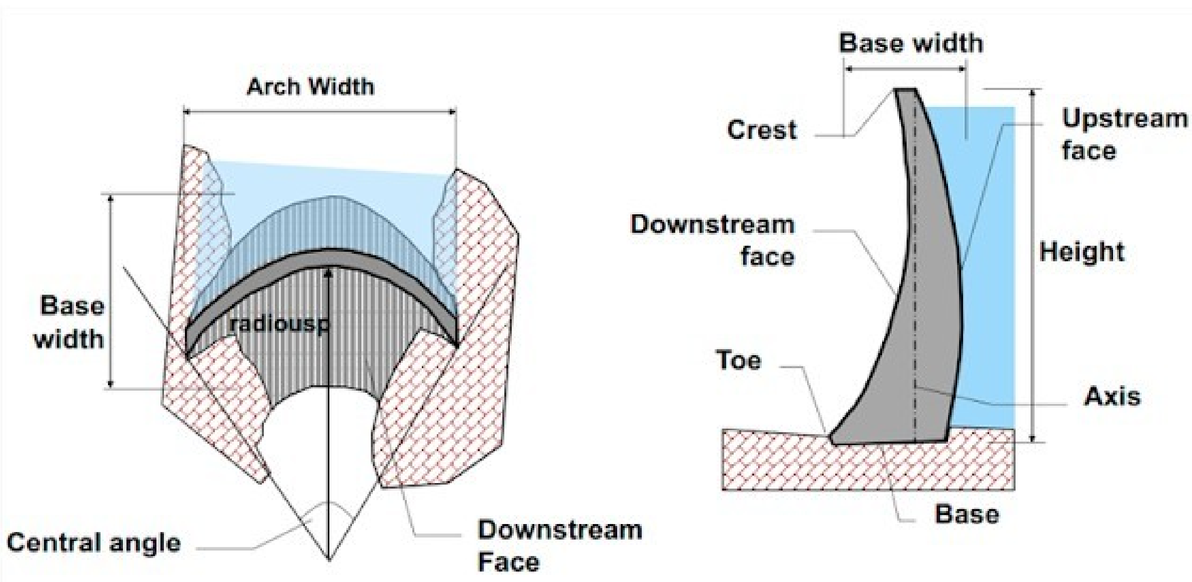
1. The major advantage derived from Arch dam is minimal amount of concrete / filling material required as the stresses of the thrust of water is taken care by both arch action and cantilever action requiring considerably small width at the bottom.
2. These dams are best suited for a narrow canyon passage and can store water as well as generate electricity.

3. Arch dams are particularly adapted to the gorges where the length is small in proportion to the height.
4. For a given height, the section of an arch dam is much lesser than a corresponding gravity dam.
5. Hence, an arch dam requires less material and is, therefore, cheaper.
6. Because of much less base width, the problems of uplift pressure are minor.
7. Since only a small part of water load is transferred to the foundation by cantilever action, an arch dam can be constructed in moderate foundations where gravity dam requiring sound foundation rock may be unsuitable.

Disadvantages of Arch Dam.

1. Arch dams are particularly adapted to the gorges where the length is small in proportion to the height.
2. For a given height, the section of an arch dam is much lesser than a corresponding gravity dam.
3. Hence, an arch dam requires less material and is, therefore, cheaper.
4. Because of much less base width, the problems of uplift pressure are minor.
5. Since only a small part of water load is transferred to the foundation by cantilever action, an arch dam can be constructed in moderate foundations where gravity dam requiring sound foundation rock may be unsuitable.

Components a Typical Arch dam is composed of



Above is a typical plan and cross-section of an arch dam showing typical components / parts of an arch dam.

Types of Arch Dams

Shell – Arch Dam – A famous and Aesthetic marvel

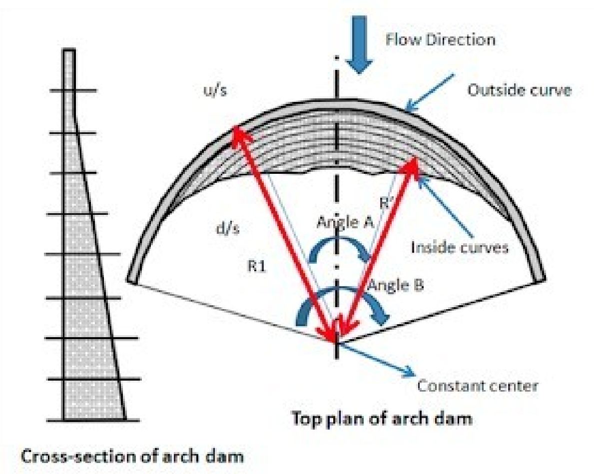
- Researchers have shown that greater the curvature of the arch dam in plan, the greater is the stresses on the abutments and thus lesser is the base width or thickness required.
- This economy can be further increased by providing curvature in the section making it a shell like or plate like structure.
- Such a non-vertical and shell like arch dam is termed as double curvature arch dam or shell-arch dam.
- Simple arch dams whose major part is distributed through the cantilever part of the arch dam can also be divided into different types as their faces can be either curvilinear or non-linear.
    1. Constant Radii Arch Dam
    2. Variable radii arch dam
    3. Constant angle arch dam

Constant Radii Arch Dam Best suited for U-Shaped Valley

The picture above clearly explains the definition of Constant Radius Arch dam;

In Constant Radius Arch Dam, the radius of the outside Circular Curve (as shown in fig R1) is constant throughout the height / elevation of the dam creating a linear upstream face of the dam.

However, the inner curves of the arch are of variable radii i.e. from top to bottom elevation of the arch dam the radius of the curves reduces creating a triangular cross-section of the arch dam as shown in the figure.



Constant Radii Arch Dam Best suited for U-Shaped Valley

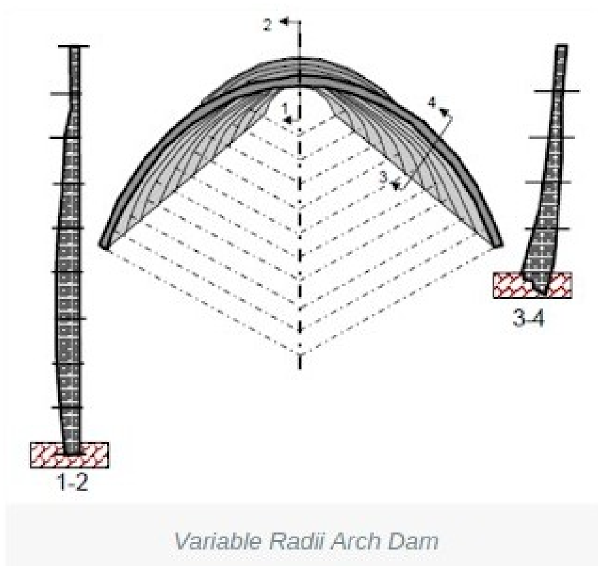
Constant Radii Arch Dam Best suited for U-Shaped Valley

- The increased thickness of the dam at the base will take care of the proportionally increasing hydrostatic thrust of the impounding water.

- It is to be noted here that the outside (upstream side) circular curves are sometimes termed as extrudes while that of inside curves are termed as intrudes.
- The constant Radii arch dam is sometimes referred as constant center arch dam; it is because of the fact that although the radii of the introdes decreases as we move down the elevation of the dam however the center of the curve is at the same line / point i.e. center of the curves are fixed.
- There is one more term associated with the arch dam which is central angle of the arch curves. If you see at the above figure; the central angle of the introdes decreases from top to bottom i.e. maximum central angle is at the top while minimum is at the bottom.
- Constant Radii or variable angle arch dams are most suitable for U-shaped valleys and is for easy construction providing vertical upstream face for efficient stress taking capacity.

Variable Radii Arch Dam Best Suited for V-shaped valley

- Now after reading Constant radii arch dam you might be able to define what is variable radii arch dam. If not let me do this for you; a Variable radii arch dam is the one in which the radius of the introdoes as well that of extrodos vary along the height.
- Making it maximum at the top and minimum at the bottom elevations. Along with the radius the central angle also become bigger and wider making it more effective and economical.



Variable Radii Arch Dam

- In a typical design of such a dam, the downstream face of the dam at the central line (crown) is vertical; while at all other locations, there is a batter on both the sides except at the abutments, where again, the upstream side becomes vertical.
- If overhangs are permitted, due to availability of stronger foundations, then the faces at the crown as well as abutments, may be provided with overhangs, affecting saving in the designed thickness.

- Evidently, since in such an arch dam, the centers of the various arch rings at different elevations, do not lie on the same vertical line; it is also known as variable center arch dam. Such dams are preferred for V-shaped valleys.

Constant Angle Arch Dams – An intermediate and most economic

- The constant angle arch dam is a special type of variable radius arch dam, in which the centeral angles of the horizontal arch rings are of the same magnitude at all elevations.
- The design of such a dam can, thus be made by adopting best central angle of 133 degrees and 34 minutes; and hence such a dam proves to be the most economical, out of the three types of ordinary arch dams.
- However, the design of such a dam usually involves providing overhangs at abutments, which require stronger foundations, and hence such a type cannot be used if the foundations are weak.

Design of Arch Dams – a complex hectic job

- As already explained above that the arch dam is a complex structure which is a bit difficult to design and construct.
- The design procedure adopted is a hit and trial type, a hydraulic dam design is proposed which is carried out through lengthy calculations for testing and checking through different criterions thus after several tries an economical, feasible and safe working design of the dam is developed.
- As far as the loads are concerned, the arch dam is designed for the same types of loads a concrete gravity dam is designed for. These loads includes :-
    - Water Pressure
    - Earthquake pressure
    - Wave pressure
    - Ice pressure
    - Temperature forces
    - Silt load
- However it is important here to understand that importance of above mentioned factor may be a bit for some types of loads and may be not for the other one.
- Like in case of arch dams we know the base width is comparatively very small to that of the concrete gravity dam, thus the uplift forces will be small.
- Mostly in the design of arch dams the uplift forces are neglected.

Arch dams are designed and engineered by three famous methods :-

1. Thin cylinder theory
2. Theory of elastic arches
3. The trial load method

Diversion headwork.

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

The purposes of diversion headwork.

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

LOCATION OF DIVERSION HEADWORKS

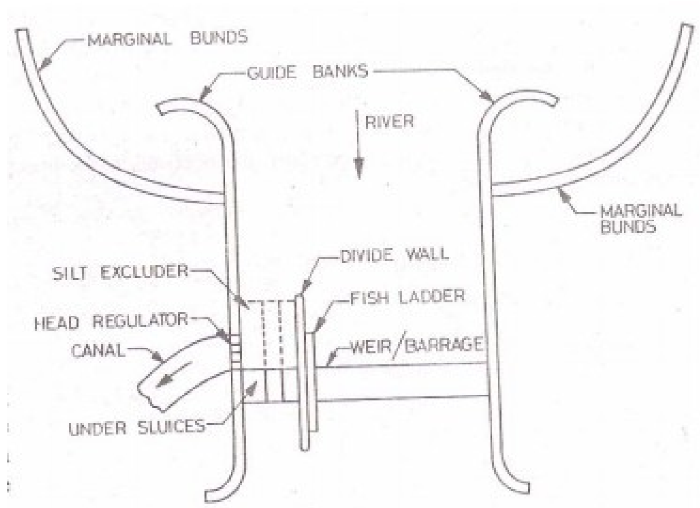
- The diversion headworks are generally located in the boulder stage or trough stage of the river at a site which is close to the commanded area of the offtaking canals.
- If there are a number of sites which are suitable, the final selection is done on the basis of cost.
- The site which gives the most economical arrangement for the diversion head works and the distribution works (canals) is usually selected.

1. The river section at the site should be narrow and well-defined.
2. The river should have high, well-defined, inerodible and non-submersible banks so that the cost of river training works is minimum.
3. The canals taking off from the diversion head works should be quite economical and should have a large commanded area.
4. There should be suitable arrangement for the diversion of river during construction.
5. The site should be such that the weir (or barrage) can be aligned at right angles to the direction of flow in the river.
6. There should be suitable locations for the undersluices, head regulator and other components of the diversion headworks.
7. The diversion headworks should not submerge costly land and property on its upstream.
8. Good foundation should be available at the site.
9. The required materials of construction should be available near the site.
10. The site should be easily accessible by road or rail.
11. The overall cost of the project should be a minimum.

