UGMIT RAYAGADA

Lecture Note

Of

H&IE, 4th Sem.

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S.No	Date	Title	Page No.	Teacher's Sign/ Remarks

* Lecture notes on
Hydraulics & Irrigation Engg.

① Hydrostatics

(1.1) Properties of fluid :-

(a) Density or Mass density

It is defined as the ratio of the mass of a fluid to its volume. Denoted by ρ (rho)

SI unit is kg/m^3

Mathematically, $\rho = \frac{\text{Mass of fluid}}{\text{Volume of fluid}}$

eg. :- value of density of water is $1\ gm/cm^3$ or $1000\ kg/m^3$.

(b) Specific weight or weight density

It is defined as the ratio between the weight of a fluid to its volume

Mathematically, $w = \frac{\text{wt. of fluid}}{\text{volume of fluid}}$

and $w = \rho g$

eg :- value of sp. weight or wt. density (w) of water is $9.81 \times 1000\ N/m^3$

(c) Specific volume :

It is defined as volume of fluid occupied by a unit mass or volume per unit mass of a fluid.

Mathematically, sp. volume $= \frac{\text{Volume of fluid}}{\text{Mass of fluid}}$

$= \frac{1}{\rho}$

unit is m^3/kg. commonly applied to gases

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(d) Specific Gravity (or relative density)

It is defined as the ratio of the weight density (density) of a fluid to the weight density (density) of a standard fluid. Standard fluid is water for liquid and air for gases.

Mathematically sp. gravity (S)

for liquid = $\frac{\text{wt. density (density) of liquid}}{\text{wt. density (density) of water}}$

for gases = $\frac{\text{wt. density (density) of gas}}{\text{wt. density (density) of air}}$

So wt. density of liquid = S × wt. density of water

= S × 1000 × 9.81 N/m^3

& density of liquid = S × density of water

= S × 1000 kg/m^3

eg:- sp. gravity of mercury = 13.6

so density of mercury = 13.6 × 1000

= 13600 kg/m^3

Problem 1: Calculate the density, sp. wt. and wt. of 1 L. of petrol of S.G = 0.7.

Ans:- Given; volume = 1 l = $\frac{1}{1000}$ = 0.001 m^3

($\because$ 1 m^3 = 1000 l so 1 l = $\frac{1}{1000}$ = 0.001 m^3)

S.G, S = 0.7

So, (i) density (ρ) = S × 1000 kg/m^3

= 0.7 × 1000 = 700 kg/m^3.

(ii) specific wt. (w) = ρg = 700 × 9.81 N/m^3

= 6867 N/m^3

(iii) wt. (W)

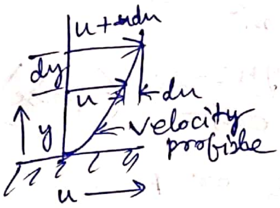
as w = $\frac{\text{wt.}}{\text{volume}}$ so wt(W) = w × volume

= 6867 × 0.001 = 6.867 N.

(Ans)

(e) Viscosity

It is defined as the property of fluid which offers resistance to the movement of one layer of fluid over another adjacent layer of fluid.



When two fluid layers at a distance 'dy' apart, move one over other at a velocity u & u+du then there is shear stress acting between the fluid layers.

shear stress τ is proportional to rate of change of velocity w.r.t. y.

Mathematically, $\tau \propto \frac{du}{dy}$

or $\tau = \mu \frac{du}{dy}$

μ = constant of proportionality known as coefficient of dynamic viscosity or only viscosity.

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$\frac{du}{dy}$ = rate of shear strain or rate of shear deformation or velocity gradient.

or $\mu = \frac{\tau}{\left(\frac{du}{dy}\right)}$

So viscosity is also defined as the shear stress required to produce unit rate of shear strain.

units of viscosity

Dimensions of $\mu = \frac{\text{Force} \times \text{time}}{(\text{Length})^2}$

MKS unit of $\mu = \frac{\text{kgf-sec}}{m^2}$

CGS unit of $\mu = \frac{\text{dyne-sec}}{cm^2}$ (= poise)

SI unit of $\mu = \frac{Ns}{m^2}$ or Pa.s.

$1 \frac{\text{kgf-sec}}{m^2} = \frac{9.81\ Ns}{m^2} = 98.1$ poise.

So $1 \frac{Ns}{m^2} = \frac{98.1}{9.81}$ poise = 10 poise

or 1 poise = $\frac{1}{10} \frac{Ns}{m^2}$

1 centipoise (CP) = $\frac{1}{100}$ Poise (P)

eg:- viscosity of water at 20°C is 0.01 poise or 1.0 centipoise

→ kinematic viscosity :-

It is defined as the ratio between the dynamic viscosity and density of fluid. Denoted by ν (nu)

so $\nu = \frac{\mu}{\rho}$

Dimensions of $\nu = \frac{(\text{Length})^2}{\text{Time}}$

unit in MKS & SI → m^2/sec.

CGS → cm^2/s (stoke)

1 stoke = $1\ cm^2/s = \left(\frac{1}{100}\right)^2 m^2/s = 10^{-4} m^2/s$

1 centistoke = $\frac{1}{100}$ stoke

→ Newton's law of viscosity :-

It states that the shear stress (τ) on a fluid element layer is directly proportional to the rate of shear strain

$\tau = \mu \frac{du}{dy}$

Fluid which obeys Newton's law and viscosity is Newtonian fluid & which does not obey is Non-Newtonian fluid.

NB :- Viscosity of liquid decreases with increase in temperature while viscosity of gases increases with increase in temperature.

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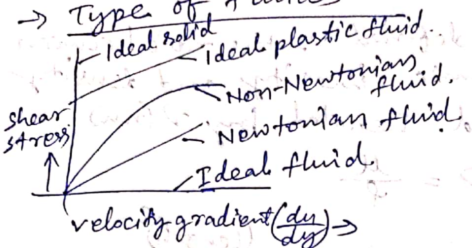
→ Type of fluids



(a) Ideal fluid → An incompressible and non-viscous fluid is known as Ideal fluid. It is Imaginary.

(b) Real fluid
Fluid with viscosity are real fluid.

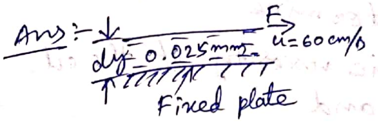
(c) Newtonian fluid
Real fluid obeying Newton's law of viscosity is Newtonian fluid.

(d) Non-Newtonian fluid :-
Real fluid which doesn't obey Newton's law of viscosity.

(e) Ideal-plastic fluid. :-
A fluid in which shear stress is more than the yield value and shear stress is proportional to the rate of shear strain (velocity gradient) is known as Ideal plastic fluid.

Problem-2 A plate 0.025 mm distant from a fixed plate, moves at 60cm/s and requires a force of 2N/area i.e., 2N/m² to maintain this speed. Determine the fluid viscosity between the plates.

Ans :-



Given, distance b/w plates, $dy = 0.025 mm = 0.025 \times 10^{-3} m$

velocity of upper plate, $u = 60 cm/s = 0.6 m/s$

Force on upper plate, $F = 2 N/m^2 = \tau$

So du = change of velocity $= u - 0 = u = 0.60 m/s$.

dy = change of distance $= 0.025 \times 10^{-3} m$.

using relation, $\tau = \mu \frac{du}{dy}$

$\Rightarrow 2 = \mu \frac{0.60}{0.025 \times 10^{-3}}$

$\Rightarrow \mu = 8.33 \times 10^{-5} \frac{Ns}{m^2}$

$= 8.33 \times 10^{-4}$ poise

$(\because 1 \frac{Ns}{m^2} = 10 \text{ poise})$

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Problem-3 The space b/w two square flat parallel plates is filled with oil. Each side of the plate is 60 cm. The thickness of oil film is 12.5 mm. The upper plate moves at 2.5 m/s requires a force of 98.1 N to maintain the speed. Determine:

(i) the dynamic viscosity of the oil in poise, and

(ii) The kinematic viscosity of the oil in stokes if S.G of oil is 0.95.

Ans → Given:

Each side of sq. plate = 60 cm = 0.60 m.

Area = A = 0.6 × 0.6 = 0.36 m^2

thickness of oil film, $dy = 12.5$ mm $= 12.5 \times 10^{-3}$ m

velocity of upper plate, $u = 2.5$ m/s.

∴ change of velocity b/w plates, $du = 2.5$ m/s

Force required on upper plate, $F = 98.1$ N.

∴ shear stress, $\tau = \frac{F}{A} = \frac{98.1}{0.36} \frac{N}{m^2}$.

(i) we know, $\tau = \mu \frac{du}{dy}$

or $\frac{98.1}{0.36} = \mu \times \frac{2.5}{12.5 \times 10^{-3}}$

$\Rightarrow \mu = 1.3635 \frac{Ns}{m^2} = 13.635$ P

(ii) S.G of oil, $s = 0.95$

Let ν = kinematic viscosity of oil

so $\rho = S \times 1000 = 0.95 \times 1000 = 950$ kg/m^3

So $\nu = \frac{\mu}{\rho} = \frac{1.3635 \left(\frac{Ns}{m^2}\right)}{950}$

$= 0.001435$ m^2/s $= 0.001435 \times 10^4$ cm^2/s

$= 14.35$ stokes (∵ 1 cm^2/s = 1 stoke)

Problem-4

The velocity distribution for flow over a flat plate is given by $u = \frac{3}{4}y - y^2$ in which u is velocity in m/s at a distance y m above plate. Determine the shear stress at y = 0.15 m. Take dynamic viscosity of fluid as 8.6 poise.

Ans ⇒ Given, $u = \frac{3}{4}y - y^2$

$\frac{du}{dy} = \frac{3}{4} - 2y$

at $y = 0.15$, $\frac{du}{dy} = \frac{3}{4} - 2 \times 0.15 = 0.45$

viscosity, $\mu = 8.5$ poise $= \frac{8.5}{10} \frac{Ns}{m^2}$

we know, $\tau = \mu \frac{du}{dy} = \frac{8.5}{10} \times 0.45$

$= 0.3825 \frac{N}{m^2}$ (Ans)

(f) Surface tension and capillarity

→ Surface tension :-

It 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.

Denoted by σ (sigma)

unit → MKS → kgf/m

SI → N/m

All the molecules on the free surface experience a downward force. This free surface of liquid acts like a very thin film under tension of the surface of liquid act as an elastic membrane under tension.

→ Surface tension on Liquid Droplet :-

Consider a small spherical droplet of liquid of radius 'r'. On entire surface of droplet S.T. acts.

Let σ = S.T of liquid

p = pr. intensity inside droplet

d = dia of droplet

The forces acting on one half will be

(i) Tensile force due to S.T around the circumference $= \sigma \times$ circumference

$= \sigma \times \pi d$.

(ii) Pr. force on the area, $\frac{\pi}{4} d^2$

$= p \times \frac{\pi}{4} d^2$

under equilibrium condn,

$p \times \frac{\pi}{4} d^2 = \sigma \times \pi d$

or $\boxed{p = \frac{4\sigma}{d}}$

So with decrease in diameter of droplet pr. intensity inside the droplet increases.

→ S.T. on hollow Bubble :-

Hollow bubble like a soap bubble in air has two surfaces in contact with air, one inside & other outside.

So under equilibrium condn,

$p \times \frac{\pi}{4} d^2 = 2 \times (\sigma \times \pi d)$

$\Rightarrow \boxed{p = \frac{8\sigma}{d}}$

→ S.T. on Liquid jet :-

Force due to pr. = $p \times$ area of semi jet

$= p \times L \times d$.

Force due to S.T. $= \sigma \times 2L$

Equating, $p \times L \times d = \sigma \times 2L \Rightarrow \boxed{p = \frac{\sigma \times 2L}{Ld}}$

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Problem-5 The pr. outside the droplet of water of dia 0.04 mm is 10.32 N/cm² (atm. pr.). Calculate the pr. within the droplet if S.T. is given as 0.0725 N/m of water.

Ans:- Given, dia of droplet, $d = 0.04$ mm $= 0.04 \times 10^{-3}$ m

pr. outside droplet $= 10.32$ N/cm² $= 10.32 \times 10^{4}$ N/m²

S.T., $\sigma = 0.0725$ N/m

pressure inside droplet (excess of outside pr.)

$$p = \frac{4\sigma}{d} = \frac{4 \times 0.0725}{0.04 \times 10^{-3}} = 7250 \text{ N/m}^2 = \frac{7250}{10^4} = 0.725 \text{ N/cm}^2$$

pr. inside droplet
$= p +$ pr. outside $= 0.725 + 10.32$
$= 11.045$ N/cm²

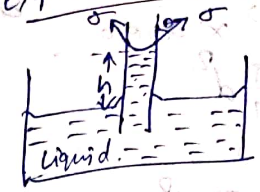
(g) **Capillarity** :-

It is defined as a phenomenon of rise or fall of a liquid surface in a small tube relative to adjacent general level of liquid when the tube is vertical in liquid.

Rise of liquid surface is capillary rise & fall is capillary depression.

unit of capillarity is cm or mm.

→ **Expression for capillary rise**



Glass tube dia $= d$
h = ht of liquid in tube
σ = surface tension of liquid
θ = angle of contact b/w liquid and glass tube

wt. of liquid of ht. h in tube
$= \frac{\pi}{4} d^2 \times h \times \rho g$ (ρ = density of liquid) —(1)

vertical component of surface tensile force $= (\sigma \times \text{circumference}) \times \cos\theta$
$= \sigma \times \pi d \times \cos\theta$ —(2)

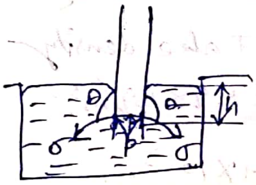
Equating 1 & 2 we have,

$$\frac{\pi}{4} d^2 \times h \times \rho g = \sigma \times \pi d \times \cos\theta$$

or $$\boxed{h = \frac{4\sigma\cos\theta}{\rho g d}}$$

for contact b/w water and clean glass take $\theta = 0°$.

→ **Expression for capillary fall** :-



In this case the level of mercury will be lower than general level of outside liquid.
h = depression in tube

Forces acting on mercury are
(i) Due to S.T. acting in downward direction $= \sigma \times \pi d \times \cos\theta$

(ii) Due to hydrostatic force in upward direc^n & equal to intensity of pr. at depth 'h' × area.

$= p \times \frac{\pi}{4} d^2 = \rho g h \frac{\pi}{4} d^2$

Equating we have;

$\sigma \times \pi d \times \cos\theta = \rho g h \frac{\pi}{4} d^2$

$\Rightarrow \boxed{h = \frac{4\sigma \cos\theta}{\rho g d}}$

value of θ for mercury and glass tube is 128°.

Problem-6 Calculate the capillary effect in mm in a glass tube of 4mm dia, when immersed in (i) water and (ii) mercury. The temp. of liquid is 20°C and values of S.T. of water and mercury at 20°C in contact with air are 0.073575 N/m and 0.51 N/m respectively. The angle of contact for water is 0 and mercury is 130°. Take density of water at 20°C = 998 kg/m³

Ans → Given, $d = 4mm = 4 \times 10^{-3} m$.

Capillary rise or depression is,

$h = \frac{4\sigma \cos\theta}{\rho g d}$

(i) Capillary rise for water :-

$\sigma = 0.073575$ N/m, $\theta = 0°$.

$\rho = 998$ kg/m³ at 20°C

$h = \frac{4\sigma \cos\theta}{\rho g d} = \frac{4 \times 0.073575 \times \cos 0°}{998 \times 9.81 \times 4 \times 10^{-3}}$

$= 7.51 mm$

(ii) Capillary fall for mercury:-

$\sigma = 0.51$ N/m, $\theta = 130°$

$\rho = S \times 1000 = 13.6 \times 1000 = 13600$ kg/m³

$h = \frac{4\sigma \cos\theta}{\rho g d} = \frac{4 \times 0.51 \times \cos 130°}{13600 \times 9.81 \times 4 \times 10^{-3}}$

$= -2.46 mm$ (depression)

1.2 Pressure and its measurement :-

→ **Intensity of pressure** :-

If we consider a small area dA in large mass of fluid and let force dF acting on area dA, then ratio

$\frac{dF}{dA} = p$ = pressure intensity.

If force (F) is uniformly distributed over the area (A), then $p = \frac{F}{A}$

unit of pr. → in MKS → kgf/m² or kgf/cm²

in SI → N/m² & N/mm².

1 N/m² = 1 Pascal or 1 Pa.

1 bar = 10^5 Pa

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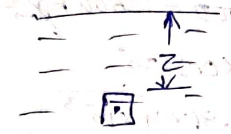
→ **Pascal's law** :-

It states that the pr. intensity at a pt. in a static fluid is equal in all directions.

If p_x, p_y, p_z are pr. intensity in fluid element in x, y, z directions, then, according to pascal's law,

$$p_x = p_y = p_z$$

→ **Pressure head** :-



Consider small fluid element at depth z from surface.

Then $\frac{\partial p}{\partial z} = \rho g = w$

i.e., Rate of increase of pr. in a vertical direction is equal to weight density of the fluid at that point (Hydrostatic Law)

Integrating above, we have,

$$p = \rho g z \quad \text{or} \quad \boxed{z = \frac{p}{\rho g}}$$

z = pressure head

Problem-6 The pressure intensity at a point in a fluid is given as 3.924 N/cm². Find the corresponding ht. of fluid when the fluid is (a) water (b) oil of S.G 0.9.

Ans :- Given, $p = 3.924 \ N/cm^2$
$= 3.924 \times 10^4 \ N/m^2$

So $z = \frac{p}{\rho g}$

(a)- for water $\rho = 1000 \ kg/m^3$

$\therefore z = \frac{p}{\rho g} = \frac{3.924 \times 10^4}{1000 \times 9.81}$
= 4m of water

(b) For oil, sp.gr. = 0.9

$\therefore$ Density of oil, $\rho_{oil} = 0.9 \times 1000 = 900 \ kg/m^3$

$z = \frac{p}{\rho_{oil} \times g} = \frac{3.924 \times 10^4}{900 \times 9.81} = 4.44m$ of oil.

Problem-7 An oil of sp. gr. 0.9 is contained in a vessel. At a pt. the ht. of oil is 40m. Find the corresponding ht. of water at the pt.

Ans :- Given, $S_{oil} = 0.9$,

ht of oil, $z_{oil} = 40m$.

Density of oil, $\rho_{oil} = S \times 1000$
$= 0.9 \times 1000$
$= 900 \ kg/m^3$

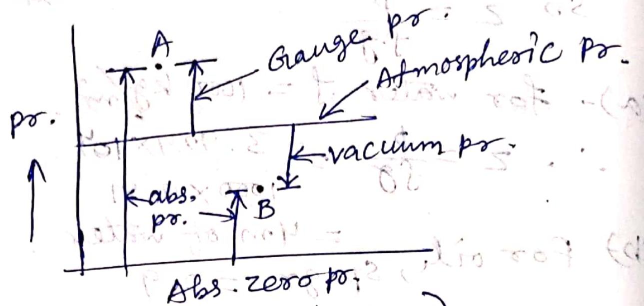
So $p = \rho_{oil} \times g \times z_{oil} = 900 \times 9.81 \times 40 \ N/m^2$

$\therefore$ Corresponding ht. of water

$z = \frac{p}{\rho_{water} \times g} = \frac{900 \times 9.81 \times 40}{1000 \times 9.81}$

= 36m of water.

→ Absolute, Gauge, Atmospheric and vaccum pr.



(Relationship b/w pr.)

(a) Absolute pr. :- It is defined as the pr. which is measured with reference to absolute vaccum pr.

(b) Gauge pr. :- It is defined as the pr. which is measured with the help of a pr. measuring instrument, in which atmospheric pressure is taken as datum. Atmospheric pr. on the scale is marked as zero

(c) Vaccum pr. :- It is defined as the pr. below the atm. pr.

Mathematically,

(i) Absolute pr.
= Atmospheric pr. + Gauge pr.
or $P_{abs} = P_{atm} + P_{gauge}$

(ii) Vaccum pr. = Atm. pr. − Abs. pr.

Note - (a) Atm. pr. at sea level at 15°C is $101.3\ kN/m^2$ or $10.13\ N/cm^2$ (SI) is $1.033\ kgf/cm^2$ (MKS)

(b) Atm. pr. head is 760mm of mercury or 10.33m of water.

→ Measurement of pr :-

Pr. of a fluid is measured by (i) Manometers, (ii) Mechanical Gauges

(i) Manometers :- These are devices used for measuring the pr. at a pt. in a fluid by balancing the colum of fluid by the same or another colum of fluid. They are of 2 types, (a) Simple manometer.
(b) Differential manometer.

(ii) Mechanical Gauges :-
These are devices used for measuring pr. by balancing fluid column by spring or dead wt.
Types ⇒ (a) Diaphragm pr. gauge
(b) Bourdon tube pr. gauge
(c) Dead-wt. pr. gauge
(d) Bellows pr. gauge

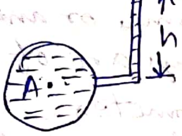
→ Simple Manometer

It consists of a glass tube with one end connected to a pt. where pr. to be measured and other end open to atm.

type → piezometer, U-tube manometer, single column Manometer.

→ piezometer :-

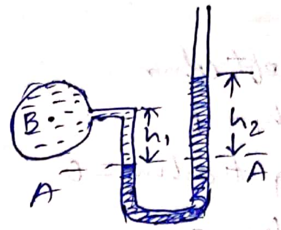
It is simplest form to measure gauge pr. The rise of liquid gives pr. head at that point.



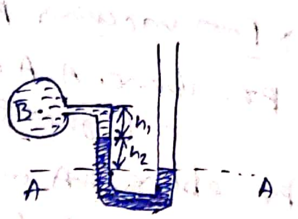
$p_A = \rho g h \ \frac{N}{m^2}$

→ U-Tube Manometer

It consists of a glass tube bent in U-shape, one end connected to pt. at which pr. is to be measured and other end remain open to atm. Tube contains mercury or other liquid whose sp. gr. is greater than sp. gr. of liquid whose pr. is to be measured.



For gauge pr.



For vacuum pr.

→ For gauge pr.

Let B is the pt. at which pr. is to be measured, whose value is p.
Datum line is A-A

Let h_1 = ht. of lighter liquid above datum line

h_2 = ht. of heavier liquid above datum

S_1 = sp. Gr. of lighter liquid

ρ_1 = density of lighter liquid $= 1000 \times S_1$

S_2 = sp. gr. of heavy liquid.

ρ_2 = density of heavy liq. $= 1000 \times S_2$

As the pr. is same for the horizontal surface

So pr. above horizontal datum line A-A in left column and in right column of U-tube manometer should be same.

pr. above A-A in left column $= p + \rho_1 g h_1$

pr. above A-A in right column $= \rho_2 g h_2$

Equating 2 prs.

$p + \rho_1 g h_1 = \rho_2 g h_2 \Rightarrow p = \rho_2 g h_2 - \rho_1 g h_1$

→ For vacuum pr. :-

pr. above A-A in left column
$= \rho_2 g h_2 + \rho_1 g h_1 + p$

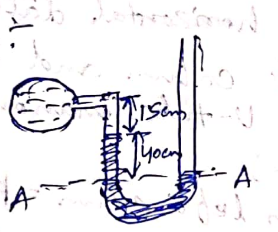
pr. above AA in right column = 0.

$\therefore \rho_2 g h_2 + \rho_1 g h_1 + p = 0$

$\Rightarrow p = -(\rho_2 g h_2 + \rho_1 g h_1)$

Problem-8 A simple U-tube manometer containing mercury is connected to a pipe in which a fluid of sp. gr. 0.8 and having vacuum pr. is flowing. The other end of the manometer is open to atm. Find the vacuum pr. in pipe, if the diff. of mercury level in the two limbs is 40 cm and the height of fluid in the left from centre of pipe is 15 cm below.

Ans :-



S.G of fluid, $S_1 = 0.8$

S.G of mercury $S_2 = 13.6$

$\rho_1 = 800\ kg/m^3$

$\rho_2 = 13.6 \times 1000$
$= 13600\ kg/m^3$

$h_2 = 40cm = 0.4\ m$.

$h_1 = 15\ cm = 0.15\ m$.

Equating pr. above datum line A-A,

$\therefore \rho_2 g h_2 + \rho_1 g h_1 + p = 0$

$\Rightarrow p = -[\rho_2 g h_2 + \rho_1 g h_1]$

$= -[13.6 \times 1000 \times 9.81 \times 0.4 + 800 \times 9.81 \times 0.15]$

$= -54543.6\ N/m^2 = -5.454\ N/cm^2$

→ Single column Manometer :

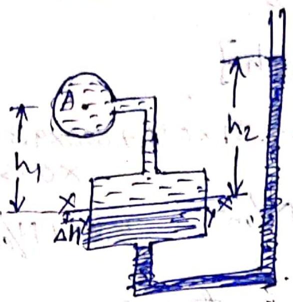
It is a modified form of a U-tube manometer in which a reservoir, having large c/s area (about 100 times) as compared to the area of tube is connected to one of the limbs. So due to large c/s area of reservoir, for any variation in pr., the change in the liquid level in reservoir will be very small which may be neglected and hence the pr. is given by the height of liquid in the other limb.

It is of 2 types :-

1) Vertical single column Manometer

2) Inclined single column Manometer

1) vertical single column Manometer :



Let Δh = fall of heavy liquid in reservoir

h_2 = Rise of heavy liquid in right limb.

h_1 = height of centre of pipe above x-x

p_A = pr. at A, to be measured

A = c/s area of reservoir.

a = c/s area of right limb.

S_1 = sp. gr. of Liquid in pipe.

S_2 = sp. gr. of heavy liquid in reservoir and right limb.

ρ_1 = density of liquid in pipe.

ρ_2 = Density of liquid in reservoir.

Fall of heavy liquid in reservoir will cause a rise in heavy liquid level in right limb.

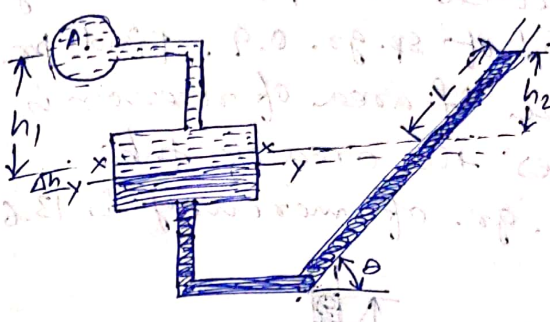
$\therefore \quad A \times \Delta h = a \times h_2 \Rightarrow \Delta h = \frac{a h_2}{A}$

$p_A = \frac{a h_2}{A}(\rho_2 g - \rho_1 g) + h_2 \rho_2 g - h_1 \rho_1 g$

As area A is very large as compared to a so $\frac{a}{A}$ is very small & neglected.

Then $p_A = h_2 \rho_2 g - h_1 \rho_1 g$.

(2) Inclined single column Manometer :



Due to inclination the distance moved by the heavy liquid in right limb will be more. This is more sensitive.

L = Length of heavy liquid moved in right limb from x-x.

θ = inclination of right limb with horizontal

h_2 = vertical rise of heavy liquid in right limb x-x = $L\sin\theta$.

$p_A = h_2 \rho_2 g - h_1 \rho_1 g$

$= L\sin\theta . \rho_2 g - h_1 \rho_1 g$

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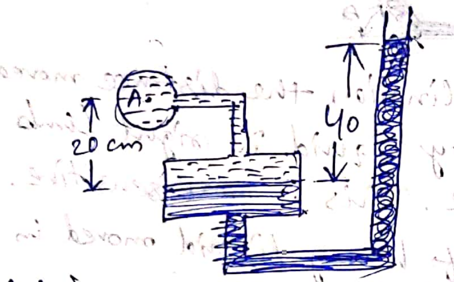
Problem-9 : A single column manometer is connected to a pipe containing a liquid of sp. gr. 0.9. Find pr. in pipe if area of reservoir is 100 times the area of tube. The sp. gr. of mercury is 13.6



Ans :-

Given, $S_1 = 0.9$, so $\rho_1 = 900\ kg/m^3$

$S_2 = 13.6$, $\rho_2 = 13600\ kg/m^3$

$$\frac{\text{Area of reservoir}}{\text{Area of limb}} = \frac{A}{a} = 100$$

height of liquid, $h_1 = 20cm = 0.2m$

Rise of mercury in right limb

$h_2 = 40cm = 0.4m$

$$p_A = \frac{a}{A} h_2 [\rho_2 g - \rho_1 g] + h_2 \rho_2 g - h_1 \rho_1 g$$

$$= \frac{1}{100} \times 0.4 [13600 \times 9.81 - 900 \times 9.81]$$

$$+ 0.4 \times 13.6 \times 1000 \times 9.81 - 0.2 \times 900 \times 9.81$$

$$= 5.21\ N/cm^2$$

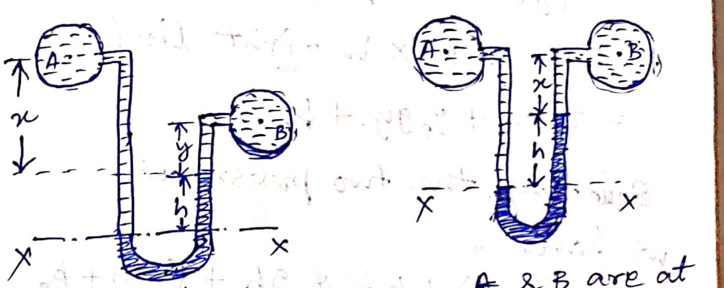
→ **Differential Manometers :-**

These are devices used for measuring the difference of pr. b/w two points in a pipe or in two different pipes. So it consists of a U-tube, containing heavy liquid, whose two ends are connected to the points, whose difference of pr. is to be measured.

Type → 1) U-tube differential manometer

2) Inverted U-tube differential manometer.

1) **U-tube Differential manometer :-**



(Two pipes at diff. levels)

A & B are at the same level

If two points A & B are at different level and contain liquids of different sp. gr. these points are connected to the U-tube differential manometer. Pr at A & B are p_A & p_B.

h = Difference of mercury level in the U-tube.

y = Distance of the centre of B, from mercury level in right limb

x = Distance of centre of A, from mercury level in right limb

ρ_1 = density of liquid at A.

