Fluid Mechanics

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Properties of Fluids

Fluid mechanics is that branch of science which deals with the behavior of the fluids (liquids or gases) at rest as well as in motion. Thus this branch of science deals with the static, kinematics and dynamic aspects of fluids. The study of fluids at rest is called fluid statics. The study of fluids in motion, where pressure forces are not considered, is called fluid kinematics and if the pressure forces are also considered for the fluids in motion, that branch of science is called fluid dynamics.

Density or Mass density

Density or mass density of a fluid is defined as the ratio of the mass of a fluid to its volume. Thus mass per unit volume of a fluid is called density. It is denoted the symbol ρ (rho), the unit mass density in SI unit is kg per cubic meter (kg/m^3).

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

The value of density of water is 1 g/cm^3 or 1000 kg/m^3

Specific gravity

Specific gravity is defined as the ratio of the weight density of a fluid to the weight density of a standard fluid. For liquids, the standard fluid is taken water and for gases, the standard fluid is taken air. Specific gravity is also called relative density. If the specific gravity of a fluid is known, then the density of the fluid will be equal to specific gravity multiplied by the density of water.

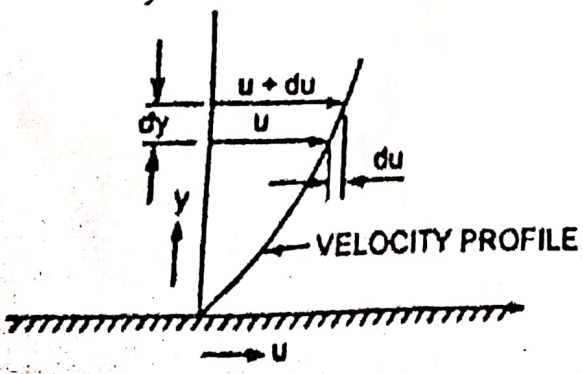
$$S = \frac{\text{weight density of liquid}}{\text{weight density of water}}$$

Viscosity

Viscosity is defined as the property of a fluid which offers resistance to the movement of one layer of fluid over another adjacent layer of the fluid. This shear stress is proportional to the rate of change of velocity with respect to y. It is denoted by symbol τ called Tau.

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

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



Velocity variation near a solid boundary.

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Where, μ (called mu) is the constant of proportionality and is known as the coefficient of dynamic viscosity or only viscosity. du/dy represents the rate of shear strain or rate of shear deformation or velocity gradient. Unit of viscosity is Ns/m^2 or Pa. s or poise.

Kinematic viscosity: It is defined as the ratio between the dynamic viscosity and density of fluid. It is denoted by the ν called 'nu'. Unit of kinematic viscosity is cm^2/s or stoke.

$$\nu = \frac{\text{Viscosity}}{\text{Density}} = \frac{\mu}{\rho}$$

Newton's Law of Viscosity: It states that the shear stress (t) on a fluid element layer is directly proportional to the rate of shear strain. The constant of proportionality is called the co-efficient of viscosity. Mathematically, it is expressed as given by equation,

$$\mu = \frac{\tau}{du/dy} = \frac{\text{Shear stress}}{\text{rate of shear strain}}$$

Fluids which obey the above relation are known as Newtonian fluids and the fluids which do not obey the above relation are called Non-Newtonian fluids.

Classification of Fluids

The fluids may be classified into the following five types:

1. Ideal fluid.
2. Real fluid.
3. Newtonian fluid.
4. Non-Newtonian fluid, and
5. Ideal plastic fluid.

1. Ideal Fluid: A fluid, which is incompressible and is having no viscosity, is known as an ideal fluid. Ideal fluid is only an imaginary fluid as all the fluids, which exist, have some viscosity.

2. Real Fluid: A fluid, which possesses viscosity, is known as real fluid. All the fluids, in actual practice, are real fluids.

3. Newtonian Fluid: A real fluid, in which the shear stress is directly, proportional to the rate of shear strain (or velocity gradient), is known as a Newtonian fluid.

4. Non-Newtonian Fluid: A real fluid, in which the shear stress is not proportional to the rate of shear strain (or velocity gradient), known as a Non-Newtonian fluid.

5. 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 (or velocity gradient), is known as ideal plastic fluid.

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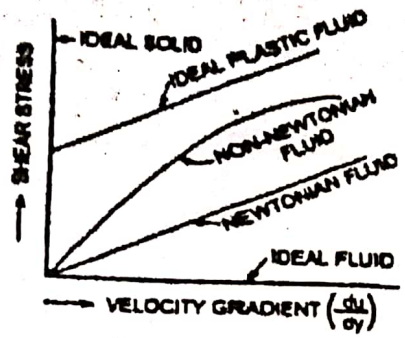

Fig. Types of fluids.

Question: A plate separated by 0.5 mm from a fixed plate moves at 0.50 m/s under a force per unit area of 4.0 N/m^2. Determine the viscosity of the fluid between the plates.

Solution:

Shear stress, $\tau = \mu \frac{du}{dy}$

Viscosity, $\mu = \tau \frac{dy}{du} = 4 \times \frac{0.0005}{0.5} = 0.004$ N. s/m^2 = 0.004 Pa. s

Question: If co-efficient of dynamic viscosity, μ = 1 N-sec/m^2 and velocity distribution, $v = 0.9 - y^2$. Determine the shear stress if y = 0.45. (BWDB – 2016)

Solution:

Shear stress, $\tau = \mu \frac{du}{dy} = \mu \frac{d}{dy}(0.9 - y^2)$

$\tau = \mu (0 - 2y) = -2 \mu y = -2 \times 1 \times 0.45 = -0.9$ N/m^2

Question: Determine the intensity of shear of an oil having viscosity = 1 poise, the oil is used for lubricating the clearance between a shaft of diameter 10 cm and its journal bearing. The clearance is 1.5 mm and the shaft rotates at 150 $r.p.m.$

Solution:

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

Dia of shaft, $D = 10$ cm $= 0.1$ m

$dy = 1.5$ mm $= 1.5 \times 10^{-3}$ m

Tangential speed of shaft, $u = \frac{\pi D N}{60} = \frac{\pi \times 0.1 \times 150}{60} = 0.785$ m/s

du = change of velocity between shaft and bearing $= u - 0 = u$

$\tau = \mu \frac{du}{dy} = \frac{1}{10} \times \frac{0.785}{1.5 \times 10^{-3}} = 52.33$ N/m^2

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Question: A flat plate of area $1.5 \times 10^6\ mm^2$ is pulled with a speed of 0.4 m/s relative to another plate located at a distance of 0.15 mm from it. Find the force and power required to maintain this speed, if the fluid separating then is having viscosity as 1 poise.

Solution:

Area of the plate, $A = 1.5 \times 10^6\ mm^2 = 1.5\ m^2$

Speed of plate relative to another plate, $du = 0.4$ m/s

Distance between the plates, $dy = 0.15$ mm $= 0.15 \times 10^{-3}$ m

Viscosity, $\mu = 1$ poise $= \frac{1}{10} \frac{Ns}{m^2}$

$$\tau = \mu \frac{du}{dy} = \frac{1}{10} \times \frac{0.4}{0.15 \times 10^{-3}} = 266.66\ N/m^2$$

Shear force, $F = \tau \times$ Area $= 266.66 \times 1.5 = 400$ N

Power required to move the plate at the speed 0.4 m/s $= F \times \mu = 400 \times 0.4 = 160$ W

Question: Find the kinematic viscosity of an oil having density 981 kg/m^2. The shear stress at a point in oil is 0.2452 N/m^2 and velocity gradient at that point is 0.2 per second.

Solution:

Shear stress, $\tau = \mu \frac{du}{dy}$

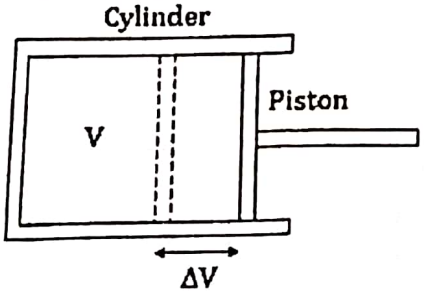
Viscosity, $\mu = \tau \frac{dy}{du} = 0.2452 \times 0.2 = 1.226\ N.s/m^2$

Kinematic viscosity, $\nu = \frac{\mu}{\rho} = \frac{1.226}{981} = 0.125 \times 10^{-2}\ m^2/s$

$\nu = 0.125 \times 10^{-2} \times 10^4\ cm^2/s = 12.5\ cm^2/s = 12.5$ stoke

Compressibility and bulk modulus

Compressibility is the reciprocal of the bulk modulus of elasticity, K which is defined as the ratio of compressive stress to volumetric strain.



$$\text{Bulk modulus, } K = \frac{\text{Increase of pressure}}{\text{Volumetric strain}} = \frac{dP}{-dV/V} = -\frac{dP}{dV} V$$

$$\text{Compressibility} = \frac{1}{K}$$

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Question: What is the bulk modulus of elasticity of a liquid which is compressed in a cylinder from a volume of 0.0125 m^3 at 80 N/cm^2 pressure to a volume of 0.0124 m^3 at 150 N/cm^2 pressure?

Solution:

Initial volume, $V = 0.0125\ m^3$

Final volume $= 0.0124\ m^3$

Decrease in volume, $dV = 0.0125 - 0.0124 = 0.0001\ m^3$

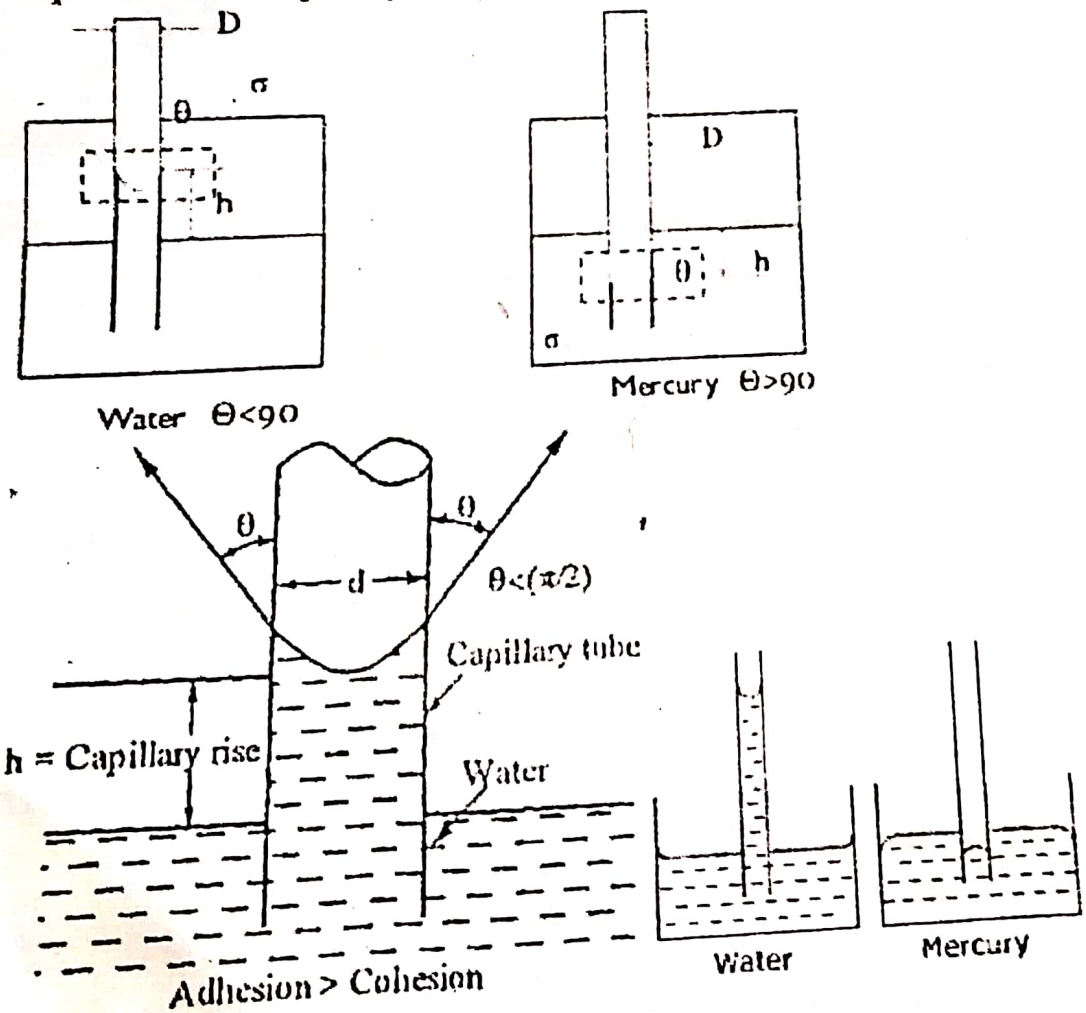
Increase in pressure, $dP = 150 - 80 = 70\ N/cm^2$

$$K = \frac{dP}{-dV/V} = \frac{70}{-0.0001/0.125} = 8.75 \times 10^3\ N/cm^2$$

Capillarity: Capillarity is defined as a phenomenon of rise or fall of a liquid surface in a small tube relative to the adjacent general level of liquid when the tube is held vertically in the liquid. Simply, it is the rise or fall of liquid in small diameter tube due to surface tension and adhesion between liquid and solid. The rise of liquid surface is known as capillary rise while the fall of the liquid surface is known as capillary depression.

Cohesion: It is the attraction force between the molecules of same material.
Adhesion: It is the attraction force between the molecules of different materials.

Expression for capillary rise /fall



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Pressure and its measurement

Manometer: An instrument for measuring the pressure acting on a column of fluid, consisting of a U-shaped tube of liquid in which a difference in the pressures acting in the two arms of the tube causes the liquid to reach different heights in the two arms.

Identify two points that have the same pressure, i.e., that are at the same elevation in the same liquid, such as points 2 and 3. Then we can write

$P_2 = P_3$

$P_1 + \gamma_{water}\, h = P_4 + \gamma_{Hg}\, H$

Since point 4 is shown to be open to the atmosphere, the pressure there is zero gage pressure: $P_4 = 0$. Thus, the manometer would measure the pressure P_1 in the pipe to be

$P_1 = \gamma_{Hg}\, H - \gamma_{water}\, h$

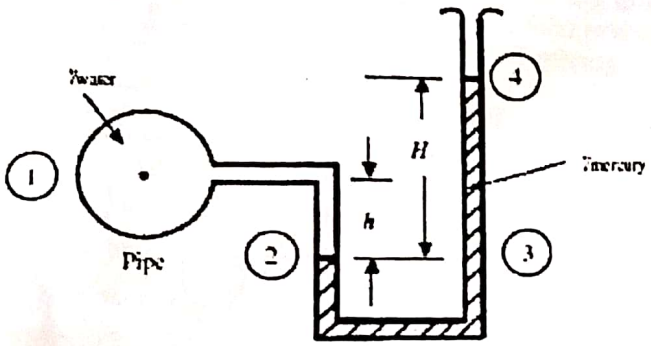
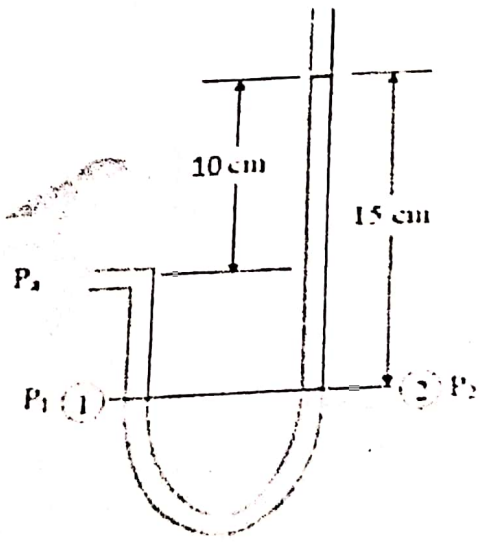


Figure: A U-tube manometer using water and mercury.

Question: What is manometer? A U-tube nanometer containing Hg (sp. gr. 13.6) has its right limb opened to the atmosphere as shown in the figure. The left limb is full of water and is connected to a pipe containing water under pressure. Find the pressure of water in the pipe above atmosphere for the manometer reading as shown in figure. (BWDB – 2019)

Solution:



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The curved surface that develops in tube is called meniscus.

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

The value of θ between water and clean glass tube is approximately equal to zero & $\cos 0^0 = 1$

Hence, the rise of water, $h = \frac{4\sigma}{\rho g d}$

h = height of liquid in the tube
σ = Surface tension of liquid
d = diameter of the tube
ρ = density of liquid
θ = Angle of contact between liquid and glass tube
Value of θ for mercury and glass tube is 128^0

Question: Calculate the capillary rise in a glass tube of 2.5 mm diameter when immersed vertically (a) water (b) mercury. Take surface tension $\sigma = 0.0725$ N/m for water and $\sigma = 0.52$ N/m for mercury in contact with air. The specific gravity for mercury is given as 13.6 and angle of contact = 130^0.

Solution:

Sp. gr. of mercury = 13.6

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

Capillary rise for water ($\theta = 0$)

The rise of water, $h = \frac{4\sigma}{\rho g d} = \frac{4 \times 0.0725}{1000 \times 9.81 \times 25 \times 10^{-3}} = 0.0118\text{m} = 1.18\text{ cm}$

For mercury ($\theta = 130^0$)

$h = \frac{4\sigma \cos\theta}{\rho g d} = \frac{4 \times 0.52 \times \cos 130^0}{13.6 \times 1000 \times 9.81 \times 2.5 \times 10^{-3}} = -0.004\text{ m} = -0.4\text{ cm}$

The negative sign indicates the capillary depression.

Question: Find out the minimum size of glass tube that can be used to measure water level if the capillary rise in the tube is to be restricted to 2 mm. Consider surface tension of water in contact with air as 0.073575 N/m.

Solution:

Capillary rise, h = 2 mm = 2×10^{-3} m

Let the diameter of tube = d

The rise of water, $h = \frac{4\sigma}{\rho g d}$

$d = \frac{4\sigma}{\rho g h} = \frac{4 \times 0.073575}{1000 \times 9.81 \times 2 \times 10^{-3}} = 0.015\text{ m} = 1.5\text{ cm}$

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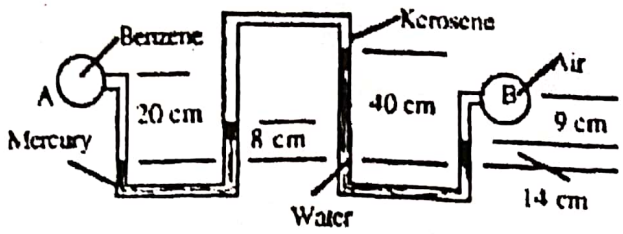
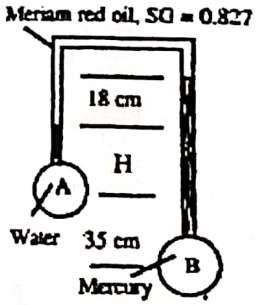


Solution:

$P_A + 8640 \times 0.2 - 133100 \times 0.08 - 7885 \times 0.32 + 9790 \times 0.26 - 12 \times 0.09 = P_B$

$P_A - P_B = 8900$ Pa

Question: For the manometer of figure, all fluids are at 20°C. If $P_A - P_B = 97$ kPa, determine the height H in centimeters. Take the specific weights of mercury 133100 N/m^3 and water 9790 N/m^3.



Solution:

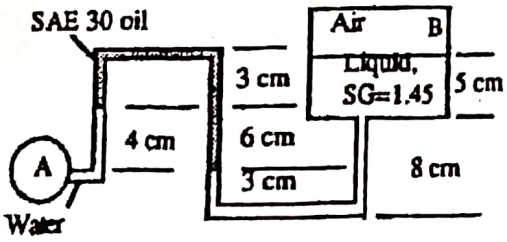
Specific weight of red oil $= 0.827 \times 9790 = 8096$ N/m^3

$P_A - 9790 \times H - 8096 \times 0.18 + 133100\,(0.18 + H + 0.35) = P_B$

$P_A - 9790\,H - 1457.28 + 70543 + 133100\,H = P_A + 9700$

$H = 0.226$ m $= 22.6$ cm

Question: In figure the pressure at point A is 25 psi. All fluids are at 20°C. What is the air pressure in the closed chamber B? Take $\gamma = 9810$ N/m^3 for water, 8720 N/m^3 for SAE 30 oil.



Solution:

Specific weight of liquid $= 1.45 \times 8720 = 14196$ N/m^3

$$P_A = 25 \text{ psi} = \frac{25 \times 39.37 \times 39.37 \times 9.81}{2.204} = 172475.83 \text{ N/m}^2$$

$172475.83 - 9810 \times 0.04 + 8720 \times 0.06 - 14196 \times 0.1 = P_B$

$P_B = 171187$ N/m^2

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Equating the pressure at 1 – 2

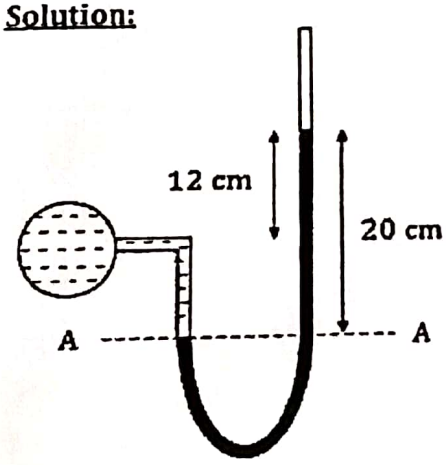
$P_a + \rho_1 g h_1 = \rho_2 g h_2$

$P_a + 1000 \times 9.81 \times 0.05 = 13.6 \times 1000 \times 9.81 \times 0.15$

$P_a = 20012.4 - 490.5 = 19521.90$ N/m² of water

Question: The right limb of a simple U-tube manometer containing mercury is open to the atmosphere while the left limb is connected to a pipe in which a fluid of sp. Gr. 0.9 is flowing. The centre of the pipe is 12 cm below the level of mercury in the right limp. Find the pressure of fluid in the pipe if the difference of mercury level in the two limbs is 20 cm.

Solution:



Sp. gr. of fluid, $S_1 = 0.9$

Density of fluid, $\rho_1 = S_1 \times 1000 = 0.9 \times 1000 = 900$ kg/m³

Sp. gr. of mercury, $S_2 = 13.6$

Density of mercury, $\rho_2 = S_2 \times 1000 = 13.6 \times 1000 = 13600$ kg/m³

Difference of mercury level, $h_2 = 20$ cm $= 0.2$ m

Height of fluid from $A - A$, $h_1 = 20 - 12 = 8$ cm $= 0.08$ m

Let, $P =$ Pressure of fluid in pipe

Equating the pressure above $A - A$,

$P + \rho_1 g h_1 = \rho_2 g h_2$

$P + 900 \times 9.81 \times 0.08 = 13600 \times 9.81 \times 0.2$

$P = 25977$ N/m² $= 2.5977$ N/cm²

Question: Determine Δp between points A and B. All fluids are at 20°C. Take the specific weights of Benzene 8640 N/m³, Mercury 133100 N/m³, kerosene 7885 N/m³, water 9790 N/m³ and air 12 N/m³.

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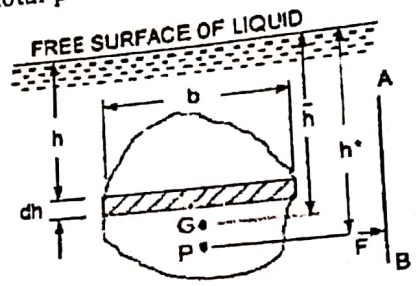
Hydrostatic forces on surfaces

Total pressure 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 the surface.

Centre of pressure is defined as the point of application of the total pressure on the surface. There are four cases of submerged surfaces on which the total pressure force and centre of pressure is to be determined.



- Horizontal plane surfaces.

Total pressure, $F = \gamma A \bar{h} = \rho g A \bar{h}$

Here, $\bar{h}$ = Distance of C.G. from the free surface.

- Vertical plane surfaces.

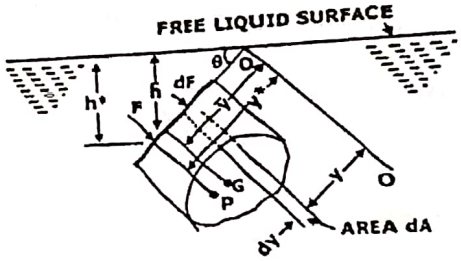
Total pressure, $F = \gamma A \bar{h} = \rho g A \bar{h}$

Center of pressure for vertical plane surfaces, $h^* = \frac{I_G}{A \bar{h}} + \bar{h}$

- Inclined plane surfaces.

Total pressure, $F = \gamma A \bar{h} = \rho g A \bar{h}$

Center of pressure for inclined plane surfaces, $h^* = \frac{I_G \sin\theta^2}{A \bar{h}} + \bar{h}$



- Curved surfaces.

The total force on the curved surfaces, $R = \sqrt{{F_H}^2 + {F_V}^2}$

The direction of the resultant force on the curved surface with the horizontal,

$$tan\,\theta = \frac{F_V}{F_H} \text{ or } \theta = tan^{-1}\left(\frac{F_V}{F_H}\right)$$

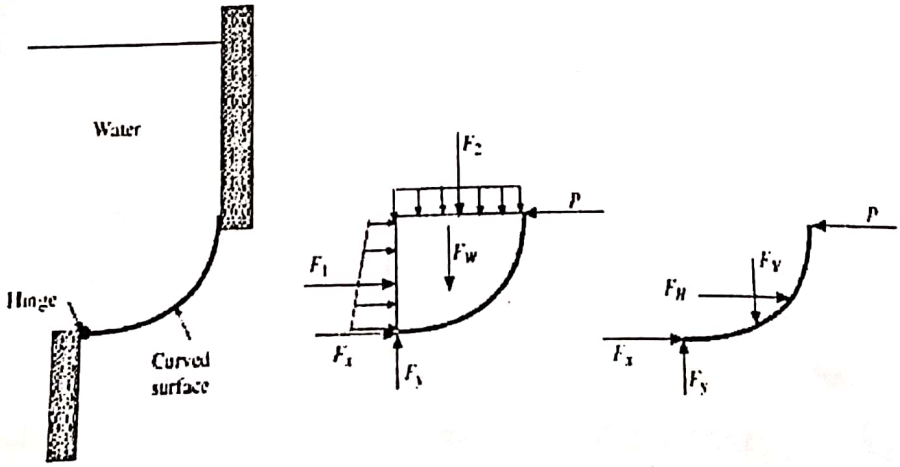


Figure: Forces on a curved surface:

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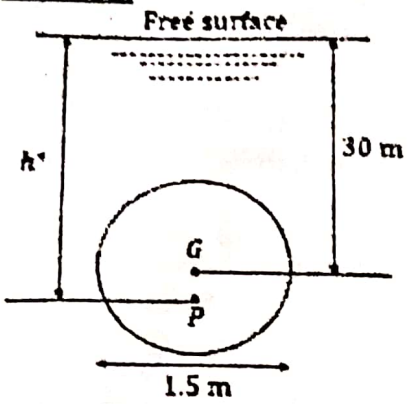
Question: Determine the total pressure on a circular plate of diameter 1.5 m which is placed vertically in water in such a way that the centre of the plate is 3 m below the free surface of water. Find the position of centre of pressure also.

Solution:



Area, $A = \frac{\pi d^2}{4} = \frac{\pi \times 1.5^2}{4} = 1.767 \text{ m}^2$

Total pressure, $F = \rho g A \bar{h} = 1000 \times 9.81 \times 1.767 \times 3 = 52002.81 \text{ N}$

Moment of inertia, $I_G = \frac{\pi d^4}{64} = \frac{\pi \times 1.5^4}{64} = 0.2485 \text{ m}^4$

Position of centre of pressure, $h^* = \frac{I_G}{A\bar{h}} + \bar{h} = \frac{0.2485}{1.767 \times 3} + 3 = 3.0468 \text{ m}$

Question: Find the resultant force on one side of a 25 mm diameter vertical circular plate standing at the bottom of 3 m pool water? (Density of water = 1000 kg/m^3, g = 9.8 m/s^2) (SGDL – 2017)

Solution:

Here, $\bar{h} = 3 - \frac{25}{2 \times 1000} = 2.9875 \text{ m}$

$$A = \frac{\pi d^2}{4} = \frac{\pi}{4}\left(\frac{25}{1000}\right)^2 = 4.9 \times 10^{-4} \text{ m}^2$$

$$P = \bar{h} \rho g = 2.9875 \times 1000 \times 9.81 = 29307.375 \text{ N/m}^2$$

$$F = P A = 29307.375 \times 4.9 \times 10^{-4} = 14.36 \text{ N}$$

Question: 25 mm diameter circular disk is on the base of a swimming pool of depth 2.5 m, width 3 m. Find the force on the circular disk. g = 9.8 m/s^2. (BHP – 2017)

Solution:

Here, $\bar{h} = 2.5 - \frac{25}{2 \times 1000} = 2.4875 \text{ m}$

$$A = \frac{\pi d^2}{4} = \frac{\pi}{4}\left(\frac{25}{1000}\right)^2 = 4.9 \times 10^{-4} \text{ m}^2$$

$$P = \bar{h} \rho g = 2.4875 \times 1000 \times 9.81 = 24402.375 \text{ N/m}^2$$

$$F = P A = 24402.375 \times 4.9 \times 10^{-4} = 11.96 \text{ N}$$

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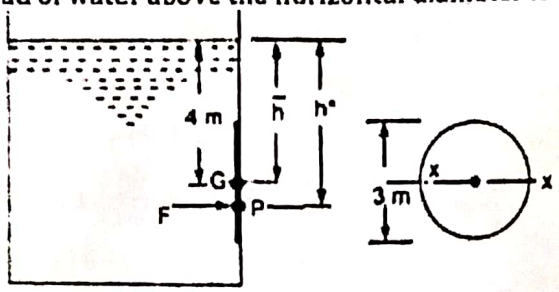
Question: A circular opening, 3 m diameter in a vertical side of a tank is closed by a disc of 3 m diameter which can rotate about a horizontal diameter. Calculate (i) the force on the disc and (ii) the torque required to maintain the disc in equilibrium in the vertical position when the head of water above the horizontal diameter is 4 m.