COMPONENT PARTS OF A DIVERRSION HEADWORK

A diversion headwork consist of the following component parts

1. Weir or barrage
2. Undersluices
3. Divide wall
4. Fish ladder
5. Canal head regulator
6. pocket or approach channel
7. Silt excluders/ Silt prevention devices/
8. River training works (Marginal bunds and guide banks)

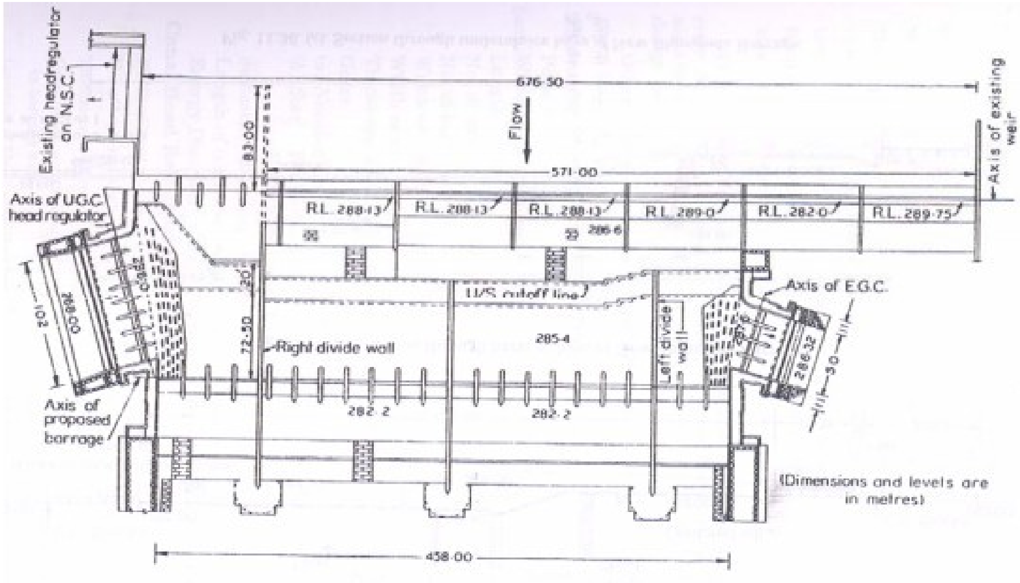


Undersluices

- Undersluice sections are provided adjacent to the canal head regulators.
- The undersluices should be able to pass fair weather flow for which the crest shutters on the weir proper need not be dropped.
- The crest level of the undersluices is generally kept at the average bed level of the river.

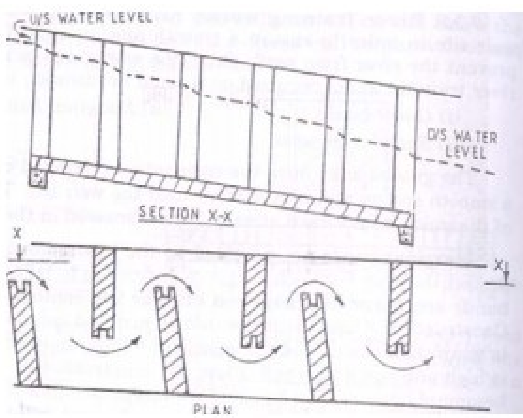
Divide Wall

- A divide wall is a wall constructed parallel to the direction of flow of river to separate the weir section and the undersluices section to avoid cross flows.
- If there are undersluices at both the sides, there are two divide walls.



Fish Ladder

- A fish ladder is a passage provided adjacent to the divide wall on the weir side for the fish to travel from the upstream to the downstream and vice versa.
- Fish migrate upstream or downstream of the river in search of food or to reach their sprawling places.
- In a fish ladder the head is gradually dissipated so as to provide smooth flow at sufficiently low velocity.
- Suitable baffles are provided in the fish passage to reduce the flow velocity.



Canal Head Regulator

- A canal head regulator is provided at the head of the canal offtaking from the diversion headworks.
- It regulates the supply of water into the canal, controls the entry silt into the canal, and prevents the entry of river floods into canal.

Silt Excluder

- A silt excluder is a structure in the undersluices pocket to pass the silt laden water to the downstream so that only clear water enters into the canal through head regulator.

- The bottom layer of water which are highly charged with silt pass down the silt excluder an escape through the undersluices.

Guide Banks and Marginal Bunds

- Guide banks are provided on either side of the diversion headworks for a smooth approach and to prevent the river from outflanking.

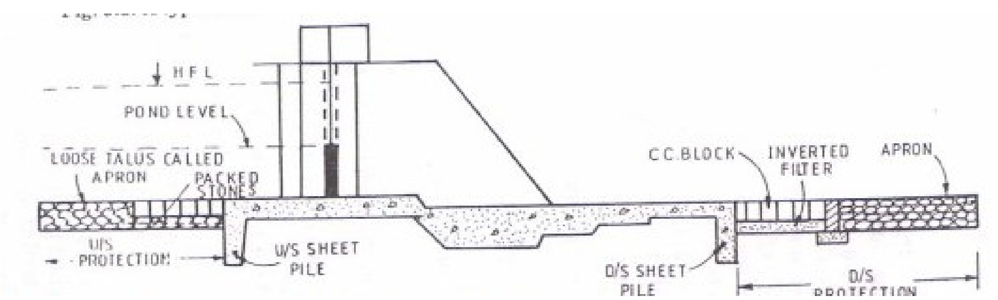
- Marginal bunds are provided on either side of the river upstream of diversion headworks to protect the land and property which is likely to be submerged during ponding of water in floods.

- **Weir or Barrage**

- A diversion head works is a structure constructed across a river for the purpose of raising water level in the river so that it can be diverted into the offtaking canals.

- A weir is a raised concrete crest wall constructed across the river.

- It may be provided with small shutters (gates) on its top. In the case of weir, most of the raising of water level or ponding is done by the solid weir wall and little with by the shutters.



- A barrage has a low crest wall with high gates. As the height of the crest above the river bed is low most of the ponding is done by gates. During the floods the gates are opened so afflux is very small.

- A weir maintains a constant pond level on its upstream side so that the water can flow into the canals with the full supply level (F.S.L.). If the difference between the pond level and the crest level is less than 1·5 m or so, a weir is usually constructed.

- On the other hand, if this difference is greater than 1·50 m, a gate-controlled barrage is generally more suitable than a weir. In the case of a weir, the crest shutters are dropped during floods so that the water can pass over the crest.

- During the dry period, these shutters are raised to store water upto the pond level. Generally, the shutters are operated manually, and there is no mechanical arrangement for raising or dropping the shutters.

- On the 'other hand, in the case of a barrage, the control of pondage and flood discharge is achieved with the help of gates which are mechanically operated

ADVANTAGES AND DISADVANTAGES OF WEIRS AND BARRAGES

1. Weirs Advantages: The initial cost of weirs is usually low.

Disadvantages:

i. There is a large afflux during floods which causes large submergence.

ii. Because the crest is at high level, there is great silting problem

iii. The raising and lowering of shutters on the crest is not convenient. Moreover, it requires considerable time and labour.

iv. The weir lacks an effective control on the river during floods.

2. Barrages Advantages

i. The barrage has a good control on the river during floods. The outflow can be easily regulated by gates.

ii. The afflux during floods is small and, therefore, the submerged area is less.

iii. There is a good control over silt entry into the canal.

iv. There is a good control over flow conditions, shoal formations and crosscurrents on the upstream of the barrage.

v. There are better facilities for inspection and repair of various structures.

vi. A roadway can be conveniently provided over the structure at a little additional cost.

Disadvantages:

The initial cost of the barrage is quite high.

Conclusion: A barrage is generally better than a weir. Most of the diversion headworks these days usually consist of barrages.

TYPES OF WEIRS

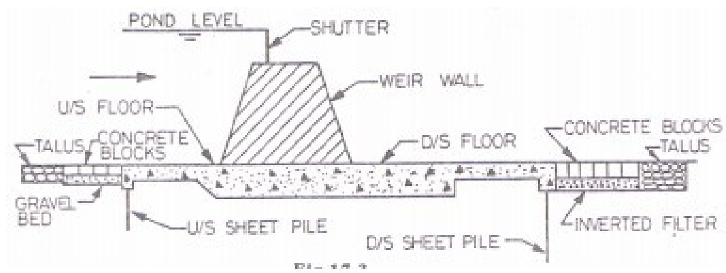
The weirs may be broadly divided into the following types

1. Vertical drop weirs.
2. Rockfill weirs.
3. Concrete glacis or sloping weirs.

1. Vertical drop weirs

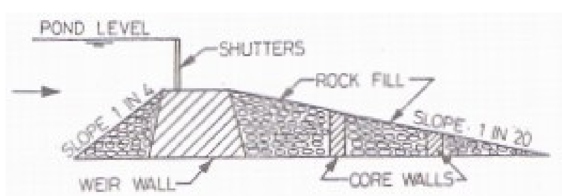
- A vertical drop weir consists of a masonry wall with a vertical (or nearly vertical) downstream face and a horizontal concrete floor.

- The shutters are provided at the crest, which are dropped during floods so as to reduce afflux. The water is ponded upto the top of the shutters during the rest of the period.
- Vertical drop weirs were quite common in early diversion headworks, but these are now becoming more or less obsolete.
- The vertical drop weir is suitable for hard clay foundation as well as consolidated gravel foundations, and where the drop is small.
- The upstream and downstream cutofIwalls (or piles) are provided upto the scour depth. The weir floor is designed as a gravity section.



2. Rockfill weirs:

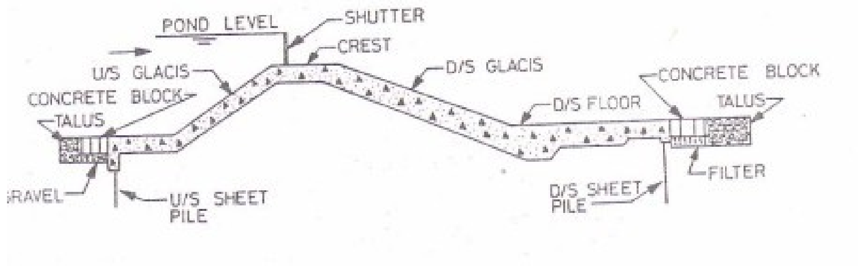
- In a rockfill type weir, in addition to the main weir wall, there are a number of core walls. The space between the core walls is filled with the fragments of rock (called rockfill).
- A rockfill weir requires a lot of rock fragments and is economical only when a huge quantity of rockflll is easily available near the weir site.
- It is suitable for fine sand foundation. The old Okhla Weir across the Yamuna river is a rockfill weir.
- Such weirs are also more or less obsolete these days.



3. Concrete sloping weir :

- Concrete sloping weirs (or glacis weirs) are of relatively recent origin.
- The crest has glacis (sloping floors) on upstream as well as downstream.
- There are sheet piles (or cut off walls) driven upto the maximum scour depth at the upstream and downstream ends of the concrete floor.
- Sometimes an intermediate pile is also driven at the beginning of the upstream glacis or at the end of downstream glacis.

- The main advantage of a sloping weir over the vertical drop weir is that a hydraulic jump is formed on the d/s glacis for the dissipation of energy.
- Therefore, the sloping weir is quite suitable for large drops.



Modes of Failure :

- Irrigation structures (or hydraulic structures) for the diversion and distribution works are weirs, barrages, head regulators, distributary head regulators, cross regulators, cross-drainage works, etc.
- These structures are generally founded on alluvial soils which are highly pervious. Moreover, these soils are easily scoured when the high velocity water passes over the structures.
- The failures of weirs constructed on the permeable foundation may occur due to various causes, which may be broadly classified into the following two categories:

  1. Failure due to- subsurface flow

  2. Failure due to surface flow

1. Failure due to subsurface flow:

The failure due to subsurface flow may occur by piping or by rupture of floor due to uplift.

(a) Failure by piping:

- Piping (or undermining) occurs below the weir if the water percolating through the foundation has a large seepage force when it emerges at the downstream end of the impervious floor.
- When the seepage force exceeds a certain value, the soil particles are lifted up at the exit point of the seepage.
- With the removal of the surface soil particles, there is further concentration of flow in the remaining portion and more soil particles are removed.
- This process of backward erosion progressively extends towards the upstream side, and a pipe-like hollow formation occurs beneath the floor.
- The floor ultimately subsides in the hollows so formed and fails. This type of failure is known as piping failure.