ρ_2 = Density of liquid at B.

ρ_g = density of heavy liquid or mercury.

pr. above X-X in ~~right~~ left limb.

$= \rho_1 g(h+x) + p_A$

pr. above X-X in right limb

$= \rho_g gh + \rho_2 gy + p_B$

Equating the two pressures, we have,

$$\rho_1 g(h+x) + p_A = \rho_g gh + \rho_2 gy + p_B$$

or $p_A - p_B = hg(\rho_g - \rho_1) + \rho_2 gy - \rho_1 gx$.

This is the difference of pr. at A&B

If the two points A & B are at same level and contain same liquid of density ρ_1, then,

pr. above XX in right limb $= \rho_g gh + \rho_1 gx + p_B$

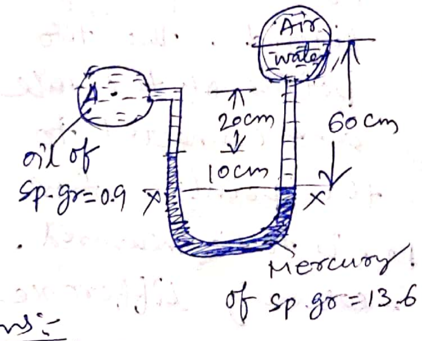
pr. above X-X in left limb $= \rho_1 g(h+x) + p_A$

Equating the two pr.

$$\rho_g gh + \rho_1 gx + p_B = \rho_1 g(h+x) + p_A$$

$$\Rightarrow p_A - p_B = g \cdot h(\rho_g - \rho_1)$$

Problem-10 A differential manometer is connected at two points A & B as shown in fig. At B air pressure is 9.81 N/cm² (abs), find the absolute pr. at A.



Ans:- Given, Air pr. at B = 9.81 N/cm²

or $p_B = 9.81 \times 10^4$ N/m²

$\rho_1 = 0.9 \times 1000 = 900$ kg/m³

$\rho_g = 13.6 \times 1000 = 13600$ kg/m³

$\rho_2 = 1000$ kg/m³

pr. at A = p_A

pr. above xx in right limb

$= 1000 \times 9.81 \times 0.6 + p_B = 5886 + 98100$
$= 103986 \ N/m^2$

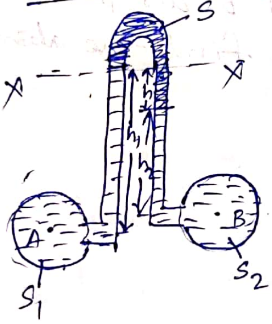
pr. above x-x in left limb

$= 13.6 \times 1000 \times 9.81 \times 0.1 + 900 \times 9.81 \times 0.2 + p_A$

Equating, $103986 = 13341.6 + 1765.8 + p_A$

$\Rightarrow p_A = 8.887 \ N/cm^2$ = absolute pr. at A

→ **Inverted U-tube differential Manometer**



It consists of an inverted U-tube, containing a light liquid. The two points of the tube are connected to the points whose difference of pr. is to be measured. It is used for measuring difference of low pressures.

Let the $p_A > p_B$

h_1 = ht. of liquid in left limb below X-X.

h_2 = ht. of liquid in right limb.

h = difference of light liquid

ρ_1 = density of liquid at A

ρ_2 = density of liquid at B

ρ_s = density of light liquid

pr. in left limb below X-X

$= p_A - \rho_1 g h_1$

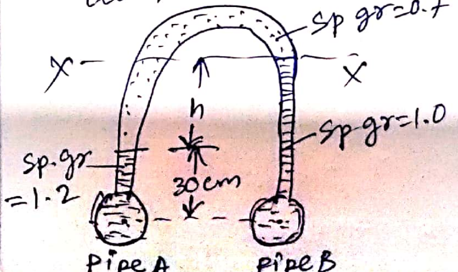
pr. in right limb below X-X

$= p_B - \rho_2 g h_2 - \rho_s g h$

Equating, $p_A - \rho_1 g h_1 = p_B - \rho_2 g h_2 - \rho_s g h$

$\Rightarrow p_A - p_B = \rho_1 g h_1 - \rho_2 g h_2 - \rho_s g h$

Problem-11 Find out the differential reading 'h' of an inverted U-tube manometer containing oil of sp. gr. 0.7 as the manometric fluid when connected across pipes A & B as in fig., conveying liquids of sp. gr. 1.2 & 1.0 and immiscible with manometric fluid. Pipes A & B are located at the same level and assume the pr at A & B are equal.



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Ans → Given $p_A = p_B$

density of liquid in A $= S_A \times 1000$
$= 1.2 \times 1000$
$= 1200\ kg/m^3$

density of liquid in B $= S_B \times 1000$
$= 1000\ kg/m^3$

$\rho_s = 0.7 \times 1000 = 700\ kg/m^3$.

pr. below X-X in left limb
$= p_A - 1200 \times 9.81 \times 0.3 - 700 \times 9.81 \times h$

pr. below X-X in right limb
$= p_B - 1000 \times 9.81 \times (h + 0.3)$

Equating,

$p_A - 1200 \times 9.81 \times 0.3 - 700 \times 9.81 \times h$
$= p_B - 1000 \times 9.81 \times (h + 0.3)$
$(\because p_A = p_B)$

Solving we have $h = 20\ cm$

(1.3) Hydrostatic forces on Surfaces :-

→ Pr. exerted on an immersed surface

In fluid at rest condn,

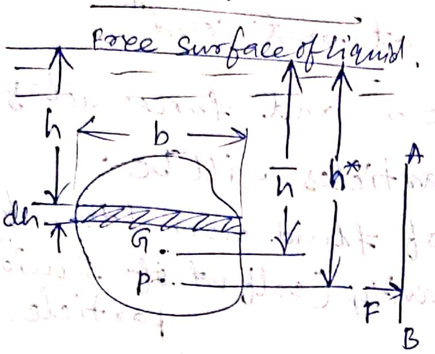
$\tau = \mu \frac{du}{dy} = 0$ and forces acting on fluid particles will be :

1) due to pr. of fluid normal to surface
2) due to gravity (self wt. of fluid particle)

→ Total pr. ⇒ It is defined as the force exerted by a static fluid on a surface either plane or curved when the fluid comes in contact with the surfaces. This force always acts normal to surface.

→ Centre of pr. :- It is defined as the pt. of application of total pr. on the surface. The submerged surface may be vertical, horizontal inclined or curved surface.

(a) vertical plane surface submerged in liquid :-



Consider a plane vertical surface of arbitrary shape immersed in liquid
Let A = Total area of surface
$\bar{h}$ = distance of CG of area from free surface of liquid
G = C.G of plane area
P = centre of pr
h^* = distance of C.P from free surface of liquid

(a) Total pr (F) :-
Total pr may be determined by dividing the entire surface into a number of small parallel strips & force on small strip is calculated and total pr. force on whole area is calculated by integrating the force on small strip

consider a strip of thickness dh & width b at depth h from free surface
pr. intensity on strip = p = ρgh
area of strip, dA = b×dh
Total pr force on strip, dF = p×Area
= ρgh×b×dh

Total pr. force on whole surface
$F = \int dF = \int \rho g h \times b \times dh = \rho g \int b \times h \times dh$
but $\int b \times h \times dh = \int h \times dA$
= Moment of surface area about free surface
= area of surface × distance of CG from free surface
= $A\bar{h}$

or $\boxed{F = \rho g A \bar{h}}$

(b) centre of pr. (h^*) :-
It is calculated by principle of moments
The resultant force F act at P at distance h^* from surface ..
So moment of F about surface = $F \times h^*$
Moment of dF about surface
= $dF \times h = \rho g h \times b \times dh \times h$
Sum of moment of all such forces about free surface
= $\int \rho g h \times b \times dh \times h = \rho g \int b h \times h dh$

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$= \rho g \int b h^2 dh = \rho g \int h^2 dA$.

but $\int b h^2 dh$ = M.O.I. of surface about free surface $= I_o$

Sum of moments about surface $= \rho g I_o$

So $F \times h^* = \rho g I_o$.

but $F = \rho g A \bar{h}$

So $\rho g A \bar{h} \times h^* = \rho g I_o$

or $h^* = \dfrac{I_o}{A\bar{h}}$

By theorem of parallel axis,

$I_o = I_G + A\bar{h}^2$

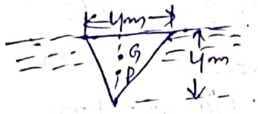
So $h^* = \dfrac{I_G + A\bar{h}^2}{A\bar{h}} = \dfrac{I_G}{A\bar{h}} + \bar{h}$

So C.P. i.e. h^* lies below C.G of the vertical surface.

and distance of C.P from free surface of liquid is independent of density of liquid.

Problem-12 | Determine the total pr. and C.P on an isosceles Δ plate of base 4m and altitude 4m when it is immersed vertically in an oil of sp. gr. 0.9. The base of plate coincides with the free surface of oil.

Ans:-



Given, base of ~~pate~~ plate $= b = 4m$

ht. of plate $= h = 4m$

Area, $A = \dfrac{bh}{2} = \dfrac{4 \times 4}{2} = 8m^2$

S of oil $= 0.9$, $\rho = 900 \text{ kg/m}^3$.

$\bar{h} = \frac{1}{3} h = 1.33m$.

$F = \rho g A \bar{h} = 900 \times 9.81 \times 8 \times 1.33$

$= 95947.6$ N.

$h^* = \dfrac{I_G}{A\bar{h}} + \bar{h}$

$I_G = \dfrac{bh^3}{36} = \dfrac{4 \times 4^3}{36} = 7.11 m^4$

$h^* = \dfrac{7.11}{8 \times 1.33} + 1.33 = 1.99m$ (Ans)

Problem-13 A tank contains water upto a ht. of 0.5m above the base. An immiscible liquid of sp.gr. = 0.8 is filled on the top of water upto 1m height. Calculate:

(i) total pr on one side of tank,
(ii) position of C.P for one side of tank, which is 2m wide.

Ans :- Given, Depth of water = 0.5 m

Depth of liquid = 1 m

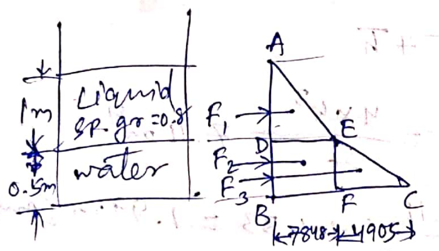
Sp. gr. of liquid = 0.8

density, $\rho_1 = 0.8 \times 1000 = 800\ kg/m^3$

density of water, $\rho_2 = 1000\ kg/m^3$

width of tank = 2m.

(i) Total pr. on one side (by Pr. diagram)



pr. intensity on top, $p_A = 0$

$p_D = \rho_1 g h_1$

$= 800 \times 9.81 \times 1 = 7848\ N/m^2$

for BC, $p_B = \rho_1 g h_1 + \rho_2 g \times 0.5$

$= 7848 + (1000 \times 9.81 \times 0.5) = 12753\ N/m^2$

Now force, F_1 = Area of $\triangle ADE$ × width of tank

$= \frac{1}{2} \times AD \times DE \times 2.0 = \frac{1}{2} \times 1 \times 7848 \times 2$

$= 7848\ N$.

Force F_2 = area of rectangle DBFE × width of tank

$= 0.5 \times 7848 \times 2 = 7848\ N$

F_3 = area of $\triangle EFC$ × width of tank

$= \frac{1}{2} \times EF \times FC \times 2.0$

$= \frac{1}{2} \times 0.5 \times 4905 \times 2.0 = 2452.5\ N$.

Total pr., $F = F_1 + F_2 + F_3$

$= 7848 + 7848 + 2452.5$

$= 18148.5\ N$

(ii) centre of pr (h^*).

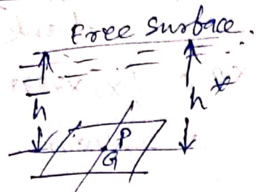
Taking moments of all force about A, we get,

$F \times h^* = F_1 \times \frac{2}{3} AD + F_2 (AD + \frac{1}{2} BD) + F_3 [AD + \frac{2}{3} BD]$

$\Rightarrow 18148.5 \times h^* = 7848 \times \frac{2}{3} \times 1 + 7848 (1.0 + \frac{0.5}{2}) + 2452.5 (1 + \frac{2}{3} \times .5)$

$\Rightarrow h^* = 1.009\ m$ from top.

(b) Horizontal plane surface submerged in liquid :-



Consider plane surface immersed in a static fluid. As every point of the surface is at the same depth from free surface, pr. intensity will be equal on the entire surface and equal to $p = \rho g h$, where h is depth of surface.

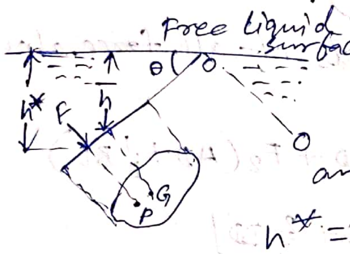
Let A = total area of surface

Then total force, F, on surface

$= p \times Area = \rho g h \times A = \rho g A \bar{h}$

here $\bar{h} = h^* = h$.

(c) Inclined plane surface submerged in liquid :-



Total pr.,

$F = \rho g A \bar{h}$

and centre of pr.

$$h^* = \frac{I_G \sin^2\theta}{A\bar{h}} + \bar{h}$$

I_G = MOI of inclined surface about an axis passing through G and parallel to O–O.

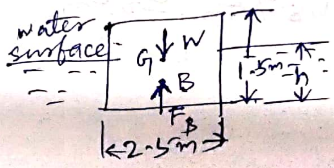
Buoyancy and Floatation

→ **Buoyancy** :- When a body is immersed in a fluid, an upward force is exerted by the fluid on the body. This upward force is equal to weight of the fluid displaced by the body and is called force of buoyancy or simply buoyancy.

→ **Centre of buoyancy** :-

It is that point through which the force of buoyancy is supposed to act. It is the centre of gravity of the fluid displaced.

[Problem-1] Find the volume of water displaced and position of centre of buoyancy for a wooden block of width 2.5 m and depth 1.5 m, when it floats horizontally in water. Density of wooden block is 650 kg/m³ and its length 6 m.



Given, width = 2.5 m

Depth = 1.5 m

Length = 6 m.

volume = 2.5 × 1.5 × 6

= 22.5 m³

Density of wooden block

$\rho = 650 \text{ kg/m}^3$

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Weight of block $= \rho \times g \times$ Volume.
$= 650 \times 9.81 \times 22.5 = 143471\,N$

For equilibrium, the wt. of wt. of water displaced = wt. of wooden block.
$= 143471\,N$.

$\therefore$ volume of water displaced
$= \dfrac{\text{wt. of water displaced}}{\text{wt. density of water}}$

$= \dfrac{143471}{1000 \times 9.81} = 14.625\,m^3$.

Position of centre of Buoyancy.

Volume of wooden block in water
= volume of water displaced.

or $2.5 \times h \times 6.0 = 14.625$
(where h = depth of block in water.)
$\Rightarrow h = 0.975\,m$

So centre of buoyancy
$= \dfrac{0.975}{2} = 0.4875\,m$ from base.

[Problem-2] A body of dimensions 1.5m × 1m × 2m, weighs 1962N in water. Find its weight in air. What will be its sp. gr.?

Ans → Given, volume of body
$= 1.5 \times 1 \times 2 = 3\,m^3$

wt. in water $= 1962\,N$

volume of water displaced
= volume of body $= 3\,m^3$

$\therefore$ wt. of water displaced.
$= 1000 \times 9.81 \times 3 = 29430\,N$.

For equilibrium of body,
[wt. of body in air − wt. of water displaced = wt. in water.

$\Rightarrow W_{air} - 29430 = 1962$

$\Rightarrow W_{air} = 31392\,N$

Mass ~~bo~~ of body $= \dfrac{\text{wt. in air}}{g}$

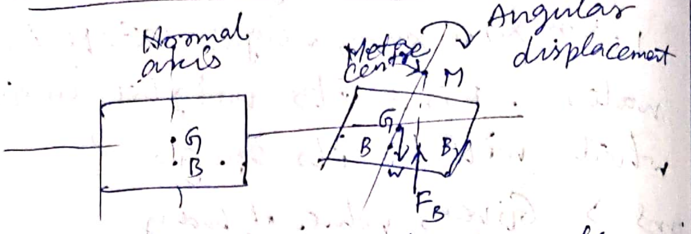
$= \dfrac{31392}{9.81} = 3200\,kg$.

ρ of body $= \dfrac{\text{Mass}}{\text{volume}} = \dfrac{3200}{3} = 1066.67$

$\therefore$ Sp. gr. of body $= \dfrac{1066.67}{1000}$

$= 1.067$.

→ Meta-centre



Metacentre is defined as the point about which a body starts oscillating when the body is tilted by a small angle. It is that pt. at which line of action of force of buoyancy will meet the normal axis of the body when body is given a small angular displacement.

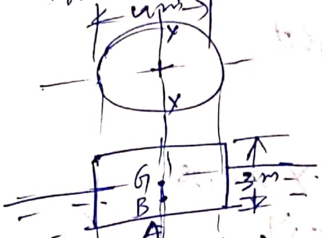
→ Meta-centric ht.

Distance MG, i.e., the distance between the meta-centre of a floating body and CG of body.

$$MG = \frac{I}{\forall} - BG$$

(where $\forall$ = volume of body submerged in water)

[problem →] A solid cylinder of dia 4m has a height of 3m. Find the meta-centric ht. of cylinder when it is floating in water with its axis vertical. The sp. gr. of the cylinder = 0.6



Given, Dia of cylinder $D = 4m$

Ht of cylinder, $h = 3m$

Sp gr. of cylinder = 0.6

Depth of immersion of cylinder $= 0.6 \times 3 = 1.8m$

Density of cylinder $= 0.6 \times 1000 = 600\ kg/m^3$

wt. of cylinder $= \rho g V = 600 \times 9.81 \times \frac{\pi}{4}(4)^2 \times 3$

$= 221896.97\ N.$

we know wt. of water displaced = wt. of cylinder

$= 221896.97\ N.$

So volume of water displaced

$= \frac{\text{wt. of water displaced}}{\text{wt. density of water}}$

$= \frac{221896.97}{1000 \times 9.81} = 22.62\ m^3$

volume of cylinder in water = volume of water displaced

$$\Rightarrow \frac{\pi}{4}(4)^2 \times h = 22.62$$

$$\Rightarrow h = 1.79 \simeq 1.8m.$$

So $AB = \frac{1.8}{2} = 0.9m$

and $AG = \frac{3}{2} = 1.5m$

$BG = AG - AB = 1.5 - 0.9 = 0.6m.$

So Metacentric height

$$GM = \frac{I}{\forall} - BG$$

I = MOI about Y-Y axis

$$= \frac{\pi D^4}{64} = \frac{\pi}{64} \times (4)^4$$

$\forall$ = volume of cylinder in water

$$= \frac{\pi}{4}(D)^2 \times \text{depth of immersion}$$

$$= \frac{\pi}{4}(4)^2 \times (1.8)\ m^3$$

So $$GM = \frac{\frac{\pi}{64} \times (4)^4}{\frac{\pi}{4} \times (4)^2 \times 1.8} - 0.6 = -0.05m$$

−ve sign means metacentre M is below the C.G (G)

→ Stability of submerged body

Stable Equilibrium — unstable Equilibrium — Neutral Equilibrium

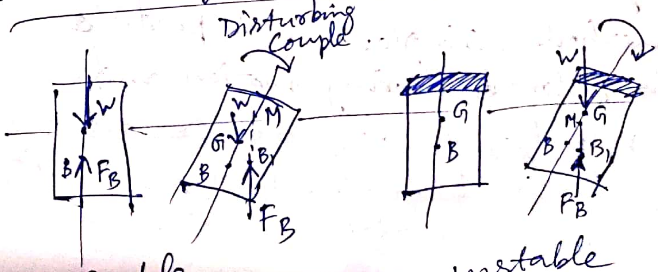
Consider a baloon submerged in air. If the lower portion have heavier material then G lies below B when angular displacement applied in clockwise direcⁿ then W & F_B constitute a couple in anti-clockwise direction. This is stable Equilibrium.

stable → when $W = F_B$ & B is above G,

unstable → when $W = F_B$ & B is below G.

Neutral → If $F_B = W$ & B & G are at same point.

→ Stability of floating body



Stable — unstable

Stable ⇒ If M is above G

Unstable ⇒ If M is below G.

Neutral :- If M & G are at same pt.

Equilibrium	Floating	Submerged
Stable	M is above G	B is above G
unstable	M is below G	B is below G
Neutral	M & G coincide	B & G coincide

(2) kinematics of fluid flow

(2.1) Basic equation of fluid flow and their application :-

kinematics is defined as that branch of science which deals with motion of particles without considering the forces causing the motion.

Fluid motion is described by two methods. In Lagrangian method, single fluid particle is followed during motion and velocity, acc^n, etc are describes.

In Eulerian method, velocity, acc^n, etc. are described at a point in flow field.

(2.3) Types of fluid flow :-

(a) Steady and Unsteady flows

Steady flow is defined as that type of flow in which fluid characteristics like velocity, pressure, density, etc., at a point don't change with time.

Mathematically $\left(\frac{\partial v}{\partial t}\right)_{x_0,y_0,z_0} = \left(\frac{\partial p}{\partial t}\right)_{x_0,y_0,z_0}$

(x_0, y_0, z_0) is a fixed point in fluid field

$= \left(\frac{\partial \rho}{\partial t}\right)_{x_0,y_0,z_0} = 0$

Unsteady flow is that type of flow in which velocity, pr, etc. at a point changes wrt time.

$\left(\frac{\partial v}{\partial t}\right)_{x_0,y_0,z_0} \neq 0, \left(\frac{\partial p}{\partial t}\right)_{x_0,y_0,z_0} \neq 0$, etc.

(b) uniform & non-uniform flow

uniform flow is that type of flow in which velocity at any given time does not change wrt. space. $\left(\frac{\partial v}{\partial s}\right)_{t=const} = 0$

Non-uniform → velocity at any given time changes wrt space

$\left(\frac{\partial v}{\partial s}\right)_{t=const} \neq 0$

(c) Laminar & Turbulent flow

Laminar → fluid particles move along well-defined or stream line and all stream lines are straight and parallel. particles move in laminas or layers gliding smoothly over adjacent layer.

Turbulent → fluid particles move in zig-zag way. Eddies formation responsible for high energy loss.

For pipe flow this type of flow is determined by Reynold's No. $\left(\frac{Vd}{\nu}\right)$ If $R_e < 2000$ → Laminar

$2000 < R_e < 4000$ → Laminar or Turbulent.

$R_e > 4000$ → Turbulent flow

(d) Compressible and Incompressible flow

Compressible → ~~flow~~ ρ is not const.

$\rho \neq$ const.

Incompressible → ρ = const.

(e) Rotational and Irrotational flow

Rotational → fluid particles flow along stream-lines & also rotate about their own axis

Irrotational → If fluid particles flowing along stream lines don't rotate about their axis.

(f) 1, 2 & 3D flows :-

1D → flow parameter like velocity is a function of time and one space co-ordinate only.

$u = f(x)$, $v = 0$, $w = 0$

u, v, w are velocity components along x, y, z directions respectively

2D → ~~flow~~ velocity is func^n of time and two rectangular space co-ordinates

$u = f_1(x, y)$, $v = f_2(x, y)$, and $w = 0$

3D → velocity is function of time and three mutually ⊥^r direc^ns.

$u = f_1(x, y, z)$, $v = f_2(x, y, z)$

$w = f_3(x, y, z)$

(2.1) Rate of flow (discharge) (Q) :-

It is defined as quantity of fluid flowing per second through a section of a pipe or a channel.

For incompressible fluid, rate of flow is volume of fluid/s. and for compressible fluid → wt. of fluid per second

∴ For liquids unit of Q are m^3/s or l/s

For gases unit of Q is kgf/s or N/s

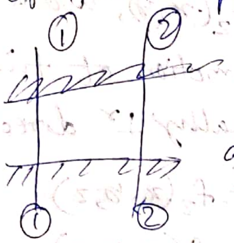
$Q = A \times V$ where A = C/s area of pipe

V = avg. velocity of fluid across secⁿ.

→ **Continuity Eqⁿ**

This eqⁿ is based on the principle of conservation of mass.

In a pipe the fluid flowing at all the C/s, the quantity of fluid per second is constant.



Rate of flow at section 1-1 = $\rho_1 A_1 V_1$

and Rate of flow at secⁿ 2-2 = $\rho_2 A_2 V_2$

According to law of conservation of mass,

Rate of flow at secⁿ 1-1

= Rate of flow at secⁿ 2-2

or $\rho_1 A_1 V_1 = \rho_2 A_2 V_2$

If fluid is incompressible → $\rho_1 = \rho_2$

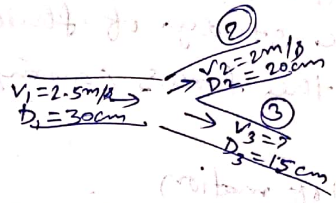
or $A_1 V_1 = A_2 V_2$

where A_1 & A_2 = Area of pipe at 1-1 & 2-2

V_1 & V_2 = Avg. velocity at any C/s 1-1 & 2-2

Problem-1 A 30 cm diameter pipe, conveying water, branches into two pipes of diameters 20 cm & 15 cm respectively. If the average velocity in the 30 cm diameter pipe is 2.5 m/s, find the discharge in this pipe. Also determine the velocity in 15 cm pipe if the average velocity in 20 cm dia pipe is 2 m/s.

Ans :-



Given, $D_1 = 30\text{ cm} = 0.30\text{ m}$

So $A_1 = \frac{\pi}{4} D_1^2 = \frac{\pi}{4}(.3)^2 = 0.07068\text{ m}^2$

$V_1 = 2.5\text{ m/s}$

$D_2 = 20\text{ cm} = 0.20\text{ m}$

$A_2 = \frac{\pi}{4}(.2)^2 = 0.0314\text{ m}^2$

$V_2 = 2\text{ m/s}$,

$D_3 = 0.15\text{ m}$

$A_3 = \frac{\pi}{4}(0.15)^2 = 0.01767\text{ m}^2$

According to continuity Eqⁿ

$Q_1 = Q_2 + Q_3$

but $Q_1 = A_1 V_1 = 0.07068 \times 2.5$
$= 0.1767 \, m^3/s.$

$Q_2 = A_2 V_2 = 0.0628 \, m^3/s$

So $0.1767 = 0.0628 + Q_3$

$\Rightarrow Q_3 = 0.1139 \, m^3/s = A_3 V_3$

$\Rightarrow 0.1139 = 0.01767 \times V_3$

$\Rightarrow V_3 = 6.44 \, m/s$

→ **Dynamics of fluid flow**

It is the study of fluid motion with the forces causing flow.

→ **Bernoulli's Eqⁿ of motion**

Statement :- It states that in a steady, ideal flow of any incompressible fluid, the total energy at any point of the fluid is constant. The total energy consists of pressure energy, kinetic energy and potential or datum energy.

These energies per unit wt. of the fluid are :

pr. Energy $= \frac{p}{\rho g}$

kinetic Energy $= \frac{v^2}{2g}$

Datum or potential Energy $= Z$

Mathematically,

$$\frac{p}{\rho g} + \frac{v^2}{2g} + Z = \text{const}$$

Assumptions

(i) Fluid is ideal (viscosity is zero)
(ii) Flow is steady.
(iii) Flow is incompressible.
(iv) Flow is irrotational.

Bernoulli's Eqⁿ for Real fluid

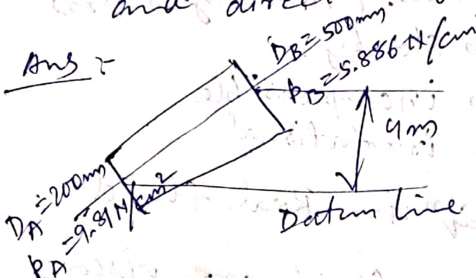
All real fluids are viscous and hence offer resistance to flow. So there is some losses in fluid flow. So considering these losses Bernoulli's Eqⁿ for real fluid is.

$$\frac{p_1}{\rho g} + \frac{v_1^2}{2g} + z_1 = \frac{p_2}{\rho g} + \frac{v_2^2}{2g} + z_2 + h_L$$

h_L = loss of energy b/w 1 & 2.

problem-2 A pipeline carrying oil of sp. gr. 0.87, changes in diameter from 200mm dia at position A to 500 mm dia at a position B which is 4m at a higher level. If the pr. at A & B are 9.81 N/cm² and 5.886 N/cm² respectively and the discharge is 200 l/s, determine the loss of head and direction of flow.

Ans :-



Given, $Q = 200\ l/s = 0.2\ m^3/s$

sp. gr. of oil = 0.87

ρ for oil $= 0.87 \times 1000 = 870\ kg/m^3$

At sec^n A, $D_A = 200\ mm = 0.2m$

area, $A_A = \frac{\pi}{4}(D_A)^2 = \frac{\pi}{4}(0.2)^2$

$= 0.0314 m^2$

$p_A = 9.81\ N/cm^2$

$= 9.81 \times 10^4\ N/m^2$

If datum line passing through A,

then $Z_A = 0$

$\therefore V_A = \frac{Q}{A_A} = \frac{0.2}{0.0314} = 6.369 m/s$

at sec^n B, $D_B = 0.50m$

$A_B = \frac{\pi}{4}(0.5)^2 = 0.1963 m^2$

$p_B = 5.886 \times 10^4\ N/m^2$

$Z_B = 4m$, $V_B = \frac{Q}{Area} = \frac{0.2}{0.1963}$

$= 1.018 m/s$

Total energy at A

$= E_A = \frac{p_A}{\rho g} + \frac{V_A^2}{2g} + Z_A$

$= \frac{9.81 \times 10^4}{870 \times 9.81} + \frac{(6.369)^2}{2 \times 9.81} + 0$

$= 13.557 m.$

Total energy at B

$= E_B = \frac{p_B}{\rho g} + \frac{V_B^2}{2g} + Z_B$

$= \frac{5.886 \times 10^4}{870 \times 9.81} + \frac{(1.018)^2}{2 \times 9.81} + 4$

$= 10.948 m.$

as $E_A > E_B$ so flow from A to B

Loss of head, $h_L = E_A - E_B = 2.609 m$

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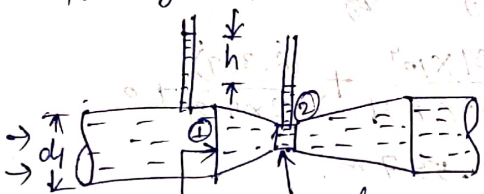
Practical Applications of Bernoulli's Theorem :-

① **Venturimeter** :- It is a device used for measuring the rate of flow of a fluid flowing through a pipe. It has 3 parts:

(i) A short converging part.
(ii) Throat
(iii) Diverging part.