Solution:

Area, $A = \frac{\pi d^2}{4} = \frac{\pi \times 3^2}{4} = 7.0685 \text{ m}^2$

Location of CG, $\bar{h} = 4$ m

Force on the disc, $F = \gamma_w \bar{h} A = 9810 \times 4 \times 7.0685 = 277368$ N

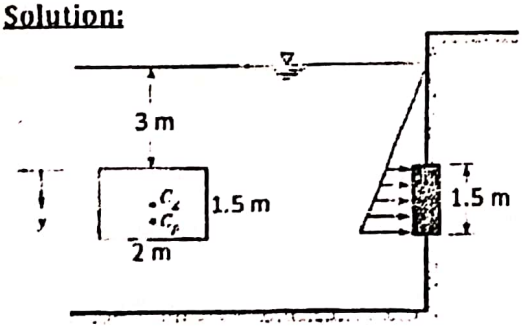
Moment of inertia, $I_G = \frac{\pi d^4}{64} = \frac{\pi \times 3^4}{64} = 3.9760 \text{ m}^4$

Position of centre of pressure, $h^* = \frac{I_G}{A\bar{h}} + \bar{h} = \frac{3.9760}{7.0685 \times 4} + 4 = 4.140$ m from free surface

Moment about horizontal diameter $X - X = F \times (h^* - \bar{h}) = 277368 \times 0.14 = 38831 \text{ N} - \text{m}$

Question: A rectangular gate 2 m wide and 1.5 m high is in a vertical plane. The gate is immersed vertically downwards, the top side being at a depth of 3 m below the free surface. Find the force exerted by the water on the gate and the position of center of pressure. (BB AD – 2018)

Solution:



Area, $A = 2 \times 1.5 = 3 \text{ m}^2$

Location of CG, $\bar{y} = 3 + \frac{1.5}{2} = 3.75$ m

Resultant force on gate, $F = \gamma A \bar{y} = 9.81 \times 3 \times 3.75 = 110.3625$ KN

Moment of inertia about CG, $I_G = \frac{b h^3}{12} = \frac{2 \times 1.5^3}{12} = 0.5625 \text{ m}^4$

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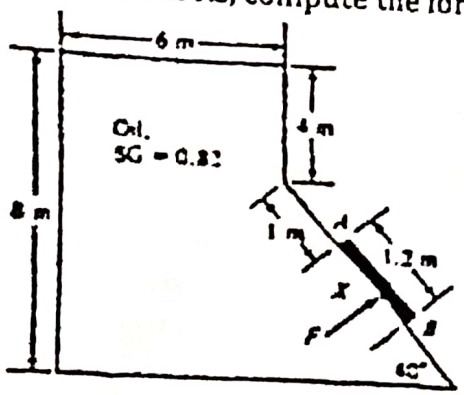
Centre of pressure, $h^* = \bar{y} + \frac{I_G}{A\bar{y}} = 3.75 + \frac{0.5625}{3 \times 3.75} = 3.8$ m

Question: Gate AB in figure is 1.2 m long and 0.8 m into the paper. Neglecting atmospheric-pressure effects, compute the force F on the gate and its center of pressure position *X*.



Solution:

Area, $A = 1.2 \times 0.8 = 0.96$ m²

Location of CG, $h_{CG} = 4 + (1 + 0.6)\sin 40^\circ = 5.028$ m

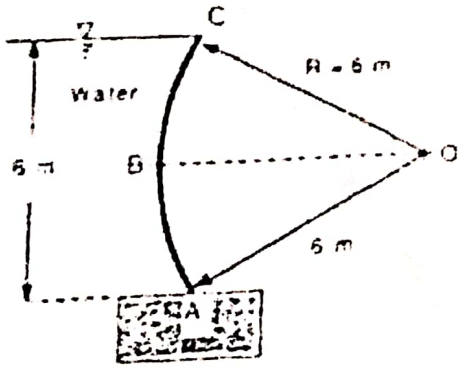
Force on gate, $F = \gamma_{oil}\, h_{CG}\, A = (0.82 \times 9790) \times 5.028 \times 0.98 = 38750$ N

Moment of inertia about CG, $I_G = \frac{b\,h^3}{12} = \frac{0.8 \times 1.2^3}{12} = 0.1152$ m⁴

Centre of pressure from free surface, $h_{CP} = \frac{I_G \sin^2\theta}{h_{CG}\, A} + h_{CG}$

$$h_{CP} = \frac{0.1152 \times \sin 40^2}{5.028 \times 0.96} + 5.028 = 5.037 \text{ m}$$

Question: Circular-arc tainter gate ABC pivots about point O. For the position shown, determine (a) the hydrostatic force on the gate (per meter of width into the paper) and (b) The line of action of resultant force (BEPZA – 2016)



Solution:

The horizontal hydrostatic force is based on vertical projection:

$F_H = \gamma\, h_{CG}\, A_{vert} = 9790 \times 3 \times (6 \times 1) = 176220$ N

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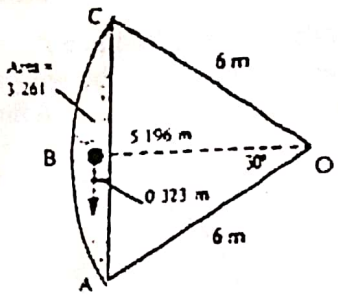
The vertical (upward) hydrostatic force on gate ABC:

F_V = Weight of imaginary water of $ACBA$ = γ x width of gate x A_{ABC}

F_V = γ x width of gate x (Volume of OABC – Volume of OAC)

$$= 9810 \times 1 \times \left[\frac{60}{360} \times \pi \times 6^2 - \frac{1}{2} \times 6 \times 6 \cos 30^0\right]$$

$$= 9810 \times (18.84 - 15.58) = 31980.6 \text{ N}$$

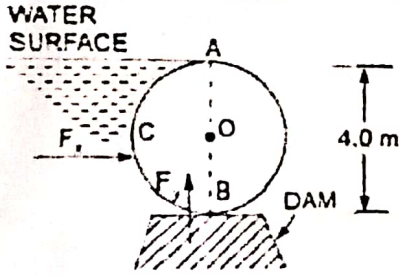


Resultant force, $R = \sqrt{F_H^2 + F_V^2} = \sqrt{176220^2 + 31980.6^2} = 179098.42$ N

Resultant passes through point O and acts at an angle of action:

$$\theta = \tan^{-1}\left(\frac{31980.6}{176220}\right) = 10.28^0$$

Question: Find the magnitude and direction of the resultant force due to water acting on a roller gate of cylindrical form of 4.0 m diameter, when the gate is placed on the dam in such a way that water is just going to spill. Take the length of the gate as 8 m.



Solution:

Area, $A = 4 \times 8 = 32$ m^2

Location of CG, $h_{CG} = 2$ m

Horizontal force on gate, $F = \gamma_{water}\, h_{CG}\, A = 9810 \times 2 \times 32 = 627840$ N

Vertical force on gate, F_V = Weight of imaginary water of the curved surface ACB

$$F_V = \gamma \text{ x width of gate x } A_{ACB} = 9810 \times 8 \times \frac{\pi \times 2^2}{2} = 493104 \text{ N}$$

Resultant force, $R = \sqrt{F_H^2 + F_V^2} = \sqrt{627840^2 + 493104^2} = 798328$ N

Direction of resultant force, $\theta = \tan^{-1}\left(\frac{493104}{627840}\right) = 31^0\ 8'$

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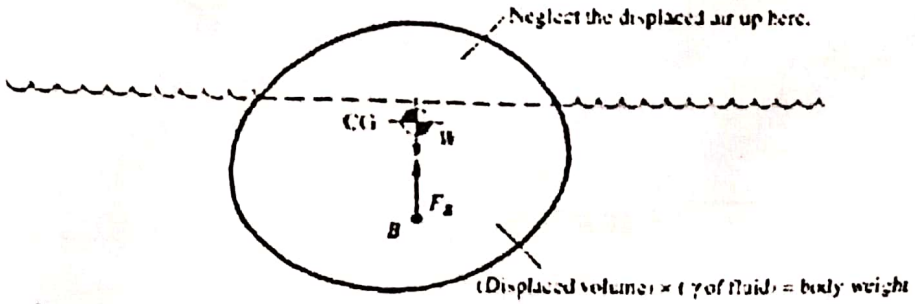
Buoyancy and floatation

When a body in immersed in a fluid, an upward force is exerted by the fluid on the body. This upward force is equal to the weight of the fluid displaced by the body is called the force of buoyancy or simply buoyancy. A floating body displaces its own weight in the fluid in which it floats.



In the case of a floating body, only a portion of the body is submerged, thus:
$F_B = \gamma$ (displaced volume) = weight of the floating body

Question: Find the volume of the water displaced and position of centre of buoyancy for a wooden block of width 2.5 m and of depth 1.5 m. when it floats horizontally in water. The density of wooden block is 650 kg/m^3 and its length 6.0 m.

Solution:

Volume of the block = 2.5 x 1.5 x 6 = 22.50 m^3

Weight of block = ρ x g x volume = 650 x 9.81 x 22.50 N = 143471 N

For equilibrium the weight of water displaced = Weight of wooden block = 143471 N

$$\text{Volume of water displaced} = \frac{\text{weight of water displaced}}{\text{weight density of water}} = \frac{143471}{1000 \times 9.81} = 14.625 \text{ m}^3$$

Position of centre of buoyancy:

Volume of wooden block in water = Volume of water displaced

$2.5 \times h \times 6.0 = 14.625$ Where h is depth of wooden block in water

$$h = \frac{14.625}{2.5 \times 6.0} = 0.975 \text{ m}$$

$$\text{Centre of buoyancy} = \frac{0.975}{2} = 0.4875 \text{ m from base}$$

Question: A wood of 5 kg weight flows in water; if the wood contain 60% of its volume at the time of floating then G_s=? (BWDB – 2018)

Solution:

For equilibrium, weight of wooden block = weight of water displaced

$\gamma_{wood} \times V_{wood} = \gamma_{water} \times V_{water}$

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$$\rho_{wood} \times V_{wood} \times g = \frac{60}{100} \times V_{wood} \times 1000$$

$$\rho_{wood} = 600 \text{ kg/m}^3$$

$$G_s = \frac{\gamma_{wood}}{\gamma_{water}} = \frac{600}{1000} = 0.6$$

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Dynamics of fluid flow

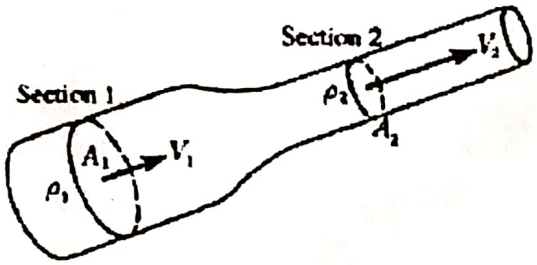
Continuity equation: If steady flow exists in a channel and the principle of conservation of mass is applied to the system, there exists a continuity of flow, defined as: "The mean velocities at all cross sections having equal areas are then equal, and if the areas are not equal, the velocities are inversely proportional to the areas of the respective cross sections." Thus if the flow is constant in a reach of channel the product of the area and velocity will be the same for any two cross sections within that reach.

$Q = A_1 V_1 = A_2 V_2$

Where: Q = the volumetric flow rate

A = the cross sectional area of flow

V = the mean velocity



Question: If the velocity in a 12 in pipe is 1.65 ft/s, what is the velocity in a 3 in diameter jet issuing from attached to the pipe?

Solution:

$$A_1 = \frac{\pi}{4} \times \left(\frac{12}{12}\right)^2 = 0.7854 \text{ ft}^2$$

$$A_2 = \frac{\pi}{4} \times \left(\frac{3}{12}\right)^2 = 0.049 \text{ ft}^2$$

We know, $A_1 V_1 = A_2 V_2$

$0.7854 \times 1.65 = 0.049 \times V_2$

$V_2 = 26.44$ ft/s

Bernoulli (Energy) Equation for steady incompressible flow

For a non-viscous, incompressible fluid in steady flow, the sum of pressure, potential and kinetic energies per unit volume is constant at any point.

$$Z_1 + \frac{P_1}{\gamma} + \frac{V_1^2}{2g} = Z_2 + \frac{P_2}{\gamma} + \frac{V_2^2}{2g} + \text{Head loss}$$

Energy and Hydraulic Grade Lines

The energy grade line (EGL) and the hydraulic grade line (HGL) provide a graphical interpretation of Bernoulli's equation. The EGL represents the total head available with respect to a chosen datum (i.e., a reference line, as shown in the figure).

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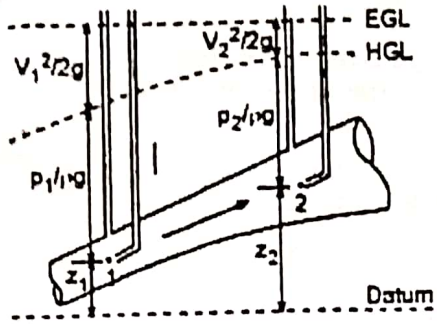
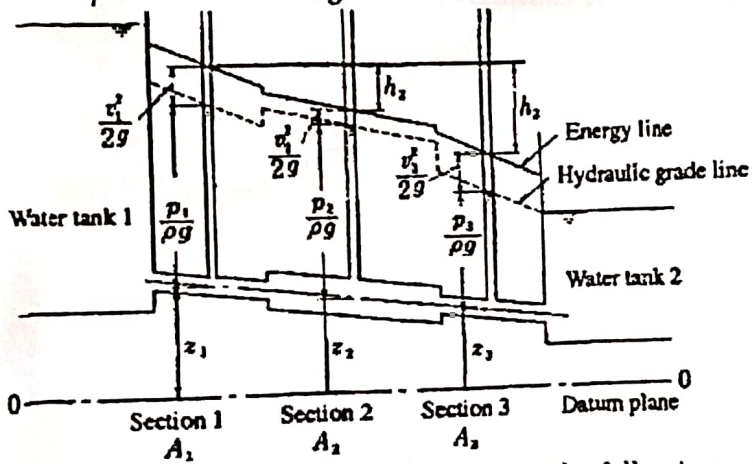


Energy and Hydraulic Grade Lines

$$EGL = \frac{P}{\gamma} + \frac{V^2}{2g} + Z$$

The EGL is a constant for frictionless flow where no work or heat is associated with the process. On the other hand, the HGL is the sum of static pressure and elevation head.

$$HGL = \frac{P}{\gamma} + Z = EGL - \frac{V^2}{2g}$$



Question: Write Bernoulli's equation for the following condition-[Considering two points on a pipeline] (a) With friction and horizontal (b) Without friction and horizontal (c) With friction and inclined. (50 BMA)

Solution:

a) With friction and horizontal=

$$P_1 + \frac{1}{2}\rho V_1^2 = P_2 + \frac{1}{2}\rho V_2^2 + H_f$$

b) Without friction and horizontal

$$P_1 + \frac{1}{2}\rho V_1^2 = P_2 + \frac{1}{2}\rho V_2^2$$

c) With friction and inclined.

$$\frac{P_1}{\rho g} + \frac{V_1^2}{2g} + h_1 + H_{pump} = \frac{P_2}{\rho g} + \frac{V_2^2}{2g} + h_2 + H_{pump}$$

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Question: Oil with specific gravity 0.750 is flowing through a 6 in pipe under a pressure of 15 psi. If the total energy relative to a datum plane 8.00 ft below the center of the pipe is 58.6 ft.lb/lb, determine the flow rate of the oil.

Solution:

$$A = \frac{\pi}{4} \times \left(\frac{6}{12}\right)^2 = 0.196 \text{ ft}^2$$

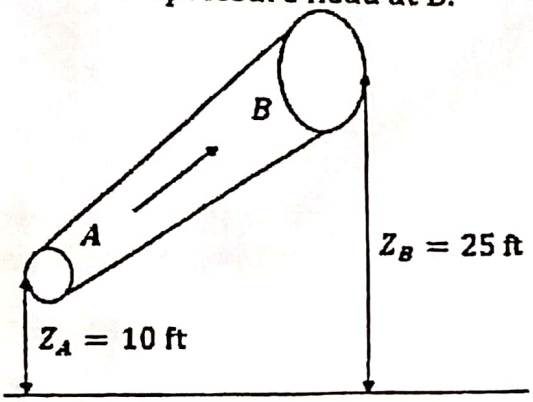
We know, $H = Z + \frac{P}{\gamma} + \frac{V^2}{2g}$

$$58.6 = 8 + \frac{15 \times 144}{0.750 \times 62.4} + \frac{V^2}{2 \times 32.2}$$

$$V = 19.62 \text{ ft/s}$$

$$Q = AV = 0.196 \times 16.92 = 3.32 \text{ ft}^3/\text{s}$$

Question: Water flows from A, where the diameter is 12 in, to B, where the diameter in 24 in, at the rate of 13.2 cfs. The pressure head at A is 22.1 ft. considering no loss of energy from A to B, find the pressure head at B.



Solution:

$$A_A = \frac{\pi}{4} \times \left(\frac{12}{12}\right)^2 = 0.7854 \text{ ft}^2$$

$$A_B = \frac{\pi}{4} \times \left(\frac{24}{12}\right)^2 = 3.1416 \text{ ft}^2$$

$$V_A = \frac{Q}{A_A} = \frac{13.2}{0.785} = 16.81 \text{ ft/s}$$

$$V_B = \frac{Q}{A_B} = \frac{13.2}{3.1416} = 4.202 \text{ ft/s}$$

$$Z_A + \frac{P_A}{\gamma} + \frac{V_A^2}{2g} = Z_B + \frac{P_B}{\gamma} + \frac{V_B^2}{2g}$$

$$10 + 22.1 + \frac{16.81^2}{2 \times 32.2} = 25 + \frac{P_B}{\gamma} + \frac{4.202^2}{2 \times 32.2}$$

$$\frac{P_B}{\gamma} = 11.2 \text{ ft}$$

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Question: In a horizontal pipe of 30 cm diameter, water flows with a velocity 2 m/s and the pressure is 52 KN/m². If the diameter of the pipe is 60 cm in the other end, what is the pressure? (BWDB – 2019)

Solution:

Here, $A_1 = \frac{\pi \times 0.3^2}{4} = 0.0706\ m^2$

$A_2 = \frac{\pi \times 0.6^2}{4} = 0.2827\ m^2$

We kmow, $A_1 V_1 = A_2 V_2$

$A_2 = \frac{0.0706 \times 2}{0.2827} = 0.5\ m/s$

Now, $\frac{P_1}{\gamma} + \frac{V_1^2}{2g} = \frac{P_2}{\gamma} + \frac{V_2^2}{2g}$

$\frac{52}{9.81} + \frac{2^2}{2 \times 9.81} = \frac{P_2}{9.81} + \frac{0.5^2}{2 \times 9.81}$

$P_2 = 53.875\ kN/m^2$

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Flow Through Pipes

The Reynolds number (Re) of a flowing fluid is calculated by multiplying the fluid velocity by the internal pipe diameter and then dividing the result by the kinematic viscosity.

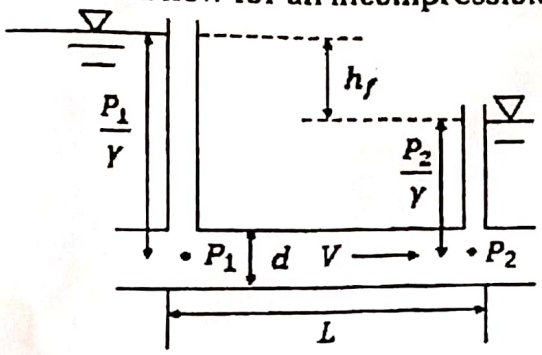
$$\text{Reynolds number} = \frac{\text{Fluid velocity x Internal pipe diameter}}{\text{Kinematic viscosity}}$$

$$R_e = \frac{V D}{v} = \frac{V D \rho}{\mu}$$

$R_e < 2000$ – the flow is laminar
$2000 < R_e < 4000$ – the flow is transitional
$R_e > 4000$ - the flow is turbulent

Darcy-Weisbach equation

In fluid dynamics, the Darcy–Weisbach equation is an empirical equation, which relates the head loss, or pressure loss, due to friction along a given length of pipe to the average velocity of the fluid flow for an incompressible fluid.



For pipe flow Darcy – weisbach equation is, $h_f = \frac{f' L V^2}{2 g d}$ or $\frac{4 f L V^2}{2 g d}$ (Where, $f' = 4 f$)

h_f = Head loss due to friction in a pipe of diameter d and Length L

V = Mean velocity of flow

f' = Darcy – weisbach friction factor

f = Coefficient of friction

For laminar flow, $f' = \frac{64}{R_e}$ and $f = \frac{16}{R_e}$ $\quad R_e < 2000$

For turbulent flow in smooth pipe, $f = \frac{0.0791}{R_e^{1/4}}$ $\quad 4000 \leq R_e \leq 10^6$

Question: Glycerin ($\rho = 2.44$, $\mu = 3.11 \times 10^{-2}$) at 68°F flows 120 ft through a 6 in diameter new wrought iron pipe at a velocity of 10 ft/s. determine the head loss due to friction.

Solution:

Reynold's number, $R_e = \frac{\rho d v}{\mu} = \frac{2.44 \times 6 \times 10}{12 \times 3.11 \times 10^{-2}} = 392$

Since $R_e < 2000$, the flow is laminar and $f' = \frac{64}{R_e} = \frac{64}{392} = 0.1633$

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Head loss due to friction, $h_f = \frac{f' L V^2}{2 g d} = \frac{0.1633 \times 120 \times 12 \times 10^2}{6 \times 2 \times 32.2} = 60.9$ ft

Question: For axial flow through a circular tube, the Reynolds number for transition to turbulence is approximately 2300, based upon the diameter and average velocity. If d = 5 cm and the fluid is kerosene at 20°C, find the volume flow rate in m^3/s which causes transition. For kerosene at 20°C, take $\rho = 804$ kg/m³ and $\mu = 0.00192$ kg/m·s.

Solution:

$$R_e = \frac{\rho V d}{\mu}$$

$$2300 = \frac{804 \times V \times 0.05}{0.00192}$$

$$V = 0.11 \text{ m/s}$$

$$Q = V A = 0.11 \times \frac{\pi}{4} (0.05)^2 = 2.16 \times 10^{-4} \text{ m}^3/\text{s}$$

Question: A fluid flows through a pipe having diameter 0.5 m, length 100 m and discharge 0.4 m^3/s. If frictional coefficient is 0.005 find the head loss due to friction. (51 BMA)

Solution:

Head loss due to friction, $h_f = \frac{4 f L V^2}{2 g d} = \frac{4 f L Q^2}{2 g \left(\frac{\pi}{4}\right)^2 d^5}$

$$h_f = \frac{4 \times 0.005 \times 100 \times 0.4^2}{2 \times 9.81 \times 0.616 \times 0.5^5} = 0.847 \text{ m}$$

Question: A fluid flows through a pipe having diameter 0.6 m and length 100 m. If the frictional coefficient is 0.004 and the head loss is 25 m find out the discharge. (DSCC – 2019)

Solution:

Head loss due to friction, $h_f = \frac{4 f L V^2}{2 g d} = \frac{4 f L Q^2}{2 g d A^2} = \frac{4 f L Q^2}{2 g \left(\frac{\pi}{4}\right)^2 d^5}$

Now, $Q^2 = \frac{2 g \left(\frac{\pi}{4}\right)^2 d^5 h_f}{4 f L}$

Discharge, $Q = \sqrt{\frac{2 g \left(\frac{\pi}{4}\right)^2 d^5 h_f}{4 f L}} = \sqrt{\frac{2 \times 9.81 \times 0.616 \times 0.6^5 \times 25}{4 \times 0.004 \times 100}} = 3.83 \text{ m}^3/\text{s}$

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Question: A crude oil of kinematic viscosity 0.4 stoke is flowing through a pipe of diameter 300 mm at the rate of 300 liters per sec. Find the head lost due to friction for a length of 50 m of the pipe.

Solution:

Kinematic viscosity, $v = 0.4$ stoke $= 0.4\ cm^2/s = 0.4 \times 10^{-4}\ m^2/s$

Diameter of pipe, $d = 300$ mm $= 0.30$ m

Discharge, $Q = 300$ liters/s $= 0.3\ m^3/s$

Length of pipe, $L = 50$ m

Velocity of flow, $V = \frac{Q}{A} = \frac{0.3}{\pi \times 0.3^2/4} = 4.24$ m/s

Reynolds number, $R_e = \frac{V D}{v} = \frac{4.24 \times 0.30}{0.4 \times 10^{-4}} = 3.18 \times 10^4$

$f = \frac{0.0791}{R_e^{1/4}} = \frac{0.0791}{(3.18 \times 10^4)^{1/4}} = 0.00591$ $\qquad 4000 \leq R_e \leq 10^6$

Head loss due to friction, $h_f = \frac{4 f L V^2}{2 g d} = \frac{4 \times 0.00591 \times 50 \times 4.24^2}{2 \times 9.81 \times 0.3} = 3.61$ m

Question: An oil of sp. gr. 0.9 and viscosity 0.06 poise is flowing through a pipe of diameter 200 mm at the rate of 60 liters/s. Find the head lost due to friction for a 500 m length of pipe. Find the power required to maintain this flow.

Solution:

Sp. gr. of oil $= 0.9$

Viscosity, $\mu = 0.06$ poise $= 0.006\ Ns/m^2$

Diameter of pipe, $d = 200$ mm $= 0.20$ m

Discharge, $Q = 60$ liters/s $= 0.06\ m^3/s$

Length of pipe, $L = 500$ m

Density, $\rho = 0.9 \times 1000 = 900\ kg/m^3$

Velocity, $V = \frac{Q}{A} = \frac{0.06}{\pi \times 0.2^2/4} = 1.91$ m/s

Reynolds number, $R_e = \frac{V D \rho}{\mu} = \frac{1.91 \times 0.2 \times 900}{0.006} = 57300$

$f = \frac{0.0791}{R_e^{0.25}} = \frac{0.0791}{(57300)^{0.25}} = 0.0051$ $\qquad 4000 \leq R_e \leq 10^6$

Head loss due to friction, $h_f = \frac{4 f L V^2}{2 g d} = \frac{4 \times 0.0051 \times 500 \times 1.91^2}{2 \times 9.81 \times 0.2} = 9.48$ m

Power required $= \gamma_w h_f Q = 9.81 \times 9.48 \times 0.06 = 5.02$ kW

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Question: Calculate the discharge through a pipe of diameter 200 mm when the difference of pressure head between the two ends of a pipe 500 m apart is 4 m of water. Take the value of coefficient of friction as 0.009

Solution:

Diameter of pipe d = 200 mm = 0.20 m

Head loss due to friction, $h_f = \frac{4 f l V^2}{2 g d}$

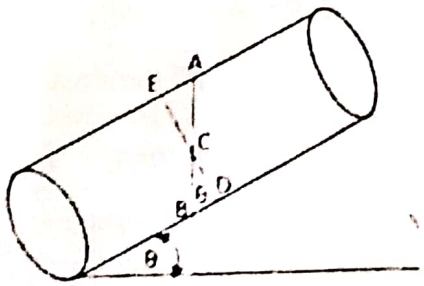
$$4 = \frac{4 \times 0.009 \times 500 \times V^2}{2 \times 9.81 \times 0.2}$$

$$V = \sqrt{\frac{4 \times 2 \times 9.81 \times 0.2}{4 \times 0.009 \times 500}} = 0.934 \text{ m/s}$$

Discharge, $Q = A V = 0.934 \times \frac{\pi \times 0.2^2}{4} = 0.0293 \text{ m}^3/\text{s}$

Question: Diameter of a pipe 4 ft. Angle of inclination θ = 30°. The pressure is the center of pipe 5 psi. Determine the pressure in top and bottom level. (BCPCL – 2016)

Solution:



Diameter of pipe = 4'

CD = CE = 2'

$\text{Cos } 30^\circ = \frac{CD}{BC}$

$BC = \frac{2}{\text{Cos } 30^\circ} = 2.31 = AC$

$P_C = 5 \text{ psi}$

Now, $P_C = P_A + \gamma AC$

$P_A = P_C - \gamma AC = 5 \times 122 - 62.4 \times 2.31 = 575.85 \text{ psf} = 4 \text{ psi}$

$P_B = P_C + \gamma BC = 5 \times 122 + 62.4 \times 2.31 = 863.22 \text{ psf} = 6 \text{ psi}$

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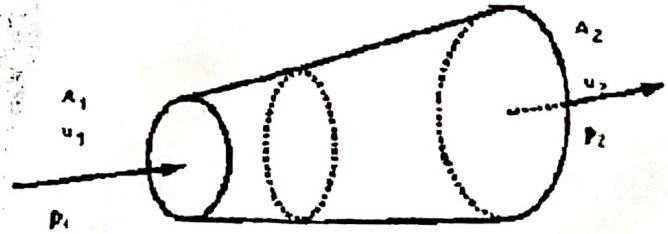
Momentum Equation

We have all seen moving fluids exerting forces. A jet of water from a hose exerts a force on whatever it hits. In fluid mechanics the analysis of motion is performed in the same way as in solid mechanics - by use of Newton's laws of motion. Account is also taken for the special properties of fluids when in motion.

Newton's second Law can be written: *The Rate of change of momentum of a body is equal to the resultant force acting on the body, and takes place in the direction of the force.*

To determine the rate of change of momentum for a fluid we will consider a streamtube as we did for the Bernoulli equation. We start by assuming that we have steady flow which is non-uniform flowing in a stream tube.



In time δ_t a volume of the fluid moves from the inlet a distance $u\,\delta_t$, so the volume entering the streamtube in the time δ_t is

Volume entering the stream tube = Area x distance = $A_1\,u_1\,\delta_t$

Mass entering the stream tube = volume x density = $\rho_1\,A_1\,u_1\,\delta_t$

Momentum of fluid entering stream tube = mass x velocity = $\rho_1\,A_1\,u_1\,\delta_t\,u_1$

Momentum of fluid leaving stream tube = mass x velocity = $\rho_2\,A_2\,u_2\,\delta_t\,u_2$

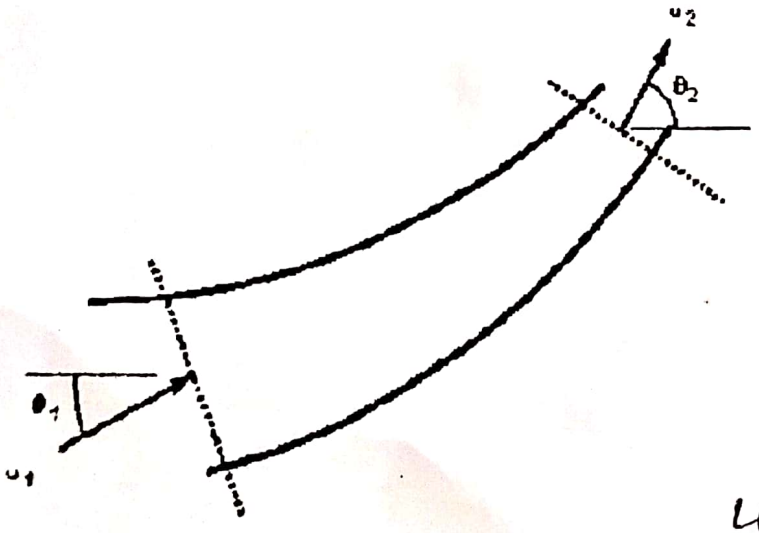
Force = rate of change of momentum

$$F = \frac{\rho_2\,A_2\,u_2\,\delta_t\,u_2 - \rho_1\,A_1\,u_1\,\delta_t\,u_1}{\delta_t}$$

From continuity equation, $Q = A_1\,u_1 = A_2\,u_2$ and if we have a fluid of constant density

$$F = \rho\,Q\,(u_2 - u_1)$$

At the inlet the velocity vector u_1 makes an angle θ_1 with the x-axis, while at the outlet u_2 make an angle θ_2. In this case we consider the forces by resolving in the directions of the co-ordinate axes.