(b) Failure by rupture of floor:

- The water percolating through the foundation exerts an upward pressure on the impervious floor, called the uplift pressure.
- If the weight of the floor is not adequate to counterbalance the uplift pressure, it may fail by rupture.

2. Failure due to surface flow

The failure due to surface flow may occur by suction pressure due to hydraulic jump or by scouring of the bed.

(a) Failure by suction pressure :

- In the glacis type of weirs, a hydraulic jump is formed on the d/s glacis. In this case, the water surface profile in the hydraulic jump trough is much lower than the subsoil H.G.L.
- Therefore uplift pressure occurs on the glacis. This uplift pressure is known as the suction pressure. If the thickness of floor is not adequate, the rupture of floor may occur.

(b) Failure by scour :

- During floods, scouring occurs in the river bed. The bed of the river may be scoured to a considerable depth.
- If no suitable measures are adopted, the scour may cause damage to the structure and may lead to the failure.

Design aspects

The basic principles for the design of all irrigation structures on pervious foundations are as follows:

(a) Subsurface flow

1. The structure should be designed such that the piping failure does not occur due to subsurface flow.
2. The downstream pile must be provided to reduce the exit gradient and to prevent piping.
3. An impervious floor of adequate length is provided to increase the path of percolation and to reduce the hydraulic gradient and the seepage force.
4. The seepage path is increased by providing piles and impervious floor to reduce the uplift pressure.
5. The thickness of the floor should be sufficient to resist the uplift pressure due to subsurface flow. The critical section is d/s of the weir/crest wall.

6. A suitably graded inverted filter should be provided at the downstream end of the impervious floor to check the migration of soil particles along with water. The filter layer is loaded with concrete blocks. Concrete blocks are also provided at the upstream end.

(b) Surface flow

1. The piles (or cutoff walls) at the upstream and downstream ends of the impervious floor should be provided upto the maximum scour level to protect the main structure against scour.

2. The launching aprons should be provided at the upstream and downstream ends to provide a cover to the main structure against scour.

3. A device is required at the downstream to dissipate energy. For large drops, hydraulic jump is used to dissipate the energy.

4. Additional thickness of the impervious floor is provided at the point where the hydraulic jump is formed to counterbalance the suction pressure.

5. The floor is constructed as a monolithic structure to develop bending resistance (or beam action) to resist the suction pressure.

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UNIT – 4

CANAL IRRIGATION

Canal regulations – direct sluice - Canal drop – Cross drainage works-Canal outlets – Design of prismatic canal-canal alignments-Canal lining - Kennedy's and Lacey's Regime theory-Design of unlined canal

CANAL:-

A canal is an artificial channel generally trapezoidal in shape constructed on the ground to carry water to the field either from the river of from a reservoir.

Canal regulations

Any structure constructed to regulate the discharge, full supply level or velocity in a canal is known as Regulation Work.

Types & Location:

1. Head Regulator or Head Sluice → at Barrage/Weir, Dam
2. Cross Regulator → on Parent Canal
3. Distributory Head Regulator → on Off-take Canal
4. Canal Fall → along Parent Canal or Off-take Canal
5. Canal Escape → on any type of canal
6. Canal Outlet → on Distributing Canal

Types & Purpose:

1. Head Regulator or Head Sluice → to divert water to parent channel from a barrage or weir
2. Cross Regulator → to head up water in the parent channel to divert some of it through an off take channel or distributory canal
3. Distributory Head Regulator → to control the amount of water flowing in to off take channel
4. Canal Fall → to lower the water level of the canal
5. Canal Escape → to allow release of excess water from the canal system
6. Canal Outlet → to take out water for delivery to the field channel or water courses

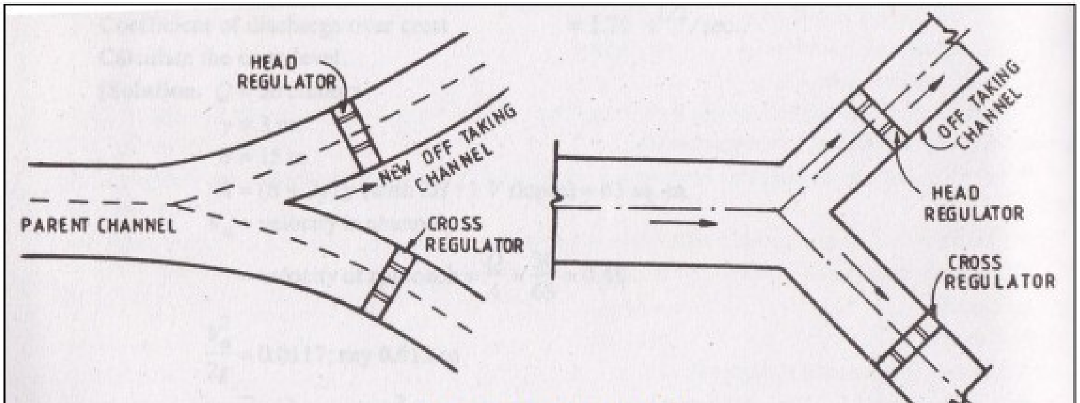


Fig: Canal Regulators

A head regulator provided at the head of the off-taking channel, controls the flow of water entering the new channel.

While a cross regulator may be required in the main channel downstream of the off-taking channel, and is operated when necessary so as to head up water on its upstream side, thus to ensure the required supply in the off-taking channel even during the periods of low flow in the main channel.

Main functions of a head regulator:

1. To regulate or control the supplies entering the off-taking canal
2. To control the entry of silt into the off-taking canal
3. To serve as a meter for measuring discharge.

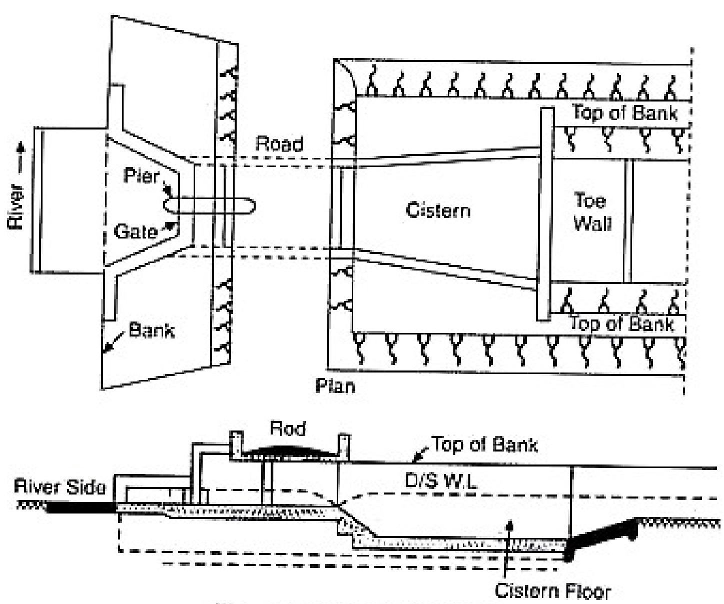


Fig. 12.11. Head regulator

- It consists of a raised crest with abutments on both sides. The crest may be subdivided in various bays by providing piers on the crest.

- The piers support roadway and a platform for operating gates.
- The gates control the flow over the crest. They are housed and operated in grooves made in the abutments and piers. Sill of the regulator crest is raised to prevent silt entry.
- Sometimes the gates are provided in tiers. Then lower tiers may be kept closed to raise the sill of the regulator.
- The head regulator is generally constructed with masonry. It should be founded on a good rock foundation. It should be safe against shear, sliding and overturning.
- It should be flanked with adequate wing walls. The head regulator should also be given proper protection by providing aprons on upstream and downstream side of the barrel.
- To prevent seepage cutoff is also essential. To take irrigation water at low velocities waterway of the head regulator should be sufficiently big.

Main functions of a cross regulator:

1. To control the entire Canal Irrigation System.
2. To help in heading up water on the upstream side and to fed the off-taking canals to their full demand.
3. To help in absorbing fluctuations in various sections of the canal system, and in preventing the possibilities of breaches in the tail reaches.
4. Cross regulator is often combined with bridges and falls, if required.

Canal Escapes:

It is a side channel constructed to remove surplus water from an irrigation channel (main canal, branch canal, or distributary etc.) into a natural drain.

The water in the irrigation channel may become surplus due to -

- Mistake
- Difficulty in regulation at the head
- Excessive rainfall in the upper reaches
- Outlets being closed by cultivators as they find the demand of water is over

Functions of Distributary Head Regulator:

- It is a hydraulic structure constructed at the head of a distributary. This regulator performs the same functions as that of a head regulator.

i. It regulates the supply of the distributary.

ii. It can be used many times as a meter.

iii. It is also a silt selective structure.

iv. Distributary head regulator controls the flow in the distributary. By closing the gates distributary can be dried to carry out repairs or maintenance works.

- The points to be considered in design are similar to those considered in the design of a head regulator.
- Only difference is that the distributary head regulator is much smaller in magnitude as compared to the head regulator.

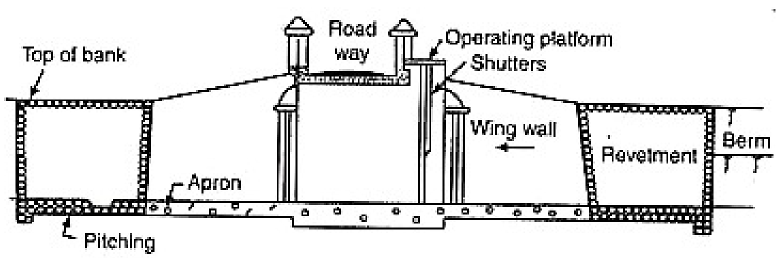
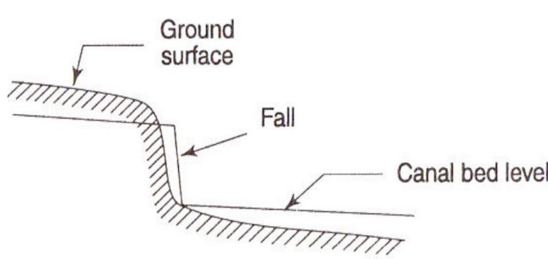


Fig. 12.13. Distributary head regulator

Canal drop or Fall

A canal fall or drop is an irrigation structure constructed across a canal to lower down its bed level to maintain the designed slope when there is a change of ground level to maintain the designed slope when there is change of ground level.

This falling water at the fall has some surplus energy. The fall is constructed in such a way that it can destroy this surplus energy



Types of Canal Fall:

1. Ogee Fall - to provide smooth transition and to reduce disturbance and impact

2. Rapid Fall - consists of a glacis sloping at 1: 0 to 1:20. Very high cost of construction

3. Stepped Fall - next development of rapid fall. Cost of construction is high

4. Notch Fall - the fall is consists of one or more trapezoidal notches

5. Vertical Drop Fall - high velocity jet enters the deep pool of water in the cistern and dissipation of energy is affected by turbulent diffusion

6. Glacis Type Fall - utilizes standing wave phenomenon for dissipation of energy

Types:

a) Straight Glacis Type

b) Parabolic Glacis Type of Montague type

Ogee Fall

In this type of fall, an ogee curve (a combination of convex curve 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 the canal.

- The fall consists of a concrete vertical wall and concrete bed.
- Over the concrete bed the rubble masonry is provided in the shape of ogee curve.
- The surface of the masonry is finished with rich cement mortar (1:3).
- The upstream and downstream side of the fall is protected by stone pitching with cement grouting.
- The design consideration of the ogee fall depends on the site condition.

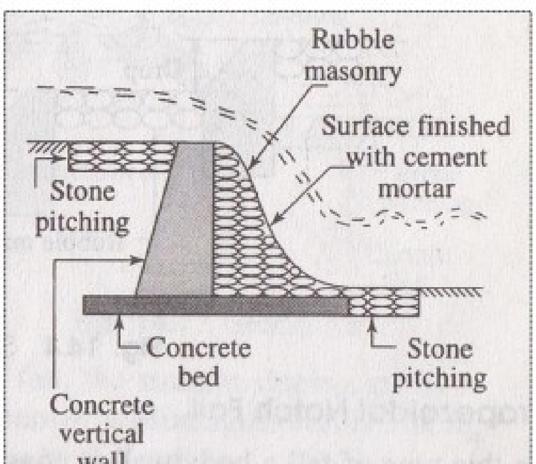


Fig: Ogee Fall

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 from 1 in 10 to 1 in 20.