→ Expression for rate of flow through venturimeter :-



Let d_1 = dia. at inlet or at sec^n (1)

p_1 = pr. at sec^n (1)

v_1 = velocity of fluid at sec^n (1)

a_1 = area at sec^n (1) $= \frac{\pi}{4} d_1^2$

d_2, p_2, v_2, a_2 are values at sec^n (2).

Applying Bernoulli's eq^n at sec^n (1) & (2)

$$\frac{p_1}{\rho g} + \frac{v_1^2}{2g} + z_1 = \frac{p_2}{\rho g} + \frac{v_2^2}{2g} + z_2$$

As pipe is horizontal so $z_1 = z_2$

So $$\frac{p_1 - p_2}{\rho g} = \frac{v_2^2}{2g} - \frac{v_1^2}{2g}$$

but $\frac{p_1 - p_2}{\rho g} = h$ = pr. head diff. at 1 & 2

So $$h = \frac{v_2^2}{2g} - \frac{v_1^2}{2g}$$

we know $a_1 v_1 = a_2 v_2$ or $v_1 = \frac{a_2 v_2}{a_1}$

putting v_1 we have,

$$h = \frac{v_2^2}{2g} - \frac{\left(\frac{a_2 v_2}{a_1}\right)^2}{2g}$$

$$\Rightarrow v_2^2 = 2gh \frac{a_1^2}{a_1^2 - a_2^2}$$

or $$v_2 = \frac{a_1}{\sqrt{a_1^2 - a_2^2}} \sqrt{2gh}$$

$$Q = a_2 v_2 = \frac{a_1 a_2}{\sqrt{a_1^2 - a_2^2}} \times \sqrt{2gh}$$ (Theoretical discharge)

$$Q_{act.} = C_d \times \frac{a_1 a_2}{\sqrt{a_1^2 - a_2^2}} \times \sqrt{2gh}$$

C_d = coefficient of venturimeter and its value is less than 1. (If C_d not given then take 0.98)

Value of 'h' by differential U-tube manometer

Case-1 : when manometer contains liquid heavier than the flowing liquid.

Let S_h = sp. gr. of heavier liq.

S_o = sp. gr. of liquid flowing through pipe

x = Difference of heavier liquid column in U-tube

Then $h = x\left[\frac{S_h}{S_o} - 1\right]$

Case-2 : when manometer contains liquid lighter than flowing liquid.

$h = x\left[1 - \frac{S_l}{S_o}\right]$

S_l = sp. gr. of lighter liquid in U-tube.

Case-3 : Inclined venturimeter with differential U-tube manometer. Manometer has heavier liquid

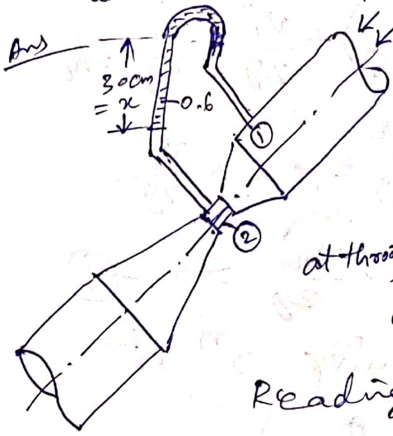
then $h = x\left[\frac{S_h}{S_o} - 1\right]$

Case-4 → Inclined venturimeter & lighter liquid,

$h = x\left[1 - \frac{S_l}{S_o}\right]$

Problem-3 | Find the discharge of water flowing through a pipe 30 cm dia. placed in an inclined position where a venturimeter is inserted, having a throat diameter of 15 cm. The difference of pr. b/w the main and throat is measured by a liquid of sp. gr. 0.6 in an inverted U-tube which gives a reading of 30 cm. The loss of head b/w the main and throat is 0.2 times the kinetic head of the pipe.

Ans



Given,

Dia at inlet $d_1 = 30$ cm

$a_1 = \frac{\pi}{4}(30)^2 = 706.85 cm^2$

at throat, $d_2 = 15$ cm

$a_2 = \frac{\pi}{4}(15)^2 = 176.7 cm^2$

Reading of differential manometer, $x = 30$ cm

Difference of pr. head, h is given by

$$\left(\frac{p_1}{\rho g} + z_1\right) - \left(\frac{p_2}{\rho g} + z_2\right) = h$$

& $h = x\left[1 - \frac{S_l}{S_o}\right]$

where $S_1 = 0.6$, $S_0 = 1$

So $h = 30\left[1 - \frac{0.6}{1.0}\right] = 12$ cm of water

Loss of head,

$h_L = 0.2 \times$ kinetic head of pipe $= 0.2 \times \frac{v_1^2}{2g}$

Applying Bernoulli's eq^n at sec^n (1) & (2) we get

$$\frac{p_1}{\rho g} + \frac{v_1^2}{2g} + z_1 = \frac{p_2}{\rho g} + \frac{v_2^2}{2g} + z_2 + h_L$$

$$\Rightarrow \left(\frac{p_1}{\rho g} + z_1\right) - \left(\frac{p_2}{\rho g} + z_2\right) + \frac{v_1^2}{2g} - \frac{v_2^2}{2g} = h_L$$

$$\Rightarrow h + \frac{v_1^2}{2g} - \frac{v_2^2}{2g} = 0.2 \times \frac{v_1^2}{2g}$$

$$\Rightarrow 12 + 0.8\frac{v_1^2}{2g} - \frac{v_2^2}{2g} = 0 \quad \text{--- (1)}$$

Applying continuity eq^n at (1) & (2) we get,

$a_1 v_1 = a_2 v_2$

$\Rightarrow v_1 = \frac{a_2}{a_1} v_2 = \frac{v_2}{4}$

Substituting value of v_1 in (1) we get,

$$12 + \frac{0.8}{2g}\left(\frac{v_2}{4}\right)^2 - \frac{v_2^2}{2g} = 0$$

$\Rightarrow v_2 = 157.4$ cm/s

$\therefore$ discharge $= a_2 v_2 = 27.8$ l/s.

(2) Orifice Meter or Orifice plate:-

It is a device used for measuring the rate of flow of a fluid through a pipe. It is a cheaper device compared to venturimeter.

It consists of a flat circular plate with circular sharp edged hole called orifice, concentric with pipe. Orifice diameter is 0.5 times the dia. of pipe & may vary from 0.4 to 0.8 times the pipe diameter.

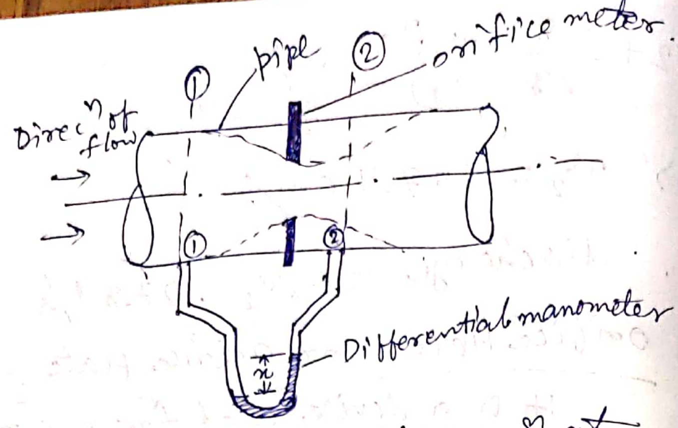
A differential manometer is connected at section (1), at a distance of about 1.5 to 2 times the pipe diameter upstream from orifice plate, and at section (2), at a distance of about half the dia. of orifice on d/s side from orifice plate.

Let p_1 = pr. at sec^n (1)

v_1 = velocity at sec^n (1)

a_1 = area of pipe at sec^n (1)

$p_2, v_2, a_2 \rightarrow$ at sec^n (2).



Appling Bernoulli's eqn at secn (1) & (2) we get,

$$\frac{p_1}{\rho g} + \frac{v_1^2}{2g} + z_1 = \frac{p_2}{\rho g} + \frac{v_2^2}{2g} + z_2$$

or $\left(\frac{p_1}{\rho g} + z_1\right) - \left(\frac{p_2}{\rho g} + z_2\right) = \frac{v_2^2}{2g} - \frac{v_1^2}{2g}$

$\Rightarrow h = \frac{v_2^2}{2g} - \frac{v_1^2}{2g}$

or $2gh = v_2^2 - v_1^2$

$\Rightarrow v_2 = \sqrt{2gh + v_1^2}$ ——— (1)

Now section (2) is at vena-contracta and a_2 represents area at vena-contracta.

a_0 = area of orifice,

$C_c = \frac{a_2}{a_0}$

C_c = Coefficient of contraction

$a_2 = a_0 C_c$ ——— (2)

By continuity eqn, we have,

$a_1 v_1 = a_2 v_2$ or $v_1 = \frac{a_2 v_2}{a_1} = \frac{a_0 C_c}{a_1} v_2$ ——— (3)

Substituting the value of v_1 in eqn-1,

$$v_2 = \sqrt{2gh + \frac{a_0^2 C_c^2 v_2^2}{a_1^2}}$$

$$\Rightarrow v_2 = \frac{\sqrt{2gh}}{\sqrt{1 - \left(\frac{a_0}{a_1}\right)^2 C_c^2}}$$

$$\left(Q = v_2 \times a_2 = v_2 \times a_0 C_c\right)$$

$$\Rightarrow v_2 = \frac{a_0 C_c \sqrt{2gh}}{\sqrt{1 - \left(\frac{a_0}{a_1}\right)^2 C_c^2}} \quad \text{——— (4)}$$

Above expression is simplified by using,

$$C_d = C_c \frac{\sqrt{1 - \left(\frac{a_0}{a_1}\right)^2}}{\sqrt{1 - \left(\frac{a_0}{a_1}\right)^2 C_c^2}}$$

$$\Rightarrow C_c = C_d \frac{\sqrt{1 - \left(\frac{a_0}{a_1}\right)^2 C_c^2}}{\sqrt{1 - \left(\frac{a_0}{a_1}\right)^2}}$$

putting C_c in eqn-4, we get,

$$Q = a_0 \times C_d \frac{\sqrt{1 - \left(\frac{a_0}{a_1}\right)^2 C_c^2}}{\sqrt{1 - \left(\frac{a_0}{a_1}\right)^2}} \times \frac{\sqrt{2gh}}{\sqrt{1 - \left(\frac{a_0}{a_1}\right)^2 C_c^2}}$$

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ii) $Q = \dfrac{C_d\, a_0 a_1 \sqrt{2gh}}{\sqrt{a_1^2 - a_0^2}}$

C_d = coefficient of discharge for orificemeter & its value is smaller than that for a venturimeter.

[Problem-4] An orificemeter with orifice dia. 15cm is inserted in a pipe of 30cm dia. The pr. difference measured by a mercury oil differential manometer on the two sides of the orificemeter gives a reading of 50cm of mercury. Find rate of flow of oil of sp. gr. 0.9 when the coefficient of discharge of orificemeter is 0.64.

Ans:- Given, dia of orifice $d_0 = 15$ cm

$a_0 = \frac{\pi}{4}(15)^2 = 176.7\ cm^2$

Dia of pipe, $d_1 = 30$cm

$a_1 = \frac{\pi}{4}(30)^2 = 706.85\ cm^2$

Sp. gr. of oil $= S_0 = 0.9$

Reading of diff. manometer $x = 50$ cm mercury

$h = x\left[\frac{S_g}{S_0} - 1\right] = 705.5$ cm oil

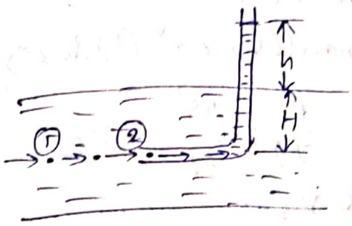
$C_d = 0.64$

$Q = C_d \dfrac{a_0 a_1 \sqrt{2gh}}{\sqrt{a_1^2 - a_0^2}}$

$= 0.64 \times \dfrac{176.7 \times 706.85 \times \sqrt{2 \times 981 \times 705.5}}{\sqrt{(706.85)^2 - (176.7)^2}}$

$= 137414.25\ cm^3/s = 137.4141\ l/s$.

③ Pitot tube



It is a device used for measuring velocity of flow at any point in a pipe or channel.

principle → If velocity of flow at a pt. becomes zero, pr. there is increased due to conversion of K.E into pressure Energy.

Pitot tube is a glass tube bent at right angle. The liquid rises up in tube due to conversion of K.E into Pr. Energy; velocity is determined by measuring the rise of liquid in the tube. Consider ① & ② points at same level. (1) is at far away from tube & (2) is at inlet of pitot tube.

Let p_1 = pr. intensity at (1)
v_1 = velocity of flow at (1)
p_2 = pr. intensity at (2)
v_2 = velocity at pt.-(2)
= 0
H = depth of tube in liquid.
h = rise of liquid in tube above free surface.

Applying Bernoulli's eqn (1) & (2)

$$\frac{p_1}{\rho g}+\frac{v_1^2}{2g}+z_1=\frac{p_2}{\rho g}+\frac{v_2^2}{2g}+z_2$$

as $z_1 = z_2$ & $v_2 = 0$

$\frac{p_1}{\rho g}$ = pr. head at (1) = H

$\frac{p_2}{\rho g}$ = pr. head at (2) = h + H,

So $H+\frac{v_1^2}{2g}=(h+H)$

or $h=\frac{v_1^2}{2g} \Rightarrow v_1=\sqrt{2gh}$

$(v_1)_{act} = C_v\sqrt{2gh}$

C_v = coefficient of pitot tube,

$h = x\left[\frac{S_g}{S_o}-1\right]$

[Problem-5] Find the velocity of flow of an oil through a pipe, when the difference of mercury level in a differential U-tube manometer connected to the two tapping of the pitot tube is 100mm. Take coefficient of pitot tube 0.98 & sp. gr. of oil = 0.8

Ans:- Given: Diff. of mercury level

x = 100mm = 0.1m

sp. gr. of oil = S_o = 0.8.

sp. gr. of mercury, S_g = 13.6

C_v = 0.98

Diff. of pr. head, $h = x\left[\frac{S_g}{S_o}-1\right]$

$= 0.1\left[\frac{13.6}{0.8}-1\right] = 1.6$m of oil.

∴ velocity of flow, $C_v\sqrt{2gh} = 0.98\sqrt{2\times9.81\times1.6}$

$= 5.49$ m/s.

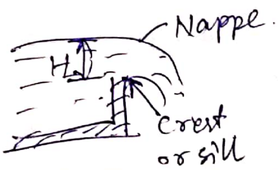
(2.2) Flow over Notches & weirs

Notch is a device for measuring the rate of flow of a liquid through a small channel or a tank. It is an opening in the side of a tank or a small channel in such a way that the liquid surface in tank or channel is below the top edge of opening.

weir is a concrete or masonry str., placed in an open channel over which the flow occurs. It is a vertical wall with sharp edge at top.

Nappe or vein :- The sheet of ~~small~~ water flowing through a notch or over a weir.

Crest or sill :- The bottom edge of a notch or a top of a weir over which water flows.



→ **classification of Notches & weirs** :-

Notches

→ according to shape of opening :-
1) Rectangular Notch
2) Triangular Notch
3) Trapezoidal Notch
4) Stepped Notch.

→ according to effect of sides on nappe.
1). Notch with end contraction.
2) Notch without end contraction or suppressed Notch.

weirs :-

→ according to shape of opening
1) Rectangular weir
2) Triangular weir.
3) Trapezoidal weir (cipolletti weir)

→ according to shape of the crest :
1) sharp-crested weir
2) Broad-crested weir
3) Narrow-crested weir
4) Ogee-shaped weir

→ According to effect of sides on the emerging nappe :
1) weir with end contraction
2) weir without end contraction.

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→ **Discharge over a Rectangular Notch or weir :**

Consider a rectangular notch or weir provided in a channel carrying water.

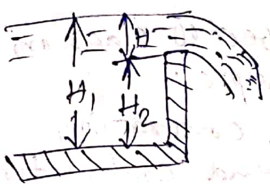
H = head of water over crest

L = length of notch or weir.

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

problem-1 Determine the height of a rectangular weir of length 6m to be built across a rectangular channel. The maxm depth of water on the upstream side of weir is 1.8m and discharge is 2000 l/s. Take $C_d = 0.6$ & neglect end contractions

Ans :-



Given, L = 6m

Depth of waters $H_1 = 1.8m$

$Q = 2000 l/s = 2 m^3/s$

$C_d = 0.6$

H = ht of water above crest of weir.

H_2 = ht of weir.

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

$$\Rightarrow 2.0 = \frac{2}{3} \times 0.6 \times 6.0 \times \sqrt{2 \times 9.81} \times H^{3/2}$$

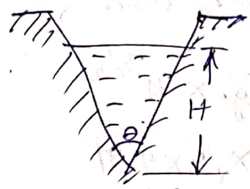
$$= 10.623\, H^{3/2}$$

$$\Rightarrow H = 0.328m$$

So H_2 = ht of weir $= H_1 - H = 1.8 - 0.328$

$= 1.472m$

→ **Discharge over a triangular Notch or weir**



H = head of water above v-notch.

θ = angle of notch

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

If θ = 90° (right angled notch) and $C_d = 0.6$

then $Q = 1.417\, H^{5/2}$

problem-2 water flows over a rectangular ~~notch~~ weir 1m wide at a depth of 150mm 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

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Find depth over the triangular weir.

Ans:- Given, For rectangular weir,

length, $L = 1m$

Depth of water, $H = 150mm = 0.15m$

$C_d = 0.62$

For triangular weir, $\theta = 90^\circ$

$C_d = 0.59$

Let depth over triangular weir $= H_1$

Discharge over rectangular weir,

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

$$= \frac{2}{3} \times 0.62 \times 1.0 \times \sqrt{2 \times 9.81} \times (.15)^{3/2}$$

$$= 0.10635 m^3/s$$

The same discharge passes through the triangular right-angled weir.

For triangular weir,

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

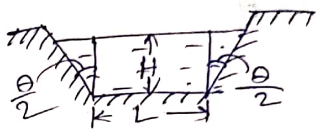
$$\Rightarrow 0.10635 = \frac{8}{15} \times .59 \times \tan\frac{90}{2} \times \sqrt{2 \times 9.81} \times H_1^{5/2}$$

$$\Rightarrow H_1 = 0.3572 m$$

→ Advantages of Δr notch over rectangular Notch:

1) Expression for Q for Δr notch is simple
2) For measuring low discharge, a Δr notch gives more accurate result.
3) In Δr notch, only H is required.
4) Ventilation of Δr notch not necessary

→ Trapezoidal Notch or weir :-



Let

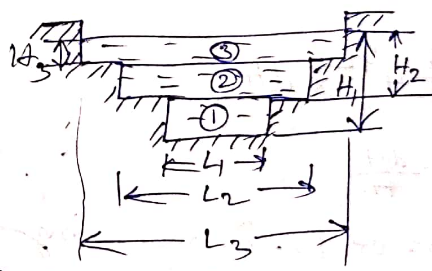
H = ht of water over notch

L = Length of crest of notch

It consists of one rectangular notch and one triangular notch with angle θ.

So
$$Q = \frac{2}{3} C_{d_1} L \sqrt{2g} \times H^{3/2} + \frac{8}{15} C_{d_2} \times \tan\frac{\theta}{2} \sqrt{2g} \times H^{5/2}$$

→ Stepped Notch



$$Q = Q_1 + Q_2 + Q_3 = \frac{2}{3} C_d \times L_1 \times \sqrt{2g}\left(H_1^{3/2} - H_2^{3/2}\right) + \frac{2}{3} C_d \times L_2 \times \sqrt{2g}\left(H_2^{3/2} - H_3^{3/2}\right) + \frac{2}{3} C_d \times L_3 \times \sqrt{2g} \times H_3^{3/2}$$

→ Effect on discharge over a notch due to error in measurement of head :-

→ For Rectangular weir or notch

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

$$= K H^{3/2}$$

Differentiating we have,

$$dQ = K \times \frac{3}{2} H^{1/2} dH$$

$$\frac{dQ}{Q} = \frac{3}{2} \frac{dH}{H}$$

So an error of 1% in measuring H will produce 1.5% error in discharge over rectangular notch.

→ For Δr notch :-

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

$$= K H^{5/2}$$

$$dQ = K \frac{5}{2} H^{3/2} \times dH$$

$$\frac{dQ}{Q} = \frac{5}{2} \frac{dH}{H}$$

So an error of 1% in measuring H will produce 2.5% error in discharge over Δr notch.

→ Time required to empty a reservoir or tank with rect. notch or weir :-

If L = Length of crest of weir or notch.

H_1 = initial ht. of liquid above crest of notch.

H_2 = Final ht. of Liquid above crest of notch

T = time required in seconds to lower the height of liquid for H_1 to H_2

$$T = \frac{3A}{C_d \times L \times \sqrt{2g}} \left[\frac{1}{\sqrt{H_2}} - \frac{1}{\sqrt{H_1}}\right]$$

where A = c/s area of tank or reservoir.

→ Time required to empty a reservoir or tank with Δr notch :-

If H_1 = Initial ht. of liquid above apex of notch

H_2 = Final ht of liquid above apex of notch

A = area of c/s of reservoir or tank.

T = time required

$$T = \frac{5A}{4 \times C_d \times \tan\frac{\theta}{2} \times \sqrt{2g}} \left[\frac{1}{H_2^{3/2}} - \frac{1}{H_1^{3/2}}\right]$$

→ **velocity of approach** :-

It is defined as the velocity with which the water approaches or reaches the weir or notch before it flows over it.

If V_a is velocity of approach, then an additional head $h_a = \frac{V_a^2}{2g}$ due to velocity of approach is acting on water flowing over notch. Initial ~~head~~ ht. of water over notch = $(H + h_a)$ and final is h_a

$$V_a = \frac{Q}{\text{Area of channel}}$$

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

→ **Empirical formula for discharge over rectangular weir** :-

If length of crest = width of channel then this type of weir is called suppressed weir.

If ~~one~~ weir is not suppressed then end contraction is taken into account.

Francis formula

Effective length of weir,

$$L = (L - 0.2H)$$

$$Q = \frac{2}{3} \times C_d \times [L - 0.2H] \times \sqrt{2g}\, H^{3/2}$$

If $C_d = 0.623$, $g = 9.81\ m/s^2$, then

$$Q = 1.84[L - 0.2 \times H]\, H^{3/2}$$

If end contractions are suppressed then $H = 1.84\, L H^{3/2}$

If velocity of approach is considered, then $Q = 1.84\, L\left[(H+h_a)^{3/2} - h_a^{3/2}\right]$

Bazin's formula

$$Q = m \times L \times \sqrt{2g} \times H^{3/2}$$

where $m = \frac{2}{3} \times C_d = 0.405 + \frac{0.003}{H}$

If velocity of approach is taken,

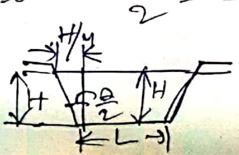
$$Q = m_1 \times L \times \sqrt{2g}\left[(H+h_a)^{3/2}\right]$$

where $m_1 = 0.405 + \frac{0.003}{(H+h_a)}$

→ **Cipolleti weir or notch** :-

It is a trapezoidal weir which has side a slope of 1 horizontal to 4 vertical

So $\tan\frac{\theta}{2} = \frac{H/4}{H} = \frac{1}{4}$.

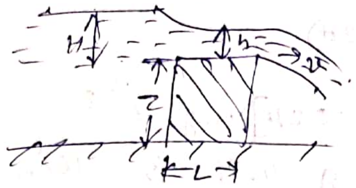


$$Q = \frac{2}{3} \times C_d \times L \times \sqrt{2g}\, H^{3/2}$$

considering V_a,

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

→ **Discharge over a Broad-crested weir** :-



L = Length of crest

H = ht of water above crest

If $2L > H$ → Broad-crested weir

If $2L < H$ → Narrow-crested weir.

h = head of water at middle of weir which is constant

$$Q = C_d \times L \times \sqrt{2g(Hh^2 - h^3)}$$

$$Q_{max} = 1.705 \times C_d \times L \times H^{3/2}$$

→ **Q over narrow crested weir** :

If $2L < H$

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

→ **Discharge over an Ogee weir** :

Here max^m height upto which crested rises is 0.115H.

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

(2.4) Losses of head of a liquid flowing through pipes :-

When pipe is running full then there is turbulent flow and if pipes are partially full then pr. inside pipe is same is equal to atmospheric pr. and is considered as open channel flow.

→ **Loss of Energy in pipes** :-

- Major Energy loss
  - ↓ Due to friction
  - Calculated by
    - (i) Darcy-Weisbach formula
    - (ii) Chezy's Formula
- Minor Energy Losses
  - → Sudden expansion of pipe
  - → Sudden contraction
  - → Bend in pipe
  - → pipe fitting
  - → obstruction in pipe

→ **Loss of Energy (or Head) due to friction** :-

(a) **Darcy-Weisbach formula** :-

h_f = loss of head due to friction

$$h_f = \frac{4fLV^2}{2gd}$$

f = coefficient of friction which is a function of Re

$= \frac{16}{Re}$ for $Re < 2000$ (viscous flow)

$= 0.079/Re^{1/4}$ $4000 < Re < 10^6$

L = length of pipe.
V = mean velocity of flow
d = dia. of pipe.

(b) Chezy's formula :-

$V = C\sqrt{mi}$

$m = \frac{A}{P} = \frac{\text{Area of flow}}{\text{wetted perimeter}}$

= hydraulic mean depth or hydraulic radius.

$m = \frac{A}{P} = \frac{\frac{\pi}{4}d^2}{\pi d} = \frac{d}{4}$ (for pipe running full)

C = chezy's constant

i = loss of head per unit length of pipe $= \frac{h_f}{L}$

Problem-1 Find the head lost due to friction in a pipe of diameter 300mm and length 50m, through which water is flowing at a velocity of 3m/s using (i) Darcy's formula, (ii) Chezy's formula for which C = 60, Take ν for water = 0.01 stoke.

Ans :- Given, Dia of pipe, d = 0.20m

L = 50m

V = 3m/s

C = 60.

$\nu = 0.01$ stoke $= 0.01\ cm^2/s$

$= 0.01 \times 10^{-4}\ m^2/s$

(i) Darcy formula,

$h_f = \frac{4fLV^2}{2gd}$

$R_e = \frac{Vd}{\nu} = \frac{3 \times 0.3}{0.01 \times 10^{-4}} = 9 \times 10^5$

$f = \frac{0.079}{R_e^{1/4}} = \frac{0.079}{(9 \times 10^5)^{1/4}} = 0.00256$

$h_f = \frac{4 \times 0.00256 \times 50 \times 3^2}{2 \times 9.81 \times 0.3} = 0.7828m$

(ii) Chezy's formula :-

$V = C\sqrt{mi}$

$C = 60$, $m = \frac{d}{4} = \frac{0.30}{4} = 0.075m$

$\therefore 3 = 60\sqrt{0.075 \times i}$

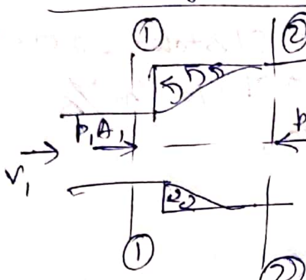
$\Rightarrow i = 0.0333$

but $i = \frac{h_f}{L} = \frac{h_f}{50} \Rightarrow h_f = 50 \times 0.0333$

$= 1.665m$

→ Minor Energy (Head) loss :-

(a)- loss of head due to sudden Enlargement :-

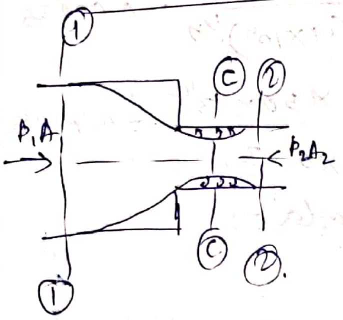


P_1, V_1, A_1 are at sec^n 1-1

& P_2, V_2, A_2 at sec^n 2-2.

$$h_e = \frac{(V_1 - V_2)^2}{2g}$$

(b)- Loss of head due to sudden Contraction



A_c = area of flow at C–C (vena contracta)

V_c = velocity of flow at C–C

h_c = loss of head due to sudden contraction

$$h_c = \frac{k V_2^2}{2g} \quad \text{where } k = \left(\frac{1}{C_c} - 1\right)^2$$

If $C_c = 0.62$ then $k = \left(\frac{1}{0.62} - 1\right)^2 = 0.375$

then $h_c = 0.375 \frac{V_2^2}{2g}$

If C_c not given then head loss is $h_c = 0.5 \frac{V_2^2}{2g}$

[problem-2] A 150 mm dia pipe reduces in diameter abruptly to 100 mm dia. If the pipe carries water at 30 l/s, Calculate the pr. loss across the contraction. Take $C_c = 0.6$

Ans :- Given, Dia of pipe, $D_1 = 0.15$ m

area of pipe, $A_1 = \frac{\pi}{4}(.15)^2$

$= 0.01767$ m^2

Dia of smaller pipe, $D_2 = 0.10$ m

$A_2 = 0.007854$ m^2

$Q = 30$ l/s $= 0.03$ m^3/s.

$C_c = 0.6$

$A_1V_1 = A_2V_2 = Q$

$\Rightarrow V_1 = \frac{Q}{A_1} = \frac{0.03}{0.01767} = 1.697$ m/s

$V_2 = \frac{Q}{A_2} = \frac{0.03}{0.007854} = 3.82$ m/s.