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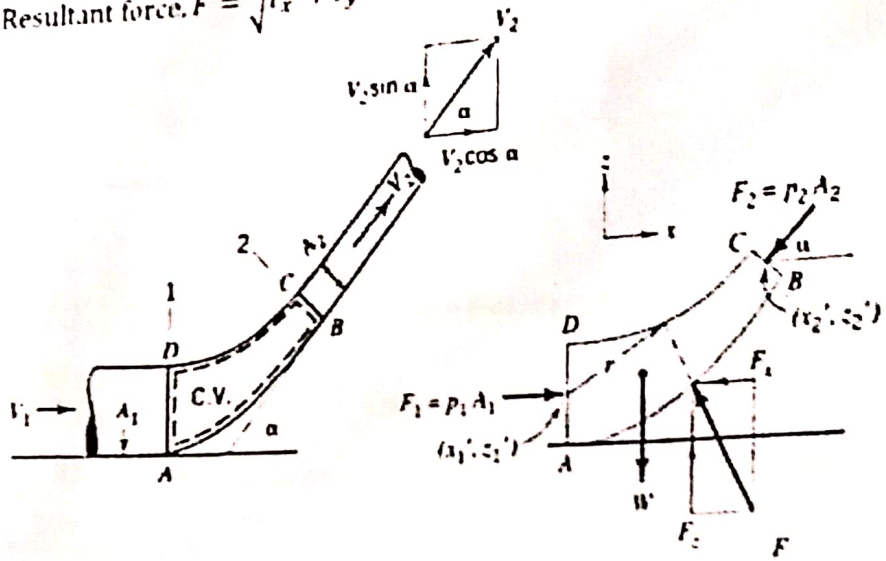
The force in the x-direction $F_x = \rho Q\,(u_2 \cos\theta_2 - u_1 \cos\theta_1) = \rho Q\,(u_{2x} - u_{1x})$

The force in the y-direction $F_y = \rho Q\,(u_2 \sin\theta_2 - u_1 \sin\theta_1) = \rho Q\,(u_{2y} - u_{1y})$

Resultant force, $F = \sqrt{F_x^2 + F_y^2}$

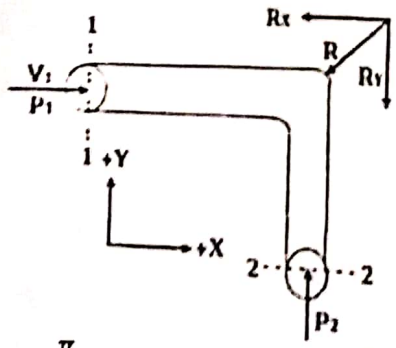


In x-direction, $F_1 - F_x - F_2 \cos\alpha = \rho Q\,(V_2 \cos\alpha - V_1)$

In y-direction, $F_z - F_2 \sin\alpha - W = \rho Q\,(V_2 \sin\alpha - 0)$

Question: The diameter of a 90° bend pipe is 16 cm at inlet and 12 cm at outlet. Neglecting friction, calculate the magnitude and direction of the force exerted on the bend by water flowing through is at 5 m/s and when the inlet pressure is 75 KN/m². (SGFCL – 2017)

Solution:



$A_1 = \frac{\pi}{4} \times 0.16^2 = 0.02\ m^2$

$A_2 = \frac{\pi}{4} \times 0.12^2 = 0.0113\ m^2$

$Q = A_1 V_1 = 0.1\ m^3/sec$

$V_2 = \frac{Q}{A_2} = 8.85\ m/s$

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By Appling Bernoulli equation to section 1 and 2, and noting that $Z_1 = Z_2$

$$\frac{P_1}{\gamma} + \frac{V_1^2}{2g} = \frac{P_2}{\gamma} + \frac{V_2^2}{2g}$$

$$\frac{75}{9.79} + \frac{5^2}{2 \times 9.81} = \frac{P_2}{9.79} + \frac{8.85^2}{2 \times 9.81}$$

$P_2 = 48.8\ kN/m^2$

By momentum equation in the X-direction

$\sum$ Forces in X direction = Rate of change of momentum in X direction

$P_1A_1 - R_x = \rho Q (V_{2x} - V_{1x})$

$P_1A_1 - R_x = \rho Q (0 - V_{1x})$

$75 \times 0.02 - R_x = -9.98 \times 0.1 \times 5$

$R_x = 6.49\ kN$

By momentum equation in the Y-direction

$\sum$ Forces in Y direction = Rate of change of momentum in Y direction

$R_y - P_2A_2 = \rho Q (V_{2y} - V_{1y})$

$R_y - P_2A_2 = \rho Q (V_{2y} - 0)$

$R_y - 48.8 \times 0.0113 = 9.98 \times 0.1 \times 8.85$

$R_y = 9.38\ KN$

Resultant force, $R = \sqrt{R_x^2 + R_y^2} = \sqrt{6.49^2 + 9.38^2} = 11.40\ KN$

Resultant force exerted by the water on bend = 11.40 KN

Direction of resultant force $= \tan^{-1}\frac{R_y}{R_x} = \tan^{-1}\frac{9.38}{6.49} = 55.32^0$

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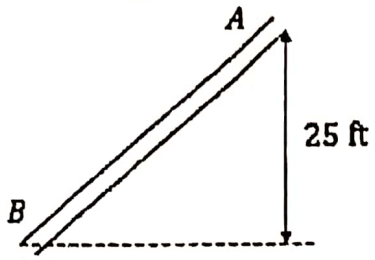
Question: The inclined pipe in fig is of uniform diameter. The pressure at A is 20 psi and at B, 30 psi. In which direction is the flow, and what is the friction loss of the fluid, if the liquid has specific weight (*a*) 30 lb/ft³ (*b*) 100 lb/ft³?



Solution:

$$Z_A + \frac{P_A}{\gamma} + \frac{V_A^2}{2g} = Z_B + \frac{P_B}{\gamma} + \frac{V_B^2}{2g} + h_L \qquad \text{assume flow is from A to B}$$

$$25 + \frac{20 \times 144}{30} = 0 + \frac{30 \times 144}{30} + h_L \qquad \text{here, } \frac{V_A^2}{2g} = \frac{V_B^2}{2g}$$

$h_L = -23$ ft. Since h_L is negative, flow is actually from B to A

$$25 + \frac{20 \times 144}{100} = 0 + \frac{30 \times 144}{100} + h_L \qquad \text{here, } \frac{V_A^2}{2g} = \frac{V_B^2}{2g}$$

$h_L = 10.6$ ft. Since h_L is positive, flow is actually from A to B

Question: Define HGL. A 15 cm diameter pipe carries 70 L/s of oil (G = 0.75). At a section 15 cm above the datum, the pressure is 2 cm vacuum of mercury. If α = 1.1 then calculate the total head in A of the oil section. (BIWTA – 2019)

Solution:

$$A = \frac{\pi}{4} \times 0.15^2 = 0.0176 \text{ m}^2$$

$$Q = 70 \text{ L/s} = 0.07 \text{ m}^3/\text{s}$$

$$V = \frac{Q}{A} = \frac{0.07}{0.0176} = 3.97 \text{ m/s}$$

$$\gamma = G\gamma_w = 0.75 \times 9.81 = 7.35 \text{ kN/m}^3$$

$$P = 2 \text{ cm vaccum of mercury} = 2 \times 1333.22 = 2666.44 \text{ Pa} = 2.66 \text{ kPa} = 2.666 \text{ kN/m}^2$$

$$\text{Head, } H = Z + \frac{P}{\gamma} + \frac{\alpha V^2}{2g} = 0.15 + \frac{2.66}{7.35} + \frac{1.1 \times 3.97^2}{2 \times 9.81} = 1.316 \text{ m}$$

Question: The value of pressure, elevation and diameter of a pipe in a point P are 12 N/m², 5 m and 25 cm. Another point Q whose elevation and diameter are 3 m and 17 cm. what will be the pressure at Q? Given discharge P to Q is 2 m³/s and head loss 1m. (BWDB – 2016, WASA – 2017)

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Solution:

For P: $Q = \frac{\pi}{4}(0.25)^2 \times V_1 = 2$

$V_1 = 40.744$ m/s

For Q: $Q = \frac{\pi}{4}(0.17)^2 \times V_1 = 2$

$V_1 = 88.11$ m/s

$\frac{P_1}{\gamma} = \frac{12}{9.8 \times 1000} = 0.00122$ m

$Z_1 + \frac{P_1}{\gamma} + \frac{V_1^2}{2g} = Z_2 + \frac{P_2}{\gamma} + \frac{V_2^2}{2g} + \text{Head loss}$

$5 + 0.00122 + \frac{40.744^2}{2 \times 9.81} = 3 + \frac{P_2}{\gamma} + \frac{88.11^2}{2 \times 9.81} + 1$

$\frac{P_2}{\gamma} = -310.39$ m

$P_2 = -310.39$ m

Question: A pipe discharge 40 m³/s. Head loss is 10% of the difference of velocity head. Find out the pressure at point B. (BWDB – 2018)

	Point P	Point Q
Elevation	1011 m	1050 m
Pipe dia.	50 cm	100 cm
Pressure	50 kpa	???

Solution:

Area of pipe at point P, $A_1 = \frac{\pi}{4} \times 0.5^2 = 0.196$ m²

Area of pipe at point Q, $A_2 = \frac{\pi}{4} \times 1^2 = 0.785$ m²

Velocity at point P, $V_1 = \frac{Q}{A_1} = \frac{40}{0.196} = 203.72$ m/s

Velocity at point P, $V_2 = \frac{Q}{A_2} = \frac{40}{0.785} = 50.93$ m/s

Difference of velocity head $= \frac{V_1^2}{2g} - \frac{V_2^2}{2g} = \frac{1}{2 \times 9.81}(203.7^2 - 50.93^2) = 1983$ m

Head loss, $h_L = 1983 \times 0.1 = 198.3$ m

$Z_1 + \frac{P_1}{\gamma} + \frac{V_1^2}{2g} = Z_2 + \frac{P_2}{\gamma} + \frac{V_2^2}{2g} + h_L$

$1011 + 2115.28 + 5.09 = 1050 + 132.2 + \frac{P_2}{\gamma} + 198.3$

$P_2 = 17176.03$ kPa

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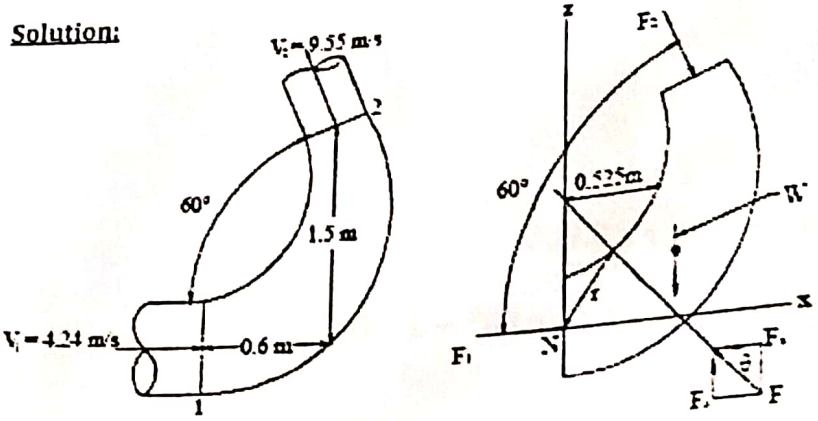
Question: When 300 liter/sec of water flow through this vertical 300 mm by 200 mm pipe bend, the pressure at the entrance is 7 t/m². Calculate the force by the fluid on the bend if the volume of the bend is 0.085 m³.

Solution:



From the continuity principal,

$$Q = V_1 A_1 = V_2 A_2$$

$$V_1 = \frac{4Q}{\pi D_1^2} = \frac{4 \times 0.3}{\pi \times 0.3^2} = 4.24 \text{ m/s}$$

$$V_2 = \frac{4Q}{\pi D_2^2} = \frac{4 \times 0.3}{\pi \times 0.2^2} = 9.55 \text{ m/s}$$

and from the Bernoulli equation,

$$\frac{p_1}{\gamma} + z_1 + \frac{V_1^2}{2g} = \frac{p_2}{\gamma} + z_2 + \frac{V_2^2}{2g}$$

$$7 + 0 + \frac{4.24^2}{19.62} = \frac{p_2}{\gamma} + 1.5 + \frac{9.55^2}{19.62}$$

$$\frac{p_2}{\gamma} = 1.77 \text{ m}, p_2 = 1.77 \text{ t/m}^2$$

Now, for the free body diagram, the pressure forces F_1 and F_2 may be computed

$$F_1 = \frac{\pi}{4} \times 0.3^2 \times 7 = 0.495 \text{ ton}$$

$$F_2 = \frac{\pi}{4} \times 0.2^2 \times 1.77 = 0.056 \text{ ton}$$

$$\Sigma F_x = 0$$

$$F_x = F_1 + \rho Q V_1 + F_2 \cos 60^0 + \rho Q V_2 \cos 60^0$$

$$F_x = 0.495 + \frac{1}{9.81} \times 0.30 \times 4.24 + \left(0.056 + \frac{1}{9.81} \times 0.30 \times 9.55\right) \cos 60^0$$

$$F_x = 0.799 \text{ ton}$$

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$$\Sigma F_z = 0$$

$$F_z = F_2 \sin 60^0 + \rho\, Q\, V_2 \sin 60^0\, \rho + \gamma \text{ x } 0.085$$

$$F_z = 0.056 \sin 60^0 + \left(\frac{1}{9.81} \text{ x } 0.30 \text{ x } 9.55\right) \sin 60^0 + 1 \text{ x } 0.085$$

$$F_z = 0.386 \text{ ton}$$

$$F = \sqrt{F_x^2 + F_z^2} = \sqrt{0.779^2 + 0.386^2} = 0.869 \text{ ton}$$

$$\tan\theta = \frac{F_z}{F_x} = \frac{0.386}{0.799} = 0.483, \theta = 25.8^0$$

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Engineering Hydrology

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Engineering hydrology

Introduction

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

Hydrological cycle

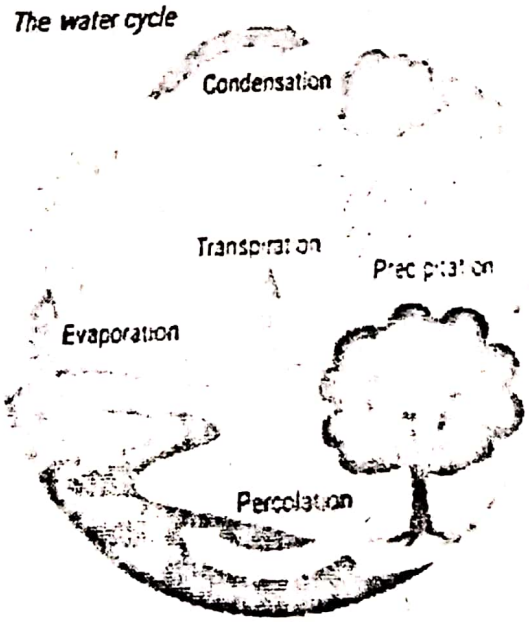
The hydrologic cycle describes the continuous re-circulating transport of the waters of the earth, linking atmosphere, land and oceans. Water evaporates from the ocean surface, driven by energy from the Sun, and joins the atmosphere, moving inland as clouds. Once inland, atmospheric conditions act to condense and precipitate water onto the land surface, where, driven by gravitational forces, it returns to the ocean through river and streams.

Hydrological Processes

1. Precipitation
2. Evaporation
3. Transpiration
4. Infiltration
5. Overland flow
6. Surface Runoff
7. Groundwater outflow

The hydrologic cycle consists of four key components

1. Precipitation
2. Runoff
3. Storage
4. Evapotranspiration



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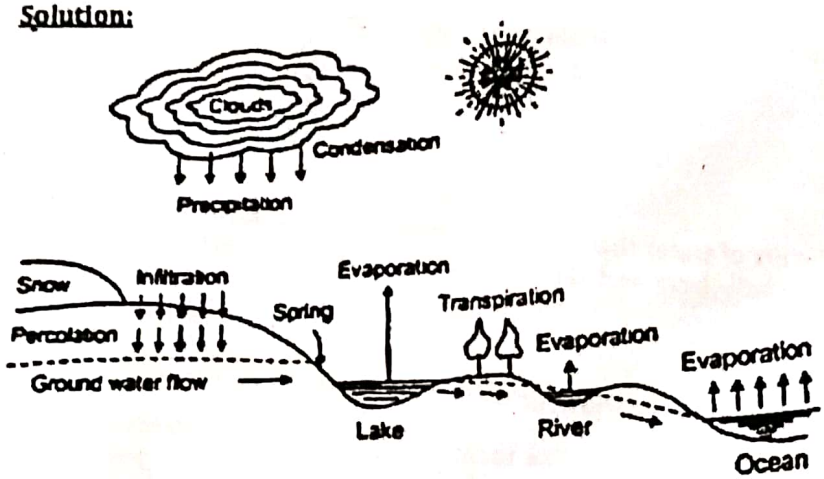
Question: What is hydrological cycle? Describe it. (BSEC – 2014)

Solution:



The hydrologic cycle begins with the evaporation of water from the surface of the ocean. As moist air is lifted, it cools and water vapor condenses to form clouds. Moisture is transported around the globe until it returns to the surface as precipitation. Once the water reaches the ground, one of two processes may occur; some of the water may evaporate back into the atmosphere or the water may penetrate the surface and become groundwater. Groundwater either seeps its way to into the oceans, rivers, and streams, or is released back into the atmosphere through transpiration. The balance of water that remains on the earth's surface is runoff, which empties into lakes, rivers and streams and is carried back to the oceans, where the cycle begins again.

Applications of Hydrology in Engineering

- Hydrology is used to find out maximum probable flood at proposed sites e.g. Dams.
- The variation of water production from catchments can be calculated and described by hydrology.
- The expected flood flows over a spillway, at a highway Culvert, or in an urban storm drainage system can be known by this very subject.
- It helps us to know the required reservoir capacity.
- Used in connection with design and operations of hydraulic structure
- Used in prediction of flood over a spillway, at highway culvert or in urban storm drainage
- Hydrology is an indispensable tool in planning and building hydraulic structures.
- Hydrology is used for city water supply design which is based on catchments area, amount of rainfall, dry period, storage capacity, runoff evaporation and transpiration.
- Dam construction, reservoir capacity, spillway capacity, sizes of water supply pipelines and effect of afforest on water supply schemes, all are designed on basis of hydrological equations

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Hydrologic equation

The hydrologic equation is simply the statement of the law of conservation of matter and is given by, $I = O + \Delta S$

Where, I = inflow

O = outflow

ΔS = change in storage

Precipitation

The term "precipitation" denotes all forms of water that reach the earth from the atmosphere. The usual forms are rainfall, snowfall, hail, frost and dew. The magnitude of precipitation varies with time and space.

Forms of precipitation:

Some of the common forms of precipitation are rain, snow, drizzle, glaze, sleet and hail.

1. Rain: It is the principal form of precipitation in BD. The term rainfall is used to describe precipitation in the form of water drops of sizes larger than 0.5mm. The maximum size of a raindrop is 6mm.

Light rain: trace to 2.5 mm/ hour
Moderate rain: 2.5mm/ hour to 7.5mm/ hour
Heavy rain: > 7.5mm/ hour

2. Snow: Snow is another important form of precipitation. Snow consists of ice crystals which usually combine to form flakes. Snow has an Initial density 0.06 to 0.15g/cm^3. Average density of 0.1g/cm^3.

3. Drizzle: A fine sprinkle of numerous water droplets of size less than 0.5mm and intensity less than 1mm/hour is known as drizzle. In this, the drops are so small that they appear to float in the air.

4. Glaze: When rain or drizzle comes in contact with cold ground at Zero degree C, the water drops freeze to form an ice coating called glaze or freezing rain.

5. Sleet: It is frozen raindrops of transparent grains which form when rain falls through air at sub-freezing temperature. In Britain, sleet denotes precipitation of snow and rain simultaneously.

6. Hail: It is a showery precipitation in the forms of irregular pellets of lump of ice of size more than 8mm. Hails occur in violent thunderstorms in which vertical currents are very strong.

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Question: What is precipitation? Write down the type of precipitation. (ARMY – 2014)

Solution:

The term "Precipitation" denotes all forms of water that reach the earth from the atmosphere
The common types of precipitation are-

- Rain
- Snow
- Drizzle
- Glaze
- Sleet
- Hail

Question: State the factors affecting hydrograph. (WRGCL – 2014)

Solution:

- Geological rock type and soil
- Vegetation
- Rainfall intensity
- Shape of drainage basin
- Topography & relief
- Heavy Storms
- Snowfall

Measurement of precipitation

Rainfall: Precipitation is expressed in terms of the depth to which rainfall water would s
on an area if all the rain were collected on it. The precipitation is collected and measured
rain gauge.

Adequacy of rain gauge stations

Optimum number of stations that should exist to have an assigned percentage of error in the estimation of mean rainfall is obtained by:

$$N = \left(\frac{C_v}{\varepsilon}\right)^2$$

N = optimal number of stations

ε = allowable degree of error in the estimate of the mean rainfall

C_v = coefficient of variation

$$C_v = \frac{100 \times \sigma_{m-1}}{\bar{P}}$$

$$\sigma_{m-1} = \sqrt{\frac{(P_i - \bar{P})^2}{m-1}}$$

σ_{m-1} = standard deviation

P_i = Precipitation magnitude in the i^{th} station

$\bar{P} = \frac{\sum_1^m P_i}{m}$ = mean precipitation

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Question: A catchment has six rain gauge stations. In a year, the annual rainfalls recorded by the gauges are as follows. For a 10% error in the estimation of the mean rainfall, calculate the optimum number of stations in the catchment.

Station	A	B	C	D	E	F
Rainfall (cm)	82.6	102.9	180.3	110.3	98.8	136.7

Solution:

$$\bar{P} = \frac{\sum P_i}{m} = \frac{82.6 + 102.9 + 180.3 + 110.3 + 98.8 + 136.7}{6} = 118.6$$

$$\sigma_{m-1} = \sqrt{\frac{(P_i - \bar{P})^2}{m-1}} = 35.04$$

$$C_v = \frac{100 \times \sigma_{m-1}}{\bar{P}} = \frac{100 \times 35.04}{118.60} = 29.54$$

Number of stations, $N = \left(\frac{C_v}{\varepsilon}\right)^2 = \left(\frac{29.54}{10}\right)^2 = 8.7$ say 9 stations

The optimum number of stations for the catchment is 9.

Hence three more additional stations are needed.

Evaporation

Evaporation is the process by which water changes from a liquid to a gas or vapor. Evaporation is the primary pathway that water moves from the liquid state back into the water cycle as atmospheric water vapor. Studies have shown that the oceans, seas, lakes, and rivers provide nearly 90 percent of the moisture in the atmosphere via evaporation, with the remaining 10 percent being contributed by plant transpiration.

Factors that affect evaporation

Factors responsible for evaporation are divided into three major classes:

1. Meteorological factors.

- Latent heat responsible for evaporation provided by sun.
- Temperature: Both of air and evaporating surface.
- Air vapor pressure as air vapor potential.
- Air humidity.
- Atmospheric pressure.
- Wind speed.

2. Nature of evaporating surface.
3. Physical factors.

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Transpiration

Transpiration is the process by which the water vapor escapes from the living plant leaves and enters the atmosphere.

Evapotranspiration

Evapotranspiration (E_t) or consumptive use (U) is the total water lost from a cropped land due to evaporation from the soil and transpiration by the plants or used by the plants in building up of plant tissue.

Potential Evapotranspiration (PET)

Potential evapotranspiration (E_{pt}) is the evapotranspiration from the short green vegetation when the roots are supplied with unlimited water covering the soil. It is usually expressed as a depth (cm, mm) over the area.

Actual Evapotranspiration (AET)

The real Evapotranspiration occurring in a specific situations called actual Evapotranspiration (AET).

Question: Define a) Evaporation b) Evapotranspiration (TGTDCL – 2014)

Solution:

Evaporation: Evaporation is a type of vaporization that occurs on the surface of a liquid as it changes into the gas phase before reaching its boiling point.

Evapotranspiration: Evapotranspiration is the total water lost from cropped land due to the evaporation from the soil transpiration from the soil plants.

Runoff

When a storm occurs, a portion of rainfall infiltrates into the ground and some portion may evaporate. The rest flows as a thin sheet of water over the land surface which is termed as overland flow. It means the draining/flowing off of precipitation from catchment area through a surface Channel. Otherwise it represents the output from the catchment in a basin unit of time.

Rainfall - Losses = Runoff

Effective rain = Rainfall – Interception loss

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Question: The total observed runoff volume during a storm of 6-hr duration with a uniform intensity of 15 mm/hour is 21.6 Mm^3. If the area of the basin is 300 km^2, find the average infiltration rate and the runoff coefficient for the basin.

Solution:

Infiltration loss, F_p = Rainfall (P) − Runoff (R)

$$= 15 \times 6 - \frac{21.6 \times 10^6}{300 \times 10^6} \times 1000 = 18 \text{ mm}$$

$$f_{av} = \frac{F_p}{t} = \frac{18}{6} = 3 \text{ mm/hr}$$

Now, Yield $= C\,A\,P$

$$21.6 \times 10^6 = C\,(300 \times 10^6)\,\frac{90}{100}$$

$$C = 0.8$$

Surface runoff

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

Subsurface runoff

Known as interflow/ subsurface flow/ through flow. The part of precipitation which infiltrates into the ground and moves laterally/horizontally in the soil and returns to the surface at some location away from the point of entry into the soil is called as interflow.

Ground water flow

Another route for the infiltrated water is to undergo deep resolution and reach the ground water storage in the soil. The part of runoff is called ground water runoff/ flow.

Direct runoff

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

Base flow

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

Rainfall excess:

The part/percentage of precipitation which is equal to the vol. of direct runoff from a basin is called rainfall excess. Effective rainfall = Direct runoff volume + subsurface runoff = Rainfall excess + Subsurface runoff

Effective rainfall > Rainfall excess

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Factors affecting runoff:

- Climatic factors
- Basin characteristics
- Basin geology
- Basin infiltration characteristics

Effective rainfall (ER): Effective rainfall (also known as Excess rainfall) is that part of the rainfall that becomes direct runoff at the outlet of the watershed. It is thus the total rainfall in a given duration from which abstractions such as infiltration and initial losses are subtracted. As such ER could be defined as that rainfall that is neither retained on the land surface nor infiltrated into the soil.

Infiltration Indices

Water entering the soil at the ground surface is called infiltration. It replenishes the soil moisture deficiency and the excess moves downward by the force of gravity called deep seepage or percolation and builds up the ground water table. The maximum rate at which the soil in any given condition is capable of absorbing water is called its infiltration capacity (*fp*). The infiltration curve expresses the rate of infiltration (cm/hr) as a function of time. The area between the rainfall graph and the infiltration curve represents the rainfall excess, while the area under the infiltration curve gives the loss of rainfall due to infiltration.

Φ Index

The φ -index is defined as that rate of rainfall above which the rainfall volume equals the runoff volume. The φ -index is relatively simple and all losses due to infiltration, interception and depression storage (*i.e.*, storage in pits and ponds) are accounted for.

$$\varphi - index = \frac{\text{basin recharge}}{\text{duration of rainfall}}$$

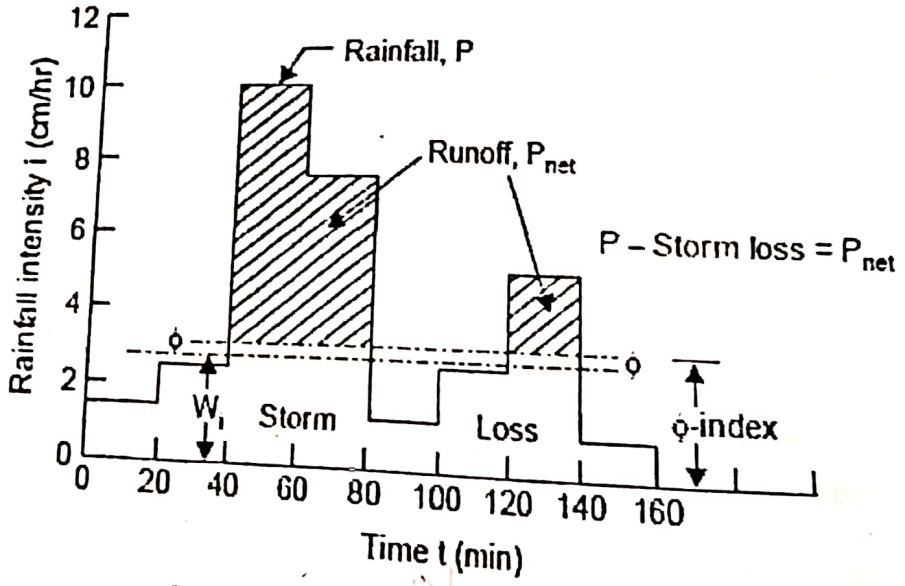

Fig. Infiltration loss by ϕ-index

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W-Index

The W-index is the average infiltration rate during the time rainfall intensity exceeds the infiltration capacity rate.

$$W - \text{Index} = \frac{F_p}{t_R} = \frac{P - Q - S}{t_R}$$

Where, P = total rainfall

Q = surface runoff

S = effective surface retention

t_R = duration of storm during which $i > f_p$

F_p = total infiltration

Question: The rates of rainfall for the successive 30 min period of a 3-hour storm are: 1.6, 3.6, 5.0, 2.8, 2.2, 1.0 cm/hr. The corresponding surface runoff is estimated to be 3.6 cm. Establish the φ-index. Also determine the W-index.