- Curtain walls are provided on the upstream and downstream side of the sloping glacis.
- The sloping bed is provided with rubble masonry.
- The upstream and downstream side of the fall is also protected by rubble masonry.
- The masonry surface is finished with rich cement mortar (1: 3).

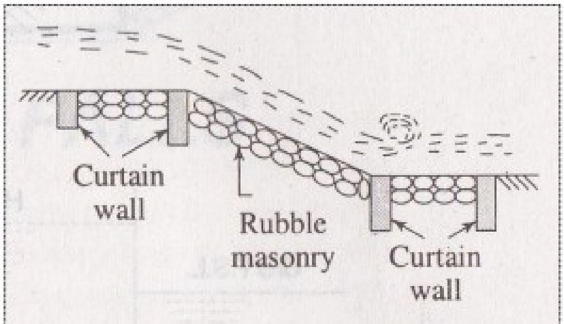


Fig: Rapid Fall

Stepped Fall

Stepped fall consists of a series of vertical drops 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 flowing water may not cause any damage to the canal bed. Brick walls are provided at each of the drops.
- The bed of the canal within the fall is protected by rubble masonry with surface finishing by rich cement mortar (1:3).

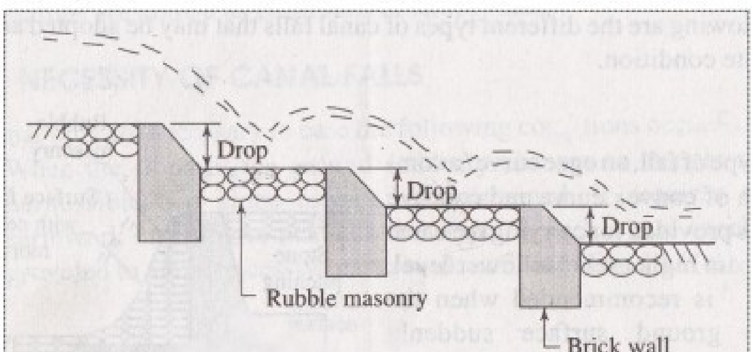


Fig: Stepped Fall

Trapezoidal Notch 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 the notches are kept at the upstream bed level of the canal.

- The body wall is constructed with masonry or concrete.
- An impervious floor is provided to resist the scoring effect of the falling water.
- The upstream and downstream side of the fall is protected by stone pitching finished by cement grouting.
- The size and number of notches depends upon the full supply discharge of the canal.

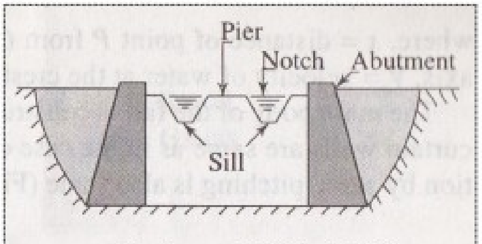


Fig: Trapezoidal Notch Fall

Vertical Drop Fall

It consists of a vertical drop walls which is constructed with masonry work. The water flows over the crest of the wall. A water eastern is provided on the downstream side which acts as a water cushion to dissipate the energy of falling water.

- A concrete floor is provided on the downstream side to control the scouring effect of the flowing water.
- Curtain walls are provided on the upstream and downstream side.
- Stone pitching with cement grouting is provided on the upstream and downstream side of the fall to protect it from scouring.

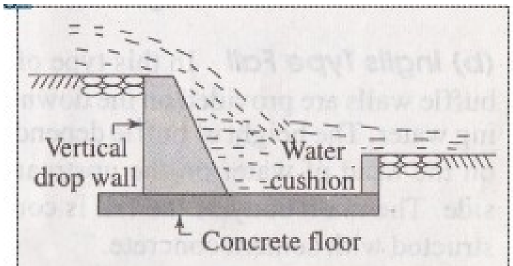


Fig: Vertical Drop Fall

Glacis Fall

It consists of a straight sloping glacis provided with a crest. A water cushion is provided on the downstream side to dissipate the energy of flowing water.

- The sloping glacis is constructed with cement concrete.
- Curtain walls and toe walls are provided on the upstream and downstream side.

- The space between the toe walls and curtain walls is protected by stone pitching.
- This type of fall is suitable for drops up to 1.5 m.

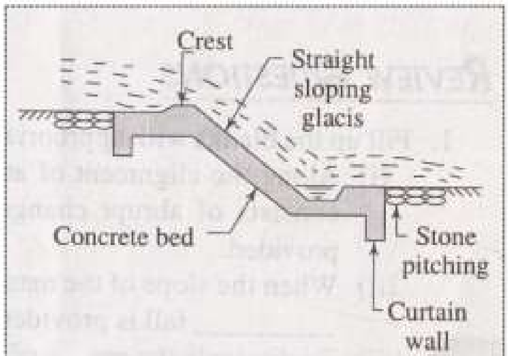


Fig: Glacis Fall

For the improvement in energy dissipation, the glacis falls have been modified as follows:

(a) Montague Type Fall

In this type of fall, the straight sloping glacis is modified by giving parabolic shape which is known as Montague profile. Taking "0" as the origin, the Montague profile is given by the equation,

$$X = v\sqrt{4\frac{y}{g} + Y}$$

Where, x = distance of point P from OX axis,

Y = distance of point P from OY axis,

υ = velocity of water at the crest,

g = acceleration due to gravity

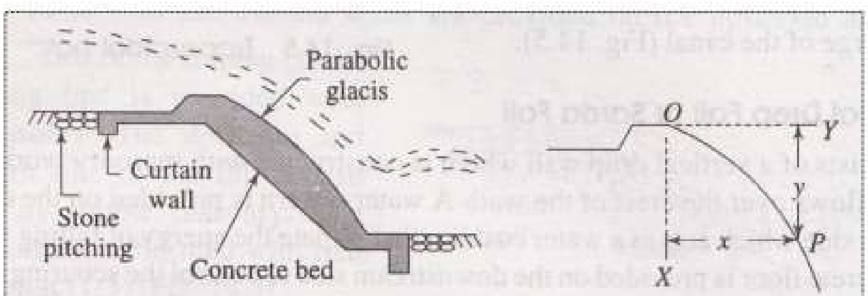


Fig: Montague Type Fall

The main body of the fall is constructed with cement concrete. Toe walls and curtain walls are same as in the case of straight sloping glacis. The bed protection by stone pitching is also same.

(b) Inglis Type Fall

In this type of fall, the gracis is straight and sloping, but buffle walls are provided on the downstream floor to dissipate the energy of flowing water.

- The height of buffle depends on the head of water on the upstream side.
- The main body of the fall is constructed with cement concrete.
- The toe walls and curtain walls are same as straight glacis.
- The protection works with stone pitching are also same. Sometimes, this fall is known as buffle fall.

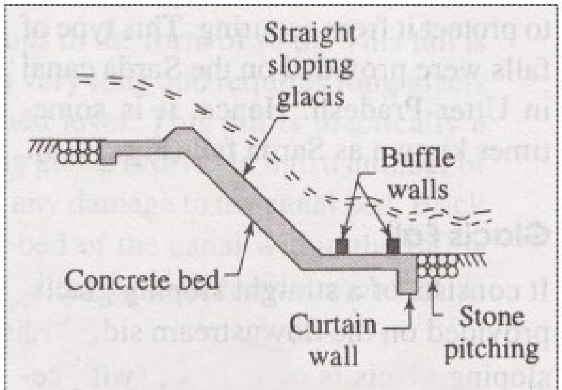


Fig: Inglis Type Fall

Cross drainage works

- In an irrigation project, when the network of main canals, branch canals, distributaries, etc. Are provided, then these canals may have to cross the natural drainages like rivers, streams, nallahs, etc at different points within the command area of the project.
- The crossing of the canals with such obstacle cannot be avoided. So, suitable structures must be constructed at the crossing point for the easy flow of water of the canal and drainage in the respective directions. These structures are known as cross-drainage works.

Necessity of Cross-drainage works:

- 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, the cross drainage works must be provided for running the irrigation system.
- At the crossing point, the water of the canal and the drainage get intermixed. So, far the smooth running of the canal with its design discharge the cross drainage works are required.
- The site condition of the crossing point may be such that without any suitable structure, the water of the canal and drainage can not be diverted to their natural directions. So, the cross drainage works must be provided to maintain their natural direction of flow.

Types of Cross-Drainage Works:

(1) Type I (Irrigation canal passes over the drainage)

a) Aqueduct

b) Siphon aqueduct

(2) Type II (Drainage passes over the irrigation canal)

a) Super passage

b) Siphon super passage

(3) Type III (Drainage and canal intersection each other of the same level)

a) Level Crossing

b) Inlet and outlet

Selection of type of cross-drainage works

- Relative bed levels
- Availability of suitable foundation
- Economical consideration
- Discharge of the drainage
- Construction problems

Aqueduct

- The aqueduct is just like a bridge where a canal is taken over the deck supported by piers instead of a road or railway.
- Generally, the canal is in the shape of a rectangular trough which is constructed with reinforced cement concrete. Sometimes, the trough may be of trapezoidal section.
- An inspection road is provided along the side of the trough.
- The bed and banks of the drainage below the trough is protected by boulder pitching with cement grouting.
- The section of the trough is designed according to the full supply discharge of the canal.
- A free board of about 0.50 m should be provided.
- The height and section of piers are designed according to the highest flood level and velocity of flow of the drainage.
- The piers may be of brick masonry, stone masonry or reinforced cement concrete.
- Deep foundation (like well foundation) is not necessary for the piers. The concrete foundation may
- be done by providing the depth of foundation according to the availability of hard soil.

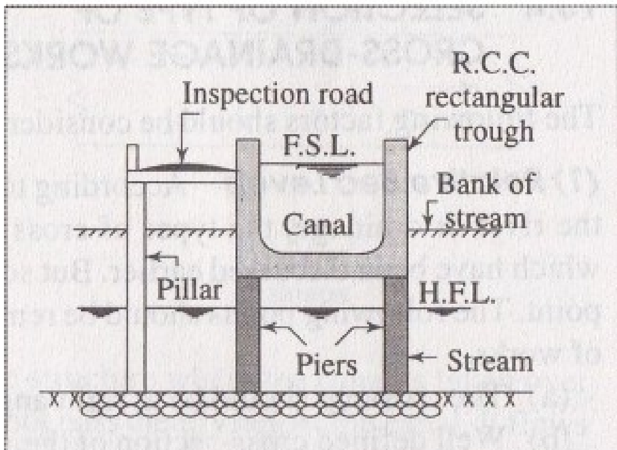


Fig: Aqueduct

Siphon Aqueduct

- The siphon aqueduct, the bed of the drainage is depressed below the bottom level of the canal trough by providing sloping apron on both sides of the crossing.
- The sloping apron may be constructed by stone pitching or cement concrete.
- The section of the drainage below the canal trough is constructed with cement concrete in the form of tunnel. This tunnel acts as a siphon.
- Cut off walls are provided on both sides of the apron to prevent scouring.
- Boulder pitching should be provided on the upstream and downstream of the cut-off walls.
- The other components like canal trough, piers, inspection road, etc. should be designed according to the methods adopted in case of aqueduct.

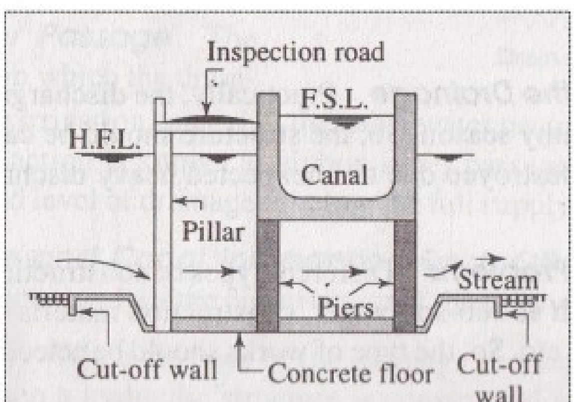


Fig: Siphon Aqueduct

Super Passage

- The super passage is just opposite of the aqueduct. In this case, the bed level of the drainage is above the fully supply level of the canal.
- The drainage is taken through a rectangular or trapezoidal trough of channel which is constructed on the deck supported by piers.