Applying Bernoulli's eq^n before & after contraction,

$$\frac{p_1}{\rho g} + \frac{V_1^2}{2g} + z_1 = \frac{p_2}{\rho g} + \frac{V_2^2}{2g} + z_2 + h_c \quad \text{—(1)}$$

but $z_1 = z_2$

$$h_c = \frac{V_2^2}{2g}\left[\frac{1}{C_c} - 1\right]^2 = 0.33$$

substituting in eqⁿ-1, we have

$$\frac{p_1}{\rho g} + \frac{(1.697)^2}{2\times 9.81} = \frac{p_2}{\rho g} + \frac{(3.82)^2}{2\times 9.81} + 0.33$$

or $\frac{p_1}{\rho g} + 0.1467 = \frac{p_2}{\rho g} + 0.7438 + 0.33$

$\therefore \frac{p_1}{\rho g} - \frac{p_2}{\rho g} = 0.7438 + 0.33 - 0.1467$

$= 0.9271$ m of water

$\therefore p_1 - p_2 = 0.909$ N/cm²

(c) Loss of head at entrance of pipe:-

This loss occurs when liquid ~~enet~~ enters a pipe connected to a large tank or reservoir.

This is similar to loss of head due to sudden contraction.

$$h_i = 0.5\frac{v^2}{2g}$$

(d) Loss of head at Exit of pipe

This loss of head due to the velocity of liquid at outlet of pipe which is dissipated either in the form of a free jet or lost in tank or reservoir.

$$h_o = \frac{v^2}{2g}$$

(e) Loss of head due to obstruction in a pipe :-

Let A = c/s of pipe (area)

a = maxⁿ area of obstruction.

Head loss due to obstruction

$$= \frac{v^2}{2g}\left(\frac{A}{C_c(A-a)} - 1\right)^2$$

(f) Loss of head due to bend in pipe:-

$$h_b = \frac{kV^2}{2g}$$

k = coefficient of bend which depends on, angle of bend, radius of curvature of bend, Diameter of pipe.

(g) Loss of head in various pipe fittings

$= \frac{kV^2}{2g}$, k = coefficient of pipe fitting.

→ Hydraulic gradient line (H.G.L.):-

It is defined as the line which gives the sum of pr. head $\left(\frac{p}{\rho g}\right)$ and datum head (z) of a flowing fluid in a pipe w.r.t some reference line or it is the line obtained by joining the top of all vertical ordinates, showing the pr. head of a flowing fluid in a pipe from centre of the pipe.

→ Total Energy line (T.E.L)

It is defined as the line which gives the sum of pr. head, datum head and kinetic head of a flowing fluid in a pipe w.r.t some reference line.

2.5 Flow through the open channels

Flow in open channels is defined as the flow of a liquid with a free surface with atmospheric pressure.

Non uniform flow in open channel is also called varied flow.

→ Rapidly varied flow is where depth of flow changes abruptly over small length of channel and Gradually varied flow is changes gradually over a long length of channel.

→ If $R_e < 500$ or 600 → Laminar flow

$R_e > 2000$ → Turbulent

$500 < R_e < 200$ → transition state.

→ If Froude No. $F_e = \frac{V}{\sqrt{gD}} < 1$ then it is subcritical flow

If $F_e = 1$ → Critical flow

$F_e > 1$ → Super critical or shooting or rapid or torrential flow

→ Discharge through open channel by Chezy's formula :-

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

problem-1 Find the slope of the bed of a rectangular channel of width 5m when depth of ~~flow~~ water is 2m and rate of flow is given as $20\,m^3/s$. Take $C = 50$.

Ans :- Given, width, $b = 5m$

depth, $d = 2m$

$Q = 20m^3/s$

$C = 50$

Bed slope $= i$

$A = b \times d = 10m^2$

$m = \frac{A}{P} = \frac{10}{b+2d} = \frac{10}{9}\,m$.

we know by chezy's formula,

$Q = AC\sqrt{mi}$

$\Rightarrow 20 = 10 \times 50 \times \sqrt{\frac{10}{9} \times i}$

$\Rightarrow i = \frac{1}{694.44}$ (Ans)

→ Empirical formulae for the value of Chezy's constant :-

1) Bazin's formula, $C = \frac{157.6}{1.81 + \frac{K}{\sqrt{m}}}$

K = Bazin's const: depends on roughness of surface of channel

2) Mannings Formula,

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

m = Hydraulic mean depth

N = Manning's constant

Problem-2 Find the discharge through a rectangular channel of width 2m, having a bed slope of 4 in 8000. The depth of flow is 1.5m and take the value of N in Manning's formula as 0.012.

Ans:- Given, width, b = 2m

d = 1.5m

$A = bd = 2 \times 1.5 = 3 m^2$

wetted perimeter, $P = b + 2d = 5m$

$m = \frac{A}{P} = \frac{3}{5} = 0.6$

Bed slope, $i = 4 \text{ in } 8000 = \frac{1}{2000}$

N = 0.012

Using Manning's formula,

$C = \frac{1}{N} m^{1/6} = \frac{1}{0.012} \times 0.6^{1/6} = 76.54$

$Q = AC\sqrt{mi}$

$= 3 \times 76.54 \sqrt{0.6 \times \frac{1}{2000}} = 3.977 m^3/s$

→ Most Economical Section of channels

A section of a channel is said to be most economical when the cost of construction of the channel is min^m i.e., the wetted perimeter for a given discharge should be minimum.

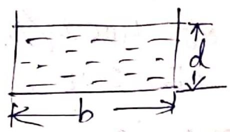
$Q = AC\sqrt{mi} = AC\sqrt{\frac{A}{P} i}$

For a given A, i & C, $Q = K\frac{1}{\sqrt{P}}$

where $K = AC\sqrt{Ai}$ = const.

Here Q will be maximum, when P is minimum.

→ Most Economical Rectangular section



b = width of channel

d = depth of flow

A = bd

P = ~~d~~ b + 2d.

$b = \frac{A}{d}$, so $P = b + 2d = \frac{A}{d} + 2d$.

For most economical section, P should be minimum for a given area, $\frac{dP}{d(d)} = 0$

$$\frac{d}{d(d)}\left(\frac{A}{d} + 2d\right) = 0 \Rightarrow A = 2d^2$$

but $A = bd \Rightarrow bd = 2d^2 \Rightarrow b = 2d$.

Now $m = \frac{A}{P} = \frac{bd}{b+2d}$

$$= \frac{2d \times d}{2d + 2d}$$

$$= \frac{d}{2}$$

So rectangular channel will be most economical when,

(i) either $b = 2d$ or width is two times depth of flow

(ii) or $m = \frac{d}{2}$ or hydraulic depth is half the depth of flow.

Problem-3

A rectangular channel carries water at the rate of 400 l/s when bed slope is 1 in 2000. Find the most economical dimensions of the channel if $C = 50$.

Ans: $Q = 400\ l/s = 0.4\ m^3/s$.

Bed slope $i = \frac{1}{2000}$

$C = 50$.

For most economical rectangular channel, $b = 2d$ & $m = \frac{d}{2}$

$A = bd = 2d \times d = 2d^2$

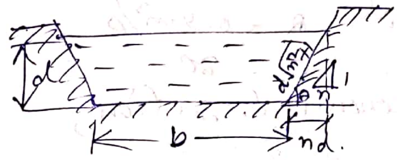
$Q = AC\sqrt{mi}$

$\Rightarrow 0.4 = 2d^2 \times 50\sqrt{\frac{d}{2} \times \frac{1}{2000}}$

$\Rightarrow d = 0.577\ m.$

& $b = 2d = 2 \times 0.577 = 1.154\ m.$

→ Most Economical Trapezoidal Channel:



Side slope is 1 vertical to n horizontal.

For most economical section,

(i) $\frac{b + 2nd}{2} = d\sqrt{n^2+1}$

i.e., half the top width must be equal to one of the sloping sides of the channel.

(ii) $m = \frac{A}{P} = \frac{d}{2}$

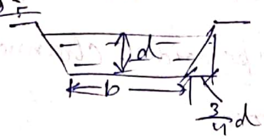
hydraulic mean depth is equal to half of d.

(ii) A semicircle drawn with radius equal to depth of flow will touch all the three sides of the channel.

Problem-4

A trapezoidal channel has side slopes of 3 horizontal to 4 vertical and slope of its bed is 1 in 2000. Determine the optimum dimensions of the channel, if it is to carry water at 0.5 m³/s. Take C = 80.

Ans:-



$n = \frac{3}{4}$

$i = \frac{1}{2000}$

$Q = 0.5 m^3/s$

$C = 80$

For most economical section,

$$\frac{b + 2nd}{2} = d\sqrt{n^2+1}$$

$$\Rightarrow \frac{b + 2\times\frac{3}{4}\times d}{2} = d\sqrt{\left(\frac{3}{4}\right)^2+1}$$

$$\Rightarrow b = 2\times 1.25d - 1.5d = d$$

& $Q = AC\sqrt{mi}$

Take $m = \frac{d}{2}$

So $0.50 = A \times 80 \times \sqrt{\frac{d}{2} \times \frac{1}{2000}}$ — (1)

Area of trapezoidal secⁿ is

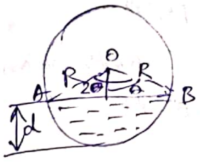
$$A = (b+nd)d = \left(d + \frac{3}{4}d\right)\times d$$

$$= 1.75d^2$$

So (from (1)) $0.50 = 1.75d^2 \times 80 \times \sqrt{\frac{d}{2}\times\frac{1}{2000}}$

$\Rightarrow b = d = 0.55m$.

→ **Flow through circular channel:-**



d = depth of water.

2θ = angle subtended by water surface AB at centre in radians

R = radius of the channel.

$$P = 2R\theta, \quad A = R^2\left(\theta - \frac{\sin 2\theta}{2}\right)$$

$$m = \frac{R}{2\theta}\left(\theta - \frac{\sin 2\theta}{2}\right)$$

& $Q = AC\sqrt{mi}$

→ **Most economical circular secⁿ:-**

→ condition for maxᵐ velocity:-

$\theta = 128°45'$

$d = 0.81D$ (D = dia. of channel)

& $m = 0.3D$.

→ condition for maxᵐ discharge:-

$\theta = 154°$, $d = 0.95D$.

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Problem-5 Determine the max^m discharge of water through a circular channel of diameter 1.5m when the bed slope of the channel is 1 in 1000. (C = 60)

Ans: Given, $D = 1.5m$, $R = 0.75m$,

$i = \frac{1}{1000}$, $C = 60$

For max^m discharge, $\theta = 154°$
$= 2.6878$ radian

$P = 2R\theta = 2 \times \frac{1.5}{2} \times 2.6878$
$= 4.0317m$.

$A = R^2\left(\theta - \frac{\sin 2\theta}{2}\right)$

$= 0.75^2\left[2.6878 - \frac{\sin(2\times154°)}{2}\right]$

$= 1.7335$

$m = \frac{A}{P} = \frac{1.7335}{4.0317} = 0.4299$

$Q = AC\sqrt{mi} = 2.1565\ m^3/s$

Pumps

(3.1)- Type of Pumps :-

The hydraulic machine which converts mechanical Energy into hydraulic energy are called pumps. The hydraulic energy is in the form of pr. energy.

Pumps are of two types:-

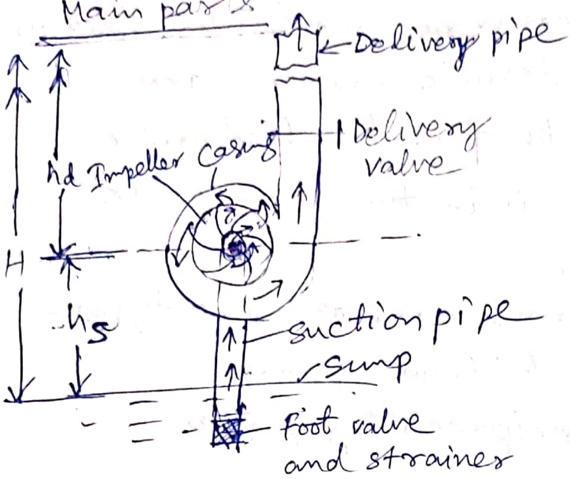
1) Centrifugal pump
2) Reciprocating pump

(3.2) Centrifugal pump :-

The centrifugal machine which converts mechanical energy into pr. energy by means of centrifugal force acting on the fluid is known as 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 pr. head of the rotating liquid takes ~~pl~~ place. The rise in pr. head at any point is proportional to the square of tangential velocity of the liquid at that point i.e., $v^2/2g$ or $w^2R^2/2g$

Main parts



main parts are,

1) Impeller
2) Casing
3) Suction pipe with foot valve and strainer
4) Delivery pipe

1) Impeller :- It is a rotating part consists of series of backward curved vanes. Impeller is mounted on a shaft which is connected to the shaft of an electric motor.

2) Casing :- Casing is an air tight passage surrounding the impeller. It is designed in such a way that the K.E of the water discharged at outlet of impeller is converted into pr. Energy before the water leaves the casing and enters the delivery pipe.

There are 3 types of casing.

1) volute casing
2) vortex casing
3) Casing with guide blades

1) volute casing :- It is a spiral type in which area of flow increases gradually & velocity decreases & increases the pressure of water flowing through the casing.

2) vortex casing : If a circular chamber is introduced b/w casing and impeller, the casing is called vortex casing. By introducing circular chamber, the loss of energy due to the formation of eddies is reduced to a considerable extent.

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and efficiency increases.

3) Casing with guide blades :-

The casing 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 such a way that the water from the impeller enters the guide vanes without shock.

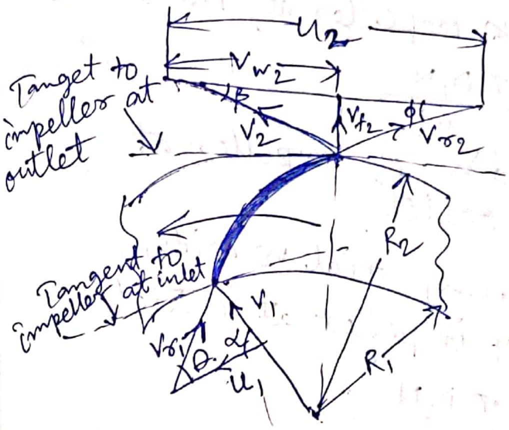
③ Suction pipe with foot valve and strainer :-

A pipe whose one end is connected to the inlet of the impeller and other end dips into the water in a sump is known as suction pipe. A foot valve is a non return valve or one way type valve fitted to lower end of suction pipe & open only in upward direction. A strainer filters water,

4) Delivery pipe

The pipe whose one end is connected to the outlet of the pump and other end delivers water at required height is known as delivery pipe.

Work done by pump on water



In case of centrifugal pump work is done by the impeller on the water. The water enters the impeller radially at inlet for best efficiency of the pump which means the absolute velocity of water at inlet makes an angle of 90° with the direction of motion at inlet.

Hence $\alpha = 90°$, $V_{w_1} = 0$

Let N = speed of impeller in rpm

D_1 = Dia. of impeller at inlet.

U_1 = tangential velocity of impeller at inlet

$= \frac{\pi D_1 N}{60}$

D_2 = dia. of impeller at outlet

U_2 = tangential velocity of impeller at outlet

$= \frac{\pi D_2 N}{60}$

work done by the impeller on the water per second per unit wt. of the water striking per second.

= − (workdone in case of turbine)

$= -\frac{1}{g}[V_{w_1}U_1 - V_{w_2}U_2]$

$= \frac{1}{g} V_{w_2}U_2$

work done by the impeller on water per second

$= \frac{W}{g}(V_{w_2}U_2)$

$= \rho Q (V_{w_2}U_2)$

Q = volume of water flowing per second.

$= \pi D_1 B_1 V_{f_1} = \pi D_2 B_2 V_{f_2}$

B_1, B_2 are width of impeller at inlet and outlet

V_{f_1}, V_{f_2} = velocities of flow at inlet and outlet

Definitions of heads & Efficiencies.

1) Suction head (h_s)

It is the vertical height of the centre line of the centrifugal pump above the water surface in the tank or sump from which water is to be lifted.

2) Delivery head (h_d)

vertical distance between the centre line of the pump and the water surface in the tank to which water is delivered.

3) Static head (H_s) :-

The sum of suction head and delivery head

$H_s = h_s + h_d$

4) Manometric head (H_m)

It is the head against which a centrifugal pump has to work.

H_m = head imparted by the impeller to the water – loss of head in pump

$= \frac{1}{g} V_{w_2} U_2$ – loss of head in impeller casing

$= \frac{1}{g} V_{w_2} U_2$ if loss of pump is zero.

→ Efficiencies :-

In centrifugal pump, the power is transmitted from shaft of electric motor to shaft of pump & then, to impeller then to water. So the power is decreasing from shaft to impeller then to water.

(a) Manometric Efficiency

The ratio of the manometric head to the head imparted by the impeller to the water is known as manometric efficiency,

$\eta_{man} = \frac{\text{Manometric head}}{\text{head imparted by impeller to water}}$

(b) Mechanical Efficiency :-

$\eta_m = \frac{\text{power at the impeller}}{\text{power at the shaft}}$

(c) overall Efficiency (η_o) :-

$\eta_o = \frac{\text{weight of water lifted} \times H_m}{1000} = \frac{W H_m}{1000}$

Reciprocating Pump :-

Difference b/w Reciprocating & centrifugal pump.

Disadvantages

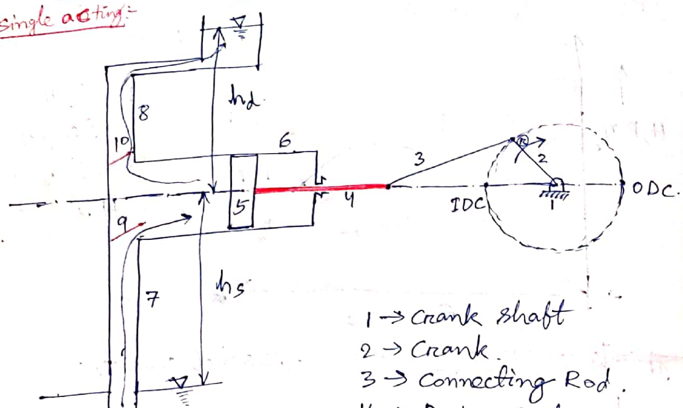
1. Reciprocating pump is having more inertia.
2. More space & requirements.
3. More wear and tear.
4. More vibrations.

Advantages over centrifugal pumps

1. More durable.
2. Can withstand high pressure.

→ Reciprocating Pump :-

→ Single acting :-



1 → Crank shaft
2 → Crank
3 → Connecting Rod.
4 → Piston Rod.
5 → Slider / Piston.
6 → Cylinder
7 → Suction Pipe.
8 → Delivery Pipe.
9 → Suction Valve.
10 → Delivery valve

IDC → Inner Dead centre.
(TDC → Top " (if vertical))
ODC → outer "
(BDC → Bottom " (If vertical))

Stroke length = L

$L = 2r$ (r = Crank radius)

(N revolution → 60 sec. ; 1 rev. → $\frac{60}{N}$ sec.)

Cross-sec. Area of piston/cylinder $A = \frac{\pi}{4} D^2$.

Volume sucked by the cylinder = (A.L)

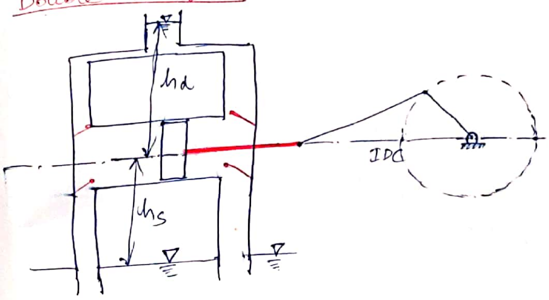
Volume Discharge = (A.L) (m^3) in 1 rev. = $\frac{60}{N}$ sec.

Q_{th} → $$Q = \frac{AL}{\frac{60}{N}} = \left(\frac{ALN}{60}\right) m^3/sec.$$

$$\dot{W} = P = \dot{m}gH = \rho Q g (h_s + h_d)$$

$$\dot{W} = \rho g \frac{ALN}{60} (h_s + h_d)$$

Double acting :-



Here. $A_1 = A = \frac{\pi}{4} D^2$.

$A_2 = A - a = \frac{\pi}{4}(D^2 - d^2)$ (D → Piston Dia. ; d → Rod dia)

$$P = \dot{W} = \rho g \frac{[A_1 + A_2] L.N.}{60} (h_s + h_d)$$

(→ Q_{th})

If $d <<<<< D$, $A_1 = A_2 \Rightarrow A = \frac{\pi}{4} D^2$

$$P = \rho g \left(\frac{2ALN}{60}\right)(h_s + h_d)$$

(→ Q_{th})

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Normally :- $Q_{act} < Q_{th}$

⇓

Slip.

⇓

$$\left(\frac{Q_{th} - Q_{ACT}}{Q_{th}}\right) \times 100 \Rightarrow \text{\% of slip}$$

↓ ↓

(positive slip)

Some times :-

when suction pipe is long.
But delivery pipe is very short.
pump is running with very high speed.

$Q_{act} > Q_{th}$

↓

Negative slip.

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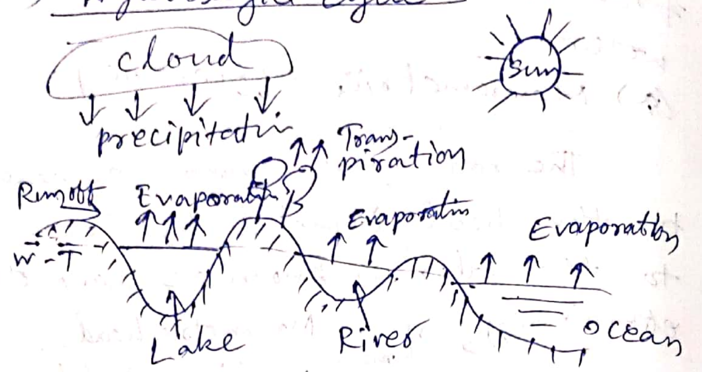
Part-B Hydrology / Irrigation Engg.

(1.1) Hydrologic cycle -



water of the universe always changes from one state to other under the effect of the sun. The water from surface sources like lake, rivers, ocean, etc. converts to vapour by evaporation due to solar heat. The vapour goes on accumulation & condensed due to sudden fall of temperature and pressure & form cloud & these cloud cause precipitation or rainfall. Some of the vapour is converted to ice at the peak of mountains & it melts in summer & flows to river to meet sea or ocean. This process of evaporation, precipitation and melting of ice go on continuously & thus a balance is maintained in atmosphere. This is called hydrologic cycle.

(1.2) Rainfall :-

From principle of hydrologic cycle, water goes on evaporation by sun & water vapours collect on atmosphere upto certain limit. when this limit exceeds & temperature and pr. falls to a certain value, the water ~~vapour~~ ~~con~~ vapour condenses and cloud formed & fall to earth in the form of water droplets known as rainfall or precipitation.

→ Types of rainfall

1) Cyclonic precipitation :-

This is caused by difference of pr. within the air mass on the surface of the earth. If low pressure is generated at some place the warm moist air rushes to the zone of low pr. with violent force & the warm moist air rises up & get condensed & heavy rainfall occur.

2> Convective Precipitation :-

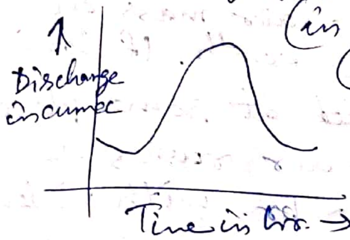
In tropical countries when on a particular hot day the ground surface gets heated unequally the warm air lifted to high altitude & cooler air takes its place & warm moist air mass condense at high altitude & cause heavy rainfall.

3> Orographic precipitation :-

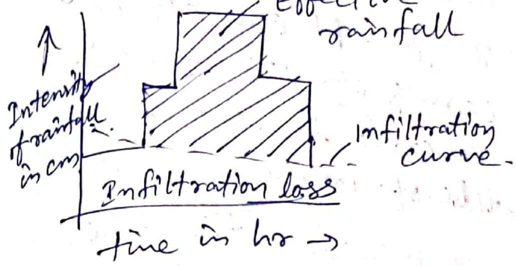
when moving warm moist air obstructed by some mountain rises up to a high altitude, then gets condensed and precipitation occur.

→ Hydrograph :-

It is a graphical representation of the discharge of a river (in cumec) against time (in hr or days).



→ Hyetograph :-

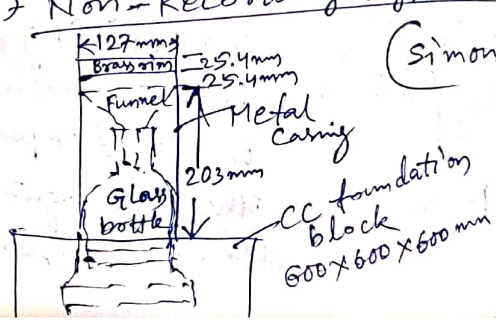


The graphical representation of rainfall and runoff is called hyetograph. Here intensity of rainfall (in cm/hr) is represented as ordinate and time (in hrs) is represented as abscissa.

→ Measurement of rainfall :-

The instrument which is used to measure the amount of rainfall is called raingauge. Raingauges are of following types :-

1> Non-Recording type raingauge (Simon's Rain gauge)



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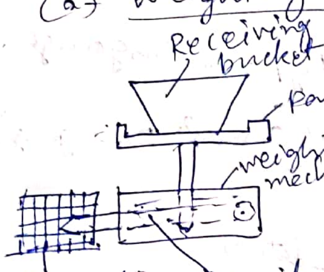
Simon's rain gauge is most commonly used as a non-recording type rain gauge. It consists of a metal casing of 127mm dia set on a concrete foundation. A glass bottle of 100mm rainfall capacity is placed within the casing. A funnel with brass rim is placed on the top of the bottle. The rainfall is recorded at every 24 hrs.

2) Recording type rain gauge :-

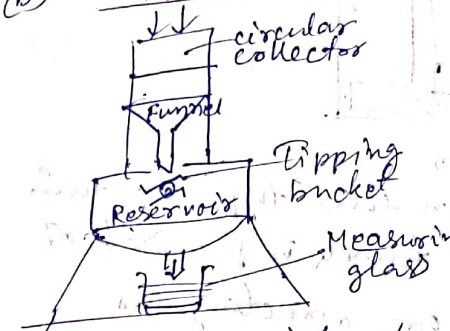
Here the amount of rainfall is automatically recorded on a graph paper by some mechanical devices. It is of 3 types.

(a) weighing bucket Raingauge:-



It consists of a receiving bucket which is placed on pan. The pan again fitted with some weighing mechanism. A pencil arm is pivoted with the weighing mechanism in such a way that movement of bucket can be traced by a pencil on the moving recording drum. The raingauge produces a graph of cumulative rainfall versus time.

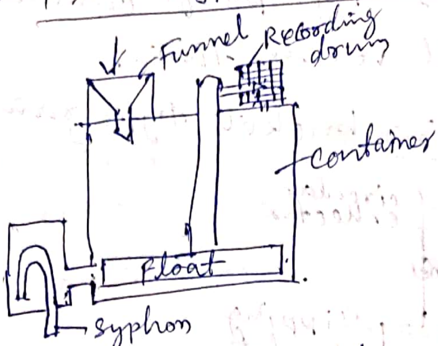
(b) Tipping bucket type raingauge.



It consists of a circular collector of diameter 30cm in which the rain water is initially collected. The rain water then passes through funnel fitted to a circular collector and gets collected in two compartment tipping buckets pivoted below funnel.

The circular motion of tipping bucket is transmitted to pen or pencil which traces a wave like curve on the sheet mounted on a revolving drum.

(c) Float type rain gauge :-



In this type of rain gauge a funnel is provided at one end of a rectangular container and a rotating recording drum is provided at other end. The float rises up as the rain water gets collected in the container & a pen connected to float, records the amount of rainfall on the graph paper.

→ Selection of site for raingauge stations :-

1) The site should be on level ground and on open space & not on sloping ground.

2) Site should be such that the distance b/w the gauge station & ~~oblikes~~ objects like tree, etc should be at least twice the height of objects.

3) In hilly area, the stn may be well shielded from high wind.

4) Site should be accessible to observer & well protected from cattles by wire fencing.

As per WMO (World Meteorological organisation) for the network of raingauge stn,

a) For plain regions - one station for every 600-900 sq.km.

b) For mountainous region - one station for every 100-250 sq. km.

c) For arid region - one station for every 1500-10,000 sq.km.

→ Average depth of precipitation :-

3 methods →

1) Arithmetic Mean Method :-

Here the rainfall values obtained from all the raingauge stations are added and divided by the number of stations to get the average value. If N number of stations and R_1, R_2, R_3 --- etc. are the rainfall values obtained from stations. Then average depth of precipitation

$$= \frac{R_1 + R_2 + R_3 + ---}{N}$$

This method is suitable where raingauge stations are uniformly distributed over the basin.

2) Thiessen polygon Method

This is suitable for large areas. here all raingauge stations has its own domain within the basin area. Domain of each raingauge stations is given by each polygon, Area of polygon is measured by graph paper. then avg. depth is calculated as $\frac{\text{rainfall depth} \times \text{Area}}{\Sigma \text{area of polygons}}$

3) Iso-hyetal Method :-

Isohyet line is line joining points of equal depths of precipitation. The area enclosed b/w two successive Iso-hyet lines is found out by graph paper & avg. depth $= \frac{\text{Avg. depth} \times \text{area}}{\Sigma \text{ Area b/w two successive isohyet lines}}$

→ Water losses :-

Matter or energy can't be lost, it only changes from one state to other. Similarly water can't be lost but changes from one state to other.

$$\text{water loss} = \text{Precipitation} - \text{Surface runoff}$$

(a) Interception :-

Due to solar head, leaves, branches, trunks of trees, etc.

gets dried up & gets capacity of absorbing water. So when rainfall occur some portion of water directly absorbed by these agents which is known as interception. It continues till the leaves, branches, etc gets completely saturated. In forest area interception is more than that of in open area.