Solution:

$$\Sigma(i - \varphi)\, t = P_{net}$$

$$[(i - \varphi) + (i - \varphi) + (i - \varphi) + (i - \varphi)]\frac{30}{60} = 3.6$$

$$\varphi = 1.6 \text{ cm/hr}$$

$$P = (1.6 + 3.6 + 5.0 + 2.8 + 2.2 + 1.0)\frac{30}{60} = 8.1 \text{ cm}$$

$$W - \text{index} = \frac{P - Q}{t_R} = \frac{8.1 - 3.6}{3} = 1.5 \text{ cm/hr}$$

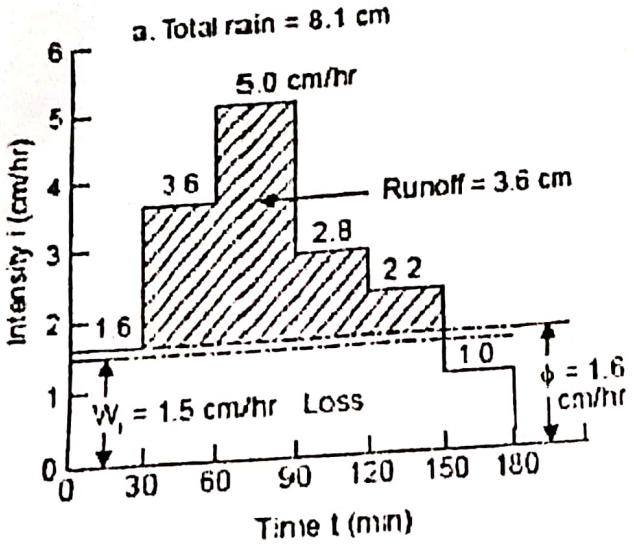

Figure: φ – index computations

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Question: The rainfall rates for successive 30-minutes intervals up to 4 hours are given below. If the surface runoff is 3.6 cm, determine φ and W-indices.

Time (minutes)	0	30	60	90	120	150	180	210	240
Rainfall intensity (cm/h)	0	1.8	2.8	4.1	3.9	2.8	2.0	1.8	0.9

Solution:

Computation of φ – index

Let i = rate of rain in cm/h and φ – index be expressed in cm/h

$P_{net} = (i - \varphi)\, t$

Assuming φ to be greater than 1.3 cm/h but less than 1.8 cm/h

$$3.6 = [(2.8 - \varphi) + (4.1 - \varphi) + (3.9 - \varphi) + (2.8 - \varphi) + (2 - \varphi) + (1.8 - \varphi)]\frac{30}{60}$$

$\varphi = 1.7$ cm/h

Since this is greater than 1.3 and less than 1.8 cm/h above computations are correct.

Computation of W – index

$$P = (1.3 + 2.8 + 4.1 + 3.9 + 2.8 + 2 + 1.8 + 0.9)\frac{30}{60} = 9.8 \text{ cm}$$

$$W - \text{index} = \frac{P - Q}{t_R} = \frac{9.8 - 3.6}{4} = 1.55 \text{ cm/hr}$$

Question: Precipitation 9 cm, runoff 5 cm, Rainfall data is given to the time. Find the value of Φ-index. (BWDB – 2018, BIWTA – 2019)

Time	1	2	3	4	5	6
Rainfall	0.4	2.2	4	2.2	1.4	0.9

Solution:

If infiltration is assumed constant, $\varphi - \text{index} = \dfrac{P - R}{t} = \dfrac{11.1 - 5}{6} = 1.017$ cm

But this infiltration rate can't be possible throughout the time period of 6 hours, because 1 hour and 6 hour rainfall rate is lesser and infiltration rate can never be more than rainfall rate. φ-index will be somewhere between 0.9 cm/hour to 1.4 cm/hour.

$$\varphi - \text{index} = \frac{11.1 - 5 - 0.4 - 0.9}{4} = 1.2 \text{ cm/hour}$$

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Question: The rainfall rates for successive 1 hour intervals up to 5 hours are given below. If the surface runoff is 5 cm, determine the φ index. (BWDB – 2019)

Time(hour)	0	1	2	3	4	5
Rainfall intensity (cm/hr.)	0	0.6	3	2	4	2.4

Solution:

We know $R = \sum(i - \varphi)\, t$

Assuming φ to be greater than 0.6 cm/h and less than 2 cm/h

$$5 = [(3 - \varphi) + (2 - \varphi) + (4 - \varphi) + (2.4 - \varphi)]\ \frac{60}{60}$$

From which we get, $\varphi = 1.65$ cm/h

Which is greater than 0.6 cm/hr. and less than 2 cm/hr.

Question: A storm with 10 cm precipitation has direct runoff 60,000 m^3 in 1 km^2 catchment area. Calculate runoff coefficient and effective rainfall. (BWDB – 2018)

Solution:

$$\text{Effective rainfall} = \text{direct runoff} = \frac{60{,}000}{1 \times 1000 \times 1000} = 0.06\ \text{m} = 6\ \text{cm}$$

$$\text{Runoff co} - \text{efficient} = \frac{\text{Runoff}}{\text{Rainfall}} = \frac{6}{10} = 0.6$$

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Hydrograph: A hydrograph of a catchment produced by a storm is a graphical representation of discharge rate of a stream with respect to the time from the commencement of storm. It is an inverted U shaped diagram.

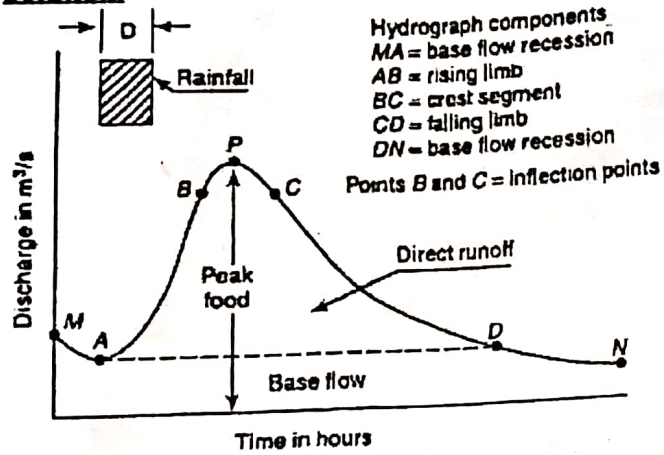
Components of Hydrograph:

Hydrograph generally contains the following three parts.

1. Rising Limb
2. Peak (or Crest) Segment
3. Falling (or Recession) Limb

Question: Draw a typical hydrograph showing different segments. (BWDB – 2013)

Solution:



Question: State the factors affecting flood hydrograph? (WRGCL – 2014)

Solution:

- Shape of the basin
- Size of the basin
- Slope
- Drainage density
- Land use
- Climate Factor

Question: Define Unit hydrograph. Why it is necessary? (BUET M.Sc. –2011)

Solution:

A unit hydrograph is defined as the hydrograph of direct runoff resulting from one unit depth (1cm) of rainfall excess occurring uniformly over the basin and at a uniform rate for a specified duration (D hours). The direct runoff hydrograph in a catchment due to a given storm can be determined if an appropriate unit hydrograph is available. The initial losses and infiltration losses are estimated and deducted from the storm hyetograph to obtain ERH.

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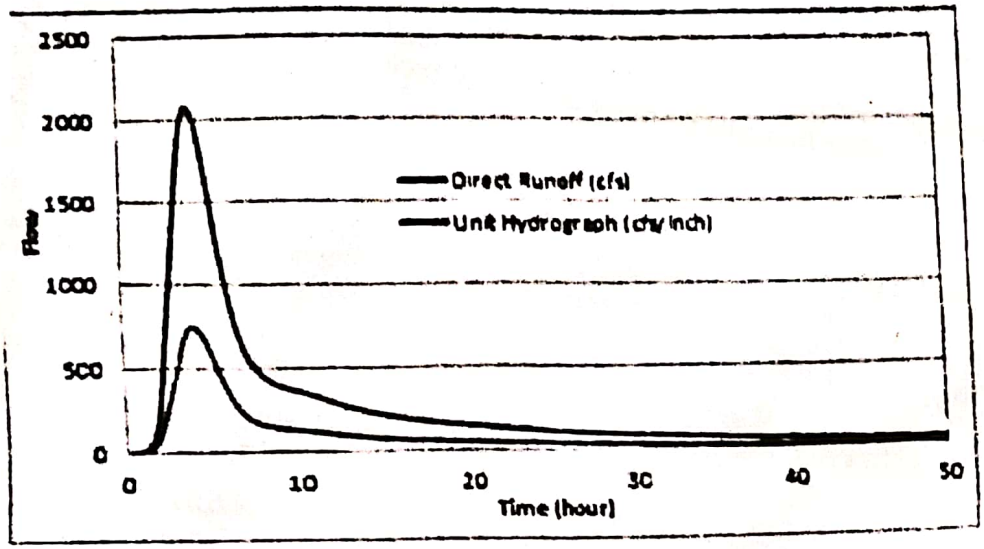
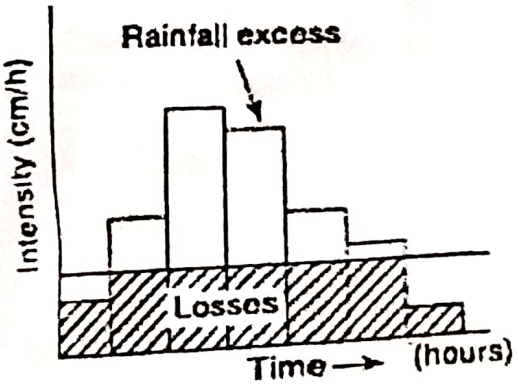


Effective rainfall: Effective rainfall also known as excess rainfall is that part of the rainfall that becomes direct runoff at the outlet of the watershed. It is thus the total rainfall in a given duration from which abstractions such as infiltration and initial losses are subtracted. The resulting hyetograph is known as effective rainfall hyetograph (ERH).



$$P_{net} = P - Losses$$

$$P_{net} = \frac{\sum Q_d t}{A} \quad (Q_d = DRO)$$

$$DRO = TRO - BFO$$

$$UGO = \frac{DRO}{P_{net}}$$

P = total rainfall

P_{net} = net precipitation or direct runoff as equivalent depth over the basin

Losses = due to infiltration (F_p)

DRO = direct runoff ordinate

TRO = total runoff ordinate

t = time interval between successive direct runoff ordinate

BFO = base flow ordinate

UGO = unit hydrograph ordinate

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Question: The design storm of a water shed has the depths of rainfall of 4.9 and 3.9 cm for the consecutive 1-hr periods. The 1-hr UG can be approximated by a triangle of base 6 hour with a peak of 50 cumec occurring after 2 hour from the beginning. Compute the flood hydrograph assuming an average loss rate of 9 mm/hour and constant base flow of 10 cumec. What is the area of water shed and its coefficient of runoff?

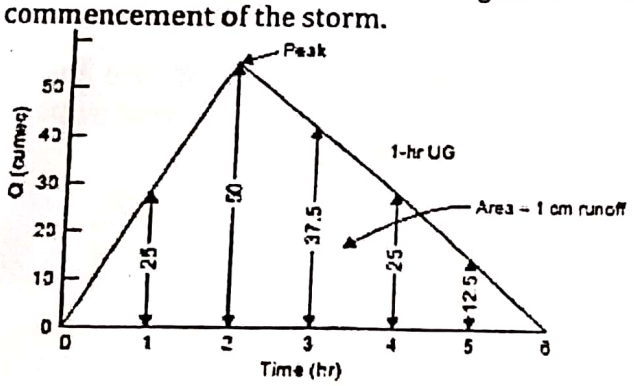
Solution:

(*i*) The flood hydrograph due to the two consecutive hourly storms is computed in Table

Time (hr)	UGO* (cumec)	DRO due to rain-fall excess (cumec)		Total (cumec)	BF (cumec)	TRO (cumec)	Remarks
		4.9 – 0.9 = 4 cm	3.9 – 0.9 = 3 cm				
1	0	0	—	0	10	10	
2	25	100	0	100	10	110	
3	50	200	75	275	10	285	
4	37.5	150	150	300	10	310	← Peak flood
5	25	100	112.5	212.5	10	222.5	
6	12.5	50	75	125	10	135	
7	0	0	37.5	37.5	10	47.5	
8	—	—	0	0	10	10	

Table: Computation of design flood hydrograph

Ordinates by proportion in the triangular UG. Peak flood of 310 cumec, after 4 hour from the commencement of the storm.



(*ii*) Area of water shed - To produce 1 cm net rain over the entire water shed (A km^2).
Volume of water over basin = Area of UG (triangle)

$$(A \times 10^6)\frac{1}{100} = \frac{1}{2}(6 \times 60 \times 60)\,50$$

$$A = 54 \text{ km}^2$$

(*iii*) Coefficient of runoff, $C = \frac{R}{P} = \frac{(4.9-0.9)+(3.9-0.9)}{4.9+3.9} = 0.795$

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Question: The peak of flood hydrograph due to a 3-h duration isolated storm in a catchment is 270 m^3/s. The total depth of rainfall is 5.9 cm. Assuming an average infiltration loss of 0.3 cm/h and a constant base flow of 20 m^3/s, estimate the peak of the 3-h unit hydrograph (UH) of this catchment. If the area of the catchment is 567 km^2 determine the base width of the 3-h unit hydrograph by assuming it to be triangular in shape.

Solution:

Duration of rainfall excess = 3 h
Loss @ 0.3 cm/h for 3 h = 0.9 cm
Total depth of rainfall = 5.9 cm
Rainfall excess = 5.9 – 0.9 = 5.0 cm

Peak of flood hydrograph = 270 m^3/s
Base flow = 20 m^3/s
Peak of DRH = 270 – 20 = 250 m^3/s

$$\text{peak of 3-h unit hydrograph} = \frac{\text{peak of DRH}}{\text{rainfall excess}} = \frac{250}{5.0} = 50 \text{ m}^3/\text{s}$$

Let, B = base width of the 3-h UH in hours
Volume represented by the area of UH = volume of 1 cm depth over the catchment area of UH
= (Area of catchment x 1 cm)

$$\frac{1}{2} \times B \times 60 \times 60 \times 50 = 567 \times 10^6 \times \frac{1}{100}$$

$B = 63$ hours

Question: The value of peak of a 6-h flood hydrograph is 35 m^3/sec and constant base flow is 10 m^3/sec. If the rainfall depth is 3.25 cm and infiltration rate is 0.125 cm/h. Determine peak of 6-h unit hydrograph and run-off co-efficient. (BWDB – 2016, MPA – 2019)

Solution:

Duration of rainfall excess = 6 h

Total depth of rainfall = 3.25 cm

Loss of infiltration rate = 0.125 x 6 = 0.75 cm

Rainfall excess = 3.25 – 0.75 = 2.5 cm

Peak of flood hydrograph = 35 m^3/sec

Base flow = 10 m^3/sec

Peak of DRH = Peak of flood hydrograph – Base flow = 35 – 10 = 25 m^3/sec

$$\text{Peak of 6-h unit hydrograph} = \frac{\text{Peak of DRH}}{\text{Rainfall excess}} = \frac{25}{2.5} = 10 \text{ m}^3/\text{sec}$$

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Question: The successive three hourly ordinates of a 6-hour UG for a particular basin are 0, 15, 36, 30, 17.5, 8.5, 3, 0 cumec respectively. The flood peak observed due to a 6-hour storm was 150 cumec. Assuming a constant base flow of 6 cumec and an average storm loss of 6 mm/hr, determine the depth of storm rainfall and the streamflow at successive 3 hour interval.

Solution:

DRO peak = Flood peak − BF = 150 − 6 = 144 cumec

$$P_{net} = \frac{DRO_{peak}}{UG_{peak}} = \frac{144}{36} = 4 \text{ cm}$$

Depth of storm rainfall, $P = P_{net}$ + losses = 4 + 0.6 x 6 = 7.6 cm

$DRO = UGO \times P_{net}$

$DRO + BF = TRO$

Hence, multiplying the given UGO by 4 cm and adding 6 cumec, the stream flow ordinates at successive 3-hour intervals are: 6, 66, 150, 126, 76, 40, 18 and 6 cumec respectively.

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Flood:

A flood is an unusually high stage in a river, normally the level at which the river overflows its banks and inundates the adjoining area. The damages caused by floods in terms of loss of life, property and economic loss due to disrupt ion of economic activity are all too well known. Thousands of crores of rupees are spent every year in flood control and flood forecasting. The hydrograph of extreme floods and stages corresponding to flood peaks provide valuable data for purposes of hydrologic design. Further, of the various characteristics of the flood hydrograph probability the most important and widely used parameter is the flood peak. At a given location in a stream, flood peaks vary from year to year and their magnitude constitutes hydrologic series which enable one to assign a frequency to a given flood-peak value. In the design of practically all hydraulic structures the peak flow that can be expected with an assigned frequency (say 1 in 100 years) is of primary importance to adequately proportion the structure to accommodate its effect The design of bridges, culvert waterways and spillways for dams and estimation of scour at a hydraulic structure are some examples wherein flood-peak values are required.

To estimate the magnitude of a flood peak the following alternative methods are available:

1. Rational method
2. Empirical method
3. Unit-hydrograph technique
4. Flood-frequency studies

Tue use of a particular method depends upon (i) the desired objective (ii) the available data and (iii) the importance of the project.

Further the rational formula is only applicable to small size (< 50 km^2) catchments and the unit-hydrograph method is normally restricted to moderate-size catchments with areas less than 5000 km^2.

Rational Equation: $Q = C\,I\,A$

Q = Peak discharge in cfs

C = Rational method runoff coefficient

I = Rainfall intensity in inch/hour

A = Drainage area in acre

Storm water quantity, $Q = \frac{C\,I\,A}{360}$

Where, Q is in m^3/sec, I is in mm/hour and A is area in hectares

$Q = 0.278\,C\,I\,A$

Where, Q is in m^3/sec, I is in mm/hour and A is area in square kilometer

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Question: Define design flood. Explain the various methods for estimating the design flood of a catchment? (33 & 38th BCS)

Solution:

Design flood is the value of the instantaneous peak discharge adopted for the design of a particular project or any of its structure. The term design flood used to denote the maximum flood flow that could be passed without damage or serious threat to the stability of engineering structures and is selected after consideration of economic and hydrologic factors. The maximum flood discharge (peak flood) in a river may be determined by the following methods:

1. Rational Method: The rational method is based on the application of the formula

$Q = C\,i\,A$

Where C is a coefficient depending on the runoff qualities of the catchment called the runoff coefficient (0.2 to 0.8). The intensity of rainfall i is equal to the design intensity or critical intensity of rainfall i_c corresponding to the time of concentration t_c for the catchment for a given recurrence interval T; the design intensity of rainfall $i\ (= i_c)$ can be found from the intensity-duration-frequency curves, for the catchment corresponding to t_c and T. If the intensity-duration-frequency curves, are not available for the catchment and a maximum precipitation of P cm occurs during a storm period of t_R hours, then the design intensity i $(= i_c)$ can be obtained from the equation

$$i_c = \frac{P}{t_R}\left(\frac{t_R + 1}{i_c + 1}\right)$$

When the time of concentration, t_c is not known, $i_c = P/t_R$

2. Empirical Method: The empirical formula used for the estimation of the flood peak are essentially regional formula based on statistical correlation of the observed peak and important catchment properties.

1. Dickens formula for moderate size basins of north and central India, $Q = C\,A^{2/3}$
C = Dickens constant with value between $6 - 30$

2. Ryves formula derived from a study of rivers in south India
$Q = C\,A^{2/3}$
Coefficient $C = 6.8$ within 80 km of coast
$C = 8.3$ for areas between 80 and 2400 km from the coast
$C = 10.0$ for limited area near the hills

3. Inglis formula for fan-shaped catchments of Bombay state (Maharashtra)

$$Q = \frac{124\,A}{\sqrt{A + 10.4}}$$

4. Myers formula $Q = 175\sqrt{A}$

In all the above formula, Q is the peak flood in cumec and A is the area of the drainage basin in km^2.

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3. Unit hydrograph method: The unit hydrograph technique used to predict the peak flood hydrograph if the rainfall producing the flood, infiltration characteristics of the catchment and the appropriate unit hydrograph are available. For design purposes, extreme rainfall situations are used to obtain the design storm, viz. the hydrograph of the rainfall excess extreme floods. The known or derived unit hydrograph of the catchment is then operated upon by the design storm to generate the desired flood.

4. Flood frequency analysis: The values of the annual maximum flood from a given catchment area for large number of successive years constitute a hydrologic data series called the annual series. The data are then arranged in decreasing order of magnitude and the probability P of each event being equaled to or exceeded is calculated by the plotting position formula,

$$P = \frac{m}{N+1}$$

Where, m = order number of the event and N = total number of events in the data. The recurrence interval, T (also called the return period or frequency) is calculated as,

$$T = \frac{1}{P}$$

The probability of occurrence of the event r times in n successive years is,

$$P_m = nCr\, P^r\, q^{n-r} = \frac{n!}{(n-r)!\,r!}\, p^r\, q^{n-r}$$

$$q = 1 - P$$

Chow has shown that most frequency distribution functions applicable in hydrologic studies can be expressed by the following equations known as the general equation of hydrologic frequency analysis,

General equation of hydrological frequency analysis, $x_T = \bar{x} + K\,\sigma$

x_T = value of the variate X of a random hydrological series with a return period T

$\bar{x}$ = mean of the variate

σ = standard deviation of the variate

K = frequency factor which depends on the the return period T and frequency distribution

Runoff coefficient(C): The coefficient C represents the integrated effect of the catchment losses and hence depends upon the nature of the surface, surface slope and rainfall intensity.

Equivalent runoff coefficient, $C_e = \frac{\sum C_i A_i}{A}$

Question: Determine designed discharge for a combined system serving population of 50000 with rate of water supply of 135 LPCD. The catchment area is 100 hectares and the average coefficient of runoff is 0.60. The time of concentration for the design rainfall is 30 min and the relation between intensity of rainfall and duration is $I = 1000/(t+20)$

Solution:

Estimation of sewage quantity

Considering 80% of the water supplied will result in wastewater generation.

The quantity of sanitary sewage = 50000 x 135 x 0.80 = 5400 m^3/day = 0.0625 m^3/s

Considering peak factor of 2.5,

The design discharge for sanitary sewage = 0.0625 x 2.5 = 0.156 m^3/s

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Estimation of storm water discharge

Intensity of rainfall, $I = \frac{1000}{(t+20)} = \frac{1000}{(30+20)} = 20$ mm/h

Storm water discharge, $Q = \frac{C\,I\,A}{360} = \frac{0.6 \times 20 \times 100}{360} = 3.33$ m^3/s

Design discharge for combined sewer = 3.33 + 0.1656 = 3.49 m^3/s

Question: Write down the four types of flood normally occurs in Bangladesh. (BWDB – 2014)

Solution:

There are several types of floods that have recently occurred regularly, affecting different areas in their own distinct way. These flood types include:

- Flash floods in hilly areas
- Monsoon floods during monsoon season
- Normal bank floods from the major rivers,
- Rain-fed floods.

Question: An urban catchment has are of 8 ha, 17 ha, 50 ha, 10 ha for different purposes. If the corresponding runoff co-efficient are 0.7, 01, 0.3 and 0.8 respectively find the equivalent runoff coefficient. (BUET M. Sc. –2017)

Solution:

Equivalent runoff coefficient, $C_e = \frac{\sum C_i A_i}{A}$

$$C_e = \frac{[(0.7 \times 8) + (0.1 \times 17) + (0.3 \times 50) + (0.8 \times 10)]}{(8 + 17 + 50 + 10)} = 0.36$$

Question: Determine the rainwater availability for a roof area of 5 cm x 5 cm, intensity 25 cm/year and runoff coefficient C is 0.70. (WASA – 2014)

Solution:

Here, $A = 25$ cm$^2 = 2.5 \times 10^{-3}$ m^2, $I = 25$ cm/year = 0.25 m/year

We know, $Q = C\,I\,A = 0.70 \times 0.25 \times 2.5 \times 10^{-3} = 4.375 \times 10^{-4}$ m^3/year

Question: Rainfall intensity is 2.4 m/year. Runoff coefficient is 0.7, Find the minimum catchment area for the daily water demand is 15 lpcd of 7 people. (BPDB – 2016, CPGCBL – 2018)

Solution:

Here, $Q = q\,N = 15 \times 7 \times 365 = 38325$ l/year = 38.325 m^3/year

We know, $Q = C\,I\,A$

$38.325 = 0.7 \times 2.4 \times A$

$A = 22.8$ m^2

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Question: Calculate the discharge of rainwater from a paved surface of 100m² when rainfall intensity is 1.5 cm per hour. Assume coefficient of runoff 0.9. (NHA – 2019)

Solution:

Here, $A = 100\ m^2 = 10^{-4}\ km^2, I = 1.5\ cm/hr = 15\ mm/hr$

We know, $Q = 0.278\ C\ I\ A = 0.278 \times 0.90 \times 15 \times 10^{-4} = 3.753 \times 10^{-4}\ m^3/s$

Question: Land area is 62 hector and underlying bed is impermeable, so no infiltration happened. Precipitation is 2 cm/hr. what is the volume of water storage in 1 hour? (DNCC – 2014)

Solution:

$$\text{Volume of water} = \frac{2}{100} \times 62 \times 10000 = 12400\ m^3$$

Question: The value of total direct runoff volume of catchment $6 \times 10^3\ m^3$. If rainfall depth is 8 cm and 0.5 is the runoff co-efficient. Determine the catchment area in km^2. (BWDB – 2016)

Solution:

Runoff depth = Runoff coefficient x Rainfall depth = $0.5 \times 8 = 4$ cm

$$\text{Catchment area} = \frac{\text{Direct runoff volume}}{\text{Runoff depth}} = \frac{6 \times 10^3}{4/100} = 150000\ m^2 = 0.15\ km^2$$

Return period: A return period is an average time or an estimated average time between events such as earthquakes, floods, landslides, or a river discharge flows to occur. Return period expressed as T.

$$\text{Return period, } T = \frac{1}{P}$$

$$\text{Probability, } P = \frac{1}{T}$$

$$\text{Probability of not happening, } q = 1 - P$$

Flood frequency studies

General equation of hydrological frequency analysis, $x_T = \bar{x} + K\,\sigma$

x_T = value of the variate X of a random hydrological series with a return period T

$\bar{x}$ = mean of the variate

σ = standard deviation of the variate

K = frequency factor which depends on the the return period T and frequency distribution

$$y_T = -\left[\ln . \ln \frac{T}{T-1}\right]$$

y_T = the value of y, commonly called the reduced variate for a given T

The variate X with a return period T, $x_T = \bar{x} + K\,\sigma$

$$\text{Where, } K_T = \frac{y_T - \overline{y_n}}{S_n} = \frac{y_T - 0.577}{1.2825}$$

$\overline{y_n}$ = reduced mean, a function of sample size N; for $N \to \infty, \overline{y_n} \to 0.577$

S_n = reduced standard deviation, a function of sample size N; for $N \to \infty, \overline{y_n} \to 1.2825$

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Question: Flood frequency computations for river at certain location by using Gumbel's distribution yielded. Peak flood is 40809 m^3/s and 46300 m^3/s for 50 year and 100 year return period respectively. Estimate the design flood magnitude for return period 500 year. Also determine the design scouring depth using Lacy's regime equation if d_{50} of bed material is 0.005 cm. (BWDB – 2020)

Solution:

$x_{100} = \bar{x} + K_{100}\,\sigma_{n-1}$ and $x_{50} = \bar{x} + K_{50}\,\sigma_{n-1}$

$(K_{100} - K_{50})\,\sigma_{n-1} = x_{100} - x_{50} = 46300 - 40809 = 5491$

$But,\ K_T = \frac{y_T}{S_n} - \frac{\overline{y_n}}{S_n}$

Where, S_n and $\overline{y_n}$ are constants for the given data series.

$(y_{100} - y_{50})\,\frac{\sigma_{n-1}}{S_n} = 5491$

$y_{100} = -\left[\ln.\ln\frac{T}{T-1}\right] = -\left[\ln.\ln\frac{100}{100-1}\right] = 4.60015$

$y_{50} = -\left[\ln.\ln\frac{T}{T-1}\right] = -\left[\ln.\ln\frac{50}{50-1}\right] = 3.90194$

$\frac{\sigma_{n-1}}{S_n} = \frac{5491}{(4.60015 - 3.90194)} = 7864$

$y_{500} = -\left[\ln.\ln\frac{T}{T-1}\right] = -\left[\ln.\ln\frac{500}{500-1}\right] = 6.21361$

$(y_{500} - y_{100})\,\frac{\sigma_{n-1}}{S_n} = x_{500} - x_{100}$

$(6.21361 - 4.60015)\ x\ 7864 = x_{500} - 46300$

$x_{500} = 58988\ m^3/s$

Question: The mean annual flood of a river is 600 m^3/s and the standard deviation of the annual flood time series is 150 m^3/s. what is the probability of a flood of magnitude 1000 m^3/s occurring in the river within next 5 years? Use Gumbel's method and assume the sample size to be very large.

Solution:

$\bar{x} = 600\ m^3/s$ and $\sigma_{n-1} = 150\ m^3/s$

$x_T = \bar{x} + K\,\sigma_{n-1}$

$K = \frac{x_T - \bar{x}}{\sigma_{n-1}} = \frac{1000 - 600}{150} = 2.6667$

$K = \frac{y_T - 0.577}{1.2825}$

$y_T = 2.6667 \times 1.2825 + 0.577 = 3.9970$

$y_T = -\left[\ln.\ln\frac{T}{T-1}\right]$

$\frac{T}{T-1} = 1.01854$

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$T = 54.9$ years say 55 years

Probability of occurrence of a flood of magnitude 1000 $m^3/s = p = \frac{1}{55} = 0.0182$

The probability of a flood of magnitude 1000 m^3/s occurring at least once in 5 years,

$p_1 = 1 - (1-p)^5 = 1 - (0.9818)^5 = 0.0877 = 11.4\%$

Risk: The probability of occurrence of an event at least once over a period of n successive years is called risk, $R = 1 - q^n$

$$\bar{R} = 1 - (1-P)^n = 1 - \left(1 - \frac{1}{T}\right)^n$$

Reliability: The probability of occurrence of an event not happening at least once over a period of n successive years is called Reliability.

$$R_e = 1 - \bar{R} = \left(1 - \frac{1}{T}\right)^n$$

Question: A bridge has an expected life of 25 years and is designed for a flood magnitude of return period 100 years, (a) what is the risk of this hydrologic design? (b) If a 10% risk is acceptable, what return period will have to be adopted? (WRE – 2015)

Solution:

Here, $n = 25$ years and $T = 100$ years

$$\bar{R} = 1 - (1-P)^n = 1 - \left(1 - \frac{1}{T}\right)^n = 1 - \left(1 - \frac{1}{100}\right)^{25} = 0.222$$

Hence the inbuilt risk in this design is 22.2%

If $\bar{R} = 10\% = 0.10$

$$0.10 = 1 - \left(1 - \frac{1}{T}\right)^{25}$$

$T = 238$ years say 240 years

Hence to get 10% acceptable risk, the bridge will have to be designed for a flood of return period $T = 240$ years.