- The section of the drainage trough depends on the high flood discharge.
- A free board of about 1.5 m should be provided for safety.
- The trough should be constructed of reinforced cement concrete.
- The bed and banks of the canal below the drainage trough should be protected by boulder pitching or lining with concrete slabs.
- The foundation of the piers will be same as in the case of aqueduct.

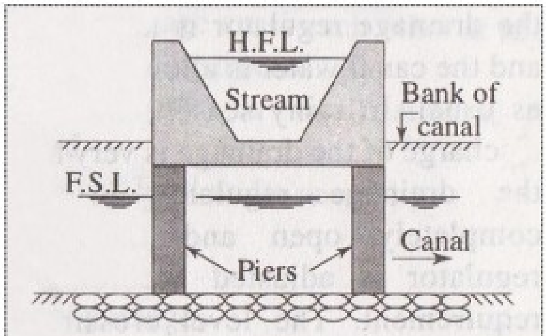


Fig: Super Passage

Siphon Super Passage

- It is just opposite siphon aqueduct. In this case, the canal passes below the drainage trough. The section of the trough is designed according to high flood discharge.
- The bed of the canal is depressed below the bottom level of the drainage trough by providing sloping apron on both sides of the crossing.
- The sloping apron may be constructed with stone pitching or concrete slabs.
- The section of the canal below the trough is constructed with cement concrete in the form of tunnel which acts as siphon.
- Cut-off walls are provided on upstream and downstream side of sloping apron.
- Other components are same as in the case of siphon aqueduct.

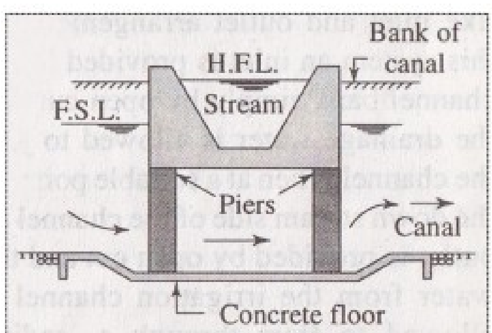


Fig: Siphon Super Passage

Level Crossing

The level crossing is an arrangement provided to regulate the flow of water through the drainage and the canal when they cross each other approximately at the same bed level.

The level crossing consists of the following components:

Crest Wall: It is provided across the drainage just at the upstream side of the crossing point. The top level of the crest wall is kept at the full supply level of the canal.

Drainage Regulator: It is provided across the drainage just at the downstream side of the crossing point. The regulator consists of adjustable shutters at different tiers.

Canal Regulator: It is provided across the canal just at the downstream side of the crossing point. This regulator also consists of adjustable shutters at different tiers.

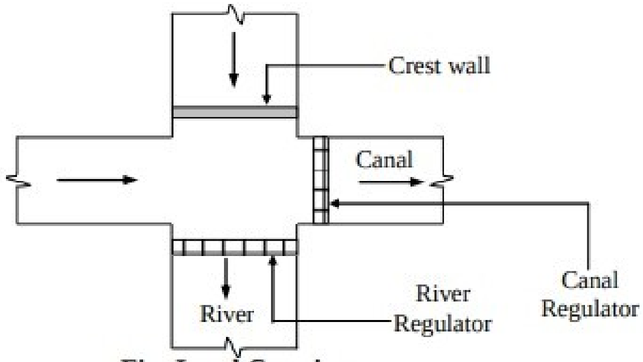


Fig: Level Crossing

Inlet and outlet

- In the crossing of small drainage with small channel no hydraulic structure is constructed. Simple openings are provided for the flow of water in their respective directions. This arrangement is known as inlet and outlet.
- In this system, an inlet is provided in the channel bank simply by open cut and the drainage water is allowed to join the channel
- At the points of inlet and outlet, the bed and banks of the drainage are protected by stone pitching.

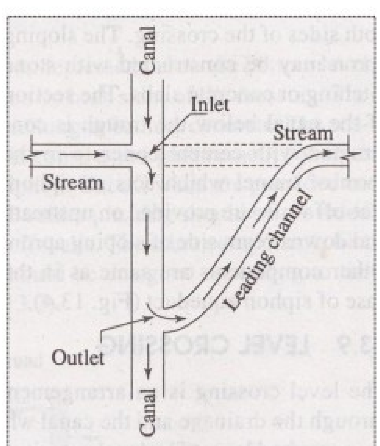


Fig: Inlet and outlet

Canal Outlets/Modules:

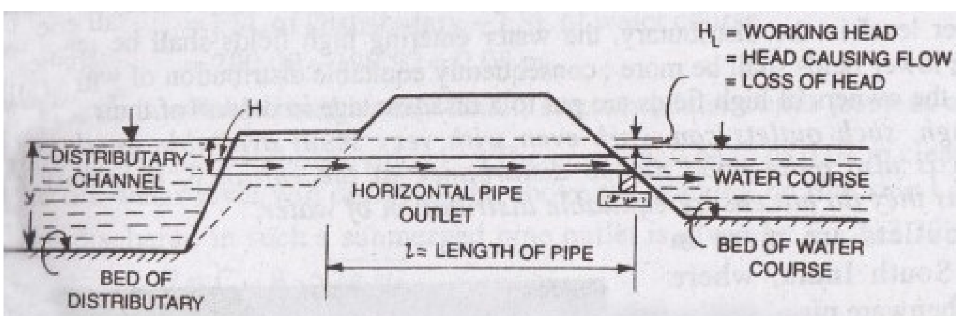
- A canal outlet or a module is a small structure built at the head of the water course so as to connect it with a minor or a distributary channel.
- It acts as a connecting link between the system manager and the farmers.

Requirements of a good module:

- It should fit well to the decided principles of water distribution.
- It should be simple to construct.
- It should work efficiently with a small working head.
- It should be cheaper.
- It should be sufficiently strong with no moving parts, thus avoiding periodic maintenance.
- It should e such as to avoid interference by cultivators.
- It should draw its fair share of silt.

Types of Outlet/modules:

(a) Non-modular modules:



Non-modular modules are those through which the discharge depends upon the head difference between the distributary and the water course.

Common examples are:

(i) Open sluice

(ii) Drowned pipe outlet

(b) Semi-modules or Flexible modules:

Due to construction, a super-critical velocity is ensured in the throat and thereby allowing the formation of a jump in the expanding flume.

The formation of hydraulic jump makes the outlet discharge independent of the water level in water course, thus making it a semi module.

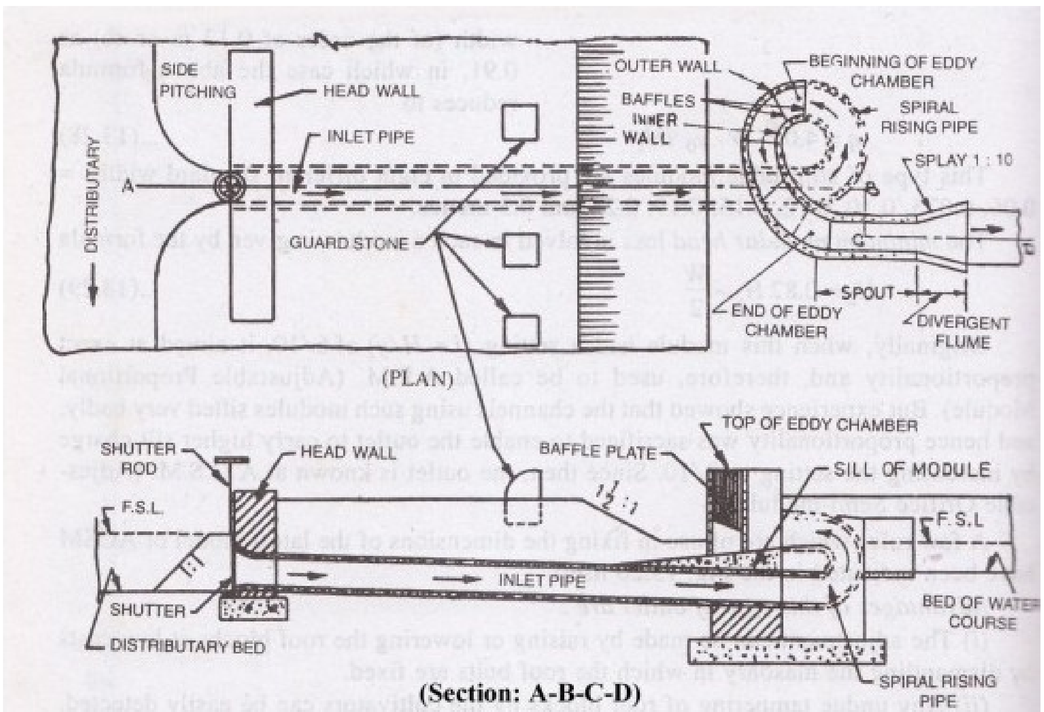
Semi-modules or flexible modules are those through which the discharge is independent of the water level of the water course but depends only upon the water level of the distributary so long as a minimum working head is available.

Examples are pipe outlet, open flume type etc.

(c) Rigid modules or Modular Outlets:

Rigid modules or modular outlets are those through which discharge is constant and fixed within limits, irrespective of the fluctuations of the water levels of either the distributary or of the water course or both.

An example is Gibb's module:



Design of prismatic canal

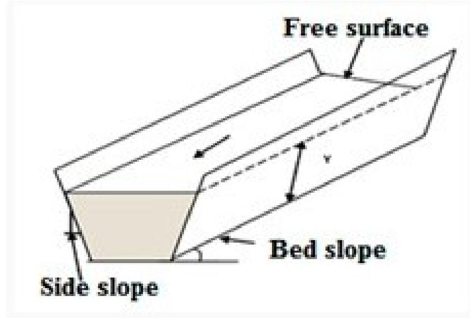
Open Channels

Irrigation water is conveyed in either open channel or closed conduits.

Open channels receive water from natural streams or underground water and convey water to the farm for irrigation.

Open channels have free surface. The free surface is subjected to atmospheric pressure.

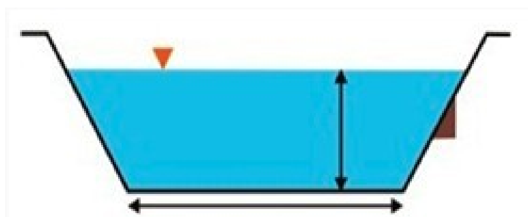
The basic equations used for water flow in open channels are continuity equation, Bernoulli equation and Darcy Weisbach equation.



A trapezoidal shaped open channel

Prismatic and Non-Prismatic Channels

- A channel in which the cross sectional shape, size and the bottom slope are constant over long stretches is termed as prismatic channel.
- Most of the man-made or artificial channels are prismatic channels.
- The rectangular, trapezoidal, triangular and half-circular are commonly used shapes in manmade channels.
- All natural channels generally have varying cross section and consequently are nonprismatic.



Sketch of a prismatic channel

Designo of Open Channel

- Open Channel is a passage through which water flows and has upper surface exposed to atmosphere.
- Open channel design involves determining cross-section dimensions of the channel for the amount of water the channel must carry (i.e., capacity) at a given flow velocity, slope and, shape or alternatively determining the discharge capacity for the given cross-section dimensions.

The terminologies used in the design of open channels of different geometry are given below:

i) Area of Cross Section (a):
Area of cross section of for a rectangular cross section, of wetted section. For a rectangular cross section, if b = width of channel and y = depth of water, the area of wetted section of channel (a) = b.y.
ii) Wetted Perimeter (p):
It is the sum of the lengths of that part of the channel sides and bottom which are in contact with water. The wetted perimeter (p) = b+2y.
iii) Hydraulic Radius (R):
It is the ration of area of wetted cross section to wetted perimeter. The hydraulic radius

$$(R) = \frac{a}{p} = \frac{by}{b+2y}$$

iv) Hydraulic Slope (S):

It is the ratio of vertical drop in longitudinal channel section (h) to the channel length (l). Hydraulic slope

$$(S) = \frac{h}{l}$$

v) Freeboard:

- It is the vertical distance between the highest water level anticipated in channel flow and the top of the retaining banks.
- This is provided to prevent over topping of channel embankments or damage due to trampling. This is provided between 15.25% of normal depth of flow.