(b) Evaporation

process of change in state of water from liquid or solid to vapour due to transfer of heat energy is called evaporation. Various factors affecting evaporation from free water surface are area of water surface, depth of water, Humidity, Temperature, wind velocity, etc. Evaporation from soil surface also occur and evaporation through the leaves of living plant is called Transpiration.

(c) Infiltration :- when it rains in a particular area, some portion of water moves downwards through soil pores under gravity this is called ~~Afltrati~~ infiltration. Infiltration capacity depends on, texture of soil, condition of soil surface, content of soil moisture, type of vegetative surface cover, soil temperature, agriculture.

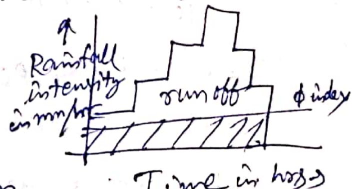
→ Infiltration indices :-

These indices are used to assess water lost by infiltration. There are 2 indices,

(a) φ-index :- It is defined as the average rate of rainfall during any storm, above which volume of rainfall is equal to the volume of direct runoff.

(b) w-index :-

It is defined as the average rate of infiltration which is calculated by the expression,



$$w\text{-index} = \frac{R-Q}{T_r}$$

R = Total rainfall.
Q = Total direct runoff
T_r = duration of rainfall in hrs.

N.B. :- For uniform rainfall the values of ϕ-index and W-index will be equal.

Problem-1 In a catchment area of 5 sq.km., the intensities of rainfall per hour for a five hour duration storm are 10, 15, 20, 22, 5 mm, the volume of direct run-off is measured as 0.50 cumec day. Determine the ϕ-index for the catchment area.

Ans :- Total rainfall in 5 hrs

$= 10+15+20+22+5 = 72 mm$

Average rate of rainfall

$= \frac{72}{5} = 14.4 mm/hr$

Total volume of rainfall over the catchment area

$= \frac{72}{1000} \times 5 \times 10^6 = 360,000 m^3$

Total volume of direct runoff

$= 0.5 \times 60 \times 60 \times 24 = 43200 m^3$

Volume of water lost

$= 360000 - 43200 = 316800 m^3$

So depth of water over the catchment $= \frac{316800}{5 \times 10^6} = 0.063360 m$

ϕ index = 63 mm

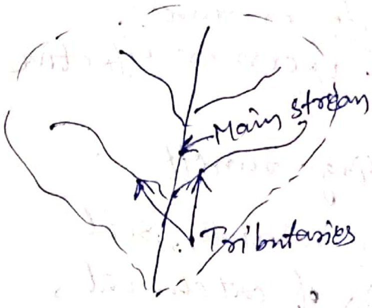
→ catchment area

The catchment area of river means the area from where the surface runoff flows to that river through the tributaries, streams, springs, etc.

Catchment is of 2 types.

1) Fan shaped

2) Fern shaped.



Fan shaped catchment

Fern shaped. catchment

Fan shaped catchments give greater runoff because tributaries are nearly of the same size, and time of flow is nearly same and is smaller, whereas in fern leaf catchments, the time of concentration is more since the discharge is distributed over a long period.

→ **Runoff** :- When it rains, some portion of rain water infiltrates into the soil, some is intercepted by vegetation, some evaporates and remaining portion flows over the ground surface to join the rivers, streams, lakes, etc. This portion of water which flows over the ground surface is known as surface runoff or runoff. or rainfall excess or effective rainfall.

→ **Factors affecting runoff** :-

Intensity of rainfall, soil characteristics of catchment, topography of the catchment, shape & size of catchment, Geological condition of catchment, cultivation and vegetative cover in catchment area, weather condition.

→ **Estimation of Run-off** :-

(a) Rational method :-

$$Q = \frac{KiA}{36}$$

- A = Catchment area in hectares
- Q = Runoff in cumec
- K = coefficient of runoff.
- i = rainfall intensity in cm/hr

(b) By Empirical formula :-

→ Inglis's Formula :-

For ghat areas, $R = 0.85P - 30.5$

R = Runoff in cm

P = Rainfall in cm

For non ghat areas

$$R = \frac{(P-17.8)}{254} \times P$$

→ Lacey's Formula

$$R = \frac{P}{1 + \frac{304.8F}{PS}}$$

F = Monsoon duration factor b/w 0.5 to 1.5

S = Catchment factor b/w 0.25 to 1.7

→ Khosla's Formula

$$R = P - \frac{T-32}{3.74}$$

T = mean temperature in °F in the catchment area.

→ **Estimation of Peak flow (Flood Discharge)**

Dicken's Formula :-

$$Q = CA^{3/4}$$

Q = discharge in cumec

A = catchment area in sq.km.

C = a constant depending upon the factor affecting the flood discharge avg. value of C is 11.5

→ Ryve's Formula,

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

C = a const. (Avg. value = 8.8)

Water requirement of Crops

(2.1) Definition of Irrigation

The process of artificial application of water to the soil for the growth of agricultural crops is called as irrigation. It includes the construction of weirs, dams, barrages and canal systems for regular supply of water to the culturable (i.e. cultivable) lands.

→ **Necessity of Irrigation**:-

(a) Insufficient rainfall → when seasonal rainfall is less than the minm requirement for the satisfactory growth of crops, the irrigation system is essential.

(b) Uneven distribution of rainfall:-
When rainfall is not evenly distributed during the crop period or throughout the culturable area, then irrigation is necessary.

(c) Improvement of perennial crops:-
Some perennial crops like sugarcane, cotton, etc. require water throughout the major part of the year. But rainfall may fulfill the water requirement in rainy season only. So for other part of year, irrigation is necessary.

(d) Development of Agriculture in Desert area:-
In desert area where the rainfall is very scanty, irrigation is required for the development of agriculture.

→ **Benefits of Irrigation**:-

(a) Yield of crops:- In the period of low rainfall or drought, the yield of crop may be increased by the irrigation system.

(b) Protection from Famine.

(c) Improvement of cash crops like vegetables, fruits, tobacco, etc.

(d) Prosperity of farmers:- Farmers can grow 2 or more crops in a year which increases their earning & living standard.

(e) Source of Revenue :-
when irrigation water is supplied to the cultivators in lieu of some taxes, it helps to earn revenue which may be spent on other development schemes.

(f) Navigation :-
Irrigation canals may be ~~utti~~ utilised for inland navigation which is further useful for communication and transportation of agricultural goods.

(g) Hydroelectric power generation.

(h) water supply.

(i) General communication along the inspection road along canal banks.

(j) Development of fishery.

→ Types of irrigation :-

1) Lift irrigation :-
when water is lifted from surface sources or underground sources by man or animal power, mechanical or electrical power and directly supplied to the agricultural land, then it is called lift irrigation.

when mechanical and electrical power are not available then lifting of water is done by following method from surface sources :-
Doon, Mote, Persian wheel, swinging Bucket, Dhenkli, Rati or pulley, Wind lass.

when mechanical or electrical power is available then underground water is ~~fi~~ lifted by pumps. Underground water may be available from the following sources :-
open well, shallow tube well, Deep tube well.

2) Flow Irrigation :-
when water flows under gravitational pull through the artificial canal towards the agricultural land, then it is termed as flow irrigation.

Types :-

(a) Inundation Irrigation System :-
In this system a canal is excavated from the bank of the inundation river

i.e., the river which overflows in rainy season but dried up in summer & winter.

In this case water flows to the agricultural land in rainy season only. There is no regulator at the head of the canal to control the flow of water. The bed level of the canal is ~~at~~ fixed at such level that water flow only when water level of river rises above canal bed & flow stops when ~~wea~~ water level of river falls below canal bed. As there is no regulator so possibility of over irrigation is there.

(b) Perennial system of Irrigation

In this system, a weir or barrage is constructed across the perennial river (i.e., the river which flows throughout the year in its full capacity) to raise the water level on the upstream side or a dam is constructed to form a storage reservoir. Then main canal is constructed on either side or both banks of the river. Regulator is constructed at the head of the canal to control ~~flo~~ the flow of water through the canal towards the agricultural land. It is of 2 types.

(i) Direct Irrigation system:-

In this system a weir is generally constructed across a perennial river to raise the water level on the upstream side upto a certain limit, so that the water can flow through the canal. Here the water level on the U/S side will remain at a constant height and the excess water flows over the weir.

(ii) Storage Irrigation system:-

In this system, a dam is constructed across a river valley to form a storage reservoir. The main canals may be taken from both sides of the dam. The flow of water through the canal is controlled by head regulator. The storage reservoir is also known as multipurpose reservoir.

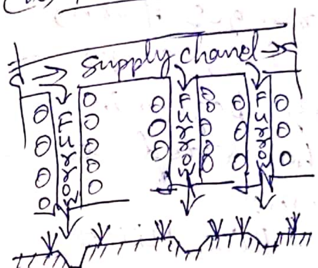
→ Methods of Distribution of water

- Surface method
  - Furrow method
  - Contour Farming method
  - Flooding method
    - Uncontrolled flooding
    - Controlled flooding
      - Free flooding
      - Basin flooding
      - Check flooding
      - Border strip
      - Zigzag
- Sub-surface method
- Sprinkler method

(A) Surface method :-

In this method, the irrigation water is distributed to the agricultural land through the small channels which flood the area upto the required depth. It is of 3 types.

(a) Furrow Method :-



In this method, the irrigation water is supplied to land by digging narrow channels known as furrows at regular intervals. The water flows through the furrows and infiltrates into the soil and spreads laterally to saturate the root zone. This method is suitable for the crops which are sown in rows like potato, ground nut, tobacco, sugarcane, etc.

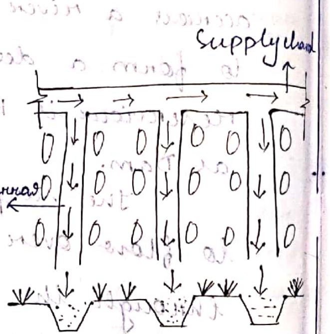
1. Surface Method :

In this method the irrigation water is distributed to the agricultural lands through small channel which flood the area upto the required depth. It is divided into three types.

- Furrow method.
- Contour farming.
- Flooding method.

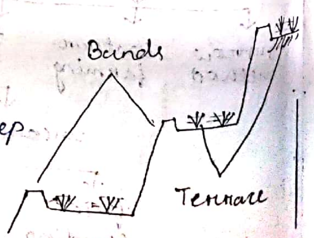
→ Furrow Method :→

In this method the irrigation water is supplied to the land by digging narrow channels known as furrow at regular interval. this method is suitable for the crop which are sown in row.



→ Contour farming :→

This method is adopted in hilly areas where the land has steep slope. Here the land is divided into series of horizontal strip



which is known as terrace. Same bunds are provided at the end of each terrace to hold water upto required depth. This method serves also the purpose of flood control and soil errosion.

→ Flooding Method :→

This method is suitable for the agricultural land which exists in a flat topography. In this method the field is flooded with water with the help of field channel. It is of two types:

1. uncontrolled flooding.
2. controlled flooding

Uncontrolled Flooding:-

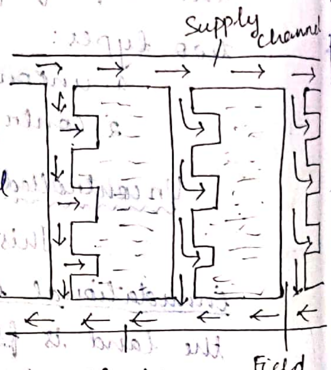
This method is applicable in innudational irrigation system. Here the land is flooded with water by innudational cannel. As there is no controll system, this type of distribution of water is known as uncontrolled flooding. This method result in wastage of water.

Controlled flooding :-

This method is applicable in perinnel irrigation system. In this method the agricultural area is flooded with water through the cannels which are provided with regulator. It is of five types.

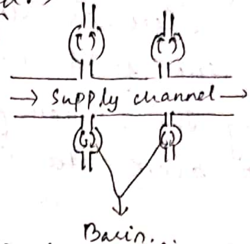
(i). free flooding.
(ii). Basin "
(iii). check "
(iv). Border strip flooding.
(v). Zig Zag method.

(i) Free Flooding :-



In this method the agricultural land is divided into small strips by a series of field channel which takes the water to the field and the extra surplus water is being passed through the waste channel.

(ii) Basin Method :-

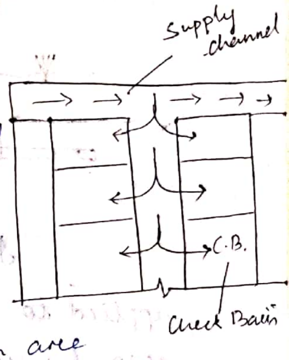


Basin flooding method :-

In this method each other tree or a group of tree are enclosed by circular channel through which the water flows.

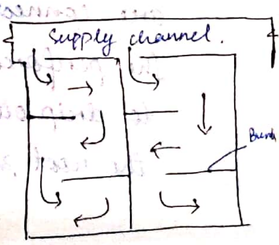
The circular channel is known as Basin. when all the basin are filled then the supplier of water is stopped & may be discharged through the drainage channel.

(iii) Check flooding :-



In this method the agricultural area is divided into small plots called check binds or check basin. The water is supplied to the check basin through the field channel which are connected to the supply channel.

(iv) Zig Zag Method :-



In this method the agricultural area is sub-divided into small plots by no. bunds in a zig zag

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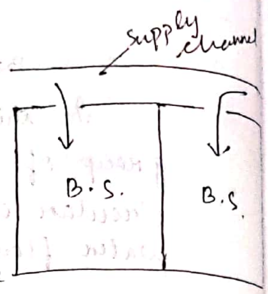
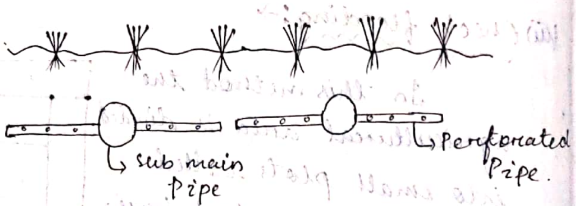
manner. The water is supplied to the plots from the field channel through the openning. The water flow in a zigzag manner to cover the entire surface.

(v) Boarder strip :–



In this method the agriculture area is divided into series of long narrow strip where the water is supplied to the required depth and then it is stopped.

2. Sub surface Method / (Drip irrigation)



In this method the water is applied to the root zone of the crop by underground network pipe which are connected with the perforated pipe. The perforated pipe allows the water to drip out slowly. Thus the soil below the root zone absorbs water continuously.

3. Sprinkle irrigation Method :

In this method the water is applied to the land in the form of spray like rain. The spraying of water is achieved by the network of main pipe, submain pipe and lateral pipes. It is of three types.

→ Perforation of lateral pipes.
→ Fixed nozzle on lateral pipe.
→ Rotating sprinkle.

* Quality of Irrigation water :–

The quality of a suitable irrigation water is verymuch influenced by the constituent of the soil which is to be irrigated. A particular water may be harmfull for irrigation on a particular soil. but the same water may be tolerable on some other soil.

The impurities which makes the water unfit for irrigation are:

→ Sediment conc. in water.
→ Total conc. of soluble salt in water.
→ Proportion of sodium ion.
→ conc. of potentially toxic element present in water.
→ Bacterial contamination.

(i) Sediment conc. in water :

The effect of sediment present in the irrigation water depends upon the type of irrigated land. When fine sediment from water is deposited on the sandy soil the fertility of the soil increases. On the other hand if the sediment has been derived from the eroded areas it may reduce the fertility or decrease the soil permeability.

(ii). Total conc. of soluble salt :→

The conc. of salt in water may not appear to be harmfull but the conc. of salt which remain in the soil after the saline water is used up by the plant is much more than the first and may proof harmfull for it.

$$C_s = \frac{C \cdot Q}{Q - [C_u - R_e]}$$

. when rain water is not considered $R_e = 0$.

C_s = Salinity conc. of soil.
Q = Total quantity of water applied
C_u = Consumptive used of water by plants
R_e = effective rainfall.
C = conc. of salt in irrigation water

C_s should be < 700 ppm then it can be used if not it is harmfull.

• The salt conc. is generally expressed in ppm if the amount exceed from 700 ppm then it is harmfull for the plant.

• The salt conc. is generally measured by determining the electrical conductivity of water which is expressed in micromhos/cm micro mho/cm.

• when its value exceeds from 250 umhos at 25°C its called low conductivity (C_1).

• If it is in between 250 – 750 then it is called medium conductivity of water (C_2)

• If 750 – 2250 then high conductivity (C_3)

• If more than 2250 then its very high conductivity of water (C_4).

(iii). Proportion of Sodium ion :-

The percentage of sodium ion is generally less than 5 percent. If this percent increase to about 10% or more than that the aggregation of soil grain break down. The soil become plastic & less permiable & it becomes plastic & sticky when it is wet. The proportion of sodium ion present in soil is generally measured by sodium absorption ratio. (SAR)

$$SAR = \frac{Na^+}{\sqrt{\frac{Ca^{++} + Mg^{++}}{2}}}$$

• when the value of SAR lies between 0 to 10 then it is called low sodium water

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If 10 - 18 medium sodium water.

- If 18-26 high sodium water.
- If more than 26 very high sodium water.

Q. what is the classification of irrigation water having the following characteristics conc. of Na, Ca, Mg are 22, 3 and 1.5 miliequivalent per lt respectively & the electrical conductivity is 200 micro mho/cm at 25°C what problem might arrive in using the water on fine textured soil.

- what remedies do you suggest to over come this struggle.

→ $Na^+ = 22$

$Ca^{++} = 3$

$Mg^{++} = 1.5$

$$SAR = \frac{Na^+}{\sqrt{\frac{Ca^{++} + Mg^{++}}{2}}}$$

$= 14.67$

i.e. is medium sodium water

The soil permiability decreases & the gets sticky.

If we add gypsum then it is prevented.

→ Crop season

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

(a) Kharif season :-

This season ranges from June to October. i.e., crops sown in the very beginning of monsoon and harvested at the end of autumn.

Kharif crops are - Rice, Millet, Maize, Jute, Groundnut, etc.

(b) Rabi season :

This season ranges from October to March. i.e., crops sown in the very beginning of winter and harvested at the end of spring.

Rabi crops are - wheat, Gram, Mustard, Rapeseed, Linseed, pulses, Onion, etc.

Some crops require more time, eg. :

(i) cotton - eight month's crop.

(ii) Sugar cane - Perennial crop.

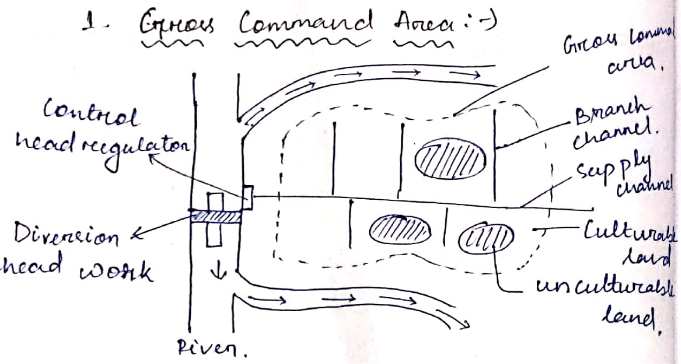
Water Requirement for Soil :-

Factors effecting the water requirement :-

(i) water table
(ii) climate condition
(iii). Ground slope
(iv). Intensity of irrigation.
(v). Type of soil.
(vi). Method of applying water.

Defination of important terms :-

1. Gross Command Area :-



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

2. Intensity of Irrigation :-

The total culturable command area may not be cultivated at the same time. So the intensity of irrigation may be defined as the ratio of cultivated land for a particular type of crop to the total culturable command area.

3. Crop season :-

The season period during which some particular type of crop can be grown every year on the same land is known as crop season. It is of two types.

(i). Khariff

The season range from June to oct.

(ii). Rubby

The season range from oct to march.

→ cotton is a crop which takes 8 months to mature & sugarcane takes 12 months. i.e. perennial crop.

4. Overlap Allowance :-

Some time a crop of one season may overlap the next crop season by a few number of days. During this period of overlaping the irrigation water is to be supplied simultaneously to the crop for both the season due to the extra demand of water during this period the discharge of cannel has to be increases. For the purpose of cannel Design a provision should be made for the extra demand of water; these provision is termed as overlap allowance.

Time factor :-

The ratio of the no. of days the canal has actually been kept open to the no. of days to the no. of days the cannal was designed to kept open during the base period is known as time factor.

$$\text{Time factor} = \frac{\text{Actual discharge}}{\text{Original discharge}}$$

Capacity factor :-

Generally a canal is designed for max discharge capacity but actually it is not required that the cannal run to the maximum capacity all the time. So the ratio of the average discharge to the maximum discharge is known as capacity factor.

Cumic Day :-

$$1 \text{ cumic day} = \frac{1m^3}{sec} \times 24 \times 60 \times 60 \text{ sec}$$

$$= 24 \times 60 \times 60 \, m^3$$

$$1 \text{ hectare} = 10,000 \, m^3$$

$$= \frac{24 \times 60 \times 60 \, m^3}{10,000 \, m^2}$$

$$= 8.64 \text{ hect meter.}$$

Base period :-

The base is defined as the period from first to last watering of the crop just before its maturity. It is known as base period. Denoted by (B). and are expressed in number of days.

Delta (Δ) :-

Each crop require certain amount of water per hectare for its maturity. The total amount of water supplied to the soil from its initial to harvesting point is known as Δ & it is expressed in m.

Duty :-

It is the duty of water is defined as number of hectare that can be irrigated by constant supply of water at the rate of 1 cumic throughout the base period is known as Duty. Expressed as hectare/cumic.

Relationship between Base, Duty & Delta

1 cumic for B day depth Δ over D hectare

1 cumic for 1 day depth $\Delta = \frac{D}{B}$ hectare

1 cumic for 1 day $= \frac{D}{B} \times \Delta$ hectare meter

1 cumic day $= \frac{D}{B} \times \Delta$ hectare met.

as we know

1 cumic day $= 8.64$ hectare met

So,

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

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

* A channel is to be designed for irrigating 5000 hectares in khariff crop & 4000 hectares in rubby crop. The water requirement for khariff & rubby are 60cm & 25cm respectively. The base period of khariff is three weeks & Rubby is 2 weeks. Determine the Discharge of the channel which is to be designed.

~~D_1 = 5000 hectare~~ $\Delta_1 = 60cm = 0.6m$

~~D_2 = 4000 hectare.~~ $\Delta_2 = 25cm = 0.25m$

$B_1 = 21$ $B_2 = 14$

~~$D_1 = 3.024$ $D_2 = 4.08$~~

Area$_1$ = 5000 hectare

Area$_2$ = 4000 hectare

as we know,

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

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

$D_1 = 302.4$ hect/cumic, $D_2 = 483.36$ hect/cumic

$$\text{discharge}(Q_1) = \frac{5000}{302.4} = 16.53 \text{ cumic}$$

$$(Q_2) = \frac{4000}{483.36} = 8.26 \text{ cumic}$$

we shall take the maximum of these values. so, $Q = 16.53$ cumic.

* The gross command area of an irrigation project is 1.5 lakh hectare where ~~40~~ 7500 hectare are unculturable. The Area of khariff crop is 60,000 hect/cumic & Rubby ... is 40,000 hect/cumic. The Duty of khariff is 30,000 hect/cumic & the duty of rubby is 40,000 hect/cumic.

(i) Find out the design discharge of channel assuming 10% transmission loss.

(ii) Find out the intensity of irrigation for khariff & Rubby.

→ (i) $Q_1 = \frac{60,000}{30,000} = 20$ cumic

$Q_2 = \frac{40,000}{40,000} = 10$ cumic.

Due to transmission loss $= 20 + 0.1 \times 20 =$ ~~22 cumic~~ 22 cumic

(ii) ~~1.5~~ $1,50,000 - 7500 = 142500$

Intensity of ~~irrig~~ khariff $= \frac{60,000}{142500} \times 100$

$= 0.42 = 42\%$

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Intensity of rubby = $\frac{40,000}{142500} \times 100$

$= 28\%$

* The gross command area of an irrigation project is 1 lakh hectare. The culturable area is 75% of gross command area the intensity of irrigation for kharif & rubby are 50% & 55% respectively. If the duty's for kharif & Rubby are 1200 hectare cumic & 1400 hectare cumic respectively. Determine the discharge at the head of cannel considering 20% provision for transmition loss, overlap allowance, evaporation loss, etc.

~~Cultura~~ Culturable area = 75000

Area of kharif = 75000 × 0.5

= 37500

Area of rubby = 41250 = 75000 × 0.55

D_1 = 1200
D_2 = 1400 } (Given)

$Q_1 = 31.25 = \frac{37500}{1200}$ cumic

$Q_2 = \frac{41250}{1400} = 29.46$ cumic

$Q = 31.25 + 31.25 \times 0.2$

$= 37.5$ cumic

* Determine the head discharge of a cannel from the following data. The value of time factor may be assumed ~~0.05~~ 0.75.

Crop	Base period	Area in hectare	Duty
(K) Rice	120	4000	1500
(R) wheat	120	3,500	2000
(P) Sugarcane	310	3,000	1200

$Q_K = \frac{A}{D} = \frac{4000}{1500} = 2.67$

$Q_R = \frac{3500}{2000} = 1.75$

$Q_P = \frac{3000}{1200} = 2.5$

$Q_1 = Q_K + Q_P = 2.67 + 2.5 = 5.17$ cumic

$Q_2 = Q_R + Q_P = 1.75 + 2.5 = 4.25$ cumic

Time factor = 0.75 = $\frac{\text{Actual } Q}{\text{Original } Q}$

Original Q = $\frac{5.17}{0.75}$

Design discharge = 6.89 Cumic

* Find out the capacity of a reservoir from the following data. The Culturable command area is 8000 hectare.

Crop	Base in day	Duty	Intensity of irrigation
Rice	120	1800	25%
wheat	120	2000	30%
Sugarcane	320	2,500	20%

assume the canal & reservoir loss as 5% & 10%.

$\Delta_R = \frac{8.64 \times B}{D}$

$= \frac{8.64 \times 120}{1800} = 0.576$

$\Delta_W = \frac{8.64 \times 120}{2000} = 0.52$

$\Delta_S = \frac{8.64 \times 320}{2,500} = 1.1$

$Capacity_R = Area \times \Delta_R$

$A_R = 0.25 \times 80000 = 20000$

$A_W = 0.3 \times 80000 = 24000$

$A_S = 0.2 \times 80000 = 16000$

$C_R = A_R \times \Delta_R$
$= 11520$

$C_W = A_W \times \Delta_W$
$= 12480$

$C_S = A_S \times \Delta_S$
$= 17600$

So, Total Capacity of field = 11520 + 12480 + 17600
= 41600

Capacity of cannel = 41600 + 0.05 × 41600
= 43680

capacity of reservoir = 43680 + 0.1 × 43680
~~= 47504.8 hectare m~~
= 48048 hectare m

Types of Soil Water :→

When water is sprayed spread over the soil either by irrigation or by rainfall the water is absorbed by the pores of the soil. This water is termed as soil water or soil moisture. It is being categorised into 5 types.

→ Gravitational water
→ Capillary water
→ Hydroscopic water
→ Field capacity
→ Permanent welting point

(a). Gravitational Water :-

When it rains or the irrigation water is supplied to the soil the water contained of the soil goes on increasing untill a saturation point is reached. at this stage the soil pores are completly saturated & no more water is absorbed by the soil.

The surface water then starts flowing downwards due to the influence of gravity. The portion of water which flows down is called as gravitational water

(b). Capillary water :-

The portion of water remained & retained by the soil after completly elemination of the gravitational water is known as capillary water. The water content goes on reducing due to evaporation and transpiration.

(c). Hygroscopic Water :→

The water content below the permanent wilting point is known as hygroscopic water. This water is retained by the soil in the form of thin film on the surface of soil particles.

(d). Field Capacity :→

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

(e) Permanent wilting point :→

It is defined as the amount of moisture held by the soil which cannot be extracted through the root zone.

(f). Consumptive use of Water :→

The consumptive use of water is defined as the total quantity of water for the growth of plant by transpiration & the amount of loss by evaporation.

(iii) Frequency of Irrigation :→

The irrigation water is applied to the field to raise the moisture content upto its field capacity. The water content also reduce gradually due to transpiration and evaporation. If the moisture content is going below the permanent wilting point then immidiatly water supply is required. The diff. between the initial & the 2nd watering of plant is known as frequency of irrigation.

$$D_w = \frac{W_s \times d}{W_w}\left[F_c - M_o\right]$$

$$F_w = \frac{D_w}{C_u}$$

where,
D_w = depth of water applied in each watering.
d = depth of root zone.
W_s = Unit wt. of soil.
W_w = unit wt. of water.
F_c = field capacity
M_o = optimum moisture content.
F_w = frequency of irrigation.
C_u = Daily consumptive use of water.

Irrigation Efficiency :→

The amount of irrigation water supplied to the land is not fully utilised so the ratio of output to input is known as irrigation efficiency. It is of four types :

(a). Water conveyance efficiency (η_c)

. It is the ratio of amount of water applied to the land to the amount of water supplied from the reservoir.

(b). Water application efficiency (η_a)

~~The total amount of water~~

It is the ratio of water stored in the root zone of the plant to the water applied to the land.

(c). Water use efficiency (η_u).

It is the ratio of total amount of water used to the total amount water supplied.

(d). Consumptive use efficiency (η_{cu})

It is the ratio of consumptive use of water to the amount of water depleted from the root zone.

———x———

FLOW IRRIGATION

5.1 INTRODUCTION

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

(a) The construction of weir or barrage across a river (known as diversion head works).

(b) The construction of dam across a river valley (to form a storage reservoir).

(c) The excavation of canal system (Network of canals to cover the command area).

This type of irrigation is popular now-a-days because a vast area can be irrigated under this system. Some important projects (such as Bhakra Nangal Project, Ukai Project, Damodar Valley Project, etc) have been implemented in India to develop agriculture and to make the country self sufficient in food. The flow irrigation may be of two types, Inundation irrigation and Perennial irrigation.

In inundation irrigation, the canals are excavated from the banks of the inundation river. The bed level of the canal is such that the water can flow in rainy season only when the water level in the river rises above the canal bed. The construction of hydraulic structures is not necessary in this system. There is no head regulator to control the flow of water through the canal. In this system water is not available throughout the year.