Question: Analysis of annual flood series of a river yielded a sample mean of 1000 m^3/s and standard deviation of 500 m^3/s. Estimate the design flood of a structure on this river to provide 90% assurance that the structure will not fail in the next 50 years. Use Gumbel's method and assume the sample size to be very large.

Solution:

$\bar{x} = 1000$ m^3/s and $\sigma_{n-1} = 500$ m^3/s

Reliability, $R_e = \left(1 - \frac{1}{T}\right)^n$

$$0.90 = \left(1 - \frac{1}{T}\right)^{50}$$

$$1 - \frac{1}{T} = (0.90)^{1/50} = 0.9978$$

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$T = 475$ years

$$y_T = -\left[\ln.\ln\frac{T}{T-1}\right] = -\left[\ln.\ln\frac{475}{475-1}\right] = 6.1622$$

$$K = \frac{y_T - 0.577}{1.2825} = \frac{6.1622 - 0.577}{1.2825} = 4.355$$

$$x_T = \bar{x} + K\,\sigma_{n-1} = 1000 + 4.355 \times 500 = 3177 \text{ m}^3/\text{s}$$

Question: Describe the different engineering measures to protect flood prone areas. (33th BCS)

Solution:

The damages due to the devastating floods can be minimised by the following flood control measures, singly or in combination.

(*i*) by confining the flow between high banks by constructing levees (flood banks) , dykes, or flood walls.

(*ii*) by channel improvement by cutting, straightening or deepening and following river training works.

(*iii*) by diversion of a portion of the flood through bypasses or flood ways. In some cases a fuse plug levee is provided. It is a low section of levee, which when once over topped, will wash out rapidly and develop full discharge capacity into the flood-way. In other locations, a concrete sill, weir or spillway controlled by stop logs or needles may be provided so that the overflow occurs at a definite river stage. Sometimes dynamiting a section of levee is resorted to bypass the flood.

(*iv*) by providing a temporary storage of the peak floods by constructing upstream reservoirs and retarding basins (detention basins).

(*v*) by adopting soil conservation measures (land management) in the catchment area.

(*vi*) by temporary and permanent evacuation of the flood plain, and flood plain zoning by enacting legislation.

(*vii*) by flood proofing of specific properties by constructing a ring levee or flood wall around the property.

(*viii*) by setting up flood forecasting—short term, long term, rhythm signals and radio, and warning centres at vulnerable areas.

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Question: Explain the principal causes of flood and possible remedial measures for flood mitigation. (38th BCS)

Solution:

There are plenty of different causes of flooding, most floods are caused by one of the following activities.

1. Heavy rainfall is the simplest cause of flooding. When there is too much rain or it happens too fast, there just isn't a place for it to go. This can result in floods like flash flooding.

2. Overflowing rivers are another cause of floods. You don't necessarily need heavy rains though to experience river flooding. River flooding can happen when there is debris in the river or dams that block the flow of the water.

3. Speaking of dams, broken dams are another cause of flooding. Older infrastructure can fail when heavy rains come and water levels rise. When dams break, they unleash torrents of water on unsuspecting households.

4. Storm surge and tsunamis also cause flooding. Storm surges from hurricanes and other tropical systems can cause sea levels to rise and cover normally dry coastal areas in several feet of water. Tsunamis on the other hand are giant waves caused by earthquakes or underwater volcanic eruptions. As these waves move inland, they build height and can push a lot of water inland in coastal areas.

5. Channels with steep banks are also to blame for flooding. Flooding often occurs when there is fast runoff into lakes, rivers, and other basins. This is often the case with rivers and other channels that feature steep sides.

6. Melting snow and ice is another common reason for flooding. When a large amount of snow and/or ice melts quickly, it often doesn't have somewhere to go except low-lying areas.

Possible remedial measures for flood mitigation

1. Introduce better flood warning systems
2. Modify homes and businesses to help them withstand floods
3. Construct buildings above flood levels
4. Tackle climate change
5. Increase spending on flood defences
6. Protect wetlands and introduce plant trees strategically
7. Restore rivers to their natural courses
8. Introduce water storage areas
9. Put up more flood barriers

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Water supply engineering

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Water Supply Engineering

Water supply system is a network of pipelines of various sizes with control valves for carrying water to all streets and supplying water to the consumers.

Water Supply Engineering is the branch of civil engineering that deals with the supply of safe water for various purposes (e.g. domestic, industrial, commercial and public supply) in sufficient amount with efficiently.

Objectives of Water Supply System

Main Objective: To bring water from source and supply safe water to the consumers in adequate quantity efficiently at low cost.

An organized water supply system has following objectives:

→ To supply safe and wholesome water.

→ To supply reliable and adequate water.

→ To make water available within easy reach of the consumers.

→ To reduce the water related diseases by obtaining general cleanliness.

Typical Components of Water Supply

(i) Transmission Line (main): Pipe line from intake to reservoir tank.

(ii) Distribution Line: Pipe line from reservoir tank to tap stand.

Question: Describe elements of water supply system. (37th BCS)

Solution:

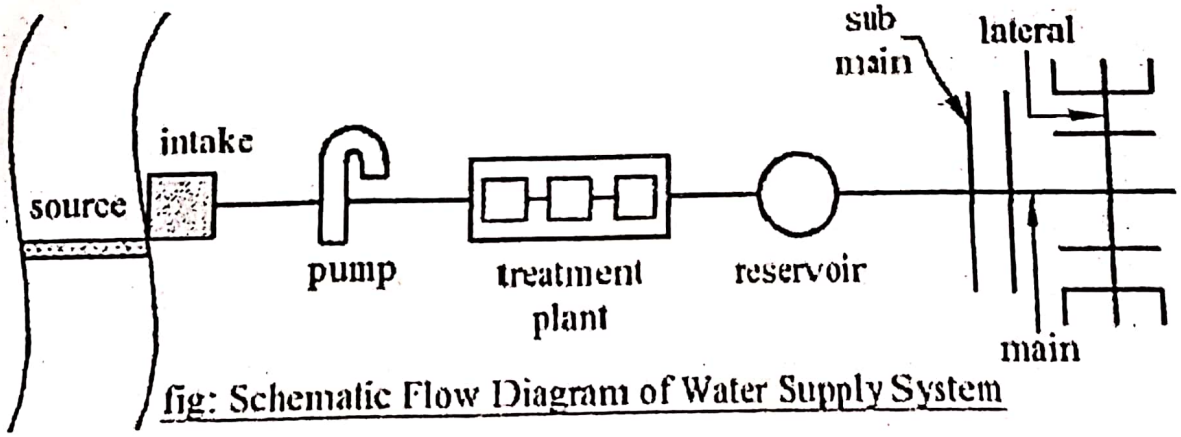

fig: Schematic Flow Diagram of Water Supply System

Following are the major components of water supply system:

1) Collection work: It includes intake works and storage. Depending upon river, a dam and storage may be constructed or direct intake can be made.

2) Transmission work: In many cases water supply schemes are far away from the city and should be conveyed to it through transmission works. It is the connection between collection and purification works through conduits, pipes, canal, aqueduct, etc.

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3) Purification work: The water collected directly from the source may not be safe for drinking because of different impurities. Aim of purification is to supply pure drinking water to the consumers.

Common components of purification works are:
(a) Screening (b) Sedimentation (c) Coagulation (d) Disinfection (e) Aeration

4) Distribution system: The treated and purified water is finally sent to consumers through suitable distribution system. In order to flow water in supply pipes under pressure, the purified water is normally stored in an elevated reservoir. More reservoirs may be needed in large systems.

Types of distribution systems:
1. Dead End or Tree System 2. Grid Iron System 3. Ring System 4. Radial System

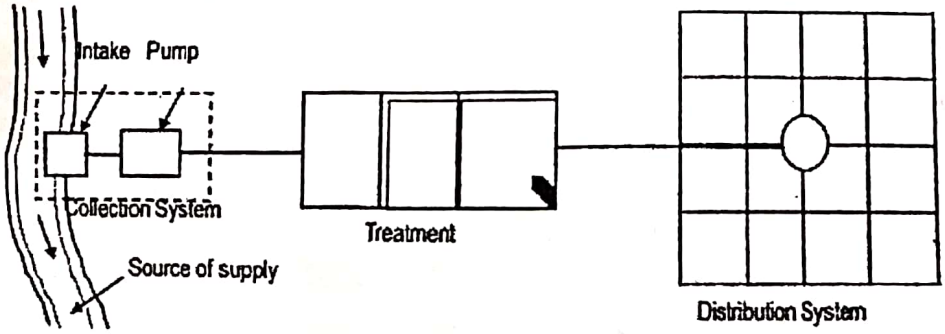
Requirements of Good Distribution System

1. Water quality should not get deteriorated in the distribution pipes.
2. It should be capable of supplying water at all the intended places with sufficient pressure head.
3. It should be capable of supplying the requisite amount of water during firefighting.
4. The layout should be such that no consumer would be without water supply, during the repair of any section of the system.
5. All the distribution pipes should be preferably laid one meter away or above the sewer lines.
6. It should be fairly water-tight as to keep losses due to leakage to the minimum.

Question: Draw the flow diagram of essential elements of water supply system.
(PGCL – 2017, BPDB – 2018, DESCO – 2019, WASA – 2017, HED – 2020, TGTDCL – 2021)

Solution:

1. Source of water
2. Collection system
3. Treatment system
4. Distribution system



Water Demand: While designing the water supply scheme for a town or city, it is necessary to determine the total quantity of water required for various purposes of the city.

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Water Quantity Estimation:

Per capita demand: It is the average quantity of water required per person per day for domestic purpose is called per capita demand. This is obtained by dividing the total quantity of water required by total population. The unit is lpcd (liters per capita demand of water).

$$\text{Per capita demand} = \frac{\text{Total yearly water requirement of the city in liters}}{365 \times \text{Design population}}$$

Total water demand = Per capita demand x Population of community
The unit of total water demand is liters/day.

Question: Define per capita consumption of water. What are the factors affecting per capita consumption of water? (33th BCS)

Solution:

It is the average quantity of water required per person per day for domestic purpose is called per capita demand. This is obtained by dividing the total quantity of water required by total population. The unit is lpcd (liters per capita demand of water).

The factors are -

- Size of the city
- Characteristic of the people
- Climate condition
- Commerce and industries
- Pressure of water
- Quality of water
- Sewage facilities
- Water rates
- Nature of supply
- Availability of private supply
- Efficiency of the management
- Number of inhabitants.

Question: What are the factor affecting per capita consumption of water?
(PGCB – 2015, 47 BMA)

Solution:

The factors are -

- Size of the city
- Characteristic of the people
- Climate condition
- Commerce and industries
- Pressure of water
- Quality of water
- Sewage facilities
- Water rates
- Nature of supply

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- Availability of private supply
- Efficiency of the management
- Number of inhabitants.

Fire Demand: Provision is made in the water works to supply sufficient quantity of water or to keep as reserve in the water main for the purpose of firefighting.

Fire Hydrants: A fire hydrant is a connection point by which fire-fighters can tap into a water supply. It is a component of active fire protection. In most cities fire hydrants are provided on the water mains at 100–150 m. apart for purpose of firefighting. The minimum water pressure at fire hydrant should be 1 – 1.5 kg/cm².

Authority	Formula
National board of fire underwriters Formula	$Q = 4637\sqrt{P}\,(1 - 0.001\sqrt{P})$
Freeman's Formula	$Q = 1136\,(P/5 + 10)$
Kuichling's Formula	$Q = 3182\sqrt{P}$
Buston's Formula	$Q = 5663\sqrt{P}$

Where, Q = Amount of water required in liters/minute, P = Population in thousands

Population Forecasting: When the design period is fixed, the next step is to determine the population in various periods, because the population of the towns generally goes on increasing.

The population forecasting is done by following standard methods:

- Arithmetic increase method
- Geometrical increase method
- Incremental increase method
- Decrease rate of growth method
- Graphical extension method
- Graphical comparison method

Arithmetic increase method

$$P_f = P_0 + n\,I$$

Where, P_f = future population at the end of n decades
P_0 = present population
n = number of decades
I = average increment for a decade

Geometrical increase method

In this method, it is assumed that percentage increase in population from decade to decade is constant.

$$P_f = P_0\left(1 + \frac{r}{100}\right)^n$$

$$\text{Growth rate, } r = \sqrt[n]{\frac{P_f}{P_0}} - 1$$

Where, r = per decade percentage increase or percentage growth rate

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P_f = future population
P_0 = present population
n = No of decades between P_0 and P_f
The average growth rate may be calculated either arithmetic average or geometric average.

Arithmetic average $= \frac{r_1 + r_2 + r_3 + \ldots + r_t}{t}$

geometric average $= \sqrt[t]{r_1 . r_2 . r_3 \ldots r_t}$

Question: A water supply scheme has to be designed for a city having a population of 100000. Estimate the important kinds of drafts which may be required to be recorded for an average water consumption of 250 lpcd. Also record the required capacities of the major components of the proposed water works system for the city using a river as the source of supply. Assume suitable figures and data where needed.

Solution:

Average daily draft = per capita demand x population

= 250 x 100000 litres/day = 25 Mld

Maximum daily draft = 1.8 x 25 = 45 Mld

Fire flow, $Q = 4637\sqrt{P}\left(1 - 0.001\sqrt{P}\right)$

Using populations in thousands, P = 100 thousands

$Q = 4637\sqrt{100}\left(1 - 0.001\sqrt{100}\right) = 71733$ litres/minute

$Q = \frac{41733 \text{ x } 60 \text{ x } 24}{10^6}$ million litres/day = 61 Mld

Coincident draft = Maximum daily draft + Fire draft = 45 + 61 = 106 Mld

Question: The total population of an area was 7000 in 2015. (*i*) Estimate the population of the area for the year 2020 (considering annual growth rate 1%) (*ii*) Determine the amount of water required for firefighting for the same area in 2025. (37th BCS)

Solution:

(i) Population in 2020, $P_{2020} = P_{2015}\left(1 + \frac{r}{100}\right)^n = 7000\,(1 + 0.01)^5 = 7357$

(ii) Fire flow, $Q = 4637\sqrt{P}\left(1 - 0.001\sqrt{P}\right)$

Population in 2025, $P_{2025} = P_{2015}\left(1 + \frac{r}{100}\right)^n = 7000\,(1 + 0.01)^{10} = 7732$

Using populations in thousands, P = 7.732 thousands

$Q = 4637\sqrt{7.732}\left(1 - 0.001\sqrt{7.732}\right) = 12858$ litres/minute

$Q = \frac{12858 \text{ x } 60 \text{ x } 24}{10^6}$ million litres/day = 18.51 Mld

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Question: Compute the population of the year 2000 and 2006 for a city whose population in the year 1930 was 25000 and in the year 1970 was 47000. Make use of geometric increase method.

Solution:

Growth rate, $r = \sqrt[4]{\frac{P_{1970}}{P_{1930}}} - 1 = \sqrt[4]{\frac{47000}{25000}} - 1 = 0.17095 = 17.095\%$ per decade

Now, $P_{2000} = P_{1970}\left(1 + \frac{r}{100}\right)^n = 47000\,(1 + 0.17095)^3 = 75459$

Population for the year 2006 means that it is after 36 years (3.6 decades) from 1970 onward,

$P_{2006} = P_{1970}\left(1 + \frac{r}{100}\right)^n = 47000(1 + 0.17095)^{3.6} = 82954$

Problem: Calculate the Peak water demand of rural community having present population of 500 & design period of 5 years. The average per capita water consumption 50 lpcd with a peak factor of 3. Population growth rate 2% and wastage 20%.

Solution:

Population, $P_0 = 500$ nos.
Design period, $n = 5$years
Population growth rate, $r = 2\%$
Wastage, $w = 20\%$.

Population after 5 year, $P_5 = P_0\left(1 + \frac{r}{100}\right)^n = 500\left(1 + \frac{2}{100}\right)^5 = 552$

Water demand, $Q = \frac{\text{Per capita consumption x Population}}{1 - 0.01\,\text{Wastage}} = \frac{50 \times 552}{1 - 0.01 \times 20} = 34500$ L/day

Peak water demand, Q = water demand x peak factor $= 34500 \times 3 = 103500$ L/day

$Q = 103.50\,m^3/day$

Question: The population of 5 decades from 1930 to 1970 are given below in table. Find out the population after one, two and three decades beyond the last known decade by using geometric increase method.

Year	1930	1940	1950	1960	1970
Population	25000	28000	34000	42000	47000

Solution:

Year	Population	Increase in population in each decade	Percentage increase in population growth rate (r)
1930	25,000		
1940	28,000	3000	$\frac{3000}{25000} \times 100 = 12\%$
1950	34,000	6000	$\frac{6000}{28000} \times 100 = 21.4\%$
1960	42,000	8000	$\frac{8000}{34000} \times 100 = 23.5\%$
1970	47,000	5000	$\frac{5000}{42000} \times 100 = 11.9\%$

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The geometric mean of the growth rates,

$r = \sqrt[4]{r_1 \times r_2 \times r_3 \times r_4} = \sqrt[4]{12 \times 21.4 \times 23.5 \times 11.9} = 16.37\%$ per decade

Assuming future population increases at this constant rate,

$P_f = P_0 \left(1 + \frac{r}{100}\right)^n = 47000\ (1 + 0.1637)^n = 47000\ (1.1637)^n$

Population after 1 decade, $P_{1980} = 47000\ (1.1637)^1 = 54694$

Population after 2 decade, $P_{1990} = 47000\ (1.1637)^2 = 63647$

Population after 3 decade, $P_{2000} = 47000\ (1.1637)^2 = 74066$

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Head, Power and Efficiency of Pumps

The total lift of pump or the total head against which the pump must operate is the algebraic some of the following quantities. The discharge lift or the vertical distance from the centre of the pump to the level to which the water is to be raised. The suction lift or the vertical distance from the level of the source of water supply to the centre of the pump, the friction head and the minor losses in the discharge and suction pipes, the velocity head.

Total head (H) = Suction head (H_s) + Delivery head (H_d) + Head loss (H_L)

Power required for the pump in lifting Q m³/sec of water by a head, $HP = \gamma_w Q H$

Where, γ_w = unit weight of water = 9.81 KN/m³

1 HP = 746 watt = 0.746 kW

Water horse power of the pump is given as:

$$WHP = \frac{\gamma_w Q H}{0.746}$$

If η is the efficiency of the pump set, then the Brake Horse Power (BHP) of the pump is:

$$BHP = \frac{\gamma_w Q H}{0.746\,\eta}$$

For FPS unit, $BHP = \dfrac{\gamma_w Q H}{550 \text{ x efficiency}}$

Q = discharge in ft³/s (cfs)
γ_w = Unit weight of water in lb/ft³
H = Total lift in feet
550 = Conversion factor from ftlb/s to HP

The actual horsepower needed is determined by dividing the theoretical horsepower by the efficiency of the pump and driving unit. The efficiencies for centrifugal pumps normally range from 50 to 85 percent. The efficiency increases with the size and capacity of the pump.

Water horsepower, $WHP = \dfrac{H Q}{3960}$

Brake horsepower, $BHP = \dfrac{H Q}{3960 \text{ x efficiency}}$

Q = volume of water to be pumped in gpm
H = Total lift or head of the pump in feet

Frictional loss in the pipe, $h_f = \dfrac{f' L V^2}{2 g d}$ (Where, $f' = 4 f$)

h_f = Head loss due to friction in a pipe of diameter d and Length L
f' = friction factor which is generally taken 0.024 – 0.03
f = friction coefficient

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Question: Design a suitable set of pumping unit to deliver 450000 gph from an intake well of a river bank to the treatment plant. Total length of rising main from the intake well to the treatment plant is 800 ft and the static head is 60 ft. Design also the cast iron main. Velocity of water = 12 fps, friction factor = 0.03 and efficiency = 70%.

Solution:

$Q = 450000 + 60 = 7500$ gpm

Again, $Q = \frac{450000}{60 \times 60 \times 6.24} = 20$ cfs

Cross – sectional area, $A = \frac{Q}{V} = \frac{20}{12} = 1.667\ ft^2$

$\frac{\pi d^2}{4} = 1.667$ (d = diameter of pipe)

$d = \sqrt{\frac{1.667 \times 4}{\pi}} = 1.5\ ft = 18$ inch

Frictional head loss, $h_f = \frac{f' l v^2}{2 g d} = \frac{0.03 \times 800 \times 12^2}{2 \times 32.2 \times 1.5} = 36$ ft

Velocity head, $h_v = \frac{V^2}{2g} = \frac{12^2}{2 \times 32.2} = 2.24$ ft

Total head, $h_s + h_f + h_v = 60 + 36 + 2.24 = 98.24$ ft

$WHP = \frac{H Q}{3960} = \frac{98.24 \times 7500}{3960} = 188$

$BHP = \frac{WHP}{\eta} = \frac{188}{0.7} = 265$

Question: A water treatment plant pumps its raw water from a reservoir next to the plant. The intake is 12 ft below the lake water surface at elevation 588 ft. The lake water is pumped to the plant influent at elevation 611 ft. Assume the suction head loss for the pump is 10 ft and the loss of head in the discharge line is 7 ft. The overall pump efficiency is 72 percent. The plant serves 44,000 persons. The average water consumption is 200 gallons per capita per day (gpcpd). Compute the horsepower output of the motor.

Solution:

Discharge, $Q = 44000 \times 200 = 8800000$ gpcpd $= 8.8$ Mgal/day

$= 8.8 \times 1.547 = 13.6\ ft^3/s$

Effective head, $H = (611 - 588) + 10 + 7 = 40$ ft

$HP = \frac{\gamma_w Q H}{550 \times \eta} = \frac{62.4 \times 13.6 \times 40}{550 \times 0.72} = 85.7$ hp

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Question: For water supply of a town, water is pumped from a river 2.3 km away into a reservoir. The maximum difference level of water in river and the reservoir is 20 m. The population of the town is 50,000 and per capita water demand is 120 liter per day. The pump is to operate for a total of 8 hours and the efficiency of the pump is 70%. Assume friction coefficient is 0.0075, the velocity of flow is 2 m/s and maximum daily demand is 1.5 times the average daily demand. Find out horse power output of the motor. (Combined Bank – 2019)

Solution:

Average daily demand = $50000 \times 120 = 6 \times 10^6$ L/day

Maximum water demand = $1.5 \times 6 \times 10^6 = 9 \times 10^6$ L/day

Since total pumping time is 8 hours/day, the capacity of pumps should be designed for a daily demand.

$$Q = \frac{24}{8} \times 9 \times 10^6 = 2.7 \times 10^7 \text{ L/day}$$

Discharge required per second, $Q = \dfrac{2.7 \times 10^7}{10^3 \times 24 \times 60 \times 60} = 0.3125 \text{ m}^3/s$

Area required, $A = \dfrac{Q}{V} = \dfrac{0.3125}{2} = 0.156 \text{ m}^2$

$\dfrac{\pi d^2}{4} = 0.156$ (d = diameter of pipe)

Diameter of the main required, $d = \sqrt{\dfrac{0.156 \times 4}{\pi}} = 0.445$ m

Head loss due to friction, $H_f = \dfrac{f' L V^2}{2 g d} = \dfrac{4 f L V^2}{2 g d}$ (Where, $f' = 4f$)

$$H_f = \frac{4 \times 0.075 \times 2.3 \times 1000 \times 2^2}{2 \times 9.81 \times 0.445} = 31.61 \text{ m}$$

Total lift, $H = 20 + 31.61 = 51.61$ m

$$P = \frac{\gamma_w H Q}{\eta \times 0.746} = \frac{9.81 \times 51.61 \times 0.3125}{0.70 \times 0.746} = 302.98 \ (BHP)$$

Question: Design the pumping unit to delivery 23700 gph of water from a well to an overhead tank against a static head of 60 ft. The total loss head due to friction, velocity etc. is estimated to be 40 ft. (PGCL – 2014, DNCC – 2016)

Solution:

Total pump head, $H = 60 + 40 = 100$ ft

Flow, $Q = 23700$ gph $= 395$ gpm

Water horsepower, $WHP = \dfrac{H Q}{3960} = \dfrac{100 \times 395}{3960} = 9.97$ WHP

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Question: You have planned to irrigate a garden by a dug well using a centrifugal pump at a rate of 10 gpm through 1 in plastic pipe. At the lowest point, the water level is 50 ft below the garden level. You will need a 75 ft long plastic pipe (including horizontal length). According to the chart, the frictional head loss per 100 ft length of 1 inch plastic pipe is 6.3 ft and the same one 90 degree below the connection loss is 16.655ft. Calculate Water horsepower and motor horse power for motor efficiency 0.5. (NPCBL – 2019)

Solution:

$$\text{Friction loss} = 75 \times \frac{6.3}{100} + 16.665 = 21.39 \text{ ft}$$

$$\text{Total loss, } H = 21.39 + 50 = 71.39 \text{ ft}$$

$$\text{Water horsepower, } WHP = \frac{H\,Q}{3960} = \frac{71.39 \times 10}{3960} = 0.18 \text{ HP}$$

$$\text{Brake horsepower} = \frac{\text{Water horsepower}}{\text{efficiency}} = \frac{0.18}{0.5} = 0.36 \text{ HP}$$

Question: A pump is installed on a well to lift water and to irrigate rice crop, sown over 3 hectares of land. If duty for rice is 864 hectares/cumec on the field and pump efficiency id 84%; determine the minimum input (H.P.) required by the pump if the lowest water level is 8 meters below the highest portion of the field. Assume negligible field channel losses. (I H.P. = 0.735 k. watt) (40th BCS)

Solution:

$$\text{Discharge required} = \frac{\text{Area}}{\text{Duty}} = \frac{3}{864} = 3.47 \times 10^{-3} \text{ cumec}$$

$$\text{Minimum static lift of pump, } H = 8 \text{ m}$$

$$\text{The output of the pump } (H.P.) = \frac{\gamma_w\, H\, Q}{0.735} = \frac{9.81 \times 8 \times 3.47 \times 10^{-3}}{0.735} = 0.37$$

$$\text{Input } H.P. \text{ of the pump} = \frac{0.37}{\eta} = \frac{0.37}{0.84} = 0.44\ H.P.$$

Question: A city has a population of 1,50,000. Water is to be supplied at the rate of 160 liters per head per day. If the static lift of the pump is 40 meters, calculate the B.H.P of motor. The rising main is 300 m long and its diameter is 50 cm. Assume that motor efficiency is 85%, pump efficiency is 60%, f = 0.01 and the peak hour demand is 1.5 times the average demand.

Solution:

$$\text{Average demand} = 150000 \times 160 = 24 \times 10^6 \text{ liters/day} = 24000 \text{ m}^3\text{/day}$$

$$= \frac{24000}{24 \times 60 \times 60} = 0.278 \text{ m}^3\text{/s}$$

$$\text{peak hour demand} = 1.5 \times 0.278 = 0.417$$

$$\text{Static lift, } h_s = 40 \text{ m}$$

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Frictional head loss, $h_f = \frac{4 f l V^2}{2 g d} = \frac{f l Q^2}{3 d^5} = \frac{0.01 \times 300 \times 0.417^2}{3 \times 0.5^5} = 5.56$ m

Total head, $H = h_s + h_f = 40 + 5.56 = 45.56$ m

Overall efficiency, $\eta = 0.85 \times 0.6 = 0.51$

$B.H.P$ of motor $= \frac{\gamma_w Q H}{0.746 \eta} = \frac{9.81 \times 0.417 \times 45.56}{0.746 \times 0.51} = 489.86$

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Sources and Conveyance of Water

Raw Water Source: The various sources of water can be classified into two categories:

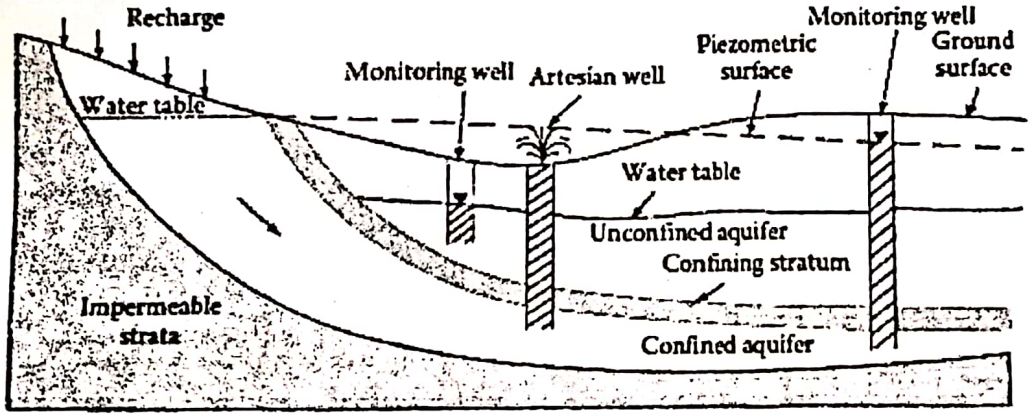
1. Surface sources, such as
   a. Ponds and lakes
   b. Streams and rivers
   c. Storage reservoirs
   d. Oceans, generally not used for water supplies, at present.
2. Sub-surface sources or underground sources, such as
   a. Springs
   b. Infiltration wells
   c. Infiltration Galleries
   d. Wells and Tube-wells.

Selection of Source of Water supply

The selection of source of water supply to a town depends upon the following factors:

- Location
- Quantity of water
- Quality of water
- Cost

Aquifer: A wet underground layer of water-bearing permeable rock or unconsolidated materials from which groundwater can be usefully extracted using a water well.



Unconfined/water table aquifer: Not confined by an upper confining layer. Upper layer is called the water table.

Confined/artesian aquifer: Water confined under a pressure greater than atmospheric by an overlying impermeable layer.

Aquicludes: Impervious formations.