Discharge Capacity of Channel

Channel capacity can be estimated by equation given as:

$$Q = \frac{(16667)(DDIR)(A)}{(HPD)(Ei)}$$

where,

Q = channel capacity (L/min)

DDIR = design daily irrigation requirement (mm/day)

A = irrigated area supplied by canal or ditch (ha)

HPD = hours per day that water is delivered

Ei= irrigation efficiency including conveyance efficiency of canal or ditch (percent).

- The velocity of flow in a canal or ditch should be non erosive and non silting that prevent the deposition of suspended substances.
- Normally flow velocity in excess of 0.6 m/s is non silting (Schwab et al., 1993).
- The maximum velocity that does not cause excessive erosion depends on the erodibility of the soil or lining material.

Economical Section of a Channel

- A channel section is said to be economical when the cost of construction of the channel is minimum.
- The cost of construction of a channel depends on depth of excavation and construction for lining.
- The cost of construction of channel is minimum when it passes maximum discharge for its given cross sectional area.
- It is evident from the continuity equation and uniform flow formulae that for a given value of slope and surface roughness, the velocity of flow is maximum when hydraulic radius is maximum.

The conditions for the most economical section of channel

1. A rectangular channel section is the most economical when either the depth of flow is equal to half the bottom width or hydraulic radius is equal to half the depth of flow.
2. A trapezoidal section is the most economical if half the top width is equal to one of the sloping sides of the channel or the hydraulic radius is equal to half the depth of flow.
3. A triangular channel section is the most economical when each of its sloping side makes an angle of 45°with vertical or is half square described on a diagonal and having equal sloping sides.

The discharge from a channel is given by

$$Q = AV = AC\sqrt{RS_0} = AC\sqrt{\frac{A}{P}S_o} = K * \frac{1}{\sqrt{P}}$$

where Q = discharge (m^3/s), A = area of cross section (m2), C = Chezys constant,

R= Hydraulic radius (m), P = wetted perimeter (m), = bed slope (fraction or m/m), K = constant for given cross sectional area and bed slope and = $A^{3/2}C\ S_o^{1/2}$ and the discharge Q will be maximum when the wetted perimeter P is minimum.

(i) Channel Shape:

Among the various shapes of open channel the semi-circle shape is the best hydraulic efficient cross sectional shape. However the construction of semicircle cross section is difficult for earthen unlined channel. Trapezoidal section is commonly used cross section.

(ii) Channel Dimensions:

The channel dimensions can be obtained using uniform flow formula, which is given by

Q = A V

Where,

V = flow velocity (m/s)

A = cross-sectional area of canal perpendicular to flow (m2)

Q = capacity of the channel (m3/s)

Velocity is computed by Manning's formula or Chezy formula.

Manning's Equation is given by

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

Chezy's equation is given by

$V = C\ R^{1/2}S^{1/2}$

Where,

n = Manning's roughness coefficient

C = Chezy's roughness coefficient

R = hydraulic radius (m)

S = bed slope (m/m)

Canal alignments

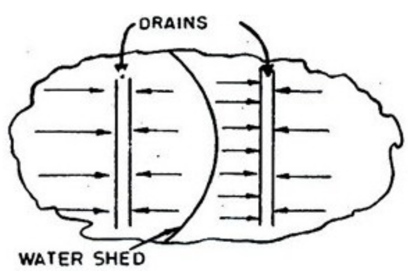
- It is now clear that irrigation water, in flow type, should reach the fields by gravity. To accomplish this requirement irrigation canal is always aligned in such a way that the water gets proper command over the whole irrigable area.
- Obviously if the canal follows a watershed or a ridge of the drainage area it will get necessary gravity flow. The watershed or the ridge is a dividing line between two drainage areas. Thus a canal which runs over the ridge gets command of area on both sides of the ridge.

Irrigation canals can be aligned in any of the three ways:

1. As watershed canal
2. As contour canal; and
3. As side slope canal

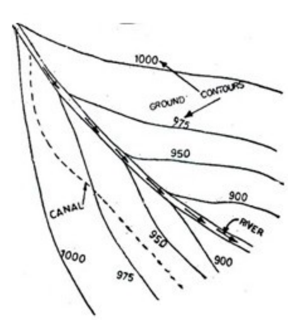
Watershed Canal

- The dividing line between the catchment area of two drains (streams) is called the watershed.
- Thus, between two major stream, there is the main watershed which divides the drainage areas of the two.
- Similarly, between any tributary and the main stream, and also between any two tributaries there, are subsidiary watersheds, dividing the drainage between the two streams on either side.



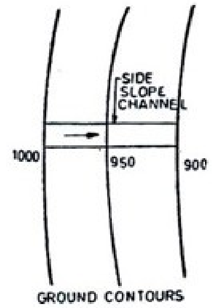
(ii) Contour Canal:

- The above arrangement of providing the canals along the watershed is not possible in hill areas.
- In the hills, the river flows in the valley, while the watershed or the ridge line may be hundred of metres above it.
- It becomes uneconomical to take the canal on top of such a ridge. The channel, in such cases, is generate sufficient flow velocities, are given to it.



iii) Side Slope Canal:

- A side slope channel is that which is aligned at right angles to the contours,i.e. along the side slopes, as shown in figure.
- Such a channel is parallel to the natural drainage flow and hence, does not intercept cross drainage, and hence no cross drainage works are required.



Precautions in Canal Alignment:

While aligning a canal following points should be considered in general:

i. The canal should be aligned on the ridge or in such a way as to obtain maximum command.

ii. So far as possible the canal alignment should be kept in the centre of the commanded area.

iii. The canal should be aligned in such a way that the length is minimum possible.

iv. The alignment should avoid inhabited places, roads, railways, properties, places of worship etc.

v. Canal should be taken through the area where subsoil formation is favourable. Water logged, alkali, saline, rocky soils create troubles.

vi. The alignment should be straight so far as possible. Where alignment is not straight simple circular curves of large radius should be provided.

vii. To ensure economy the alignment of the canal should be such that excessive cuttings and fillings are not required. The alignment should not cross hills or depressions.

Canal Linings

Canal Linings are provided in canals to resist the flow of water through its bed and sides. These can be constructed using different materials such as compacted earth, cement, concrete, plastics, boulders, bricks etc. The main advantage of canal lining is to protect the water from seepage loss.

Canal Lining is an impermeable layer provided for the bed and sides of canal to improve the life and discharge capacity of canal. 60 to 80% of water lost through seepage in an unlined canal can be saved by construction canal lining.

Types of Canal Linings

Canal linings are classified into two major types based on the nature of surface and they are:

1. Earthen type lining
2. Hard surface lining

1. Earthen Type lining

Earthen Type lings are again classified into two types and they are as follows:

i. Compacted Earth Lining
ii. Soil Cement Lining

Compacted Earth Lining

Compacted earth linings are preferred for the canals when the earth is available near the site of construction or In-situ. If the earth is not available near the site then it becomes costlier to construct compacted earth lining.

Compaction reduces soil pore sizes by displacing air and water. Reduction in void size increases the density, compressive strength and shear strength of the soil and reduces permeability. This is accompanied by a reduction in volume and settlement of the surface. Proper compaction is essential to increase the stability and frost resistance (where required) and to decrease erosion and seepage losses.

Soil Cement Lining

Soil-cement linings are constructed with mixtures of sandy soil, cement and water, which harden to a concrete-like material. The cement content should be minimum 2-8% of the soil by volume. However, larger cement contents are also used.

In general, for the construction of soil-cement linings following two methods are used.

Dry-mix method

Plastic mix method

For erosion protection and additional strength in large channels, the layer of soil-cement is sometimes covered with coarse soil. It is recommended the soil-cement lining should be protected from the weather for seven days by spreading approximately 50 mm of soil, straw or hessian bags over it and keeping the cover moistened to allow proper curing. Water sprinkling should continue for 28 days following installation.

2. Hard Surface Canal Linings

It is sub divided into 4 types and they are

i. Cement Concrete Lining
ii. Brick Lining
iii. Plastic Lining
iv. Boulder Lining

Cement Concrete Lining

Cement Concrete linings are widely used, with benefits justifying their relatively high cost. They are tough, durable, relatively impermeable and hydraulically efficient. Concrete linings are suitable for both small and large channels and both high and low flow velocities. They fulfill every purpose of lining.

There are several procedures of lining using cement concrete

i. Cast in situ lining
ii. Shortcrete lining
iii. Precast concrete lining
iv. Cement mortar lining

Brick Lining

In case of brick lining, bricks are laid using cement mortar on the sides and bed of the canal. After laying bricks, smooth finish is provided on the surface using cement mortar.

Plastic Lining

Plastic lining of canal is newly developed technique and holds good promise. There are three types of plastic membranes which are used for canal lining, namely:

a) Low density poly ethylene
b) High molecular high density polythene
c) Polyvinyl chloride

The advantages of providing plastic lining to the canal are many as plastic is negligible in weight, easy for handling, spreading and transport, immune to chemical action and speedy construction.

The plastic film is spread on the prepared sub-grade of the canal. To anchor the membrane on the banks 'V trenches are provided. The film is then covered with protective soil cover.

Boulder Lining

This type of lining is constructed with dressed stone blocks laid in mortar. Properly dressed stones are not available in nature. Irregular stone blocks are dressed and chipped off as per requirement.

When roughly dressed stones are used for lining, the surface is rendered rough which may put lot of resistance to flow. Technically the coefficient of rugosity will be higher. Thus the stone lining is limited to the situation where loss of head is not an important consideration and where stones are available at moderate cost.

Advantages of Canal Lining

1. Seepage Reduction
2. Prevention of Water Logging
3. Increase in Commanded Area
4. Increase in Channel Capacity
5. Less Maintenance

6. Safety Against Floods

1. Seepage Reduction

The main purpose behind the lining of canal is to reduce the seepage losses. In some soils, the seepage loss of water in unlined canals is about 25 to 50% of total water supplied. The cost of canal lining is high but it is justifiable for its efforts in saving of most of the water from seepage losses. Canal lining is not necessary if seepage losses are very small.

2. Prevention of Water Logging

Water logging is caused due to phenomenal rise in water table due to uncontrolled seepage in an unlined canal. This seepage effects the surrounding ground water table and makes the land unsuitable for irrigation. So, this problem of water logging can be surely prevented by providing proper lining to the canal sides.

3. Increase in Commanded Area

Commanded area is the area which is suitable for irrigation purpose. The water carrying capacity of lined canal is much higher than the unlined canal and hence more area can be irrigated using lined canals.

4. Increase in Channel Capacity

Canal lining can also increase the channel capacity. The lined canal surface is generally smooth and allows water to flow with high velocity compared to unlined channel. Higher the velocity of flow greater is the capacity of channel and hence channel capacity will increase by providing lining.

On the other side with this increase in capacity, channel dimensions can also be reduce to maintain the previous capacity of unlined canal which saves the cost of the project.

5. Less Maintenance

Maintenance of lined canal is easier than unlined canals. Generally there is a problem of silting in unlined canal which removal requires huge expenditure but in case of lined canals, because of high velocity of flow, the silt is easily carried away by the water.

In case of unlined canals, there is a chance of growth of vegetation on the canal surface but not in case of lined canals. The vegetation affect the velocity of flow and water carrying capacity of channel. Lined canal also prevents damage of canal surface due to rats or insects.

6. Safety against Floods

A line canal always withstand against floods while unlined canal may not resists and also there is chance of occurring of breach which damages the whole canal as well as surrounding areas or fields. But among the all concrete canal linings are good against floods or high velocity flows.

Kennedy's Silt Theory

RG Kennedy investigated canals systems for twenty years and come up with a Kennedy's silt theory. The theory says that, the silt carried by flowing water in a channel is kept in suspension by the eddy current rising to the surface.

The vertical component of the eddy current tries to move sediment up whereas sediment weight tries to bring it down. Therefore, if adequate velocity available to create eddies so as to keep the sediment just in suspension silting will be prevented.

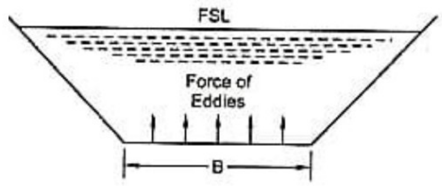


Fig.1: Eddies force according to Kennedy's silt theory

Assumptions regarding Kennedy's Silt Theory

The eddy current is generated because of friction between flowing water and the roughness of the canal bed.