In perennial irrigation either a weir or a barrage is constructed across the perennial river to raise the water level or a dam is constructed to form a storage reservoir. Then the network of canals (i.e. main, branch, distributory) is constructed from the source to the agricultural lands. Here, head regulator is constructed to control the flow of water through the canal. In this system, water is available throughout the year.

5.2 TYPES OF CANALS

1. *Based on Purpose* Based on the purpose of service, the canals are designated as (a) Irrigation canal (b) Navigation canal (c) Power canal (d) Feeder canal.

(a) Irrigation Canal The canal which is constructed to carry water from the source to the agricultural land for the purpose of irrigation is known as irrigation canal such as Bhakra Canal, Rajasthan Canal, etc.

(b) Navigation Canal The canal which is constructed for the purpose of inland navigation is known as navigation canal. This type of canal is also utilised for irrigation such as Ganga-Bramhaputra nagivation cum irrigation canal.

(c) Power Canal The canal which is constructed to supply water with very high force to the hydroelectric power station for the purpose of moving turbine to generate electric power is known as power canal or hydel canal such as Nangal Hydel Canal.

(d) Feeder Canal The canal which is constructed to feed another canal or river for the purpose of irrigation or navigation is known as feeder canal such as Farakka barrage feeder canal.

2. Based on Nature of Supply Based on the nature of supply, the canals are designated as (a) Inundation canal (b) Perennial canal.

(a) Inundation Canal The canal which is excavated from the banks of the inundation river to carry water to the agricultural land in rainy season only when the river flows to its full capacity is known as inundation canal. No regulator is provided at the head of such canal. The flow of water through the canal depends on the fluctuation of water level in the river. When the water level rises above the bed level of the canal the water starts flowing through the canal. When the water level falls below the bed level of the canal, the flow of water through the canal stops.

(b) Perennial Canal The canal which can supply water to the agricultural land throughout the year is known as perennial canal. This type of canal is taken from the up stream side of the diversion head works (weir or barrage) or from the storage reservoir with regulator at the head of the canal.

3. Based on Discharge According to the discharge capacity, the canals are designated as (a) Main canal (b) Branch canal (c) Distributory channel (d) Fie… channel.

(a) Main Canal The large canal which is taken directly from the diversion head work or from storage reservoir to supply water to the network of other small canals is known as main canal. The irrigation water is not directly supplied to the field from the main canal. The water is taken to the field through the branch canal, distributory channel and field channel. So the main canal is the backbone of the canal system (Fig. 5.1).

(b) Branch Canals The branch canals are taken from either side of the main canal at suitable points so that the whole command area can be covered by the network. The discharge capacity of the branch canal is smaller than that of the main canal. The discharge varies from 5 to 10 cumec (Fig. 5.1).

(c) Distributory Channels The distributory channels are taken from the branch canals to supply water to different sectors. The discharge capacity of these channels varies from 0.25 to 3 cumec. Again, these are designated as major distributory and minor distributory according to their function in the total network (Fig. 5.1).

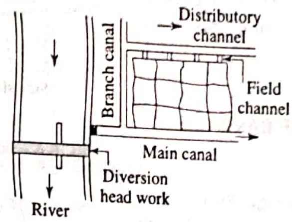


Fig. 5.1 Canal system

(d) Field Channels These channels are taken from the outlets of the distributory channels by the cultivators to supply water to their own lands. These channels are maintained by the cultivators (Fig. 5.1).

4. Based on Alignment Depending upon the alignment, the canals are designated as (a) Ridge or watershed canal (b) Contour canal, (c) Side slope canal.

(a) Ridge or Watershed Canal The canal which is aligned along the ridge line (watershed line) is known as ridge canal or watershed canal. The advantage of this type of canal is that it can irrigate the areas on both sides. Again there is no possibility of crossing any natural drainage and hence no cross-drainage work is necessary (Fig. 5.2).

(b) Contour Canal The canal which is aligned approximately parallel to the contour lines is known as contour canal. This canal can irrigate the areas on one side only. This canal may cross natural drainage and hence cross-drainage works are necessary (Fig. 5.3).

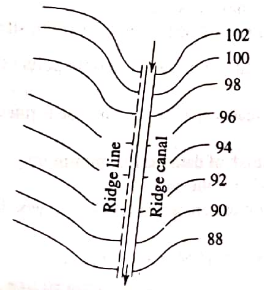


Fig. 5.2 Ridge canal

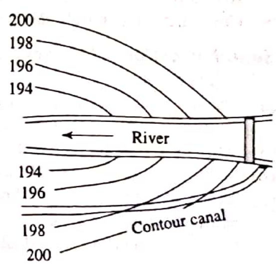


Fig. 5.3 Contour canal

(c) Side Slope Canal The canal which is aligned approximately at right angles to the contour lines is known as side slope canal. It can irrigate the areas on one side only. Again, it does not cross any natural drainage and hence the cross-drainage works are not necessary (Fig. 5.4).

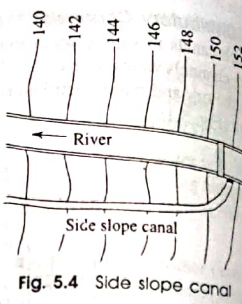


Fig. 5.4 Side slope canal

5.3 SELECTION OF BARRAGE OR DAM SITE

1. Selection of Barrage Site While fixing the site for a barrage, the following points should be considered,

(a) The site should not be on the curvature of the river.

(b) The river should be straight at least for a distance of one kilometre both on the up stream and down stream sides (Fig. 5.5).

(c) The river bank should be well defined.

(d) The width of the river should be minimum.

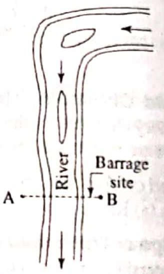


Fig. 5.5 Barrage site

(e) The storage reservoir should not submerge much valuable lands, villages, etc.

(f) The gross command area of the irrigation project should be nearer to the barrage site so that the length of main canal may be minimum to avoid transmission loss.

(g) The elevation of the barrage site should be higher than the command area so that the flow of water by gravity may be achieved.

(h) Construction materials and labours should be available near the site.

2. Selection of Dam Site While fixing the dam site, the certain points should be taken into account.

(a) The site should be on the valley so that deep reservoir may be formed with minimum surface area (Fig. 5.6).

(b) The site should be such that the length of dam may be minimum.

(c) Stable foundation should be available at the site.

(d) At the site, the rocks should not contain cracks, fissures, etc. which may cause loss due to leakage.

(e) Construction materials and labours should be available near the site.

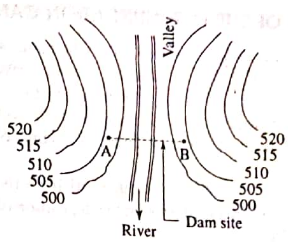


Fig. 5.6 Dam site

5.4 SELECTION OF ALIGNMENT OF PERENNIAL CANAL

The following points should be kept in mind while marking the tentative alignment of a canal,

1. The alignment should not pass through the valuable lands, religious places, villages, etc. to avoid unnecessary compensation and unwanted conflict.
2. The alignment should be short as far as possible, but to make it short the alignment should not be taken through the area where irrigation is not at all possible.
3. The alignment should be straight as far as possible.
4. If the curve is unavoidable in the alignment, then it should be provided according to IS: 5968–1970.

   Some references are given in the following table

Discharge (cumec)	–	*Radius (m)*
80–100	–	1200–1500
30–80	–	800–1000
15–30	–	400–600
5–15	–	100–150

5. The alignment should cross the natural stream, drainage, etc. approximately at right angles. At the crossing point, the width of the drainage should be minimum and the banks should be well defined.
6. The alignment should not involve heavy cutting or banking. It is preferable if balancing depth of cutting and banking may be achieved.
7. The alignment along the ridge line or watershed line is very good as the watershed canal can irrigate the areas on both the sides. Moreover, the cross-drainage works may be avoided.
8. The alignment should be such that the maximum area may be irrigated with minimum length of canal.
9. The alignment should not pass through the marshy land or water logged area, because the canal may collapse due to heavy moisture in the area.
10. The alignment should not pass through sandy soil as the percolation loss will be more and the duty of canal will be less.

5.7 ADVANTAGES AND DISADVANTAGES OF INUNDATION IRRIGATION

Advantages The following are the advantages of inundation irrigation.

1. In this system, no hydraulic structure is constructed.
2. In this system, the maintenance cost is low.
3. The water carried by the canal contains fine silt which improves the fertility of the land.
4. The canals run in rainy season only, and in other seasons the canals remain practically dry. So, there is no possibility of water-logging.

Disadvantages The following are the disadvantages of this system.

1. As there is no control over the flow of water over irrigation may spoil the crops.
2. The supply of water is uncertain.
3. It is not applicable in rabi season.
4. The head of the canal may be damaged during flood.
5. In case the river changes its course, the whole system is to be abandoned.
6. Silting is the main problem which involves recuring cleaning expenditure.

5.8 COMPARISON BETWEEN INUNDATION AND PERENNIAL IRRIGATION

Inundation irrigation	*Perennial irrigation*
1. The irrigation water is available in rainy season only.	1. The irrigation water is available throughout the year.
2. No hydraulic structure is necessary.	2. Hydraulic structures are necessary, (such as diversion headhead works, cross-drainage works, etc.).
3. The canal water contains plenty of silt which makes the land fertile.	3. The canal water contain practically no silt and hence chemical manure is essential.
4. Large area cannot be included under this system.	4. Large area can be included under this system.
5. The silting of the canal bed is a major problem.	5. Negligible silting takes place in the canal bed.
6. Water tax cannot be imposed.	6. Water tax can be imposed.
7. Initial cost is low.	7. Initial cost is high.
8. No technical persons are required for the operation of the irrigation system.	8. Technical persons are always required for the operation of the irrigation system.
9. The main canal is not provided with regulator and hence there is a possibility of over irrigation.	9. The main canal is provided with head regulator and hence there is no possibility of over irrigation.

5.9 SYSTEM OF BANDHARA IRRIGATION

This is a minor irrigation system suitable for irrigating isolated areas, up to 500 hectares. The bandhara is similar to weir which is constructed across a small stream to raise the water level on the up stream side to divert the water through the canal [Fig. 5.12(a)].

The height of the bandhara depends on the water level to be raised on the up stream side [Fig. 5.12 (b)].

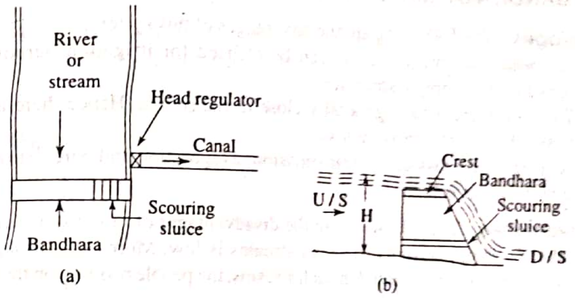


Fig. 5.12 Bandhara irrigation

It is constructed with brick masonry or stone masonry with R.C.C. crest. The crest width varies from 1 m to 2 m. The scouring sluices are provided at the bottom of the bandhara near the head reach of the canal. The function of scouring sluices is to remove the silt which may get deposited in front of the canal head. Normally, the sluices are kept closed and these are opened when the deposited silt is to be removed. The surplus water is allowed to pass over the crest of the bandhara.

In this system, the water is directly taken from the main canal and supplied to the agricultural land. The total area under a bandhara is known as *Thal*. Again, the *Thal* is divided into several zones which are known an *Phad*. That's why, sometimes this system is known as '*Phad* irrigation system.' (Fig. 5.13).

This system is suitable for small streams. Sometimes, more than one bandhara may be constructed on the same stream at a reasonable interval to

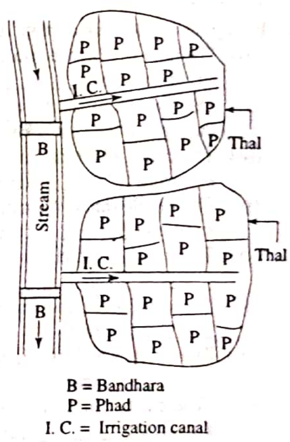


Fig. 5.13 Thal and phad

Canal Section :→

The several terms in the canal section are:

(i). canal bank
(ii). Berm
(iii). Hydrolic gradient.
(iv). counter berm
(v). Free board,
(vi). service road.
(vii). Doweel or Dowela.
(viii). Borrowe pits
(ix). spoil bank
(x). Land width.

(i). canal bank →

It is necessary to retain water in the canal to the full supply level (FSL). The cutting of the cannal are of three types:

→ Canal in fully cutting.
→ canal in partially cutting & banking
→ Canal in fully banking.

(a) Canal in fully cutting :-)

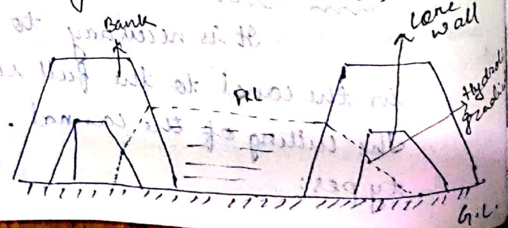
In this case the banks are constructed on both side of the canal, to provide only a inspection road. Here the hydrolic gradient has no function.

(b). Canal in partially cuttings & Banking:-

In this case the bank are contructed on the side of the canal to retain water, the height of the bank depend upon the full supply level of the canal. The hydraulic gradient should have a minimum cover of 0.5m.

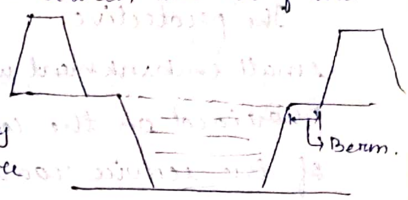
(c). Canal in fully banking :-)

In this case the canal & both the canal bank are constructed over above the ground level to minimise the cross-section of the bank a core wall is provided which deflect the hydrolic gradient downwards.



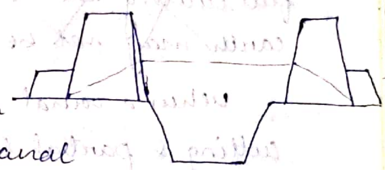
(ii). Berm :-)

The distance between the toe of the bank and the tope edge of cutting is known as Berm. Necessity of providing berms are as follows :-



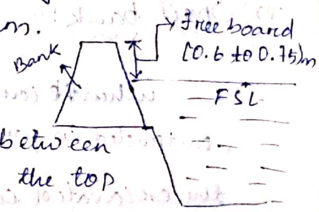
→ To protect the bank from erosion.
→ To provide space for widening of canal
→ To protect the bank from down
→ The silt deposit on the berm makes an imperivious lining.

(iii). Counter berm :-)



When the water is retained by a canal bank the hydraulic gradient line passes through the body of bank for stability of the bank a minimum projection of soil is provided on both side of the bank to control the hydraulic gradient line is known as counter berm.

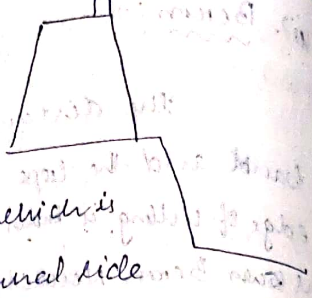
(v). Free board :-)



It is the distance between the full supply level & the top of the bank.

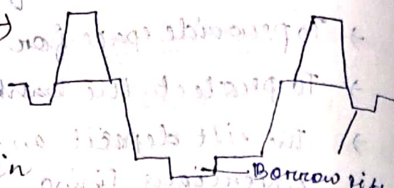
The amount of free board varies from 0.6 to 0.75 m

(vii) Dowel or Dowla :-



The protective small embankment which is provided on the canal side of the service road for the safety of the vehicles is known as dowel.

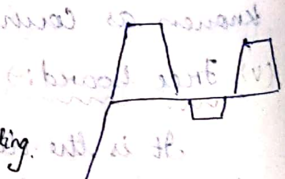
(viii) Borrow pits :-



~~when a canal is constructed in full cutting the excavated earth may not be completel~~

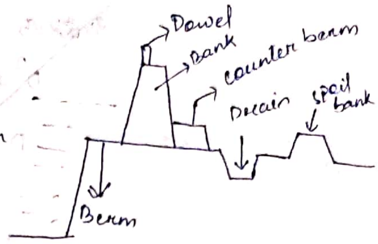
when a canal is constructed in partial cutting & partial banking the excavated earth may not be sufficient for forming the required bank. In such a case the extra earth required for construction ~~which is know~~ is taken from some pits known as borrow pits.

(ix) Spoil bank :-



when the canal is constructed in ~~part~~ full cutting the excavated earth may not be completely required for forming the bank in such case the extra earth is deposited in the form of small bank which is known as spoil bank

(x) Land width :-



The total width required for the construction of a canal is called as land width.

(xi) Balancing depth :-

In constructing a canal section if the quantity of excavated earth can be fully utilized for making the bank on both side then the section is called economical section. The depth of cutting for that ideal condition is known as balancing depth.

Find the balancing depth for a canal section in fully cutting have the following data. Bed width of canal = 10m.

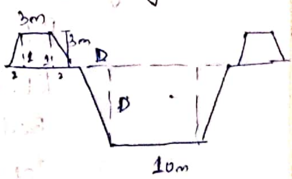
Side slope in cutting = 1:1

Side slope in banking = 2:1

Top width of bank = 3 m.

height of bank above the ground level = 3m.

~~sied ex.~~



B = 10m. D = 3m.

Area of bank = $\frac{(15+3)}{2} \times 3$

= ~~[crossed out]~~

= ~~[crossed out]~~ 45 × 2 = 90

Area of canal = (B + D) D.

~~90.22.05 = (10 + D) D.~~

~~$D^2 + 10D - 90.0 = 0$.~~

~~D = 5.72~~

Area of bank = $2 \frac{(15+3)}{2} \times 3$.

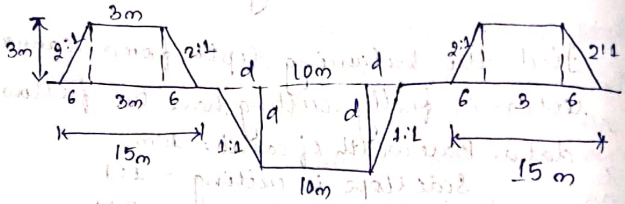
= 54 sq m.

Area of canal = (B + D) × D

54 = (10 + d) × d.

$d^2 + 10d - 54 = 0$.

d = 3.88.



*. Draw the section of a canal in fully cutting with the following data:

RL of ground level = 152.50m

RL of canal bed = 150m

Bed width of canal = 15m

Top width of bank = 3m

height of bank = 1.5m.

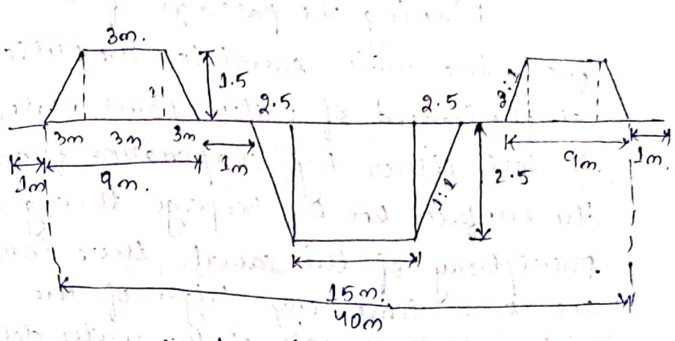
Berm = 1m.

Side slope in cutting = 1:1

Side slope in banking is 2:1

full supply depth = 2.5.

find also the land width required.



land width = 42m

= 40 + 2

= 42m.

*. Draw the section of a canal in fully banking with the following data:

RL of Ground level = 148m.

The canal bed is just at ground level

Bed width of canal = 12m

full supply depth = 2m.

Free board = 0.5 m

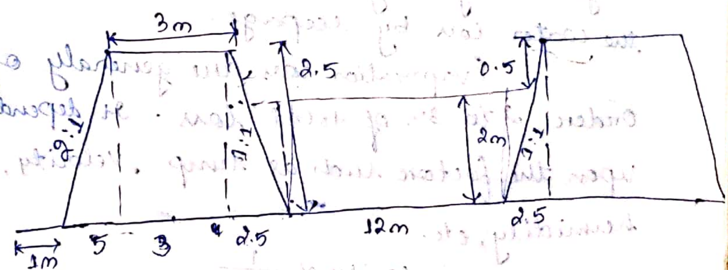
Hydraulic gradient = 1 in 4.

Top width of bank = 3m

Side slope of bank = 1:1 in canal side

& 2:1 in countryside

Find also the land width.



land width = 35m.

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Canal losses :-

During the passage of water from the main canal to the outlet at the head of water pores water may be lost either by evaporation from the surface or by seepage through the pheriphery of the canal these losses are some-time very high of the order of 25 to 50% of the water deleivered to the channel.

The losses in the canal are taken place by two categories.

(i) evaporation.
(ii) Seepage
where the seepage are divided into two types. (a) absorption.
(b) Percolation.

(i) Evaporation loss :-

The water lost by evaporation is generally very small as compared to the water loss by seepage.

Evaporation loss are generally of order 2 to 3% of total loss. It depends upon the factor such as temp., velocity, humidity, etc.

$$\text{Velocity} \propto \frac{1}{\text{loss of water}}$$

(ii) Seepage loss :-

There are of two types :

(a) Percolation:-

It is the process in which the water passes through the soil under the action of gravity towards the water table. This process is known as percolation loss.

(b) Absorption loss :-

when the water passes through the canal for which the side of the soil of the canal get wetted up to a welting point then this process is known as absorption process.

The factor depending upon the seepage are :

(i) Type of seepage loss.
(ii) Permeability of soil.
(iii) Condition of canal.
(iv) Velocity of flow.

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Lined Canal :-

It is the type of canal in which we are providing a water proofing or an impervious layer on the entire side of the cutting of the canal. Then that type of canal is known as lined canal.

Advantages of lined canal :-

→ It reduces the loss of water and hence the duty increases.

→ It controlles the water logging and hence the velocity of flow can be increased.

→ Due to the increase capacity of the discharge capacity of the canal is also increases.

→ It provides smooth surface & hence the velocity increases.

→ Due to the increase in velocity the evaporation loss is reduced.

→ It eleminates the effect of Scorring in bed.

→ It provides stable section to the canal. → It reduce the requirement of the length of the canal becoz smaller section can produce higher discharges.

→ It reduces the maintainance cost of the canal.

Types of lined canal :-

→ Cement concrete lining.
→ pre caste lining.
→ Cement mortar lining.
→ Brick lining.
→ Short crete lining.
→ Asphalt lining.
→ Bentonight & clay lining.
→ Soil cement lining.
→ Boldere lining.

7.3 CEMENT CONCRETE LINING

This lining is recommended for the canal in full banking. The cement concrete lining (cast in-situ) is widely accepted as the best impervious lining. It can resist the effect of scouring and erosion very efficiently. The velocity of flow may be kept above 2.5 m/sec. It can eliminate completely growth of weeds. The lining is done by the following steps,

(a) Preparation of sub-grade The sub-grade is prepared by ramming the surface properly with a layer of sand (about 15 cm). Then, a slurry of cement and sand (1:3) is spread uniformly over the prepared bed.

(b) Laying of concrete The cement concrete of grade M_{15} is spread uniformly according to the desired thickness (generally, the thickness varies from 100 mm to 150 mm). After laying, the concrete is tapped gently until the slurry comes on the top. The curing is done for two weeks. As the concrete is liable to get damaged by the change of temperature, the expansion joints are provided at appropriate places. Normally no re-inforcement is required for this cement concrete. But in special cases, a network of 6 mm diameter rods may be provided with spacing 10 cm centre to centre (Fig. 7.1).

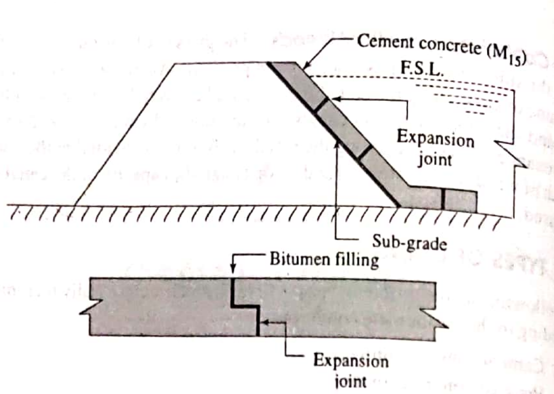

Fig. 7.1 Cement concrete lining

7.4 PRE-CAST CONCRETE LINING

This lining is recommended for the canal in full banking. It consists of pre-cast concrete slabs of size 60 cm × 60 cm × 5 cm which are set along the canal bank and bed with cement mortar (1:6). A network of 6 mm diameter rod is provided in the slab with spacing 10 cm centre of centre. The proportion of the concrete is recommended as 1:2:4. Rebates are provided on all the four sides of the slab so that proper joints may be obtained when they are placed side by side. The joints are finished with cement mortar (1:3). Expansion joints are provided at a suitable interval. The slabs are set in the following sequence,

(a) The sub-grade is prepared by properly ramming the soil with a layer of sand. The bed is levelled so that the slabs can be placed easily.

(b) The slabs are stacked as per estimate along the course of the canal. The slabs are placed with cement mortar (1 : 6) by setting the rebates properly. The joints are finished with cement mortar (1:3).

(c) The curing is done for a week (Fig. 7.2).

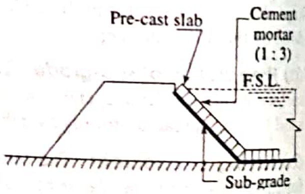

Fig. 7.2 Pre-cast concrete lining

7.5 CEMENT MORTAR LINING

This type of lining is recommended for the canal fully in cutting where hard soil or clayey soil is available. The thickness of the cement mortar (1:4) is generally 2.5 cm. The sub-grade is prepared by ramming the soil after cutting. Then, over the compacted sub-grade, the cement mortar is laid uniformly and the surface is finished with neat cement polish. This lining is impervious, but is not durable. The curing should be done properly (Fig. 7.3).

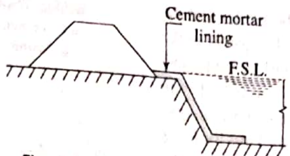

Fig. 7.3 Cement mortar lining

7.6 LIME CONCRETE LINING

When hydraulic lime, surki and brick ballast are available in plenty along the course of the canal or in the vicinity of the irrigation project, then the lining of the canal may be made by the lime concrete of proportion 1:1:6. The procedure of laying this concrete is same as that of the cement concrete lining. Here, the thickness of concrete varies from 150 mm to 225 mm and the curing should be done for longer period. This lining is less durable than the cement concrete lining. However, it is recommended because of the availability of the materials and also because of the economics.

7.7 BRICK LINING

This lining is prepared by the double layer brick flat soling laid with cement mortar (1:6) over the compacted sub-grade. The first class bricks should be recommended for the work. The surface of the lining is finished with cement plaster (1:3) (Fig. 7.4). The curing should be done perfectly.

This lining is always preferred for the following reasons,

(a) This lining is economical.
(b) Work can be done very quickly.
(c) Expansion joints are not required.
(d) Repair works can be done easily.
(e) Bricks can be manufactured from the excavated earth near the site.

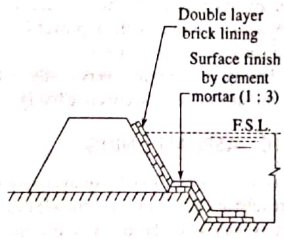

Fig. 7.4 Brick lining

However this lining has certain disadvantages,

(a) It is not completely impervious.
(b) It has low resistance against erosion.
(c) It is not so much durable.

7.8 BOULDER LINING

In hilly areas where the boulders are available in plenty, this type of lining is generally recommended. The boulders are laid in single or double layer maintaining the slope of the banks and the bed level of the canal. The joints of the boulders are grouted with cement mortar (1:6). The surface is finished with cement mortar (1:3). Curing is necessary in this lining too. This lining is very durable and impervious. But the transporting cost of the material is very high. So, it cannot be recommended for all cases (Fig. 7.5).

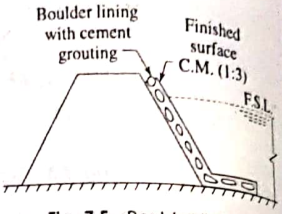

Fig. 7.5 Boulder lining

7.9 SHOT CRETE LINING

In this system, the cement mortar (1:4) is directly applied on the sub-grade by an equipment known as cement gun. The mortar is termed as shot crete and the lining is known as shot crete lining. The process is also known as guniting, as a gun is used for laying the mortar. Sometimes, this lining is known as gunited lining. The lining is done in two ways,

(a) By Dry Mix In this method, a mixture of cement and moist sand is prepared and loaded in the cement gun. Then it is forced through the nozzle of the gun with the help of compressed air. The mortar spreads over the sub-grade to a thickness which varies from 2.5 cm to 5 cm.

(b) By Wet Mix In this process, the mixture of cement, sand and water is prepared according to the approved consistency. The mixture is loaded in the gun and forced on the sub-grade.

This type of lining is very costly and it is not durable. It is suitable for resurfacing the old cement concrete lining.

7.10 ASPHALT LINING

This lining is prepared by spraying asphalt (i.e. bitumen) at a very high temperature (about 150°C) on the subgrade to a thickness varies from 3 mm to 6 mm. The hot asphalt when becomes cold forms a water proof membrane over the subgrade. This membrane is covered with a layer of earth and gravel. The lining is very cheap and can control the seepage of water very effectively but it cannot control the growth of weeds.

7.11 BENTONITE AND CLAY LINING

In this lining a mixture of bentonite and clay are mixed thoroughly to form a sticky mass. This mass is spread over the sub-grade to form an impervious membrance which is effective in controlling the seepage of water, but it cannot control the growth of weeds. This lining is generally recommended for small channels.

7.12 SOIL-CEMENT LINING

This lining is prepared with a mixture of soil and cement. The usual quantity of cement is 10 per cent of the weight of dry soil. The soil and cement are thoroughly mixed to get an uniform texture. The mixture is laid on the sub-grade and it is made thoroughly compact. The lining is efficient to control the seepage of water, but it cannot control the growth of weeds. So, this is recommended for small channels only.