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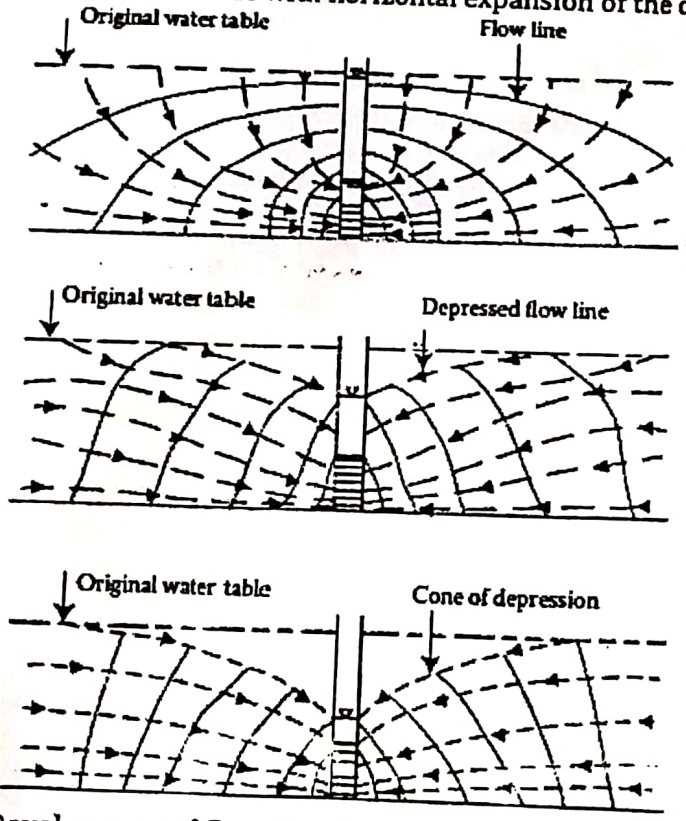
Wells: Water is removed from the aquifer surrounding the well during water pumping and the piezometric surface decreases. Instead of moving toward the natural discharge area, the groundwater within the influence of the pump flows toward the well from every direction. Drawdown is the distance by which the piezometric surface is lowered. The pumping well creates an artificial discharge area by drawing down (lowering) the water table around the well.

The conical-shaped depression of the water table around a pumping well is caused by the withdrawal of water; a valley in the water table. This area of drawdown is called the cone of depression. If pumping continues, more water must be derived from the aquifer storage at greater distances from the bore of the well and the following items are observed:

1. The cone of depression is expanded.
2. The radius of influence of the well increases due to the expansion of the cone.
3. The drawdown is incremented at any point with the increase in the depth of the cone to provide the additional head required to move the water from greater distance.
4. The cone expands and deepens more slowly with time since an increasing volume of stored water is available with horizontal expansion of the cone.



Development of flow distribution about a discharging well in an unconfined aquifer: (a) Initial stage (b) intermediate stage (c) steady-state stage.

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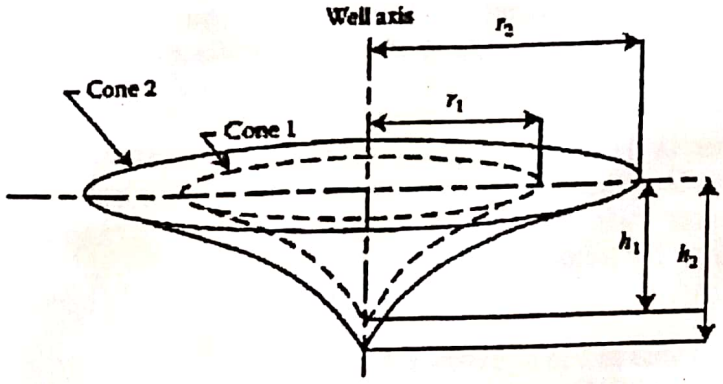


Figure: Changes in radius and depth of cone of depression.

Transmissibility, T

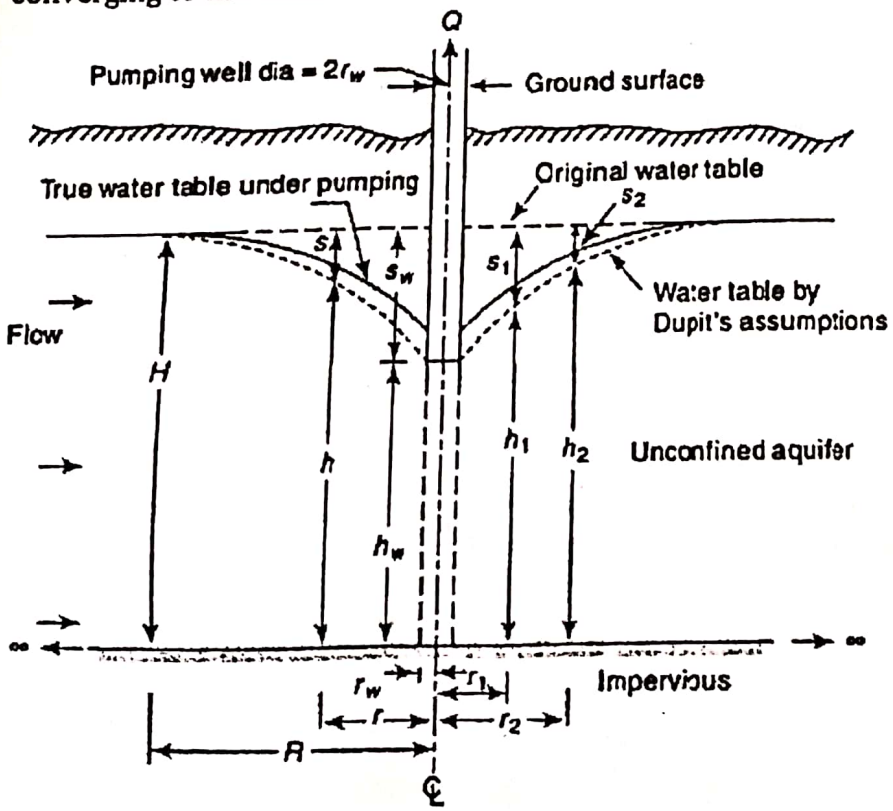
Consider an aquifer of unit width and thickness B. (i.e. depth of a fully saturated zone). The discharge through this aquifer under a unit hydraulic gradient is,

$T = K B$

This discharge is termed transmissibility, T and has the dimensions of $[L^2/T]$. Its units are m^2/s or liters per day/meter width (lpd/m)

Formula for Unconfined Aquifers

In unconfined groundwater systems, water of saturated zone is in open contact with atmospheric pressures. Groundwater release from storage occurs along permeable zones under the force of gravity as the aquifer responds to pumping. The streamline at the free surface of the aquifer is curved, and at the bottom of the aquifer it is horizontal lined, converging to the well.



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For unconfined aquifer, $Q = \frac{\pi K (h_2^2 - h_1^2)}{\ln \frac{r_2}{r_1}}$

At the edge of the zone of influence of radius R
$h_2 = H$ at $r_2 = R$ and $h_1 = h_w$ at $r_1 = r_w$

$$Q = \frac{\pi K (H^2 - h_w^2)}{\ln \frac{R}{r_w}}$$

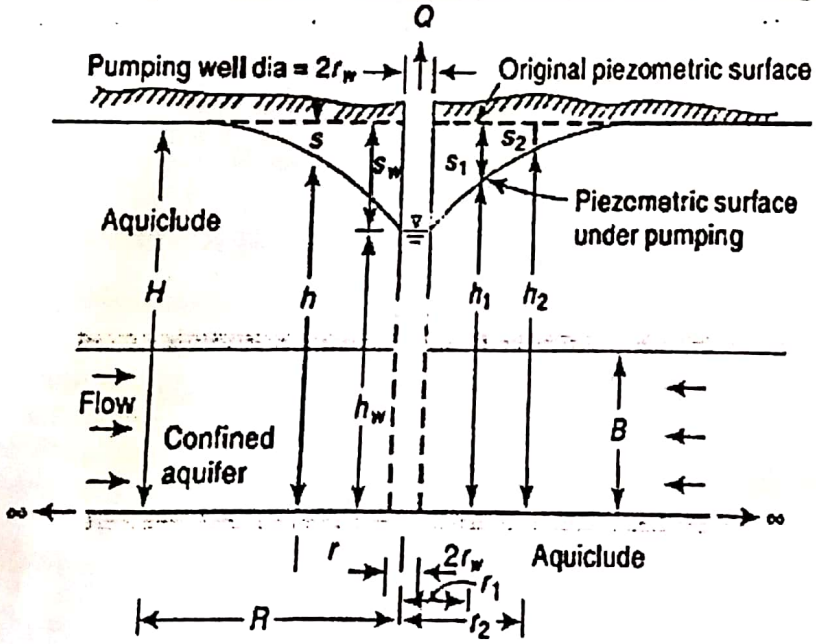
Radius of influence, $R = 3000\, S_w \sqrt{K}$
The radius of the circle of influence R varies directly as Q, for equilibrium conditions $R = C\,Q$

S_w = maximum drawdown in meters
K = hydraulic consuctivity in m/sec
If the drawdown at the pumping well $S_w = (H - h_w)$ is small relative to H then
$H^2 - h_w^2 = (H + h_w)(H - h_w) = 2\,H\,S_w$ and for unconfined aquifer, $T = K\,H$

$$Q = \frac{2\pi K H S_w}{\ln \frac{R}{r_w}} = \frac{2\pi T S_w}{\ln \frac{R}{r_w}}$$

Formula for Confined Aquifers

Due to discharge water from a completely confined aquifer by water pumping, drawdowns of the potentiometric surface occur. Water is released from storage and the fluid pressure decreases by the expulsion of water due to aquifer compaction under increased effective stress. The well discharge at any radial distance r from the pumped well is equal but by increasing the radial distance from the well, the area of discharging increases.



For confined aquifer, $Q = \frac{2\pi K B (h_2 - h_1)}{\ln \frac{r_2}{r_1}}$

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At the edge of the zone of influence of radius R
$h_2 = H$ at $r_2 = R$, $h_1 = h_w$ at $r_1 = r_w$ and for confined aquifer, $T = K\,B$

$$Q = \frac{2\pi K B (H - h_w)}{\ln \frac{R}{r_w}} = \frac{2\pi T (H - h_w)}{\ln \frac{R}{r_w}}$$

If the drawdown S_1 and S_2 at the observation wells are known,
$S_1 = H - h_1, S_2 = H - h_2$

$$Q = \frac{2\pi K B (S_1 - S_2)}{\ln \frac{r_2}{r_1}}$$

Interference among wells

If two or more wells are constructed in such a way that they are near to each other and their cones of depressions intersect, they are said to interfere. Such mutual interference of wells decreases the discharges of the interfering wells.

(a) For confined aquifers (Artesian wells)

(1) For two artesian identical wells at a distance D apart

$$Q_1 = Q_2 = \frac{2\pi K B (H - h_w)}{\ln \left(\frac{R^2}{r_w D}\right)}$$

(2) For three artesian identical wells at a distance D apart in a pattern of equilateral triangle

$$Q_1 = Q_2 = Q_3 = \frac{2\pi K B (H - h_w)}{\ln \left(\frac{R^3}{r_w D^2}\right)}$$

(3) For three artesian identical wells at a distance D apart in a straight line

$$Q_1 = Q_3 = \frac{2\pi K B (H - h_w) \ln \left(\frac{D}{r_w}\right)}{\ln \left(\frac{R}{D}\right) \ln \left(\frac{D}{r_w}\right) + \ln \left(\frac{R}{r_w}\right) \ln \left(\frac{D}{2 r_w}\right)}$$

$$Q_2 = \frac{2\pi K B (H - h_w) \ln \left(\frac{D}{2 r_w}\right)}{2 \ln \left(\frac{R}{D}\right) \ln \left(\frac{D}{r_w}\right) + \ln \left(\frac{R}{r_w}\right) \ln \left(\frac{D}{2 r_w}\right)}$$

Where Q_1 and Q_3 are the discharge of the outerwells and Q_2 is the discharge of the middle well.
Total discharge $= Q_1 + Q_2 + Q_3$

(b) For unconfined aquifers (Gravity wells)

(1) For two identical gravity wells at a distance D apart

$$Q_1 = Q_2 = \frac{\pi K (H^2 - h_w^2)}{\ln \left(\frac{R^2}{r_w D}\right)}$$

(2) For three identical gravity wells at a distance D apart in a pattern of equilateral triangle

$$Q_1 = Q_2 = Q_3 = \frac{\pi K (H^2 - h_w^2)}{\ln \left(\frac{R^3}{r_w D^2}\right)}$$

(3) For three identical gravity wells at a distance D apart in a straight line

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$$Q_1 = Q_3 = \frac{\pi K (H^2 - h_w^2) \ln\left(\frac{D}{r_w}\right)}{\ln\left(\frac{R}{D}\right) \ln\left(\frac{D}{r_w}\right) + \ln\left(\frac{R}{r_w}\right) \ln\left(\frac{D}{2r_w}\right)}$$

$$Q_2 = \frac{\pi K (H^2 - h_w^2) \ln\left(\frac{D}{2r_w}\right)}{2\ln\left(\frac{R}{D}\right) \ln\left(\frac{D}{r_w}\right) + \ln\left(\frac{R}{r_w}\right) \ln\left(\frac{D}{2r_w}\right)}$$

Where Q_1 and Q_3 are the discharge of the outerwells and Q_2 is the discharge of the middle well.
Total discharge $= Q_1 + Q_2 + Q_3$

Question: Three wells, each having a diameter of 10 cm are installed at the vertices of an equilateral triangle 12 m apart in a confined aquifer. The radius of influence of each well is 400 m and coefficient of permeability K is 20 m/day. The drawdown in each well is 2 m. The thickness of the confined aquifer is 15 m, find the discharge of each well and the percentage decrease in discharge because of well interference.

Solution:

$r_w = \frac{10}{2}$ cm $= 5$ cm $= 0.05$ m

Here, $D = 12$ m, $R = 400$ m, $K = 20$ m/day, $B = 15$ m
Drawdown in well, $S = (H - h_w) = 2$ m
For three artesian identical wells at a distance B apart in a pattern of equilateral triangle

$$Q_1 = Q_2 = Q_3 = \frac{2\pi K B (H - h_w)}{ln\left(\frac{R^3}{r_w D^2}\right)} = \frac{2\pi \times 20 \times 15 \times 2}{ln\left(\frac{400^3}{0.05 \times 12^2}\right)} = 235.76 \text{ m}^3/\text{day}$$

Discharge of each well without interference,

$$Q = \frac{2\pi K B (H - h_w)}{ln\left(\frac{R}{r_w}\right)} = \frac{2\pi \times 20 \times 15 \times 2}{ln\left(\frac{400}{0.05}\right)} = 419.73 \text{ m}^3/\text{day}$$

Hence, reduction in discharge due to interference

$$\text{Percent reduction} = \left(\frac{419.73 - 235.76}{419.73}\right) \times 100\% = 43.83\%$$

Question: A well penetrates an unconfined aquifer. Before pumping, the water table is 25 m. After a long period of pumping at a constant rate of 0.5 m^3/s, the drawdowns at distances of 50 m from the well were observed 3 m. If $K = 7.26 \times 10^{-2}$ m/s, find the drawdowns at distances of 150 m from the well.

Solution:

Here, $h_1 = 25 - 3 = 22$ m and $h_2 = 25 - S_2$

$$h_2^2 - h_1^2 = \frac{Q}{K\pi} \ln\frac{r_2}{r_1}$$

$$(25 - S_2)^2 - h_1^2 = \frac{0.5}{\pi \times 7.26 \times 10^{-2}} \ln\frac{150}{50}$$

$S_2 = 2.46$ m

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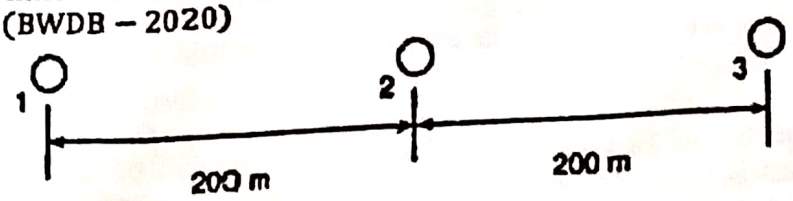
Question: Three pumping wells located along a straight line are spaced at 200 m apart. What should be the steady state pumping rate from each well so that the near steady state drawdown in each well will not exceed 2 m? The transmissibility of the confined aquifer, which all the wells fully penetrate is 2400 m^2/day and all the wells are 40 cm in diameter. The thickness of the aquifer is 40 m and the radius of influence of each well is 800 m. (BWDB – 2020)



Solution:

The following information is given in the above problem statement $S_1 \leq 2$ m, $S_2 \leq 2$ m and $S_3 \leq 2$ m, $T = 2400\ m^2/day = 27.8 \times 10^{-3}\ m^2/s$, $r_w = 0.2$ m, $b = 40$ m, $r_0 = 800$ m and $r = 200$ m. Let Q be the pumping rate from each well and h_0 be the head before pumping started. For well I, $S_1 = S_{11} + S_{12} + S_{13}$ where S_{ij} is the drawdown in well i due to pumping in well j. Thus, for the other wells, $S_2 = S_{21} + S_{22} + S_{23}$ and $S_3 = S_{31} + S_{32} + S_{33}$. By symmetry, $S_1 = S_3$. The drawdown in well 1 due to pumping in wells 1, 2 and 3 respectively.

$$S_{11} = \frac{Q \ln\left(\frac{r_0}{r_w}\right)}{2\pi T} = \frac{Q \ln\left(\frac{800}{0.2}\right)}{2\pi \times 27.8 \times 10^{-3}} = 47.78\,Q$$

$$S_{12} = \frac{Q \ln\left(\frac{r_0}{r_{12}}\right)}{2\pi T} = \frac{Q \ln\left(\frac{800}{200}\right)}{2\pi \times 27.8 \times 10^{-3}} = 7.94\,Q$$

$$S_{13} = \frac{Q \ln\left(\frac{r_0}{r_{13}}\right)}{2\pi T} = \frac{Q \ln\left(\frac{800}{400}\right)}{2\pi \times 27.8 \times 10^{-3}} = 3.97\,Q$$

The drawdown in wells 1 & 3 are identical so total drawdown in the wells is $S_1 = S_3 = 47.48Q + 7.94Q + 3.97Q = 59.39\,Q$. The drawdown in well 2 due to pumping in wells 1, 2 and 3 are respectively,

$$S_{21} = \frac{Q \ln\left(\frac{r_0}{r_{12}}\right)}{2\pi T} = \frac{Q \ln\left(\frac{800}{200}\right)}{2\pi \times 27.8 \times 10^{-3}} = 7.94\,Q$$

$S_{22} = S_{11} = 47.48\,Q$

$S_{23} = S_{21} = 7.94\,Q$

The total drawdown in well 2 is $S_2 = 7.94Q + 47.48Q + 7.94Q = 63.36\,Q$. The relationships for $S_1 = 59.39Q$ and $S_2 = 63.36Q$ show that for the same discharge from all the wells, more drawdown at middle well; so using $S_2 \leq 2$ or $63.36Q \leq 2$, then the steady state pumping rate from each well should be $Q \leq 3.16 \times 10^{-2}\ m^3/s = 113\ m^3/hr$.

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Question: A 100 mm diameter tube well is sunk to 35 m below in an unconfined aquifer. The depth of water in the tube well while pumping is 33 m and the radius of circle of influence is 30 m. If the discharge is 33.4 lps, calculate the co-efficient of permeability of the aquifer. (WARPO – 2017)

Solution:

Given, $H = 35$ m, $h_w = 33$ m, $R = 30$ m, $r_w = \frac{100}{2} = 50$ mm $= 0.05$ m

Discharge, $Q = 33.4$ lps $= \mathbf{0.0334\ m^3/s}$

For unconfined aquifer, $Q = \frac{\pi K (H^2 - h_w^2)}{\ln(R/r_w)}$

$$0.0334 = \frac{3.1416 \times K \times (35^2 - 3^2)}{\ln(30/0.05)}$$

$$K = 5 \times 10^{-4}\ \text{m}^3/\text{min} = 0.5\ \text{lps/m}$$

Question: An artesian aquifer 10 m thick with piezometric surface 40 m above the bottom confining layer is being pumped by a fully penetrating well. The aquifer is medium sand (hydraulic conductivity 1.5×10^{-4} m/s). Steady state drawdowns of 5.00 m and 1.00 m are observed at two non-pumping wells located 20 m and 200 m respectively, from the pumped well. Determine the discharge at the pumped well in m^3/s. (SGFL – 2017, BIWTA – 2019)

Solution:

Here, $B = 10$ m, $S_1 = 5$ m, $r_1 = 20$ m, $S_2 = 1$ m, $r_2 = 200$ m

For confined aquifer, $Q = \frac{2\pi K B (S_1 - S_2)}{\ln(r_2/r_1)}$

$$Q = \frac{2 \times 3.1416 \times 1.5 \times 10^{-4} \times 10\ (5 - 1)}{\ln(200/20)} = 0.0163\ \text{m}^3/\text{s}$$

Question: A 100 mm diameter tube well is sunk from a 10 m thick confined aquifer having a co-efficient of permeability equal to 0.75 lps/m^2, The depth of water below the piezometric level is 30 m and it falls 2 m in tube well while pumping. Calculate the discharge of the tube well when radius of circle of influence is 30 m. (BCIC – 2017, BUET M.Sc – 2019)

Solution:

Here, $B = 10$ m, $H = 30$ m, $h_w = 30 - 2 = 28$ m, $R = 30$ m

$r_w = 100/2 = 50$ mm $= 0.05$ m, $K = 0.75$ lps/m^2 $= 7.5 \times 10^{-4}$ m/s

For confined aquifer, $Q = \frac{2\pi K B (H - h_w)}{\ln(R/r_w)}$

$$Q = \frac{2 \times 3.1416 \times 7.5 \times 10^{-4} \times 10\ (30 - 28)}{\ln(30/0.05)}$$

$$Q = 0.00163\ \text{m}^3/\text{s} = 17.74\ \text{liters/sec}$$

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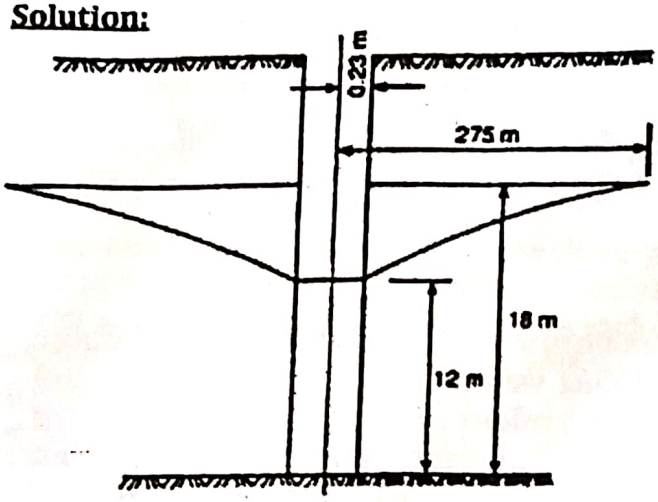
Question: A tube well is 0.46 m in diameter. The unconfined aquifer is of 18 m depth. After drawdown depth of water is 12 m in the well. Permeability of soil is 24.50 m/day. Radius of circle of influence is 275 metres. Calculate discharge of the tube well. (50 BMA)

Solution:



Permeability of soil, $K = 24.50$ m/day

Radius of circle of influence, $R = 275$ m

Tube well diameter, $r_w = 0.23$ m

$H = 18$ m, $h_w = 12$ m

For unconfined aquifer, $Q = \dfrac{\pi K (H^2 - {h_w}^2)}{\ln(R/r_w)}$

$$Q = \frac{3.1416 \times 24.50 \times (18^2 - 12^2)}{\ln(275/0.23)} = 2000 \text{ m}^3/\text{day}$$

Question: A 30 cm diameter well penetrates 25 m below the static water table. After 24 hours of pumping @ 5400 liters/minute, the water level in a test well at 90 m is lowered by 0.53 m and in a well 30 m away the drawdown is 1.11 m. What is the transmissibility of the aquifer?

Solution:

Here, $h_1 = H - S_1 = 25 - 1.11 = 23.89$ m

$h_2 = H - S_2 = 25 - 0.53 = 24.47$ m

$r_1 = 30$ m, $r_2 = 90$ m

$Q = 5400$ l/min $= 5.4$ m^3/min $= 0.09$ m^3/s

$$Q = \frac{\pi K (h_2^2 - h_1^2)}{\ln(r_2/r_1)}$$

$$K = \frac{Q \ln(r_2/r_1)}{\pi (h_2^2 - h_1^2)} = \frac{0.09 \times \ln(90/30)}{3.14 (24.47^2 - 23.89^2)} = 1.121 \times 10^{-3} \text{ m/s}$$

$$T = K H = 1.121 \times 10^{-3} \times 25 = 0.028 \text{ m}^2/\text{s}$$

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Question: A gravity well of 12 inch diameter is discharging 150 gal/min with drawdown of 10 ft. It discharges 500 gal/min with a drawdown of 50 ft. State depth of the water in the well as 150 ft. What will be the discharge from the well with drawdown of 20 ft? (EED – 2019)

Solution:

Case 1: $Q_1 = 150$ gal/min, $H = 150'$, $S_w = 10'$, $h_w = 150' - 10' = 140'$

$$Q_1 = \frac{\pi K (H^2 - h_w^2)}{\ln \frac{C Q_1}{r_w}}$$

$$150 = \frac{\pi K (150^2 - 140^2)}{\ln \frac{C \times 150}{0.5}}$$

Case 2: $Q_2 = 500$ gal/min, $H = 150'$, $S_w = 50'$, $h_w = 150' - 50' = 100'$

$$Q_2 = \frac{\pi K (H^2 - h_w^2)}{\ln \frac{C Q_2}{r_w}}$$

$$500 = \frac{\pi K (150^2 - 100^2)}{\ln \frac{C \times 500}{0.5}}$$

Solving for C and K, $C = 0.21$ and $K = 0.093$

For case 3: $H = 150'$, $S_w = 20'$, $h_w = 150' - 20' = 130'$

$$Q_3 = \frac{\pi K (H^2 - h_w^2)}{\ln \frac{C Q_2}{r_w}}$$

$$Q_3 = \frac{\pi \times 0.093 (150^2 - 130^2)}{\ln \frac{0.21 \times Q_3}{0.5}}$$

By trial and error, $Q_3 = 257$ gal/min

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Water resources engineering

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Water Resources Engineering

Question: What is the River training? (EED – 2015)

Solution:

'River training' refers to the structural measures which are taken to improve a river and its banks. River training is an important component in the prevention and mitigation of flash floods and general flood control, as well as in other activities such as ensuring safe passage of a flood under a bridge.

Question: What is meant by river training? State the objectives of river training? (34th BCS).

Solution:

River training means adopting various measures on a river to stabilize the river channel along a certain alignment for the given/planned cross-sectional dimensions. A river can be trained by diverting its flow into a secondary channel or by performing artificial cutoffs on the main river to reduce flood levels.

The various objectives of river training are

1. To guide the axis of flow at ordinary and low stages and safe passage of floods without overtopping the banks.
2. To protect the banks from erosion and generally improve their alignment by stabilizing the river channel.
3. To train the river flow along a safe course, thereby avoiding damage by flooding or erosion of valuable lands, habitation, crops, factories, etc.
4. To prevent outflanking of a bridge, barrage or weir by directing the flow in a defined stretch of the river.
5. To prevent river from changing its course.
6. To confine a river channel which has become too wide by swinging from side to side and to reclaim the land from river bed.
7. To deflect the river away from the bank which it may attack.
8. To trap bed load in areas of superfluous width.
9. To transport the bed & suspended sediments load with a maximum efficiency
10. To provide sufficient depth of flow for safe navigation.
11. To correct disorderly banks or flow conditions.

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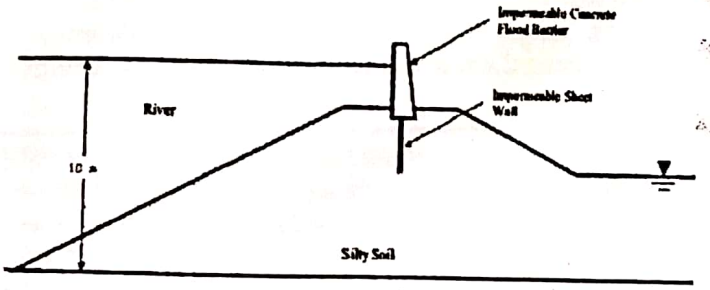
Question: বন্যা থেকে শহর রক্ষার জন্য আমরা কি কি Structuralmeasure গ্রহন করি একটি শহর রক্ষা বাঁধের ?cross-section অংকন করুন। (LGED – 2019)

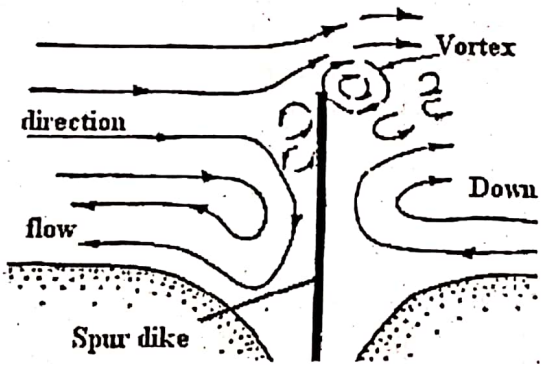
Solution:

Some methods of flood control have been practiced for a long time. Including others system there's also some structural techniques include the construction of levees, lakes, dams, reservoirs, retention ponds to hold extra water during times of flooding etc. Major of them described below.

Flood barriers: Flood barriers is uses a range of temporary flood barriers to provide additional protection to flood-prone areas. Lightweight sectional metal barriers are relatively inexpensive, can be placed in various configurations, and can be removed completely when waters receded.



Spurs: Spurs is the structure constructed transverse to the river flow to protect the bank by deflecting the current away from the bank. That protects river banks and contains flood waters. They can be built by communities using stones and wire.



Levee: A levee is an embankment running parallel to the river and is constructed to protect the area on one side of it from flooding. The method of constructing levees on one or both sides of a river to contain the flood within the leveed portion is the oldest and most commonly used method of flood control. The alignment of levees for a river is decided by the location of important cities, industries, and other areas along the river which need to be protected against floods. Closely-spaced levees will be very high and, hence, massive and uneconomic.