The quality of the suspended silt is proportional to bed width.

The theory is applicable to those channels which are flowing through the bed consisting of sandy silt or same grade of silt.

Critical velocity based on Kennedy's Silt Theory

Critical velocity is the mean velocity which will just make the channel free from silting and scouring. The velocity is based on the depth of the water in the channel. The general form of critical velocity is as follow: Where

$Vo = C\, D^{n}$ -----------------(1)

Vo = Critical velocity

D = full supply depth

C & n: Constants which found to be 0.546 and 0.64, respectively.

Thus, Equation 1 rewritten as follow:

$Vo = 0.546\, D^{0.64}$ --------------- (2)

Moreover, Equation 2 further improved upon realization that silt grade influences critical velocity. So, a factor termed as critical velocity ratio introduced and the equation became as follows:

$Vo = 0.546\, m\, D^{0.64}$ -------------- (2)

Where

m: critical velocity ratio which equal to actual velocity (V) divided by critical velocity (Vo), value of m provided in Table 1.

Table 1 Values of m based on the type of silt

Channel lining	N values
Earth	0.0225
Masonry	0.02
Concrete	0.013 to 0.018

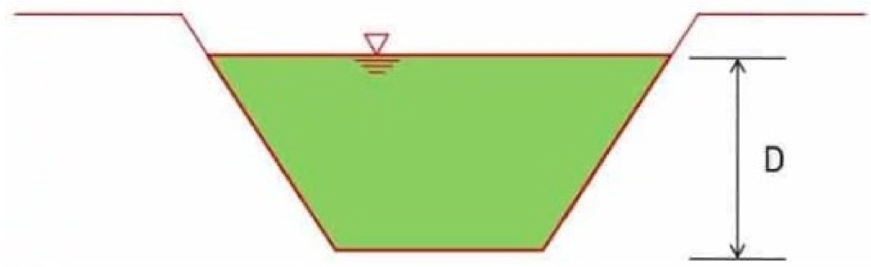


Fig.2: Depth of water in canal

Limitations of Kennedy's Silt Theory

Trial and error method used for the canal design using Kennedy's Silt Theory.

There is no equation for bed slope assessment, so the equation developed by Kutter used to compute bed slope.

The ratio of channel width (B) to its depth (D) has no significance in Kennedy's Silt Theory.

There is not perfect definition for salt grade and salt charge.

Complex phenomenon of silt transportation is not fully accounted and only critical velocity ratio (m) concept is considered sufficient.

Procedure of Canal design using Kennedy's Silt Theory

There are two cases of canal design using Kennedy's Silt Theory dependent on the given data. Both cases presented below:

Case 1

The following data shall be available before hand:

discharge (Q), rugosity coefficient (N), Critical velocity ratio (m) and bed slope of the channel (s).

1. Assume suitable full supply depth (D).
2. Then, find the mean velocity by using Kennedy's equation (Equation 3).
3. After that, find the area of cross section by using continuity equation:Where:

$$Q=A\ V$$

Q: Discharge

A: cross section area

V: mean velocity computed in step 2

4. Assume the shape of channel section with side slopes (0. 5V:1H)

5. Find out the value of base width of channel (B).

6. Then, find the perimeter of the channel (P). Which helps to find out the hydraulic mean depth of channel (R).

$$R = A/P \quad \textit{Equation 5}$$

Where:

R: hydraulic mean depth

A: canal cross section area

P: perimeter of the section

7. Finally, calculate the mean velocity (V) using kutter's formula:

$$V = \left(\frac{1/N(23+0.00155/s)}{1+(23+0.00155/s)(N/\sqrt{R})}\right)\sqrt{Rs} \quad \textit{Equation 6}$$

Where:

N: rugosity coefficient based on type of canal lining material. Table 2 provide N values for different lining condition.

S: bed slope as 1 in 'n'.

Both the values of V computed using equation 3 and V computed employing equation 6 must be the same. Otherwise repeat the above procedure by assuming another value of D.

Generally, the trial depth is assumed between 1 m to 2 m. If the condition is not satisfied within this limit, then it may be assumed accordingly.

Table 2 N values based on the channel lining material

Channel lining	N values
Earth	0.0225
Masonry	0.02
Concrete	0.013 to 0.018

Case 2

When discharge (Q), rugosity coefficient (N), Critical velocity ration (m) and B/D ratio are given.

1. Assume B/D = X
2. By using the Kennedy's equation find "V" in terms of D.
3. Find the area of cross section of the channel in terms of D2.
4. By using continuity Equation 4, find the value of D. and then Find the base width (B).
5. Find hydraulic mean depth (R) with Equation 5.
6. Finally, find the value of "V" using Equation 3.
7. Substitute the value of V in step 6 in Equation 6 will gives the longitudinal slope of the channel (S). This case will done by trial and error method.

Lacey's Silt Theory of Canals

Lacey investigated the stability conditions of different alluvial channels and came up with Lacey's silt theory which explains about the different regime conditions of a channel such as true regime, initial regime, and final regime and the design procedure of canal.

Lacey stated that a channel may not be in regime condition even if it is flowing with non-scouring and non-silting velocity. Therefore, he distinguished three regime conditions as follows :

1. True regime
2. Initial Regime
3. Final Regime

1. True regime

A channel is said to be in regime condition if it is transporting water and sediment in equilibrium such that there is neither silting nor scouring of the channel. But according to Lacey, the channel should satisfy the following conditions to be in regime condition.

1. Canal discharge should be constant.
2. The channel should flow through incoherent alluvium soil, which can be scoured as easily as it can be deposited and this sediment should be of the same grade as is transported.
3. Silt grade should be constant.
4. Silt charge, which is the minimum transported load should be constant.

If the above conditions are satisfied, then the channel is said to be in true regime condition. But this is not possible in actual practice. Hence lacey defined two other conditions which are initial and final regime conditions.

2. Initial Regime

A channel is said to be in initial regime condition when only the bed slope of channel gets affected by silting and scouring and other parameters are independent even in non-silting and non-scouring velocity condition. It may be due to the absence of incoherent alluvium. According to Lacey's, regime theory is not applicable to initial regime condition.

3. Final Regime

If the channel parameters such as sides, bed slope, depth etc. are changing according to the flow rate and silt grade then it is said to be in final regime condition. The channel shape may vary according to silt grade as shown in the figure below :

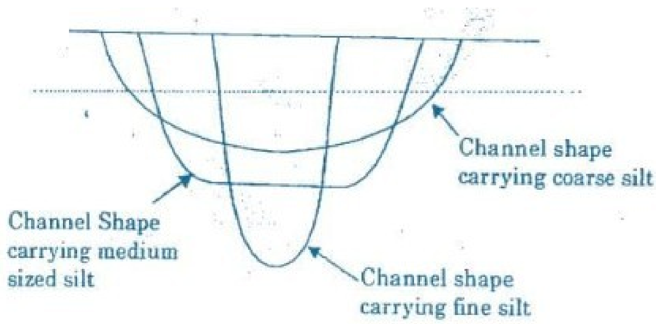


Fig 2: Channel Shape vs Silt Grade

Lacey's specified that the regime theory is valid for final regime condition only and he also specified that semi-ellipse is the ideal shape of regime channels.

Canal design using Lacey's Silt Theory

According to lacey's, the design procedure to build canal is as follows :

Canal discharge (Q) and mean particle size (dm) should be known.

From the mean size or diameter of the particle (dm), silt factor is first calculated using the below expression :

$$Silt\ factor, f = 1.76\sqrt{d_m}$$

Silt factor values for different types of soils are tabulated here.

S.No	Soil Type	Silt Factor, f
1	Fine silt	0.5 – 0.7
2	Medium silt	0.85
3	Standard silt	1
4	Medium sand	1.25
5	Coarse sand	1.5

Using discharge and silt factor, velocity (V) can be calculated by the expression as follows :

Velocity of flow $Velocity\ of\ flow, V = [\frac{Qf^2}{140}]^{1/6}$

After attaining the velocity of canal flow, find the area of the canal by dividing discharge with velocity. Also, find the mean hydraulic depth (R) of the canal and wetted perimeter (P) of the canal.

$$Area = \frac{Q}{V}$$

$$Hydraulic\ Mean\ Depth, R = \frac{5V^2}{2f}$$

$$Wetted\ Perimeter, P = 4.75\sqrt{Q}$$

Assume the bed slope (S) value or find by substituting the values of silt factor and canal discharge in the following formula :

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

Drawbacks of Lacey's Silt Theory

- Lacey did not explain the properties that govern the alluvial channel.
- In general, flow is different at bed and sides of the channel which requires two different silt factors but Lacey derived only one silt factor.
- The semi-elliptical shape proposed by Lacey as the ideal shape of the channel is not convincing.
- Lacey did not consider the silt concentration in his equations.
- Attrition of silt particles is ignored by Lacey.
- Lacey did not give proper definitions for the silt grade and silt charge.

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UNIT – 4

WATER MANAGEMENT IN IRRIGATION

Modernization techniques- Rehabilitation – Optimization of water use-Minimizing water losses-On form development works-Participatory irrigation management- Water resources associations-Changing paradigms in water management-Performance evaluation-Economic aspects of irrigation.

Modernization techniques

- Improving irrigation water management, in order to increase productivity and minimize adverse effects such as salinization, is one of the main contemporary issues in the agricultural sector.
- A considerable effort is being made to improve irrigation operations and to reduce costs.
- Society in general and water user associations, particularly where they have to bear the cost of irrigation, are demanding that irrigation become more cost-effective.

Rehabilitation, which consists of re-engineering a deficient infrastructure to return it to the original design. Although rehabilitation usually applies to the physical infrastructure, it can also concern institutional arrangements.

Process improvement, which consists of intervening in the process without changing the rules of the water management. For instance, the introduction of modern techniques is a process improvement.

Modernization, which is a more complex intervention implying fundamental changes in the rules governing water resource management. It may include interventions in the physical infrastructure as well as in its management.

Defining modernization

Irrigation modernization is a process of technical and managerial upgrading (as opposed to mere rehabilitation) of irrigation schemes combined with institutional reforms, with the objective to improve resource utilization (labour, water, economic, environmental) and water delivery service to farms.

The need for a consistent framework for modernization

- Increasing water productivity
- Increasing the cost-effectiveness
- Increasing the reliability in irrigation deliveries.
- Increasing the flexibility of deliveries.
- Consideration of other uses of water
- Increasing knowledge and human resources development

Obstacles in the way of modernization

Successful modernization is not straightforward, and failure to achieve targeted performance objectives, in some instances, requires further investigation of the underlying causes. As far as the technology is concerned, significant hardware and software progress has been made in irrigation system operations in the past decade, including computer facilities, information techniques, measurements, and canal control concepts

Technical gaps between the requirements needed to implement the improved method (availability of expertise, technical maintenance of equipment) and available local resources.

Financial constraints resulting from the gap between the cost of equipment for the improved method and the gain in water savings and improved services, as water is generally not priced or charges are low.

Social constraints. Human resources are relatively less expensive in developing economies than alternative technological solutions. An irrigation agency, often a large employer in the area, has some obligation to maintain local staff.

Institutional constraints. Bureaucratic centralized irrigation administrations are not well suited to service-oriented activities.

Model for the modern irrigation enterprise

- It is clear for many that the irrigation sector in general has not reached the same level of effectiveness as other sectors, such as the industrial and service sectors.
- Hence modernization can be seen as a means to create and favour modern irrigation enterprises by introducing methodologies which have proved successful in other sectors.
- We advocate that modern enterprises in irrigation require a reengineering of their processes in order to cope with the new challenges faced by irrigation.

Reengineering irrigation system operations

The reengineering of the irrigation operation should consist of designing the most cost-effective answer to the redefined water service within the scheme. It should consider:

The spatial distribution of the effective demand for the water service. The service might differ significantly with user demand, e.g. cash-crop farmers might ask for a high quality and costly service whereas farmers with an alternative source (wells) might be satisfied with a low and cheap service. The service might also differ because of other considerations such as hydrological hazards (salinization, water-logging) and opportunities (recycling of water).

The spatial distribution of the physical infrastructure characteristics. The sensitivity of the canal delivery structures, the efficiency in controlling water depth, the ease of monitoring and implementing operation - these are some of the important features that should be considered when designing an appropriate answer to meet the demand.