7.13 ADVANTAGES AND DISADVANTAGES OF CANAL LINING

Advantages

1. It reduces the loss of water due to seepage and hence the duty is enhanced.
2. It controls the water logging and hence the bad effects of water-logging are eliminated.
3. It provides smooth surface and hence the velocity of flow can be increased.
4. Due to the increased velocity the discharge capacity of a canal is also increased.
5. Due to the increased velocity, the evaporation loss also be reduced.
6. It eliminates the effect of scouring in the canal bed.
7. The increased velocity eliminates the possibility of silting in the canal bed.
8. It controls the growth of weeds along the canal sides and bed.
9. It provides the stable section of the canal.
10. It reduces the requirement of land width for the canal, because smaller section of the canal can produce greater discharge.
11. It prevents the sub-soil salt to come in contact with the canal water.
12. It reduces the maintenance cost for the canals.

Disadvantages

1. The initial cost of the canal lining is very high. So, it makes the project very expensive with respect to the output.
2. It involves much difficulties for repairing the demaged section of lining.
3. It takes too much time to complete the project work.
4. It becomes difficult, if the outlets are required to be shifted or new outlets are required to be provided, because the dismantling of the lined section is difficult.

7.14 SELECTION OF TYPE OF LINING

The selection of particular type of lining depends on the following factors,

(1) Imperviousness When the canal passes through the sandy soil, the seepage loss is maximum and the canal is unstable. So, to make the canal perfectly impervious and reasonably stable, the most impervious types of linings should be recommended such as cement concrete lining, pre-cast concrete lining, boulder lining, etc.

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Reclamination of water log in Salient soil:

In agricultural land when the soil pores within the root zone of the crop get saturated with the subsoil water, the air circulation within the soil pores get totally stop this phenomenon is known as water logging.

Causes of Water logging :-

(i). Over irrigation.
(ii). seepage from canal
(iii). Inadequate surface drainage.
(iv). obstruction in natural water course
(v). obstruction in sub-soil drainage
(vi). Nature of soil.
(vii). Incorrect method of cultivation.
(viii). Seepage from reservior.
(ix). Poor irrigation management.
(x). access rainfall
(xi). Topography of the land.
(xii) Occational flood.

(i) Over irrigation :-

In inundational irrigation since there is no control in system of water supply it may cause over irrigation.

(ii) Seepage of canal :-

In unlined canal system the water percolates through the bank of the canal & gets collected in the low - lying area along the course of the canal & thus the water table get raised.

(iii) Inadequate surface drainage :-

When the rainfall is heavy & there is no proper provision for surface drainage then the water log will be seen there.

(iv) Obstruction in natural water course :-

If the bridge or culvert are constructed across a water course with the opening with insufficient discharge, the upstream area get flooded & this cause waterlogging.

(v) Obstruction in subsoil drainage :-

If some impervious stratum exist at a lower depth below the ground surface then the movement of subsoil water get obstructed and cause water logging.

(vi) Nature of soil :-

The soil having low permiability like black cotton soil which doesnot allow the water to percolate so incase of over irrigation the water retain in this type of soil which may cause waterlogging.

(vii). Incorrect method of cultivation :-

If the agricultural land is not leveled properly and there is no arrangement for the surplus water to flow then it may cause waterlogging.

(viii) Seepage from reservoir :-

If the reservoir basin consist of permeable zone or crack which were not detected during the construction then this may cause water logging.

(ix). Poor irrigation management :-

If the main canal is kept open for a long period unnecessarily without computing the total water requirement then it may cause water logging.

(x) Excessive rainfall :-

If the rainfall is excessive & the water get no time to get drained then a pool of water is formed which cause logging to the land

(xi). Topography of the land :-

If the agricultural land is flat with no country slope, then it leads for water logging.

(xii) Occational flood :-

If an area get affected by flood every year and there is no proper drainage then the water table get raised & cause water logging.

Control of water logging :-

1. Prevention from percolation from canal.
2. Control of intensity of irrigation.
3. Economic use of water.
4. Fixing of crop pattern.
5. Providing drainage system.
6. Pumping of ground water.
7. Construction of sump well.

1. Prevention from percolation from canal :-

The irrigation canal should be lined with an impervious lining to prevent the percolation of water through the bed and banks of canal.

2. Control of intensity of irrigation :-

The intensity of irrigation may cause water logging so it should be controlled in a planned way so that there is no possibility of water logging.

3. Economic use of water:-

If the water is used economically then it may be controlled the water-logging & the yield of the crop will be improved

4. Fixing of crop pattern:-

Soil survey should be conducted to fix the crop pattern. the crop having high rate of evaporation should be recomended for the area effected by water logging.

5 Providing drainage system:-

Suitable drainage system should be provided in low line areas so that the rainwater does not stand for long days

6. Pumping of ground water:-

A number of open well or tube well are constructed in the waterlogged area & the ground water is pumped out & untill the water table goes down safely.

7 Construction of sump well:-

Sump well may be constructed within the water logged area and helps to collect the surface water.

LAND RECLEMATION:-

The reclimation of land is the process of making a land culturable after it gets converted to culturable area due to the bad effect of water logging.

Methods of land Reclimation:-

- Leaching.
- Addition of chemical agents
- Surface drainage
- Sub surface drainage
- Addition of waste product
- Excavation of Ponds
- Pumping of water from the tube well.

1. Leaching:

It is a process for reclemation of saline soil. In this process the agricultural land is flooded with water to a depth of 20-30 cm. The salt deposits on the surface are dissolved. Some portion of the salt is then drained off through the subsoil drainage system and some portion is removed by surface drainage system. This operation is repeated several time at a specific interval.

2. Addition of chemical agent :->

For improving the alkaline soil a chemical like gypsum is generally added with the irrigation water. The gypsum neutralised the alkaline effect of the soil and the yield of the crop is improved. The application of gypsum is not necessary every year.

3. Surface drainage :->

Proper surface drainage system should be provided in the agricultural land so that the water doesnot accumulate for a long time. The surface drain also help in draining the saline water in case of leaching operation.

4. Sub-surface drainage :->

The sub-surface drainage system of an agricultural land should also be provided for draining the excess water from the root zone. It also help in draining of saline water incase of leaching operation.

5. Addition of Waste water :-> Product :->

Waste product like groundnut shell, cane dust, etc. are added to alkaline soil & these are very effective in removing the salinity of the soil.

6. Excavation of pond :->

Ponds are excavated at suitable places within the waterlogged area. The excess runoff is collected in the pond. The pore water also flow towards the pond and thus the saturation in the root zone of the crop is reduced. Infact This pond controll the waterlogging in rainyseason & in dry season. The water of the pond may be utilised for lift irrigation.

7. Pumping of water from tube well :->

Some tube well are sunk with in the waterlogged area. The water is pumped continuously from the tube well. initially this water is discharged to a river or pond. when the reclimation of the land is complete the water may be utilized for lift irrigation.

Diversion Head Work

INTRODUCTION :-

The water flow through the irrigation canal under the force of gravity. So the elevation of the head of the canal must be higher than the command area of the irrigation project. Now to form a storage reservoir or to raise the water level of the head of the canal some structure are constructed which is known canal head work.

The canal head work are of two types
(i) Storage head work (dam)
(ii) Diversion head work

Diversion head work :-

A weir or barrage is constructed over a perennial river to raise the water level and to divert the water to the canal then it is known as diversion head work.

Objects of diversion head work :-

To raise the water level at the head of the canal.

To form a storage by constructing dikes on both of the bank of the river so that the water is available through out the year.

To control the entry of silt into the canal & to control the deposition of silt at the head of the canal.

To control the fluctuation of water level in the river during different season.

* Component part of diversion head work :-

- Weir or Barrage
- Divide wall
- Scouring sluices.
- Fish ladder.
- Canal head regulator.
- Silt excluder.
- Guide bank
- Marginal embankment or dyke.

(ii). Divide wall :-

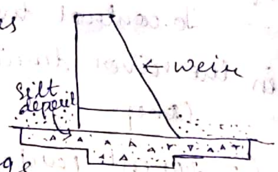
It is long wall constructed at right angle to the weir or barrage. It may be constructed with stone masonry or cement concrete masonry.

→ To form a still water pocket in front of the canal head so that the suspended silt can be settled down which then later be cleaned through the scoring sluices from time to time

-> It recists the overturning effect of the weir & barrage.

(ii). Scoring sluices :-)

The scoring sluices are the opening provided at the base of the weir & barrage

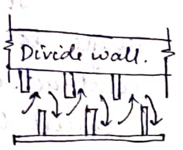


these opening are provided with adjustable gates. Normally the gates are kept close the suspended silt goes on deposition infront of the canal head regulator. when the silt deposition becomes appreciable the gate are opened. & the deposited silt is loosed with an agitator mounting on a boat.

But at the period of flood the gate are generally kept open.

(iii). Fish ladder :-)

It is provided just by the side of divide wall for the free movement of fish. In general the tendency of fish is to move from upstream to downstream in wind winter & downstream to upstream in monsoon.



(Fish ladder)

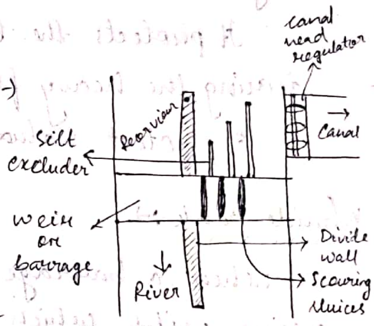
For the free movement of the fishes along the course of the river. The fish ladder is essential. In the fish ladder the baffle wall are constructed in zig-zag manner to control the velocity.

(iv). Canal Head Regulator :->

A structure which is constructed at the head of the canal to regulate flow of water is known as canal head regulator.

(v). Sill excluder :-)

when steel water pocket is formed infront of the canal head by conteuicting the divide wall, then it is found that the lower layer of water contain heavy silt & the upper layer contain very fine silt. To eleminate the light silt the silt excluder are generally being provided. The suspended heavy silt entered the silt excluder canal & passes out



(silt excluder)

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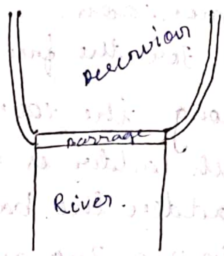
through the scoring sluices.

(vi). Marginal embankment & Dykes :-



The marginal embankment or dykes are the earthen embankment which is constructed parallel to the river bank on one or both the bank accoring to the condition.

It prevent the flood or storagewater from entering the surrounding area which may be submerged or water logged.

It protects the town & villages during the heavy flood.

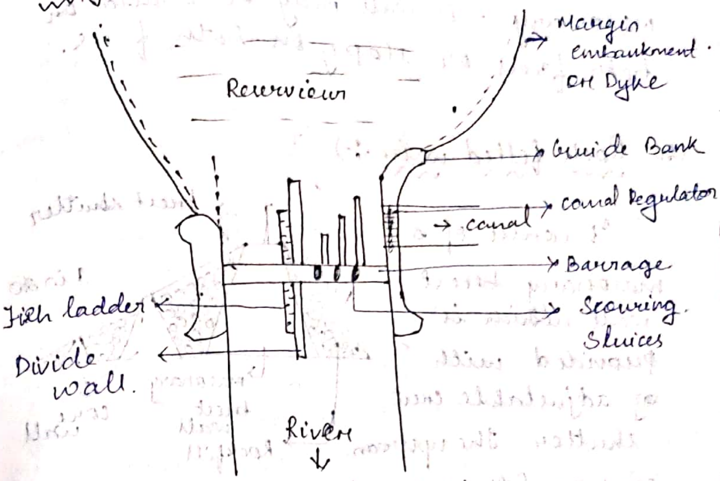
It protect valuable agricultural land.

(vii) Guide Bank :-

When a barrage is constructed across a river which flow through the alluvial soil the guide bank must be constructed on both the approaches to protect the structure from errosion.

It is an earthen embankment with curve head on both the ends

Diagram of Divergent headwork :-

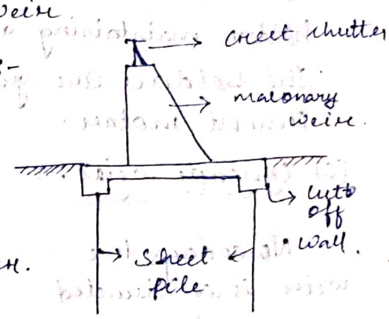


TYPES OF WEIR :-

The weir are of diff types:

(i) Masonary weir.
(ii). Rock filled weir
(iii). Concrete weir

(a). Masonary weir :-



The masonary weir wall is constructed over the impervious floor. Cutt off wall are provided at both end of the floor. Sheet pile are provided below the cut off wall. The crest shutter are provided to raise the water level if required.

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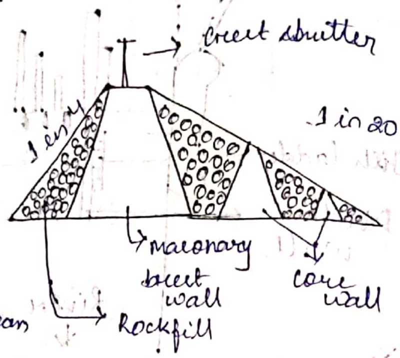
Scanned with CamScanner

The shutter are dropped during flood. The masonary weir wall may be vertical on both faces or slopy on both faces.

(b) Rock filled weir :-)

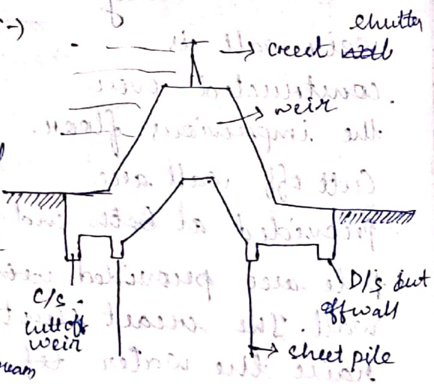
It consist of a masonary breast wall which is provided with adjustable crest shutter. The upstream



rock fill portion is constructed with bolders forming a slope of 1 in 4. The bolder are grouted with cement motar. the down stream sloping apron consist of core wall. The intermediate space between the core wall are filled with bolders maintaining a slope of 1 in 20. The bolders are grouted properly with cement motar.

(C) Concrete weir :-)

Nowadays the weir is constructed with reinforce cement concrete. The cutt off wall are provided at the upstream & downstream



end of the floor at the toe of the weir. sheet pile are provided below the cut off wall. The crest shutter are also provided which are dropped during the flood.

Canal Fall

Irrigation canal are constructed with some permissible bed slope so that there is no silting and scoring generally the slope of the natural ground surface is not uniform. in such case a vertical drop is provided to step down the canal bed and then it is continued with a permissible slope. These type of structure are called as canal fall.

Necessity of canal fall :-)

(i) when the slope of the ground suddenly changes to steeper slope, the permissible bed slope cannot be maintained it require excessive earth work, so in this case canal fall is necessary.

(ii) when the slope of the ground is more or less uniform. In that case also canal fall is required.

(iii) In cross drainage work (Like siphonic superpassage) for the smooth movement of water canal fall is necessary.

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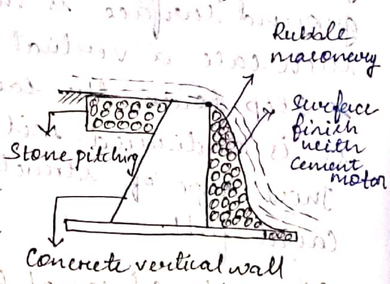
~~(iv) Types of canal fall,~~

Types of Canal fall :-)

The canal fall are of diff. types.

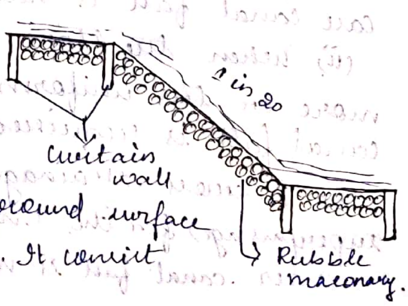
(i) Ogee fall
(ii) Rapid fall
(iii). Stepped fall.
(iv) trapizoidal notch fall.
(v). vertical drop or Sarada fall.
(vi). glacis fall.

(I). Ogee fall :-)



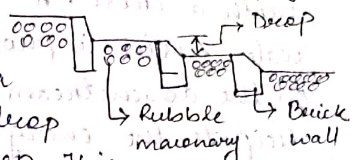
In this type of fall an ogee curve of a convex shape is provided for carrying the canal water from higher level to lower level. This fall is recomended when the natural ground surface changes to ~~depp~~ steeper slope.

(ii). Rapid fall :-)



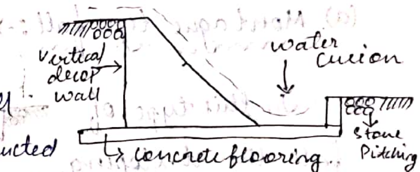
The rapid fall is suitable when the slope of the natural ground surface is even and long. It consist of a long sloping glacies with longitudinal slope which varies from 1 in 10 to 1 in 20. Curtain wall are provided on the upstream & downstream side of the slopping. The upstream & downstream side of the fall is protected by double maconary.

(iii). Stepped fall :-)



It consist of a series of vertical drop in the form of step. This fall is suitable, where the sloping ground fall is very long. This fall is pracdically a modification of rapid fall. Brick wall are provided at each of the drop. The bed of the canal with in the fall is protected by rubble maconary with surface finishing by brick cement motar.

(v). Vertical drop or sarada fall :-)



It conist of a vertical drop wall which is constructed with maconary work. The water flow over the crest of the wall. A water cusion is provided on the down stream side which is used to dissipate the energy. A concrete floore is provided on the downstream side to control the scouring effect of water. This type of fall can be provided on sarada ~~cna~~ canal in UP. Hence it is also known as Sarda fall.

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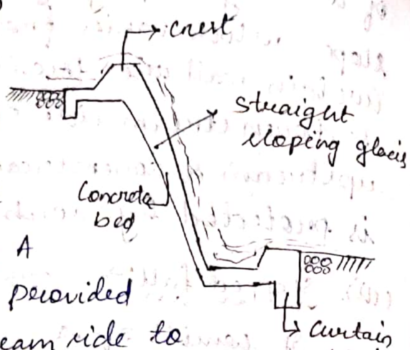
(vi). Glacis fall :-)

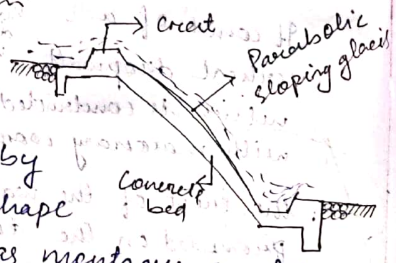


It consist of a st. slopping glacis provided with a crest. A water cusion is provided on the downstream side to decipate the energy of flowing water. The sloping glacis is constructed with cement concrete. Curtain wall are provide at the toe & the heel. This type of fall is suitable for a drop upto 1.5m.

It is of two type :-

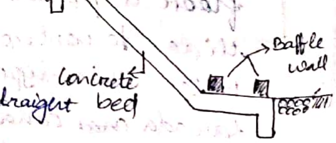
(a). Montague type fall.
(b). Inglish type fall

(a). Montague type fall :-)



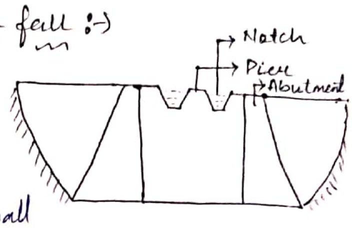
In this type of fall the st. sloping glacis is modified by giving parabolic shape which is known as montague profile.

(b). Inglish type fall :-)



In this type of fall the glacis is straight and slopy. But Baffle wall are provided on the downstream flow to decipate the energy of the flowing water. The height of baffle wall depends upon the head of water on the upstream side.

(iv). Trapizoidal Notch fall :-)



In this type of fall a body wall is constructed across the canal the bodywall consist of several trapizoidal notches between the side pier & intermediate pier. The sills of the notches are kept at the upstream bed level of the canal. An impervious floor is provided to resist the scoring effect of water. The side & number of notches depends upon the full supply discharge of canal.

Cross Drainage Work

In an irrigation project when the network of main canal, branch canal etc are provided then this canal may have to cross the natural drainage like river stream, etc at diff. point with in the command area of the project. Suitable structure must be constructed at the crossing point for the easy flow of water in the respective direction these structure are known as cross-drainage work.

Types of Cross drainage work :→

According to the relative bed level, maximum water level & relative discharge of the canal & drainage. The cross drainage work may be of following type.

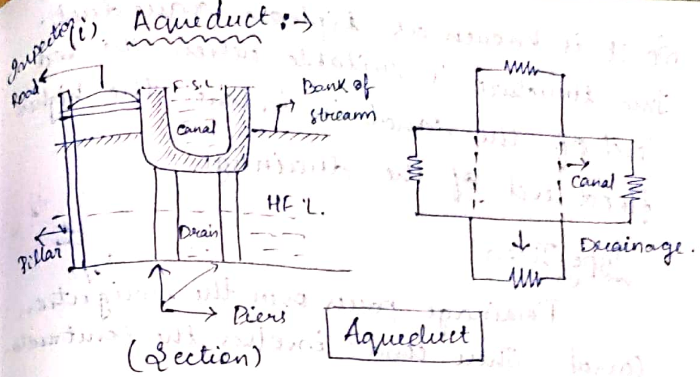
Type 1 :-

Irrigation canal passes over the drainage

(i). Aqueduct

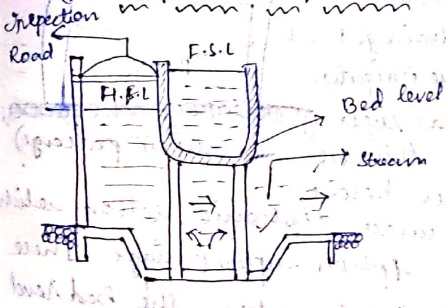
(ii). Siphon aqueduct

(i). Aqueduct :→



The hydraulic structure in which the irrigation canal is taken over the drainage is known as aqueduct. This structure is suitable when the bed level of the canal is above the highest flood level (HFL) of drainage. In this case the drainage water passes clearly below the canal.

(ii). Siphon aqueduct :→



In a hydraulic structure where the canal is taken over the drainage but the drainage water cannot pass clearly below the canal. It flows under siphonic action.

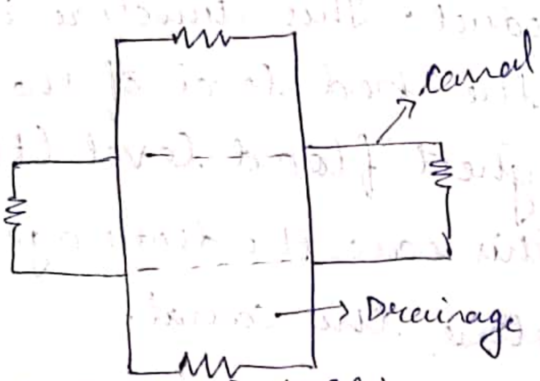
So it is known as siphon aqueduct. This structure is suitable when the bed level of the canal is below the highest flood level of the drainage.

Type-II :-)

Drainage passes over the irrigation canal. These cond^n involves the construction of :-

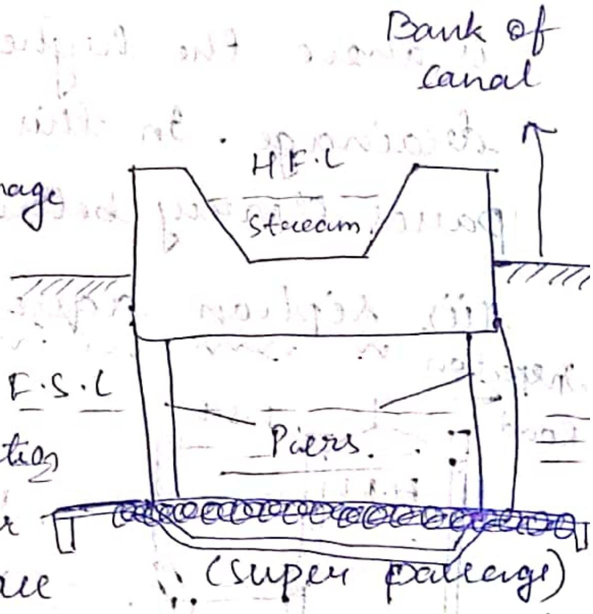
(i). Super passage.

(ii). siphon super passage



(ii) siphon super passage :-

The hydraulic structure in which the drainage F.S.L is taken over the irrigation canal is known as super passage. But in this case the water passes below the drainage under siphonic action which is known as siphon superpassage. These structure is suitable when the bed level of drainage is below the fully supply level of canal.



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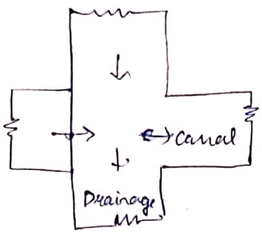
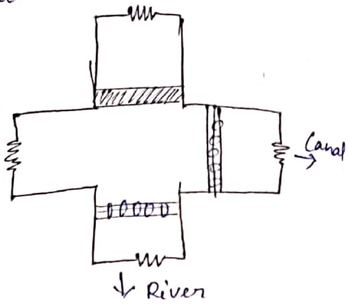
Type III :-

Drainage & canal intersect each other at same level. This condⁿ involves the construction of:

(a) level crossing.
(b). inlet & outlet.





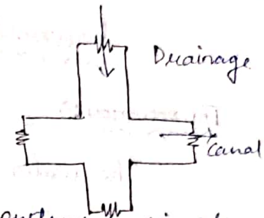
(a). Level Crossing :-

The level crossing is an arrangement provided to regulate the flow of water Through the drainage & the canal when they cross each other approximately at a same level.

In dry season when the discharge of the drainage is very low the drainage regulator is kept close & the canal water is allowed to flow as usual.

In rainy season when the discharge of the drainage is very high, the discharge regulator is kept completely open & the canal regulator is adjusted according the requirement. The level crossing is recomended for the crossing of main canal with large drainage.

(b). Inlet & Outlet :-



In the crossing of small drainage with small channel. no hydrolic structure is constructed, simple opening are provided. for the flow of water in their respective direction. This arrangement is known as inlet & outlet.

Selection of type of cross drainage work :-

→ Relative bed level:

According to the relative bed level of the canal and the river, the type of cross drainage work are generally selected. The

→ The point which should be remembered while constructing this type of work. the

(i) The crossing should be at right angle to each other

(ii) well define & cross section of river or drainage should be available

→ Available of suitable foundation:

For the construction of cross drainage work so suitable foundation is required. By boring test the selection of the foundation are generally ~~being~~ been tested.

→ Economic Consideration :-

The cost of construction of

Dam

An impervious high barrier which is constructed across a river valley to form a deep storage reservoir is known as dam.

The dam is meant for serving multipurpose functions such as, (a) irrigation, (b) Hydroelectric power generation, (c) Flood control, (d) water supply, (e) Fishery, (f) Recreation.

→ classification of dam :-

(a) Based on materials of construction

1) Rigid dam :- It is constructed with rigid materials like masonry, concrete, steel ~~and~~ or timber. designated as masonry dam, concrete dam, steel dam, timber dam.

2) Non Rigid dam :- It is constructed with non-rigid materials such as earth, clay, rock materials etc, It is designated as earthen dam, rock fill dam, composite dam.

(b) Based on structural behaviour

1) Solid gravity dam - constructed with masonry or concrete

(2) Arch dam :- It a curved masonry or concrete dam resists the forces acting on it by the principle of arch action.

3) Buttress dam :- It behaves like a retaining wall. It consists of sloping deck on U/S side supported by a number of buttresses in the form of triangular reinforced concrete wall or counterforts. It resists the forces by buttresses.

4) Embankment dam :-
It is non-rigid dam constructed simply by earthwork in trapezoidal section. Sometimes it may be of earthwork with clay core wall or rock fill. It resist forces acting on it by shear strength.

(c) Based on functions :-

1) Storage dam.

2) Detention dam -> to detain flood water temporarily in a reservoir & then released gradually so that the d/s area may not be damaged due to sudden flood.

3) Diversion dam :- To divert water from perennial river to a channel for the purpose of irrigation or hydroelectric power generation.

4) Coffer dam :- when an area in the river bed is enclosed temporarily by sheet piling for excluding water for construction of well foundation for pier foundn then it is known as coffer dam.

(d) Based on hydraulic behaviour

1) overflow dam :- It consists of crest shutters or waste weirs on the top to allow the surplus water to overflow.

2) Non overflow dam :- The dam in which spillways are provided to discharge the surplus water and the water is not allowed to flow over the crest.

Earth Dam

The earth dam are constructed purely by earth work in trapezoidal section it is the most economical hydraulic structure.

Types of Earth dam on the basis of construction:-

1). Rolled fill dam:-
2). Hydraulic fill dam
3). Semihydraulic fill dam
4). Homogeneous type dam
5). Zoned type dam.
6). Diaphragm type dam.

1). Rolled fill dam:-

In this method the dam is constructed in successive layer of earth by mechanical compaction. The selected soil is transported from borrow pit & laid on the dam section, to a layer of about 45 cm. The layer are throughly compacted by roller of recomended weight & type. when the compaction of one layer is fully achieved, the next layer is laid and compacted in the usual way. The design dam section is completed layer by layer.

2). Hydraulic fill dam:-

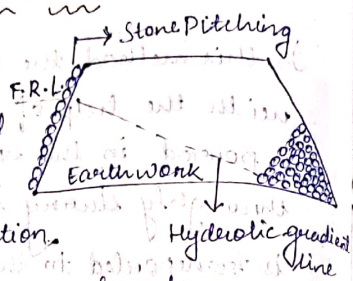
In this method the dam section is constructed with the help of water. Sufficient water is powred in the borrow pit & by plugging throughly slurry is formed. The slurry is transported to the dam side by pipeline and discharge near the upstream & downstream face of the dam. The course material gets deposited near the face & the finner material moves towards the centre and get deposited there. Thus the dam section is formed with face of course material & central core is of impervious material like clay & silt. In this case compaction is not necessary.