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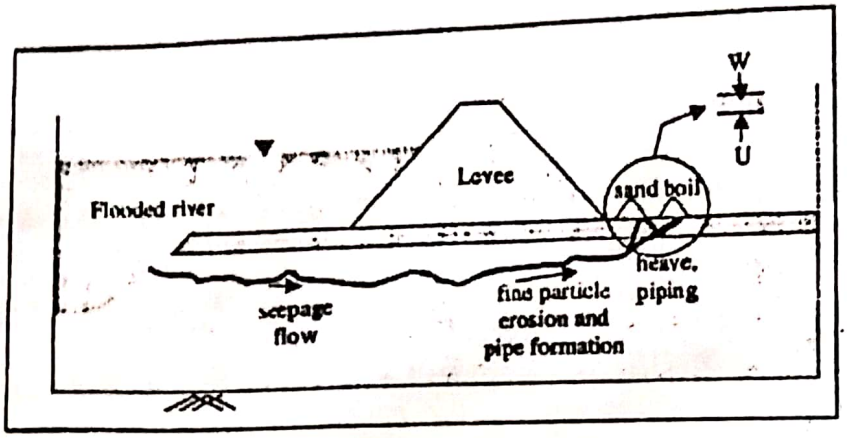
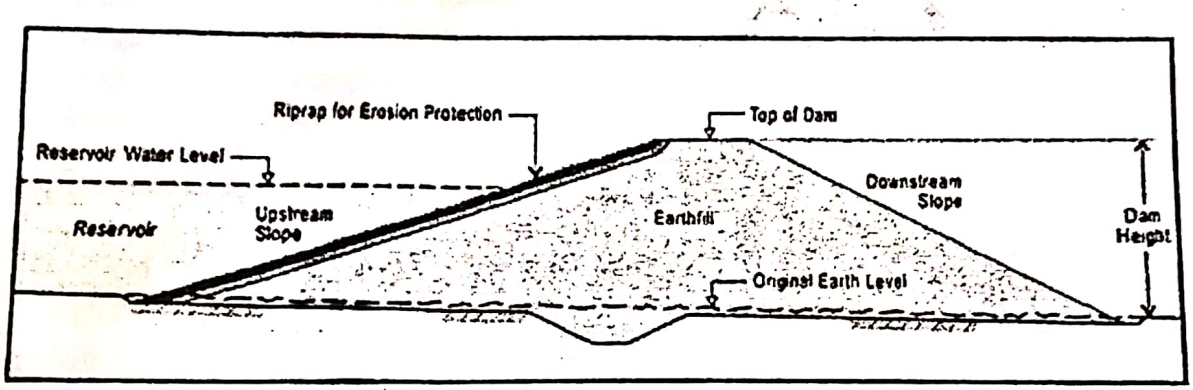


Dam: Controlled release of water stored in reservoir through sluice gates can spread discharge over a longer period (reducing peak flows) Effectiveness depends on relative scale of reservoir's catchment area to that of the whole drainage basin Geo-politics can cause problems.

Reservoir: Flood control reservoir, generally called a flood-mitigation reservoir, stores a portions of the flood flows in such a way as to minimize the flood peaks at the areas to be protected. Flood control reservoir protects valuable developments on the downstream flood plain by storing flood waters and releasing them at a rate that will reduce the downstream damage.



Question: What is meant by meandering of rivers? What are its causes? Explain different meander parameter with neat sketch. (30th BCS)

Solution:

A meander is one of a series of regular sinuous curves, bends or windings in the channel of a river, stream or other watercourse. It is produced by a stream or river swinging from side to side as it flows across its floodplain or shifts its channel within a valley. A meander is produced by a stream or river as it erodes the sediments comprising an outer concave bank (cut bank) and deposits this and other sediment downstream on an inner convex bank which is typically a point bar. The result of sediments being eroded from the outside concave bank and their deposition on an inside convex bank is the formation of a sinuous course as a channel migrates back and forth across the down-valley axis of a floodplain. The zone within which a meandering stream shifts its channel across either its floodplain or valley floor from time to time is known as a meander belt.

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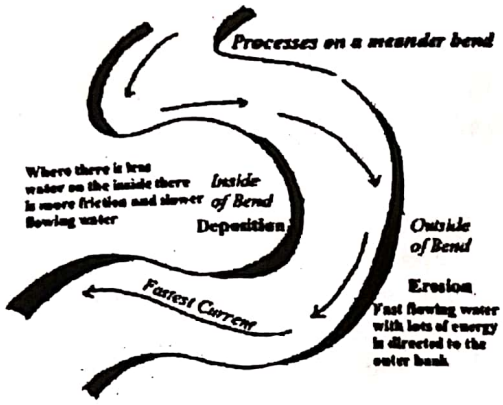
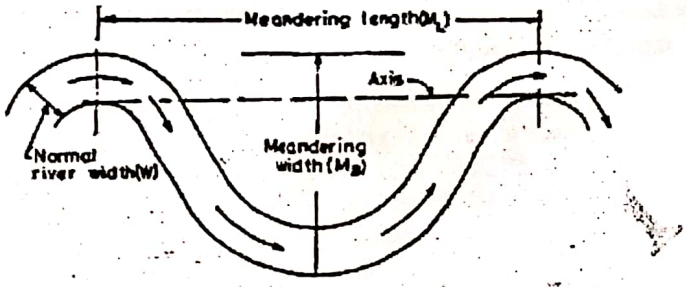
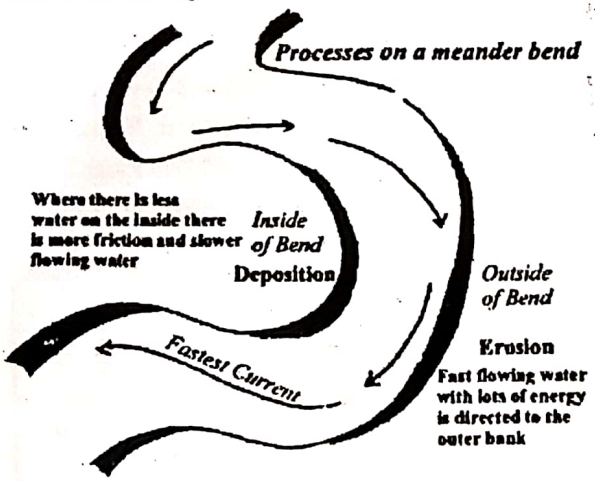


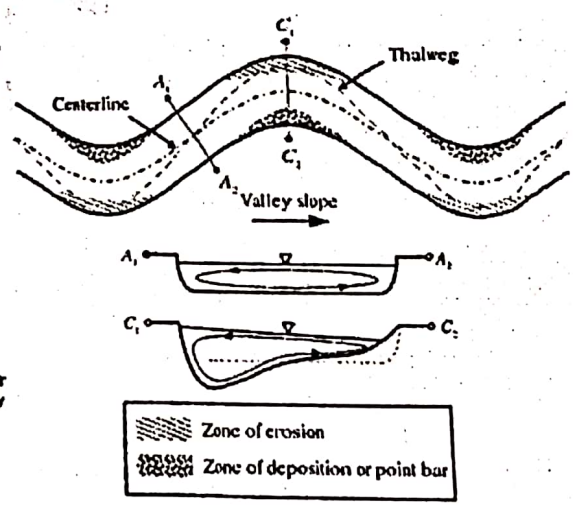


Question: What is meant by "meandering of rivers" and what are its causes? (33th BCS)

Solution:

A meander is one of a series of regular sinuous curves, bends or windings in the channel of a river, stream or other watercourse. It is produced by a stream or river swinging from side to side as it flows across its floodplain or shifts its channel within a valley. A meander is produced by a stream or river as it erodes the sediments comprising an outer concave bank (cut bank) and deposits this and other sediment downstream on an inner convex bank which is typically a point bar. The result of sediments being eroded from the outside concave bank and their deposition on an inside convex bank is the formation of a sinuous course as a channel migrates back and forth across the down-valley axis of a floodplain. The zone within which a meandering stream shifts its channel across either its floodplain or valley floor from time to time is known as a meander belt.





Question: Name the important types of river training method and indicate the purpose for which each type is adopted. (31th BCS)

Solution:

River training works is a term used to stabilize a river channel with a certain alignment and certain cross-sections.

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Embankment: These are the earthen embankment which is constructed parallel to the river banks at some suitable distance from it. These embankment walls retain the flood water and prevent them from spreading into nearby towns.

Guide banks: The guide banks guide the river flow past a bridge or any other hydraulic structure without causing damage to the work and its approaches. They protect the approach embankment for the bridge from attack of the water. They control the river and induce it to flow through the bridge more or less axially.

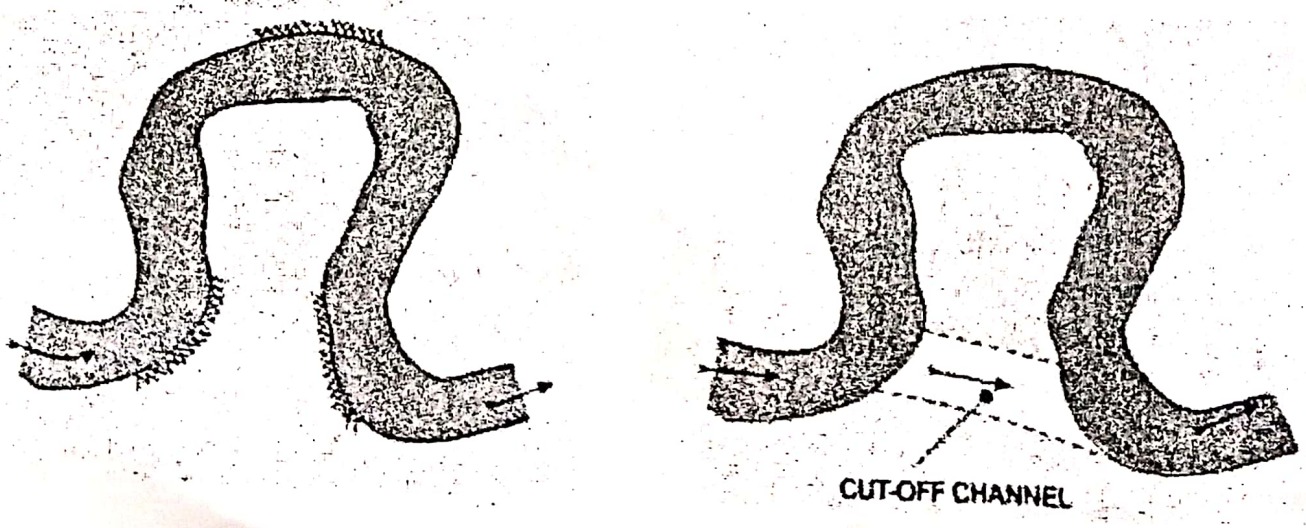
Revetment of Banks: Bank failures due to either wave action or erosive action of river flow can lead to river breach causing huge losses in terms of human lives, property, agriculture and other utilities. Measures of bank protection such as revetment, riprap, etc. provide a shield against erosion of bank material. It is a protection of boulders or concrete blocks to the river bank. Any protective work whose purpose is to maintain the stability of the bank against erosive effect of water is known as bank protection.

- To resist erosion of banks by water currents.
- To prevent sliding of soil due to drawdown of flood.
- To prevent piping of water through the banks.
- To protect hydraulic structures.
- To protect flood embankments.

Groynes or Spurs: Groynes are structures, built transverse to the river flow, extending from the bank towards the river. They protect the river banks by keeping the flow away from it. They train the river to flow along a specified course.

Pitched Islands: An island created artificially on river bed with sand in its core and protected by stone pitching on all its sides is called pitched island. It can be used to correct an oblique approach upstream of weirs, barrages and bridges.

Cutoffs: Cut-offs as river training works are to be carefully planned and executed in meandering rivers. The cut-off is artificially induced with a pilot channel to divert the river from a curved flow which may be endangering valuable land or property or to straighten its approach to a work or for any other purpose. Artificial cutoffs shorten the travel distance, and increase ease of maneuvering of a navigating vessel.



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Dredging of River: To improve navigability of the river channel the river section may need to be excavated. This excavation is carried out to increase the depth of flow even when there is flow m the river. The process of underwater excavation is termed dredging.

Question: What are the objectives and requirements of bank protection? Mention the causes of bank failure. (30th BCS)

Solution:

Revetment is a bank protection measure where the bank slope is covered with erosion resistant materials. The function of the revetment is thus to reduce the hydraulic load acting on the soil and possibly to facilitate stabilization of the soil. This type of river training works involve a protective cover of a suitable hard material applied on the slope and toe of the river bank so that the bank soil protected from the actions of erosions of erosive forces of flowing water and dynamic actions of waves.

8.5.1. Objectives of River Training. The river training works may serve the following objectives or advantages :

(*i*) To prevent the river from changing its course and to avoid outflanking of structures like bridges, weirs, aqueducts, etc.

(*ii*) To prevent flooding of the surrounding countries by providing a safe passage for the flood waters without overtopping the banks.

(*iii*) To protect the river banks by deflecting the river away from the attacked banks.

(*iv*) To ensure effective disposal of sediment load.

(*v*) To provide minimum water depth required for navigation.

Objectives of bank protection:

- To resist erosion of banks by water currents.
- To prevent sliding of soil due to drawdown of flood.
- Prevent losses of agricultural production from bank failure.
- Prevent the loss of private property and public infrastructure on flooded land.
- To prevent piping of water through the banks.
- To afford facilities for water transportation.
- To protect hydraulic structures.
- To protect flood embankments.

Requirements of bank protection:

- Effective
- Environmentally sound
- Economical
- Sustainable

Causes of bank failure

a) Failure by surface erosion of river banks: Surface erosion of river banks occurs if the driving erosive forces are exceeding the resistive forces of the individual grains or of the conglomerates in case of cohesive materials. The main impacts responsible for surface erosion at river banks are:

- Current induced shear stress

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- Wave loads (wind-generated waves; ship and boat-generated waves)
- Seepage (excessive pore pressure)
- Surface runoff and
- Mechanical action (desiccation, ship impact, activities of humans and animals)

b) Failure of protective works by mass damage of riverbanks: Mass damage of river banks can be divided into slip failures, block failures and flow slides, initiated by different processes. The actual damage of a river bank may not follow immediately after an impact. In some cases the failure process takes several days. On the other hand, a damage may occur without warning at almost any time if active surface erosion and toe scouring is prevalent or an additional (surcharge) load is applied to the bank. The risk of mass damage is increased during heavy rain and during quick fall of river stages after flood.

Question: Briefly explain the different types of River bank protection works. What are the human causes of floods in Bangladesh? (36th BCS)

Solution:

Different types of River bank protection works are given below:

Guide banks: The guide banks guide the river flow past a bridge or any other hydraulic structure without causing damage to the work and its approaches. They protect the approach embankment for the bridge from attack of the water. They control the river and induce it to flow through the bridge more or less axially.

Revetment of Banks: It is a protection of boulders or concrete blocks to the river bank. Any protective work whose purpose is to maintain the stability of the bank against erosive effect of water is known as bank protection.

- To resist erosion of banks by water currents.
- To prevent sliding of soil due to drawdown of flood.
- To prevent piping of water through the banks.
- To protect hydraulic structures.
- To protect flood embankments.

Groynes or spurs: Groynes are structures, built transverse to the river flow, extending from the bank towards the river. They protect the river banks by keeping the flow away from it. They train the river to flow along a specified course.

Other than the natural causes, human activities can as well cause an area to be prone to flooding. The main human causes for the river floods are urbanization, riverbed aggradation, ploughing and deforestation as explained below.

Human causes of floods in Bangladesh:

Rapid Urbanization: The population density in Bangladesh is increasing every year due to the drastic growth in population. This unexpected rapid growth in population can impact the drainage basin and the flooding in the watershed through urbanization. Due to the constructions of buildings, flat lands as well as bare roads, it allows more water to enter the area and increase the

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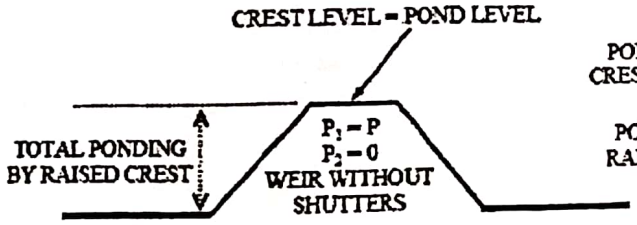
drainage basin capacity. This can be viewed from the data where stated 60% of the lands are covered with high ways and buildings. As more and more people are moving in to the cities, agricultural lands that were initially providing a better condition for floods prevention are dominated by cements and other constructions.

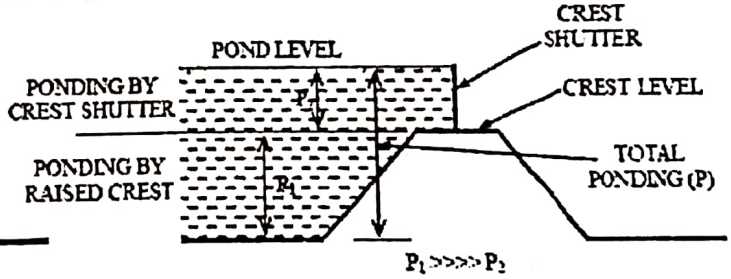
Deforestation in the Upstream Region: Along with the rapid population growth, the rate of deforestation has been increased due to the increases in food and fuel wood demands. Especially the regions with steep slope that have deforested, landslides and soil erosion occur during the monsoon season. Specifically, deforestation on the uphill regions affected on the size and the scale of the floods. Such events will support the intensity of the flooding during heavy precipitation by accelerating the speed of water. In addition, absence of trees due to deforestation minimized the interceptions of the rainwater allowing it to travel along with the eroded soil, which increases the speed of the rainwater that allowed it to reach the ground level in a short amount of time.

Stability of Flood Defenses: One minor human impact on flooding is the quality of the current flood defenses. Bangladesh is known to be in debt, which the country has insufficient expenses to construct flood protection measures. On top of that, the present flood defenses are ineffective and reconstruction are highly recommended. For instance, the level of the dyke as well as the depth of it are minimal resulting it to have insufficient storage capacity for rainwater. Furthermore, some areas of the floodplains are deforested that increased the speed of the water flow. These vulnerable flood protection schemes are then corrupted during the flooding resulting in increase in debt and economic corruption. As a result of the poor condition of the flood defenses, levees are collapsed during the period of high discharge.

Weir: If the major part or the entire ponding of water is achieved by a raised crest and a smaller part or nil part of it is achieved by the shutters, then this barrier is known as a weir.

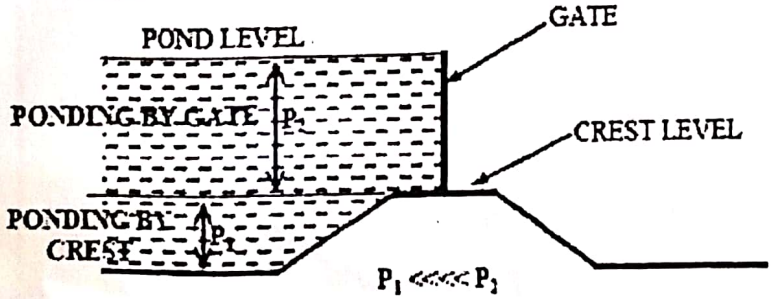


(a) Fig: Weir without shutters

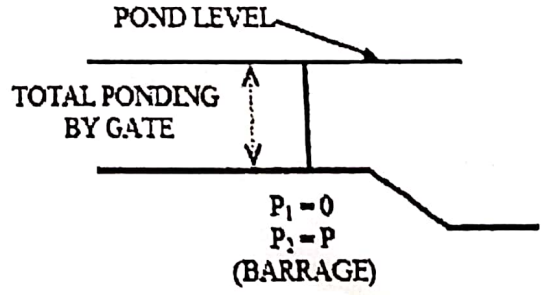


(b) Fig: Weir with shutters

Barrage: If most of the ponding is done by gates and a smaller or nil part of it is done by the raised crest, then the barrier is known as a barrage or a river regulator.



(c) Fig: Barrage with a small raised crest



(d) Fig: Barrage without any raised crest

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Layout of a Diversion Head Works and its components

A typical layout of a canal head-works is shown in figure below.

Weir proper
Under-sluices
Divide wall
River Training works
Fish Ladder
Canal Head Regulator
River Training Works e.g. Guide bank, Marginal bunds, spur and groyne etc.
Shutters and Gates
Silt Regulation Works

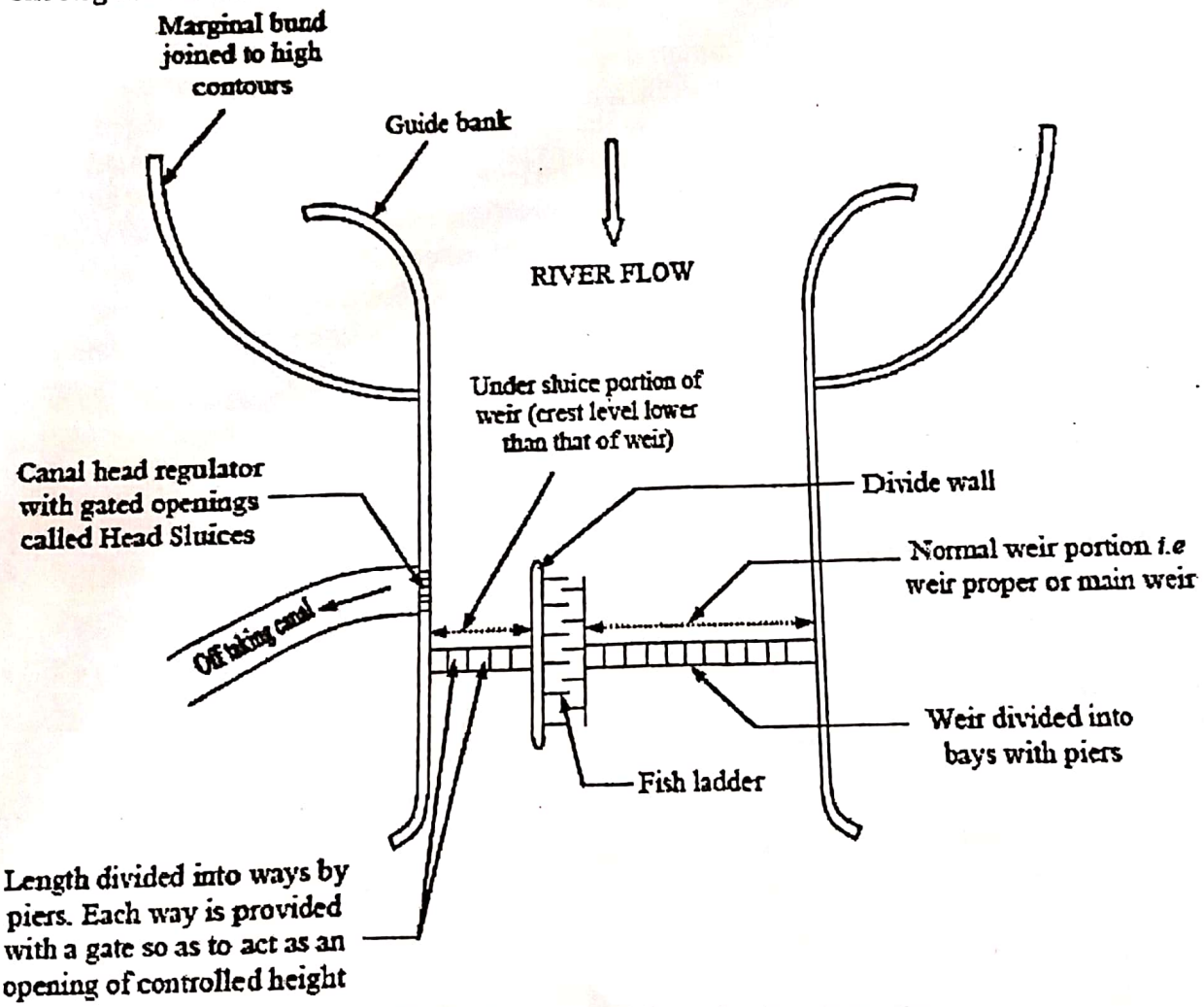

Fig: Typical layout of diversion head-works

Question: What is spur? Explain the different types of spur based on their alignment with neat sketches. (32th BCS)

Question:

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These are temporary structures permeable in nature provided on the curve of a river to protect the river bank from erosion. These are projected from the river bank towards the bed making angles 60^0 to 75^0 with the bank of the river. The length of the spurs depends on the width of the river and the sharpness of the curve. The function of the spurs is to break the velocity of flow and to form a water pocket on the upstream side where the sediments get deposited. Thus the reclamation of land on the river bank can be achieved.

The spurs may be of the following types:

(a) Bamboo Spur
(b) Timber Spur
(c) Boulder Spur

(a) **Bamboo Spur:** In this type of spur, a box like compartment is prepared by driving bamboo piles at 15 cm centre to centre. The piles are secured by bamboo bracings. The hollow space is filled up with sand bags. It is permeable in nature and water can seep through its body. This type of spur is suitable for small rivers. This is purely temporary and requires repair work every year. The length of bamboo piles depends on bed condition.

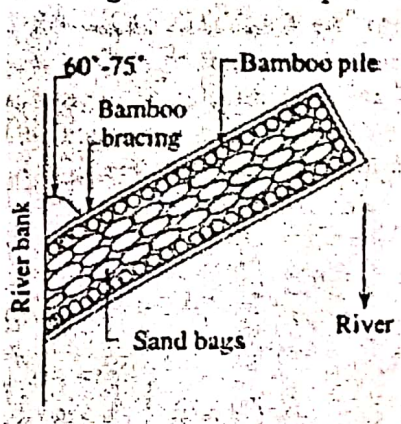

Fig: Bamboo Spur

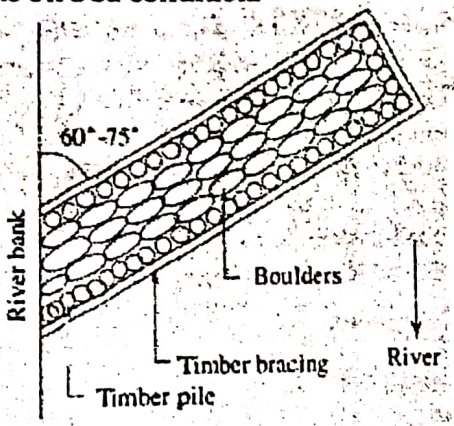

Fig: Timber Spur

(b) **Timber Spur:** In this type, a box like compartment is prepared by driving timber piles at 15 cm to 30 cm centre to centre. The piles are secured by wooden bracings. The hollow space is filled up by boulders. This spur is permeable but stable. This is recommended for bi rivers with high velocity of flow. The length of the timber piles depend on bed condition.

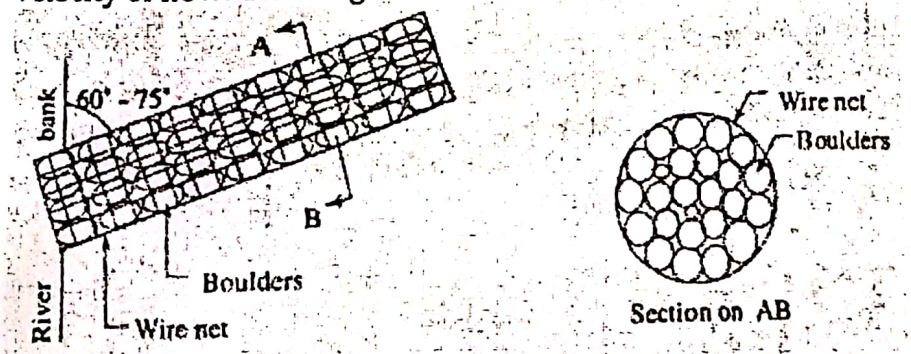

Fig: Boulder Spur

(c) **Boulder Spur:** In this type of spur, boulders are enclosed in G.I wire net in circular shape. The boulder should be heavy, varying from 30 kg to 50 kg and the wire net should be made of 4 mm diameter G.I wires. It is laid from the river bank towards the bed making an angle of 60o-75o with the bank. This type of spur is recommended for the rivers where velocity of flow is very high.

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Groynes: The function of groynes is similar to that of spur. But these are impervious permanent structures constructed on the curve of a river to protect the river bank from erosion. They extend from the bank towards the bed by making an angle of 60° to 75° with the bank. The angle may be towards the upstream or downstream. Sometimes, it is made perpendicular to the river bank. These are constructed with rubble masonry in trapezoidal section and the surface is finished with stone pitching or concrete blocks.

- The stone pitching or the concrete blocks are set with rich cement mortar.
- The length of the groyne depends on the width and nature of the river.
- The top width varies from 3 m to 4 m. The side slope may be 1½: 1 or 2:1.
- The groynes are provided in series throughout the affected length of the river bank.
- The spacing between the adjacent groynes is generally kept as $2L$, where L is the length of the groyne.
- These are recommended for the river where the permanent solution of erosion control is extremely necessary.

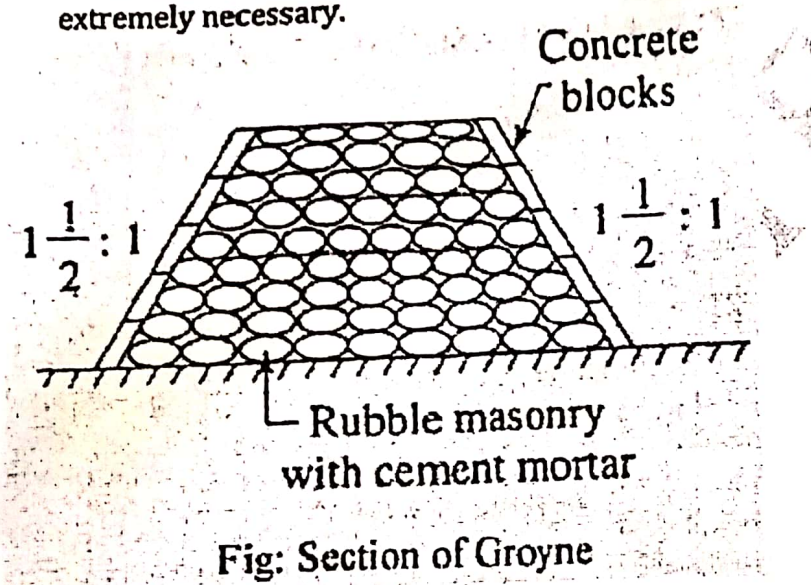

Fig: Section of Groyne

The groynes may be designated as follows:

(a) **Attracting Groyne:** The groyne which is constructed obliquely to the bank by making an angle of 60 to 75° towards the downstream is known as attracting groyne, here the flow of water is attracted towards the bank, and the velocity of flow is reduced to such an extent that it cannot cause any erosion to the bank. However, a bank protected of stone pitching is provided for safety.