Flexibility in modernization

The concept of flexibility has long been discussed and advocated in the field of irrigation modernization.

So far it has encompassed the notion of flexibility in water deliveries as opposed to rotational and fixed deliveries.

Flexible deliveries can be proposed to users in different forms (on request, free access, etc) at a cost compared to a strict rotational distribution.

This concept of flexibility leads to abandoning the homogeneous approach of irrigation systems that has so far prevailed.

Instead, a heterogeneous approach of the demand and of the efforts (inputs) to operate irrigation systems is sought for a closer match of water availability to demand requirements.

Modernization is a never-ending process of adapting activities to current constraints and objectives. The agricultural and economic contexts are permanently evolving and so are the demands from society. What was modern and up to date some decades ago might now appear to be incompatible with current needs, and this is not only true of the technical aspects of irrigation.

Low-cost technologies

The introduction of low-cost technologies, which could be part of the modernization of small-scale irrigation projects, provides another example of the site-specificity of success.

Inexpensive treadle pumps have been successful in some South Asian countries in extracting irrigation water from shallow aquifers.

These pumps have allowed poor farmers to make good use of the available labour in their households and so increase crop production and farm income.

The farmer has full control over the timing and amount of this pumped water, which given the effort involved is used sparingly.

For example, the area under irrigation by one treadle pump in West Bengal, India, varies between 0.033 and 0.13 ha. Treadle pumps have also been introduced in Africa, including the urban and peri-urban areas of Ndjamena, Chad. Here, the vegetable growers rejected the pumps in favour of mechanical pumps because they could afford the cost of fuel and spare parts.

Bucket drip-irrigation kits

Positive experience has been reported with the introduction of bucket drip-irrigation kits. These kits are suitable for the irrigation of small plots of vegetables and fruit trees in peri-urban areas (close to markets). In Kenya, the return on an investment of about US$15 for one bucket drip-irrigation kit was some US$20 per month. Farmers in Kenya have bought over 10 000 kits, although some of these farmers could not described as very poor.

Rehabilitation - the renovation of a scheme to meet its original design criteria

- Inadequate operational practices may limit improvements to water supply expected from improved infrastructure.
- Trained and motivated operational staff are needed. They must be committed to delivering a specified minimum level of service. Institutional will and government policies are needed to effect such changes.
- Farmers must be willing and able to exploit a better supply. They may need training in water use and maintenance. A formal or informal water user group must exist.
- Until the water supply is improved, it is unlikely that farmers will cooperate.

Maintenance activities in a reservoir itself comprise:

- controlling aquatic weeds,
- removing large debris (e.g. tree trunks) floating in the water that may damage hydraulic works,
- monitoring the water quality: not only from the salt content point of view but also from a biological standpoint in order to detect possible sources of pollution
- surveying the solid deposition in the bottom of a reservoir.

The retention in good working order of open drains includes the following operations:

1. light deforestation
2. weed control in the canal section
3. seeding grass in the canal section

4. maintenance of flow gauges and other measuring devices
5. removal of silt
6. maintenance of pumping stations where water cannot be evacuated by gravity.

Optimization of water use

Water efficiency of irrigation can be improved by making the right decisions regarding:

- Crop selection
- Irrigation scheduling
- Irrigation methods
- Source of water.

Improving Irrigation practices can:

- Reduce water and pumping costs
- Reduce costs for fertilizers and other agricultural chemicals
- Maintain a higher soil quality
- Increase crop yields − by as much as 100%

Irrigation scheduling

- Irrigation scheduling helps eliminate or reduce instances where too little or too much water is applied to crops.
- Scheduling is performed by all growers in one way or another.
- However, proper irrigation scheduling involves fine-tuning the time and amount of water applied to crops based on the water content in the crop root zone, the amount of water consumed by the crop since it was last irrigated, and crop development stage.
- Direct measurement of soil moisture content is among the most useful methods for irrigation scheduling.

Good irrigation scheduling requires knowledge of:

- Crop water demand at different growth cycles
- Moisture content of the soil and soil water capacity
- Weather conditions.

Soil capacity

- **Soil capacity**, which is the ability of the soil to hold water between irrigation or precipitation events, is another important factor.
- Determinants of soil capacity include soil depth, ratios of different soil particles making up the soil, soil porosity, and soil water tension.

Climatic conditions

The prevailing **climatic conditions**, such as average ambient temperature, intensity of solar radiation, humidity, and windspeed also affect both the moisture retained in the soil and the speed by which plants lose water through transpiration.

Accurate monitoring

- Accurate monitoring of water used in irrigation is an essential part of irrigation scheduling and helps reach optimal performance, saving water while enhancing yields.
- Accurate readings can be obtained through different direct measurement methods available for pipes and closed conduits
- Measurement of energy used by irrigation pumps
- End-pressure measurements in sprinkler irrigation
- Elevation differences in irrigation reservoirs or tanks
- Measurement of irrigation time and size of irrigation delivery system.

IRRIGATION METHODS

Once the quantitative and temporal characteristics of optimal water demand have been determined, a method that can make such water available in the most effective way should be selected. There are three main irrigation methods, namely:

1. Surface (or gravity) irrigation
2. Sprinkler irrigation
3. Drip irrigation.

Water losses be control:

The following are the measures that are generally taken to control the water losses from the reservoir.

1. Measure to Reduce Evaporation Loss

a) The reservoir should be constructed of less surface area and more depth.

b) Tall trees should be grown on the windward side of the reservoir which act as wind breakers and hence the rate of evaporation will be reduced.

c) The reservoir basin should be surrounded by plantation or forest area so that cooler environment exists within the reservoir area.

d) Certain chemical like cetyl alcohol is spread over the reservoir surface. It forms a thin film on water surface reducing evaporation.

2. Measure to Reduce Absorption Loss

a) The weeds and plants at the periphery of the reservoir should be removed completely.

b) The weeds from the surface of the reservoir should be removed.

3. Measure to Reduce Percolation Loss

a) Geological investigations should be carried out to locate the zones of pervious formations, cracks and fissures in the bed and periphery of the reservoir basin.

b) Suitable treatments should be adopted to stop the leakage of water through these zones.

c) Soil stabilization methods should be adopted if the basin is composed of permeable bed soil.

Water logging

- In agricultural land, when the soil pores within the root zone of the crops get saturated with the subsoil water, the air circulation within the soil pores gets totally stopped.
- This phenomenon is termed as water logging.
- The water logging makes the soil alkaline in character and the fertility of the land is totally destroyed and the yield of crop is reduced.

Effects of water logging

The following are the effects of water logging:

- Stabilization of soil
- Lack of aeration
- Fall of soil temperature
- Growth of weeds and aquatic plants
- Diseases of crops
- Difficulty in cultivation
- Restriction of root growth

Methods used for controlling water logging

The following measures may be taken to control water logging:

- Prevention of percolation from canals
- Prevention of percolation from reservoirs
- Control of intensity of irrigation
- Economical use of water
- Fixing of crop pattern
- Providing drainage system

- Improvement of natural drainage
- Pumping of ground water
- Construction of sump well

ON FORM DEVELOPMENT WORKS (OFD)

- The efficient management of irrigation water for maximizing productivity requires both, the efficient on farm water management and the optimization of the use of water and land, through appropriate methods of water application.
- The efficient on-farm water management is related to water delivery system and allied works in the command area of chak (Small irrigation block), which distributes the water to each farm.
- The items of works pertaining to on farm water management are termed as "On farm development works".

The on farm development works comprise of following,

a) Field channels for conveyance of water

b) Control structures

c) Crossings

d) Surface drainage system

e) Farm roads

f) Field channel protection works and

g) Land forming (Smoothening / grading / leveling).

Systems approach:

- The conveyance system from the dam to the farm gate is one live system and it is necessary that the designs of the different components are matched properly.
- The water management proposed to be adopted on the canal system should always be kept in view as a reference frame.

Sequence of design and execution:

(a) The ideal sequence of finalization of design would be obviously from the tail to the head.

- First, according to the topography and soil conditions, the land forming of each farm would be decided, so as to ensure efficient irrigation.
- Next, the chak water delivery system and surface drains would be designed so as to ensure adequate water deliveries to the different farms and proper drainage.

(b) The execution of OFD works shall be done only in places where canal water has actually reached. The OFD works get disturbed and deteriorated if these are not put to use immediately.

(c) After construction of OFD works, preparation of work-done drawings of OFD works (record drawings) form the basic record for planning irrigation management.

Functional utility:

- The purpose of the OFD works is to provide timely and adequate supplies of water to each holding and preserve environmental balance as well, by avoiding seepages, leakages and stagnations of water which trigger problems like water logging, causing adverse impact on environment.
- To achieve this functional utility, the planning and design of OFD works has to be hydraulically better and socially acceptable.

The functional utility of OFD works is governed by following aspects

i. Hydraulic design
ii. Economy for construction and maintenance
iii. Social acceptance i.e. User friendliness to community of farmers who will be actually using OFD works and
iv. Levels of accuracies and quality of construction. These issues need to have a proper bearing in approach to design and execution of OFD works.

Farmers Participation:

- Success of canal irrigation depends on the response of the farmer, both as an individual and as a member of the group benefited by the outlet.
- The irrigation facilities should be designed with a view to meet his requirements, particularly in respect of land forming.
- Active participation of the farmer at the stage of design should therefore be encouraged.
- This will also help in building up of an atmosphere of common purpose and thereby in the unification of the beneficiaries into a homogeneous group.

Organizational Coordination:

- The works from the canal head down to the distributory and from the distributory head to the outlet are carried out by the Construction Organization (C.O.) of the Irrigation Department.
- The outlet of a capacity of about 30 litre/second is the last Government structure on the canal system. Below outlet, OFD works are the community works.
- Design and construction proceed on the basis of the location of the outlet and its sill level.

Procedure for taking up OFD works:

The OFD works are part of CAD works. The cost estimates for OFD works are generally formulated in two parts.

a) Part I works : Cost estimates for chak water delilvery system, field channel protection works and surface drainage works in a chak.

b) Part II : Cost estimate for land forming works for each holding.

Participatory irrigation management

- The term participatory irrigation management (PIM) refers to the participation of irrigation users, i.e., farmers, in the management of irrigation systems not merely at the tertiary level of management but spanning the entire system.
- Participation should not be construed as consultation alone. The concept of PIM refers to management by irrigation users at all levels of the system and in all aspects of management. This is the simplicity and flexibility of PIM.
- There can be different forms of participation at different levels in the system with varying degrees of accountability and responsibility.
- Management by irrigation users, rather than by a government agency, is often the best solution.
- Contrary to the traditional concept that irrigation management requires a strong public-sector role, the PIM approach starts with the assumption that the irrigation users themselves are best suited to manage their own water.
- "Participation in irrigation management involves a larger role for farmers, water groups, and other stakeholders.
- It may range from offering information and opinions during consultations, to fully enabling farmers to act as principal decision makers in all or most project activities.
- There have been increasing efforts to use participation in various forms to improve the quality, effectiveness, and sustainability of irrigation systems.
- This makes it important to learn what has and has not been achieved in efforts to improve participation in irrigation management.
- Farmers' participation in irrigation management is not entirely new to India. There is considerable evidence that farmers in pre-independence years had been involved in irrigation management in different parts of the country.
- The phad system of Nasik and Dhule districts and the Malgujari tanks of Chandrapur and Bhandara districts in Maharashtra, the Ahar-Pyne system of Bihar, the Kuhl system of H.P. and the Kudimaramath of Tamilnadu are some of the important examples of PIM under traditional irrigation.
- Vestiges of these practices still survive though these have become quite weak or even extinct with the passage of time.
- A few formal water users associations were also formed from time to time like the Vadakku Kodai Melazhahian Channel Land Holders Association in Tamilnadu in December 1959, Malinagar Irrigators' Water Cooperative Society in Maharashtra in 1967, Vaishali Area Small Farmers Association in Bihar in 1971, Mohini Water Cooperative Society in Gujarat in 1978.
- These were, however, isolated examples which could be counted on fingers. Irrigation management from top to bottom remained concentrated in the hands of the government.

- It may be said that since 1972, after the establishment of CADA, a large number of farmer organisations at the outlet level were formed under the CAD projects.
- These were variously described as pipe committees, outlet committees and WUAs. These, however, lacked authority and responsibility and, therefore, could not serve any useful purpose. Many of these became non-functional after some time.