3). Semihydraulic fill dam:-

In this method the selected earth is transported from the borrow pit & dumped within the section of the dam. As done in the case of rolled filled dam, while dumping no water is used. But after dumping the water jet is forced on the dump earth. Due to the action of water. The finner material moves towards the centre and forms an impervious core. In this case compaction is not necessary.

4). Homogeneous type dam:-

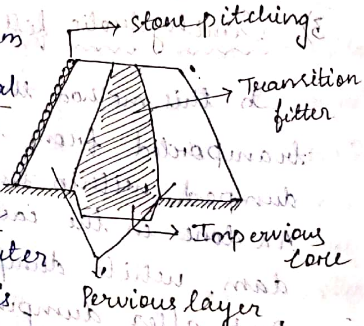
This type of dam is constructed purely with earth in trapizoidal section.



The top width & height depends on the depth of water to be retained & the gradient of the seepage line. The seepage line should pass well within the body of the dam. This type of dam is completely pervious. The upstream face of the dam is protected by stone pitching.

5). Zoned type dam:-

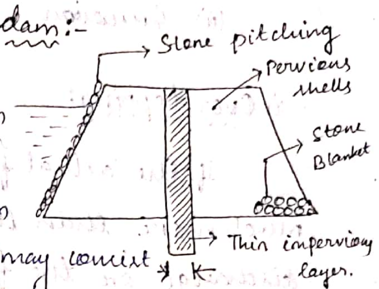
This type of dam consists of several material. The impervious core is made of puddle clay and the outer pervious shell is constructed with the mixture of earth, sand, gravel etc. The core is trapizoidal in section & its width ~~depth~~ depend



upon the seepage characteristics of the soil mixture on the upstream side.

The core is extended below the base of the dam to control the subsoil seepage.

6). Diaphragm type dam:-



In this type of dam a thin impervious core or diaphragm is provided which may consist of cement concrete or bitumenious concrete. The upstream & downstream body of the dam is constructed with pervious cell. which consist of the mixture of soil, sand gravel, etc. The thickness of the core is generally 3m or less than that. A blanket of stone is provided on the toe of the dam for the drainage of seepage water without damaging the core of the dam. The upstream face is ~~rotated~~ Protected by stone pitching.

Causes of failure in dam:-

The failure on the earth dam are caused due to various reason. It is categorised into three types:

1. Hydrolic failure.
2. Seepage failure.
3. Structural failure.

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Hydraulic Failure:-

It is of two types:

(i) Over topping.

(ii) Errosion.

(i) Over topping:

If the actual flood discharge is much more than the estimated flood discharge or the free board is kept insufficient. Then it results in the overtopping of the dam. During the overtopping the crest of the dam may be washed & the dam may totally collapse.

(ii) Errosion:

If the stone protection of on the up dam side is insufficient then the upstream face may be damaged by errosion due to wave action. The downstream side alone also may be damaged by tail water, rain water, etc. The toe of the dam may also get damage by the water flowing through the spillway.

Seepage failure:-

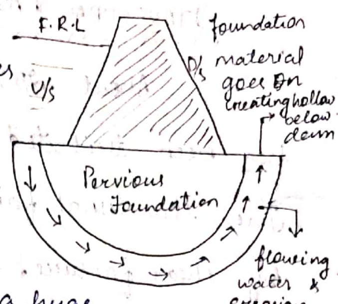
→ Pipeing through foundation.

→ Pipeing through the body of the dam.

The crumbling of the toe of the dam is called as sloughing

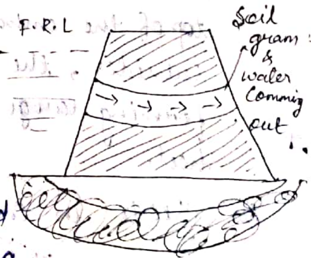
(i). Pipeing through the foundation:→

Some-time when highly permiable cavities are present in the foundation of the dam the water may start seeping at a huge rate through them. This concentrated flow at a high gradient may errode the soil. This leads to increase flow of water and soil ultimately resulting in a rush of water & soil there by creating a hollows, below the foundation. The dam may sink down into the hollow so from causing its failure.



(ii) Pipeing through the body of dam:→

when the conc. flow channel get developed in the body of the dam the soil may be removed in the same manner as was explained in the foundation piping. These flow channel may be develope due to faulty construction, insufficient compaction. crack development & in embankment, etc all these cause, are reason for piping



seepage through the dam body.

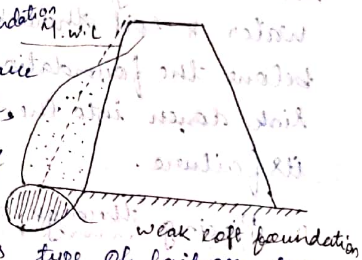
Structural Failure :-

About 25% of the dam failure has been tributed to structural failure. The structural failure are generally caused by shear failure, causing failure. So, the failure is of two types:

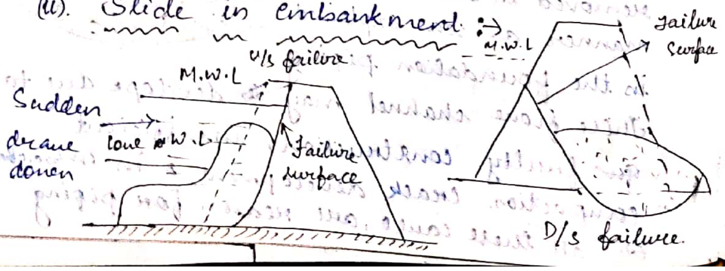
- Foundation slide
- Slide in embankment

(i) Foundation Slide :-

When the foundation of the earth dam are made of soft soil, the entire dam may slide over the foundation. In this type of failure the top of the embankment get crack & the subsides, the lower slope move outward forming large mud wave near the heel.



(ii). Slide in embankment :-



When the embankment slope are too steep for the strength of the soil. They may slide causing dam failure. The most critical condition of the slide of the upstream slope is the sudden drawdown of the reservoir. And the downstream slope is more likely to slide when the reservoir is full. The upstream slope failure leads to catastropic failure. But the downstream failure are very serious. This failure generally occur due to the development of excessive uncounted core pressure which may reduce the shearing strength.

Seepage Control In Earth Dam:

The seepage control are necessary to prevent adverse effect of water percolation through embankment & its foundation. The devices used for to control the seepage in earth dam are:

(i). Toe filter
(ii) Horizontal drainage filter.
(iii). Protective filter on the downstream of the ~~flow~~ toe

~~Chiminy drain extending upward into the embankment~~

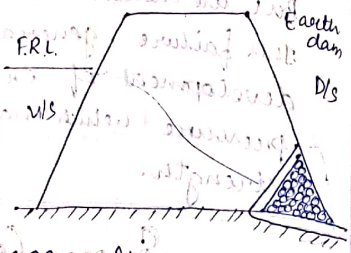
*Chiminy drain extending upward into the embankment

Foundation seepage control:

(i) upstream impervious blanket.
(ii) Downstream seepage berms
(iii) Relief walls.

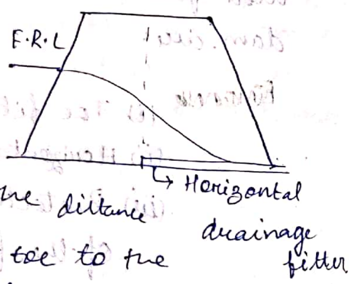
1) Toe filter :-

Here the rock toe keeps the seepage line well within the section to control the drainage. It's height is generally kept equal to 30-40% of the reservoir head.



2) Horizontal drainage filter:-

The horizontal drainage filter may extend from 25 to 100 percent of the distance from downstream toe to the centre line of the dam.



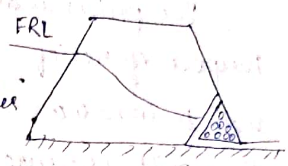
(i) It controlls the seepage line well within the embankment.

(ii) It gives greater leakage because of shorter seepage path.

It accelerate consolidation

3) Filter downstream of the toe :

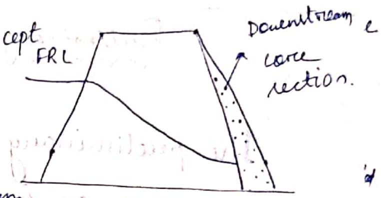
The poor provision of such filter provides addition safe & thus make the upward flow more safe.



* Downstream core section :-

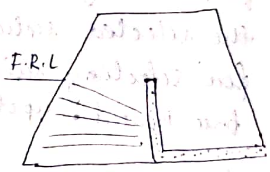
This also intercept flow through the embankment & make the downstream slope safe against piping.

It is also an earthquake resistance measure.



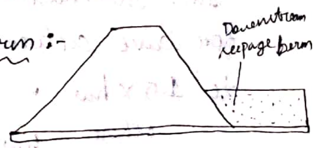
* Chimney drain:-

When there is a high degree of embankment the horizontal permeability is greater than the vertical. A correctly built vertical drain can completely intercept embankment seepage.



* Downstream seepage berm :-

Berm can be used to control seepage efficiently where the downstream top stratum



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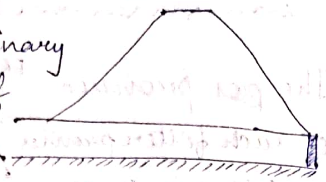
is relatively thin..

* Relief well:



The preliminary purpose of relief well is to reduce the uplift pressure in the downstream of the dam. They intercept the seepage and controll the outlet of seepage.

Preliminary Section of Earth Dam:

The preliminary section of earth dam may be done on the basis of existing dam of similar characteristic & design is finalised by checking the adequency of the selected section. A few recomendation for selecting suitable value are topwidth, free board, upstream & downstream slope.

Free board:-)

It is the vertical distance between the maxm reservoir level & the top of the dam. The maxm height of the freeboard for wave action is generally taken equal to 1.5 x hw.

where hw = height of water from the top of the creast to the bottom of the trough.

Top width:-

The top width of large earth dam should be sufficient to keep the seepage line with in the dam when the reservoir is full. It should also be sufficient to withstand earth-quake shoke.

For small dam the top width generally govern with minm roadway width requirement.

Upstream & Downstream slope:-

The side slope depend upon various factors such as the type of nature of the dam, foundation of material, height of the dam.

Type of material	upstream slope	Downstream slope
Homogeneous well graded soil	2.5 : 1	2 : 1
Homogeneous coarse silt	3 : 1	2.5 : 1
Homogeneous silty clay. (i) height < 50m	2.5 : 1	2 : 1
(ii). height > 50m	3 : 1	2.5 : 1

Module - III

GRAVITY DAM.

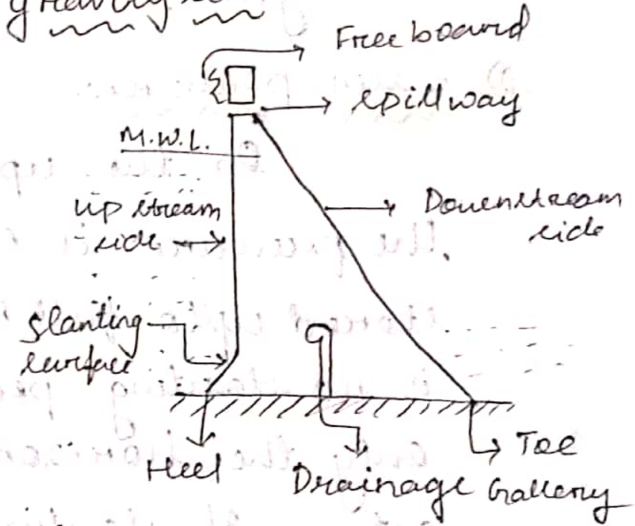
Defination :-

An impervious high barrier which is constructed across a river valley to form a deep storage reservious is called as dam.

Gravity Dam :-

It is a type of dam which can resist all the external forces acting on it through its-self weight.

Cross section of gravity dam :-



Forces acting on Gravity Dam :-

The forces that generally act on a gravity dam are

- weight of the dam.
- water pressure
- uplift pressure
- seismic force
- silt pressure.

- wave pressure
- Ice pressure
- wind pressure.

1). Self wt. of the dam :-)

The wt. of the dam is the main stabilising force which counter balance all the external force acting on it. So the dam should be constructed with specific scale wt. material. For the construction of the dam, the specific wt. of concrete & stone masonary should not be less than 2400 kg/m³ or 2300 kg/m³.

2) water pressure :-)

On the upstream face of the dam the pressure is exerted by the water stored upto full reservoir level. If there is no slanting provided in the dam then only the horizontal water pressure is acting. If slanting is provided then the horizontal & well as the vertical water pressure is acting.

3) uplift pressure :-)

The stored water on the upstream side of the dam has a tendency to seep through the soil below the foundation while seeping the flowing water exert uplift pressure on the base of the dam which depends upon the head of water.

4) Seismic force :-)

When the selected dam site comes under the seismic zone the effect of earthquake wave should be taken into account. The vertical & horizontal components of the earthquake wave are designed for a dam coming under the seismic zone.

5) Silt pressure :-)

The silt carried by the river gets deposited against the upstream base of the dam year after year. After considerable deposition of the silt it exert a pressure on the dam. So provision should be made to resist the silt pressure.

6). Wave pressure :-)

When very high wind or tornado flow over the water surface of the reservoir waves are formed which exert pressure on the upstream part of the dam. The magnitude of the wave depends upon the velocity of the wind.

7). Ice pressure :-)

This pressure should be counted only in places where the formation of ice is expected on the reservoir surface. When the sheet of ice is formed on the entire water surface of the reservoir, then it exert pressure on the dam at the point of contraction & expansion with the change in temperature.

8). Wind pressure :-

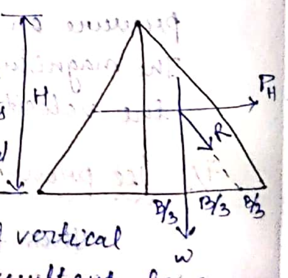
The top exposed portion of the dam is not much and the wind pressure on the surface area of this portion is negligible. But still an allowance should be made for the wind pressure at a rate of $150 \, kg/m^2$.

Causes of failure of gravity dam :-

The solid gravity dam may fail due to the following reasons :-

- By overturning
- By over stressing
- By cracking
- By sliding

(i) By Overturning :-



The solid gravity dam may fail overturning at its toe when the total horizontal force acting on the dam are greater than the total vertical force. In such case the resultant force passes through a point outside the middle third of the base of dam. For which overturning is being seen.

(ii). Over stressing :-

If the permissible working compressive stress of concrete or masonary exceed due to some adverse condition than the dam may fail by crushing due to the overstressing of concrete or masonary.

(iii). By cracking :-

The tensile stress should not be allowed to develop on the upstream face of the dam. If due to some reason the tension the tie is developed then the crack will be formed which will cause failures to the dam.

(iv). By Sliding :-

The total horizontal force acting on the dam tends to slide the entire dam at its base or along any horizontal section of the dam. The sliding may take place when the horizontal force acting on the dam are greater than the combined resistance.

Elementary and Theoritical Profile of Gravity Dam :-

Take moment about point D

$W \times B/3 = P \times H/3$

$W \times B = P \times H$

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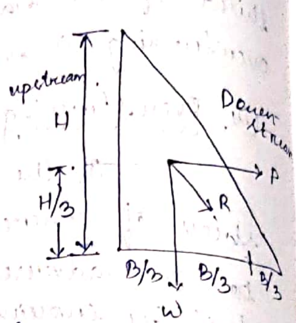
• Wave pressure ...

$$W/P = H/B$$

$$P = \frac{wH^2}{2}$$



$$W = \frac{1}{2} \times B \times H \times w \times \rho$$

$$\frac{W}{P} = \frac{\frac{1}{2} \times B \times H \times w \times \rho}{\frac{wH^2}{2}}$$

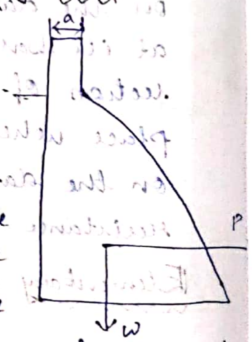
$$\frac{W}{P} = \frac{B \times \rho}{H}$$

$$\Rightarrow H/B = \frac{B \times \rho}{H}$$

$$\Rightarrow H^2 = B^2 \times \rho$$

$$B = \frac{H}{\sqrt{\rho}}$$

Practical Profile of Gravity Dam :-



In elementary profile we have seen that the maximum water level is just at the apex of dam. But in actual practice the water level may rise the maximum water level due to various reason. So some safe margin is provided at the top so that the water may not spill. This margin is known as free board. The amount of freeboard depends on intensity of wind, height of wave etc.

In normal practice 2-3 m free board is provided again some top width is necessary for providing roadway over the dam. The top width is given by

→ Top width (a) (Bligh empirical formula):

$$a = 0.552 \times \sqrt{H}$$

where,

a = top width

H = The maxm height of the reservoir

Thus the elementary profile is modified by providing free board & normal top width. This modified is known as practical profile of gravity dam:

Low & High Dam :-

Low Dam :-

A low dam is designed on the basis of elementary profile where the resultant force passes through the middle third of the base. The principal base is calculated from the elementary profile which is given as

$$\sigma = wH(\rho - C + 1)$$

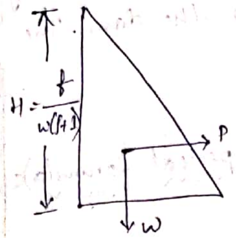
where, σ = principal stress

w = unit wt. of water (1000 kg/m³)

f = specific gravity of material

c = const.

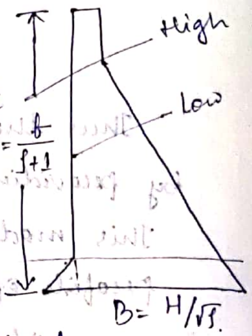
$f = wH(S - \frac{c+1}{c})$



$H = \frac{f}{w(S - \frac{c+1}{c})}$ f = Allowable working stress

$H = \frac{f}{w(S+1)}$

High Dam :-



The high dam is a complicated structure that the resultant force may pass through a point outside the middle third. In this case the section of the dam is modified by providing extra slope in the upstream and downstream when the condition is,

$$H_1 > \frac{f}{w(S+1)}$$

* To form a storage reservoir of required capacity a solid gravity dam of height 150m is to be constructed. Comment whether the dam will be designed as low dam or high dam. Taking permissible working stress as 40kg/cm² & specified gravity of material is 2.5.

$f = 40 \text{kg/cm}^2$ $w = 1000 \text{ kg/m}^2$

sp. gravity = 2.5

H = 150

$$H = \frac{40 \times 10^4}{1000 \times (2.5+1)}$$

$= 114.25$ m

* Find the maxm height of a low dam having the following data:

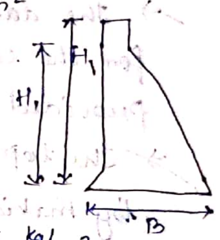
Cement concrete = 1:2:4

FOS = 4

sp. gravity of material = 2.4

Draw the section of the dam.

The ultimate compression strength of cement concrete is assumed 150 kg/cm²



$S = 2.4$

$w = 1000$

~~$a = 0.552 \times \sqrt{H}$~~

$f = \frac{150 \times 10^4}{4} = 37.5 \text{ kg/cm}^2$

$H_1 = \frac{f}{w(S+1)} = \frac{37.5 \times 10^4}{1000(2.4+1)}$

$= 110.29$ m

$B = H/\sqrt{S} = \frac{110.29}{2.4} = 69.9$

Height of water $= 110.2 - 2 = 108.29$

$a = 0.552 \times \sqrt{108.29}$

$= 5.74$ m

Spillway

The spillway are the opening provided at the body of the dam. to discharge safely the excess water or ~~xx~~ flood water when the water level rises above the normal pool level.

Necessity of Spillway :-

→ The height of the dam is always fixed according to the maximum reservoir capacity. The normal pool level indicates the maximum capacity of the reservoir.

→ The water is never stored in the reservoir above this level.

→ The dam may fail by overturning so for the safety of the dam spillway is provided.

→ The top of the dam is generally utilized by making road. The surplus water is not allowed ~~or~~ to overtop the dam. So, to stop the overtoping by ~~circular~~ surplus water, the spillway becomes extremely essential.

→ To protect the downstream ~~face~~ base & floor of the dam from the effect of scoring & errosion, the spillway are provided so that the water can pass smoothly.

Location of spillway :-

Generally the spillway are provided.

(i). The spillway may be provided with in the body of the dam.

(ii). Spillway may sometimes be provided at one side or bothsides of the dam.

(iii). Sometime bypass spillway, is provided which is completely seperated from the dam.

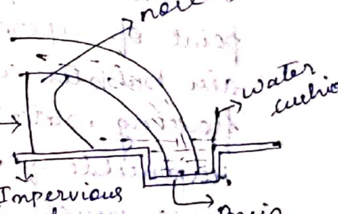
Types of Spillway :-

There are of different types.

(i). Drop spillway.
(ii). Ogee spillway.
(iii). Siphon spillway.
(iv). Chute or trough spillway.
(v). Shafte spillway.
(vi). Side channel spillway.

(i). Drop Spillway :-

In this spillway the overflowing water falls freely and almost vertically on the downstream side



This type of spillway is suitable for weir or low dam. The crest of the spillway is

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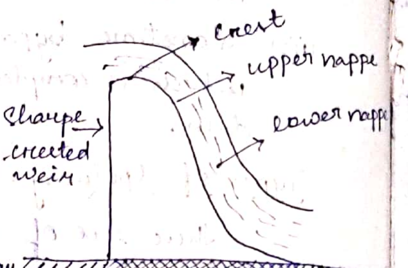
provided with a nose so that the water jet may not strike the downstream base of the structure. To protect the structure from the effect of erosion, a horizontal impervious apron is provided on the downstream side.

highdam → high impact forces → vibrations → found^n crack

(ii). Ogee Spillway :-



It is a modified form of sharpe crested weir drop spillway. Here the downstream profile of spillway is made to coincide with the shape of the lower nap of the free falling water jet from a sharpe crested weir.

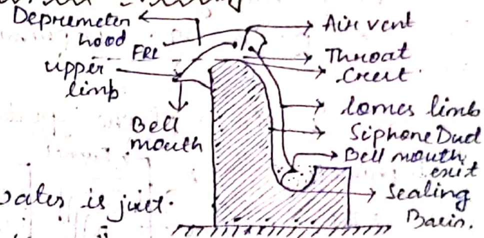
when the actual head become more than the design head. The lower nap doesnot provide the ogee profile & get seperated from the spillway surface. Thus a -ve pressure developed at the point of seperation. Due to the -ve pressure air bubbles are formed with in the flowing water. This air bubbles are responsible for frictional force which cause much damage to the spillway surface.

(iii). Siphon Spillway

The spillway which act on the principal of siphon is known as siphon spillway. It is of two types:

(i). Saddle siphon spillway
(ii) Volute siphon spillway.

1. Saddle siphon spillway :-



Function :-

→ when the water is just on the FRL (full reservoir level) there is no flow of water.

→ As the water starts rising above the FRL, the flow of water also starts over the crest.

→ when the inlet of the depremeter gets submerged the entry of air to the siphone Duct then the air ventilation stops

→ The air in the top portion of the siphone Duct is then sucked by the flowing pressure. Thus the inside pressure

Tunnel

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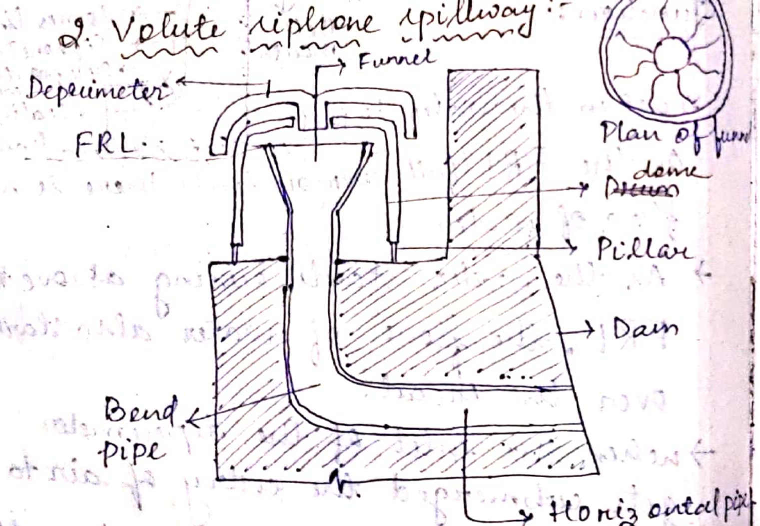
provided with a ...

is dropped below the outside atmospheric pressure. Due to this pressure difference a suction pull is created which draws more & more water over the crest.

-) Due to the gradual increase of suction pull a time comes when the siphon action starts & the siphone duct goes on running full. This phenomenon known as priming. Depriming - 532

2. Volute siphone spillway:-



It consists of a vertical shaft having a funnel at the top end & the bottom end is connected to the bend pipe. The bend pipe again is connected to a horizontal pipe which carries the flowing water away from the base of the dam. The top level of the funnel is just kept at the FRL. The funnel consists of several blades. Thus the water has a spiral motion while passing through the funnel. A circular dome is placed over the funnel where its bottom is completely open and the top end consists of a small opening which act like an air inlet. When the water rises above the FRL. Its speed over the circumference of the funnel & flow with a tip spiral motion through it. Thus a vertex is formed in the vertical shaft which induces a suction pull & a vaccume is created inside it. Thus the siphonic action is started & the shaft starts running full.

Tunnel

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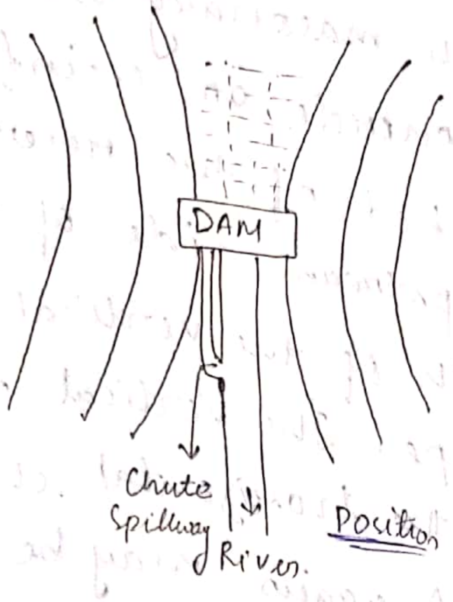
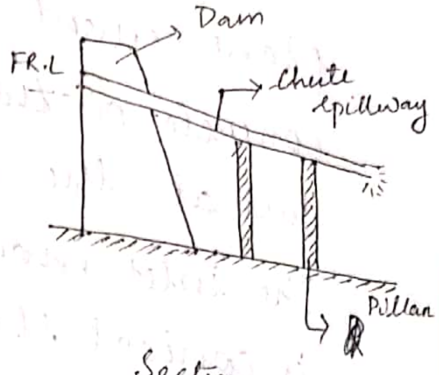
∴ completely

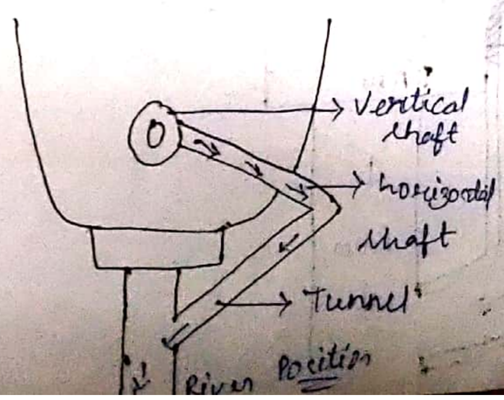
(iv). Chute & Trough Spillway:-

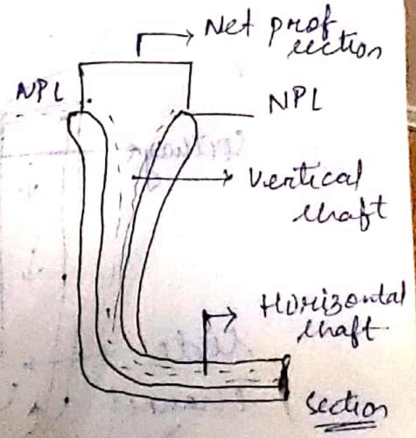




This spillway is simply a rectangular open channel provided on the dam to discharge the surplus water from the ~~reserv~~ reservoir to the same river on the downstream side. The chute is constructed by pre-cast RCC channel in a longitudinal slope of 1 in 4 or 1 in 6. The channel are supported on pillars. The section of the channel is designed according to the volume of surplus water. This spillway may be provided at one side or both side of the dam.

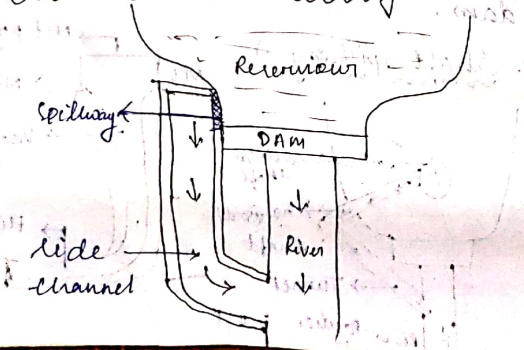
(v). Shaft spillway:-)





It consists of a vertical shaft which is constructed with masonary work or plane cement concrete or reinforce cement concrete on the bed of the reservoir just at the upstream side of the dam. The inlet mouth of the vertical shaft is conical shape. The vertical shaft is connected with the horizontal shaft. The horizontal shaft again may be taken to the body of the dam or may be connected to a tunnel outside the dam. The inlet mouth is kept at the NPL (normal pool level) of the reservoir. So, when the water level rises above the NPL it enters the shaft from all direction & flow out through it. In order to arrest the floating debries a net protection is provided on the inlet mouth.

(vi). Side channel spillway :-



The side channel spillway is completely seperate from the main body of the dam. The spillway is constructed at right angle to the dam and at any side according to the site condition. The crest of the spillway is kept at normal pool level of the reservoir. When the water rises above the NPL it spill over the crest of the spillway & flow through the side channel & ultimately meets the same river on the down stream side. This type of spillway is recomended for the site where other type of spillway are found unsuitable. The side wall of the channel may be constructed with brick or stone masonary. The longitudinal slope of the chanel depends upon the available space or men.

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