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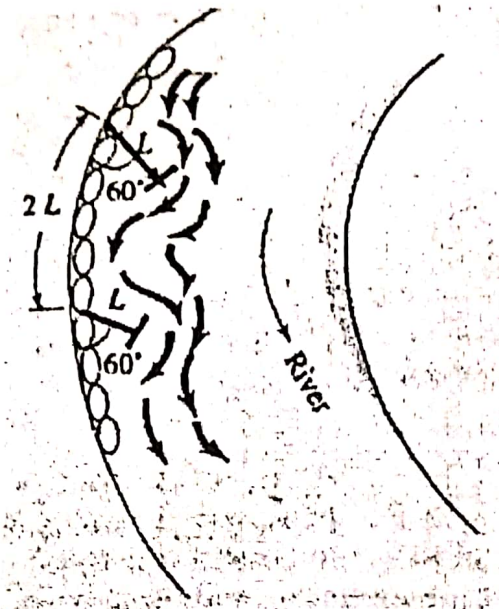

Fig: Attracting groyne

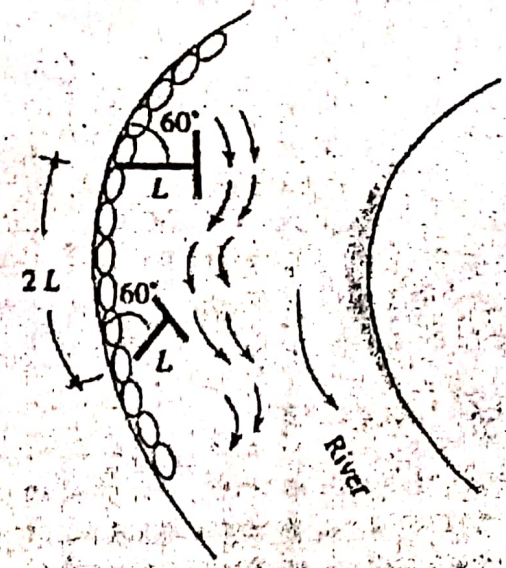

Fig: Repelling groyne

(b) **Repelling Groyne:** A groyne which is aligned towards upstream at an angle of 60° to 75° with the river bank is known as repelling groyne. A still water pocket is formed on the upstream where silting takes place. Here, the bank protection is not necessary, because the flow of water does not touch the bank and there is no effect of erosion on the bank. But still boulder pitching should be provided for safety.

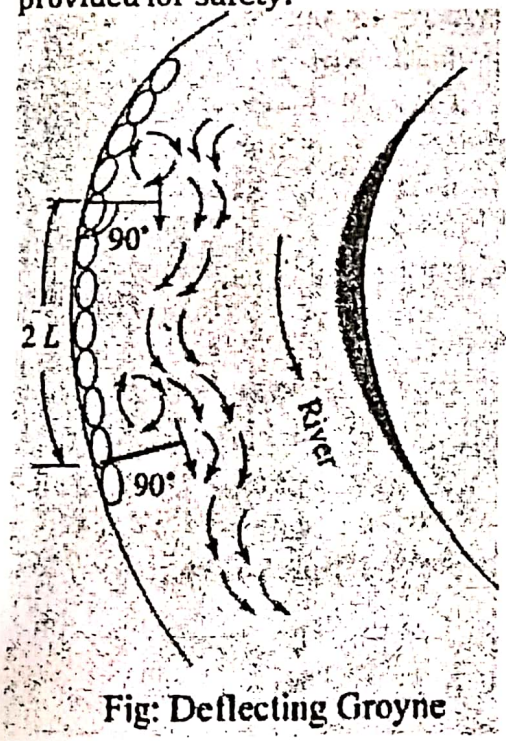

Fig: Deflecting Groyne

(c) **Deflecting Groyne:** The groyne which is constructed perpendicular to the river bank is known as deflecting groyne. Here the flow of water is deflected from bank by the perpendicular obstruction i.e. groyne. The flow of water follows an undulating path just outside the head of the groyne. An eddy current is formed on the upstream side of the groyne. This eddy current will not affect the river bank. But the bank protection is provided for safety.

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Question: Distinguish between Groyne and Guide Bank. (33th BCS)

Solution:

Embankment type structures constructed transverse to river flow, extending from the banks into the river (also transverse dykes) impervious permanent structures constructed on the curve of a river to protect the river bank from erosion by deflecting the current away from the bank. They extend from the bank towards the bed by making an angle of 60° to 75° with the bank. The angle may be towards the upstream or downstream.

Rivers in flood plains submerge very large areas during flood periods. Guide Bank are earthen embankments with stone pitching in the slopes facing water, to guide the river through the barrage. Guide banks are meant for guiding and confining the flow in a reasonable waterway at the site of the structure. The guide banks guide the river flow past a bridge or any other hydraulic structure without causing damage to the work and its approaches. The guide banks are constructed parallel or approximately parallel to the direction of flow. They extend both upstream and downstream of the abutments of the hydraulic structure. The guide banks may be provided on either side of the hydraulic structure or on one side as required.

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Open channel flow

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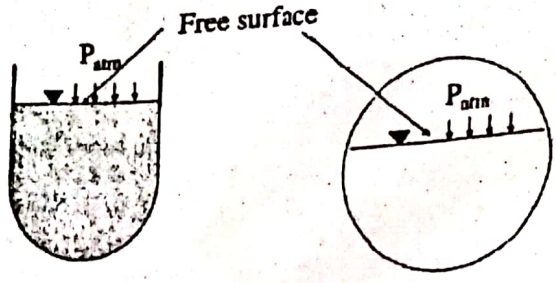
OPEN CHANNEL FLOW

An open channel is a conduit in which a liquid flows with a free surface. That is a surface on which pressure is equal to local atmospheric pressure.



Classification of Open-Channel Flows

Prismatic and Non-prismatic Channels

A channel in which the cross-sectional shape and size and also the bottom slope are constant is termed as a prismatic channel. Most of the man-made (artificial) channels are prismatic channels over long stretches. The rectangle, trapezoid, triangle and circle are some of the commonly used shapes in manmade channels. All natural channels generally have varying cross-sections and consequently are non-prismatic.

Natural flows: rivers, creeks, floods, etc.

Human-made systems: fresh-water aqueducts, irrigation channel, sewers, drainage ditches, etc.

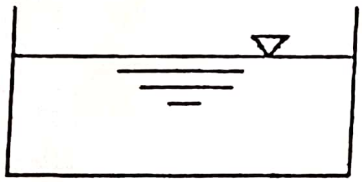


Figure: Prismatic channel

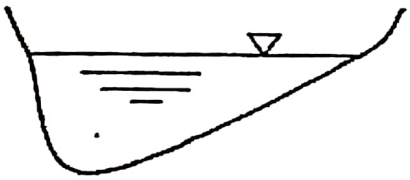
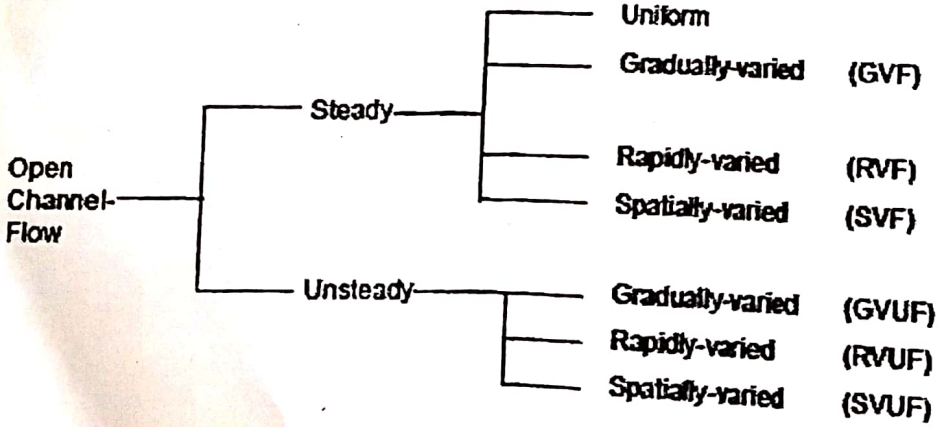


Figure: Non-prismatic channel

Classification of flows



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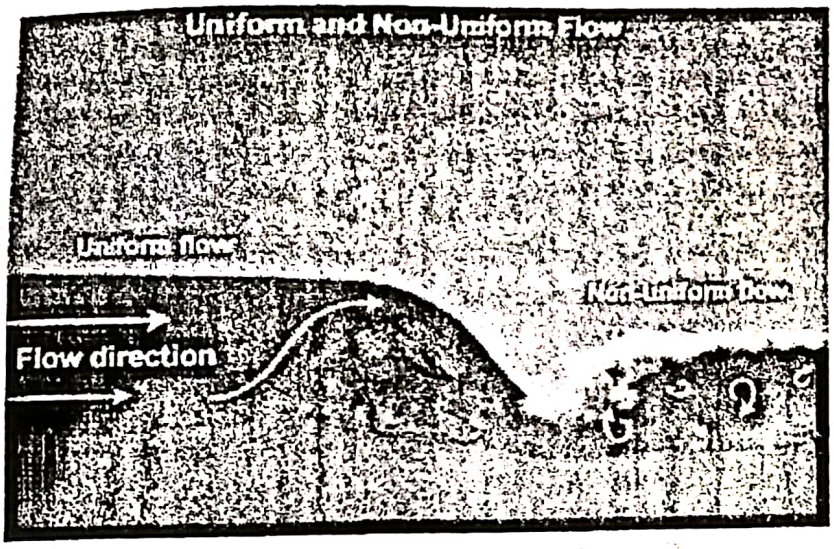
Steady and Unsteady Flows

A steady flow occurs when the flow properties, such as the depth or discharge at a section do not change with time. As a corollary, if the depth or discharge changes with time the flow is termed unsteady.

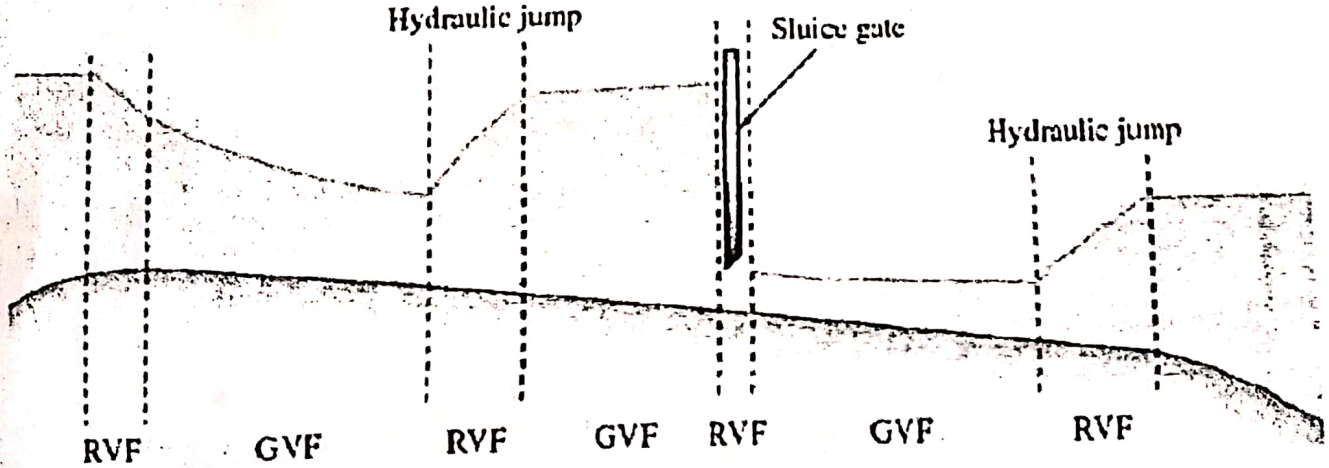
Uniform and Non-uniform Flows

If the flow properties, say the depth of flow, in an open channel remain constant along the length of the channel, the flow is said to be uniform. As a corollary of this, a flow in which the flow properties vary along the channel is termed as non-uniform flow or varied flow.



Gradually Varied and Rapidly Varied Flows

If the change of depth in a varied flow is gradual so that the curvature of streamlines is not excessive, such a flow is said to be a gradually varied flow (GVF). If the curvature in a varied flow is large and the depth changes appreciably over short lengths, such a phenomenon is termed as rapidly varied flow (RVF).



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Spatially Varied Flow

If some flow is added to or abstracted from the system the resulting varied flow is known as a spatially varied flow (SVF). SVF can be steady or unsteady.

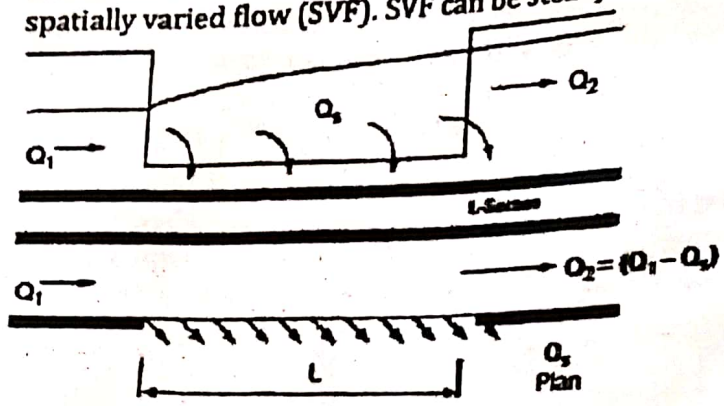
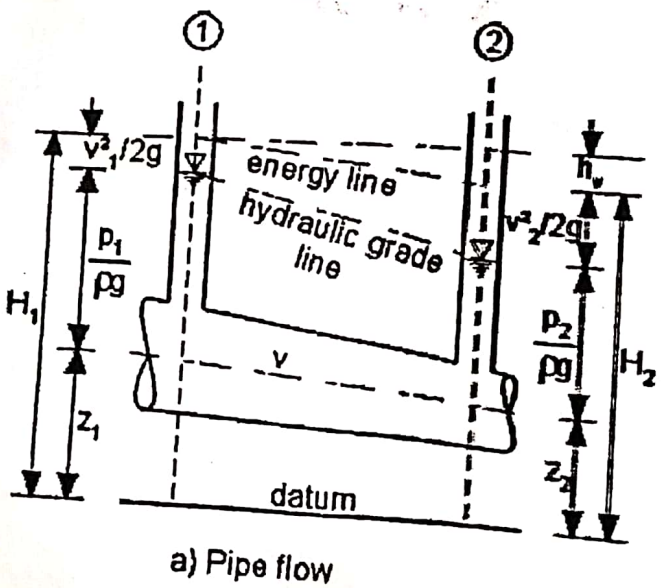


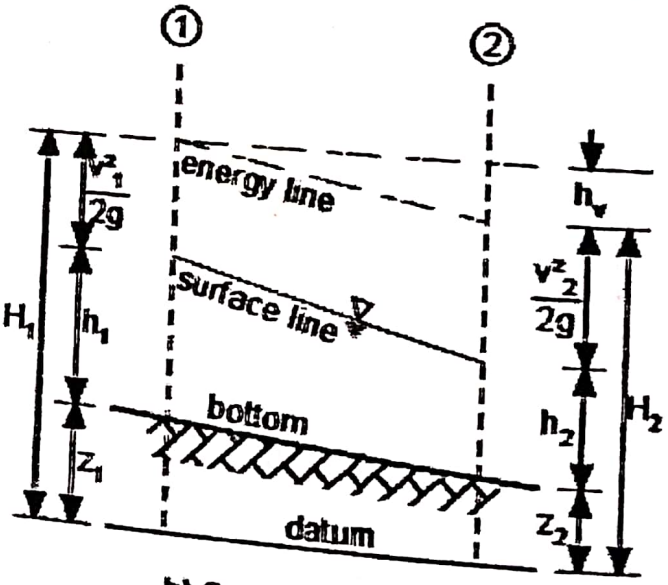
Figure: Spatially varied flow through open channel

Question: Differentiate between pipe flow and open channel flow. (TGTDCL – 2014)

OPEN CHANNEL FLOW	PIPE FLOW
Flow occurs due to gravity.	Flow occurs due to different in pressure.
The maximum velocity occurs at a little distance below the water surface.	The maximum velocity occurs at the center of the pipe.
Cross section of open channel can be trapezoidal, triangular, rectangular, circular etc.	Cross section of pipe generally round.
HGL (Hydraulic Gradient Line) coincide with water surface line.	HGL(Hydraulic Gradient Line) do not coincide top surface of the water



a) Pipe flow



b) Open channel flow

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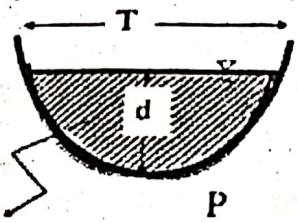
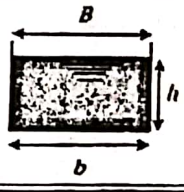
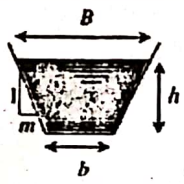
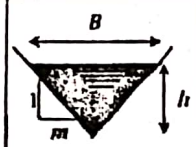
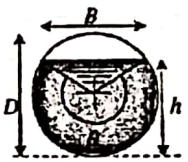
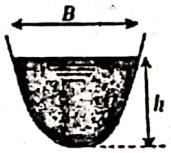
Geometric Elements of Channel Section

Top width, T: is the width of the channel section at the free surface.
The water area, A: is the cross-sectional area of the flow normal to the direction of flow.
The wetted perimeter, P: is the length of the line of intersection of the channel wetted surface with a cross-sectional plane normal to the direction of flow.
The hydraulic radius, R = A/P: is the ratio of the water area to its wetted perimeter.
The hydraulic depth, D = A/T: is the ratio of the water area to the top width.



A = A(d)

Geometric elements for different channel cross sections

	rectangular	*trapezoidal*	*triangular*	*circular*	*parabolic*
					
flow area A	bh	$(b+mh)h$	mh^2	$\frac{1}{8}(\theta-\sin\theta)D^2$	$\frac{2}{3}Bh$
wetted perimeter P	$b+2h$	$b+2h\sqrt{1+m^2}$	$2h\sqrt{1+m^2}$	$\frac{1}{2}\theta D$	$B+\frac{8}{3}\frac{h^2}{B}$ *
hydraulic radius R_h	$\frac{bh}{b+2h}$	$\frac{(b+mh)h}{b+2h\sqrt{1+m^2}}$	$\frac{mh}{2\sqrt{1+m^2}}$	$\frac{1}{4}\left[1-\frac{\sin\theta}{\theta}\right]D$	$\frac{2B^2h}{3B^2+8h^2}$ *
top width B	b	$b+2mh$	$2mh$	$\frac{(\sin\theta/2)D}{2\sqrt{h(D-h)}}$ or	$\frac{3}{2}\frac{Ah}{}$
hydraulic depth D_h	h	$\frac{(b+mh)h}{b+2mh}$	$\frac{1}{2}h$	$\left[\frac{\theta-\sin\theta}{\sin\theta/2}\right]\frac{D}{8}$	$\frac{2}{3}h$

* *Valid for* $0<\xi\le 1$ *where* $\xi = 4h/B$
If $\xi>1$ *then* $P=(B/2)\left[\sqrt{1+\xi^2}+(1/\xi)\ln\left(\xi+\sqrt{1+\xi^2}\right)\right]$

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Reynolds number: In an open-channel flow The Reynolds number is the ratio of inertial forces to viscous forces and is a convenient parameter for predicting if a flow condition will be laminar or turbulent.

$$R_e = \frac{V\,R}{\nu}$$

(Note: R is Hydraulic Radius, V = Average velocity, ν = Kinematic viscosity)

Re < 500 – Laminar flow

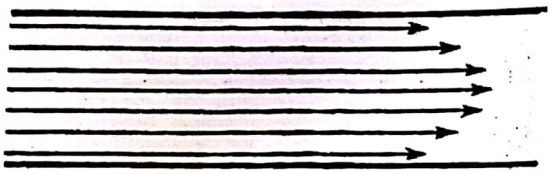
$500 < R_e < 2000$ – Transitional flow

Re > 2000 – Turbulent flow

Laminar flow or streamline flow in pipes (or tubes) occurs when a fluid flows in parallel layers, with no disruption between the layers.

Turbulent flow is a flow regime characterized by chaotic property changes. This includes rapid variation of pressure and flow velocity in space and time.

laminar flow



turbulent flow

In an open-channel flow Froude number is defined as the ratio of inertial and gravitational forces. Froude number used to describe the flow patterns.

$$F_r = \frac{V}{\sqrt{g\,D}}$$

(Note: D is Hydraulic depth, V = Average velocity)

Fr > 1 – Supercritical flow

Fr = 1 – Critical flow

Fr < 1 – subcritical flow

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Chezy equation

Introduced by the French engineer Antoine Chezy in 1768 while designing a canal for the water supply system of Paris

$V = C\sqrt{R\,S_0}$

C = Chezy coefficient

$$C = \sqrt{\frac{8\,g}{f}} = \frac{1}{n}\,R^{1/6}$$

Manning Equation

A resistance formula proposed by Robert Manning, an Irish engineer, for uniform flow in open channels, is

$$V = \frac{1}{n}\,R^{2/3}\,S_0^{1/2}$$

Values of Manning (n)

Channel type	Surface material and form	Manning's *n* range
River	earth, straight	0.02-0.025
	earth, meandering	0.03-0.05
	gravel (75-150mm), straight	0.03-0.04
	gravel (75-150mm), winding	0.04-0.08
unlined canal	earth, straight	0.018-0.025
	rock, straight	0.025-0.045
lined canal	concrete	0.012-0.017
lab. models	mortar	0.011-0.013
	Perspex	0.009

Question: Define a) Steady uniform flow, b) Darcy - Weisbarch equation (BWDB – 2014)

Solution:

Steady Uniform Flow: If the mean velocity and discharge do not change with respect to time and along the length is called Steady uniform flow.

For pipe flow, the Darcy - Weisbarch equation is,

$$h_f = \frac{f\,L\,V^2}{2\,g\,D}$$

Where, h_f = head loss due to friction in a pipe of diameter D and length L

f = Darcy-Weisbarch friction factor

Question: Water is flowing in a pipe of a diameter 12 cm at a velocity of 2 m/s. Find out the discharge. (DPDC – 2014)

Solution:

Area, $A = \frac{\pi\,d^2}{4} = \frac{3.1416 \times 0.12^2}{4} = 0.0113\ \text{m}^2$

Discharge, $Q = A\,V = 0.0113 \times 2 = 0.022\ \text{m}^3/\text{s}$

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Question: Calculate the diameter of a circular pipe that has discharges 2.5 cusec water with a velocity of 6 fps? (PGCL – 2014, PGCB – 2015, BIFPCL – 2015)

Solution:

We know, $Q = AV = \frac{\pi d^2}{4} x V$

$$d = \sqrt{\frac{4Q}{\pi V}} = \sqrt{\frac{4 x 2.5}{3.1416 x 6}} = 0.72 \text{ ft}$$

Question: Find the hydraulic depth (D) and hydraulic radius (R) of the rectangular channel of top width 10 m, height 5 m and water half of full. (NWDB – 2017)

Solution:

For rectangular channel, $b = B$
Height of water, $h = 5/2 = 2.5$ m
Area, $A = b\,h = 10 \times 2.5 = 25$ m^2
Wetted perimeter, $P = b + 2\,h = 10 + 2 \times 2.5 = 15$ m
Hydraulic depth, $D = A/B = 25/10 = 2.5$ m
Hydraulic radius, $R = A/P = 25/15 = 1.67$ m

Question: A pipe diameter of 50 cm is flowing a flow. The value of n = 0.016, S = 0.001. Determine the discharge. (WASA – 2017)

Solution:

Diameter of pipe, $d = 50$ cm $= 0.5$ m
Area, $A = \frac{\pi d^2}{4} = \frac{3.1416 \times 0.5^2}{4} = 0.196$ m^2
Perimeter, $P = \pi D = 3.1416 \times 0.5 = 1.5708$ m
Hydraulic radius, $R = A/P = 0.196/1.5708 = 0.125$ m
Discharge, $Q = \frac{1}{n} A R^{2/3} S^{1/2} = \frac{1}{0.016} \times 0.196 \times (0.125)^{2/3} \times (0.001)^{1/2} = 0.306$ m^3/s

Question: Flow velocity of a rectangular channel is 10 ft/sec. Calculate the depth of the rectangular channel from the following figure if n = 0.03 and S = 0.001 (BUET – 2017)

Solution:

We know, $V = \frac{1.49}{n} R^{2/3} S^{1/2}$

$$V = \frac{1.49}{n} \left(\frac{b\,h}{b + 2\,h}\right)^{2/3} S^{1/2}$$

$$10 = \frac{1.49}{0.03} \left(\frac{12\,h}{12 + 2\,h}\right)^{2/3} (0.001)^{1/2}$$

$h = 9.72$ ft

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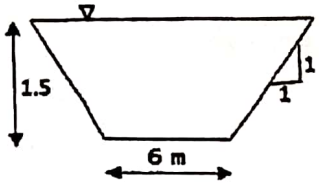
Question: A trapezoidal channel of slope 1:1 with a bottom width 6 m and depth 1.5 m. If the discharge through the channel is 15 m^3/s, find the velocity of the channel. (ERL – 2015, BCPCL – 2016, BCIC- 2016, BGFCL- 2017)

Solution:



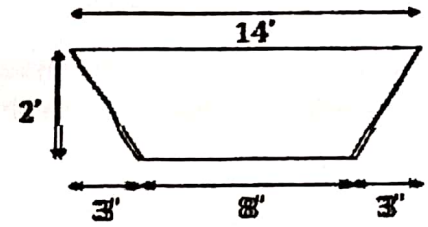
Given, $Q = 15\ m^3/s, b = 6\ m$ and $h = 1.5\ m$

$A = (b + s\,h)\,h = (6 + 1 \times 1.5)\,1.5 = 11.25\ m^2$

$$V = \frac{Q}{A} = \frac{15}{11.25} = 1.33\ m/s$$

Question: The trapezoidal channel shown has a manning's coefficient of n=0.013 and is laid at a slope of 0.0028. The depth of flow is 2 ft. What is the flow rate at ft^3/s? (SGFL – 2017)

Solution:



Given, $n = 0.013, S = 0.0028$

$$A = \frac{14 + 8}{2} \times 2 = 22\ ft^2$$

$$P = 8 + 2 \times \sqrt{3^2 + 2^2} = 15.21\ ft$$

$$R = A/P = 22/15.21 = 1.44\ ft$$

$$V = \frac{1.49}{n} R^{2/3} S^{1/2} = \frac{1.49}{0.013} 1.44^{2/3} 0.0028^{1/2} = 7.75\ ft/s$$

$$Q = A\,V = 22 \times 7.75 = 170.5\ ft^3/s$$

Question: Water flows in rectangular concrete open channel that is 12 m wide. The channel slope is 0.0028. If the velocity of the flow is 6 m/s. Find the depth of the flow (n = 0.013) (RPGCL – 2017)

Solution:

We know, $V = \frac{1}{n} R^{2/3} S^{1/2}$

$$V = \frac{1}{n}\left(\frac{b\,h}{b + 2\,h}\right)^{2/3} S^{1/2}$$

$$6 = \frac{1}{0.03}\left(\frac{12\,h}{12 + 2\,h}\right)^{2/3} (0.0028)^{1/2}$$

$$h = 2.55\ m$$

Question: A trapezoidal channel of 8 m width discharge water 56 m^3/s. Find out the depth of the channel if the side slope of the channel is 1:1.5. Consider bed slope 0.019 and manning coefficient 0.017. (AB – 2017)

Solution:

Area, $A = (b + s\,h)\,h$

Perimeter, $P = b + 2\,s\,h$

Hydraulic mean depth, $R = \frac{A}{P} = \frac{(b + s\,h)\,h}{b + 2\,s\,h}$

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We know, $Q = \frac{1}{n} A R^{2/3} S^{1/2}$

$Q = \frac{1}{n} A R^{2/3} S^{1/2}$

$A R^{2/3} = \frac{n Q}{\sqrt{S}}$

$[(b + s\,h)\,h] \times \left[\frac{(b + s\,h)\,h}{b + 2\,s\,h}\right]^{2/3} = \frac{0.017 \times 56}{\sqrt{0.019}}$

$[(8 + 1.5\,h)\,h]^{5/3} \times \left(\frac{1}{8 + 3\,h}\right)^{2/3} = 68.90$

$h = 0.881$ m

Question: Water flows through a 10 m wide rectangular channel at a velocity of 5 m/s. The channel slope is 0.0028. Find the depth of flow if $n = 0.013$ (PCCB – 2018)

Solution:

Assume, the depth of flow $= h$ m

Area, $A = b\,h$

Wetted perimeter, $P = b + 2\,h$

Hydraulic radius, $R = \frac{A}{P} = \frac{10h}{10 + 2h}$

We know, $V = \frac{1}{n} R^{2/3} S^{1/2}$

$5 = \frac{1}{0.013} \left(\frac{10\,h}{10 + 2\,h}\right)^{2/3} (0.0028)^{1/2}$

$\frac{10\,h}{10 + 2\,h} = 1.36$

$h = 1.87$ m

Question: A 24" diameter sewers with n = 0.013 is laid on a grade of 0.015. What will be the velocity and discharge when the depth of flow is 12". (BCIC – 2019)

Solution:

Area, $A = \frac{\pi}{4 \times 2}\left(\frac{24}{12}\right)^2 = 1.5708$ ft^2

Perimeter, $P = \frac{\pi \times 24}{2 \times 12} = 3.1416$ ft

Hydraulic radius, $R = A/P = 1.5708/3.1416 = 0.5$ ft

Velocity, $V = \frac{1.49}{n} R^{2/3} S^{1/2} = \frac{1.49}{0.013}(0.5)^{2/3}(0.0015)^{1/2} = 8.83$ fps

Discharge, $Q = A\,V = 8.83 \times 1.5708 = 13.87$ cfs

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Question: A rectangular troweled concrete channel 12' wide with a slope of 10' in 8000' is designed for a discharge of 600 cfs for a water temperature 40°F. Estimate the depth of the flow. (HBFC – 2018)

Solution:

Assume, $n = 0.015$ and $S = \frac{10}{8000} = 0.00125$

Area, $A = b\,h = 12\,h$

Wetted perimeter, $P = b + 2\,h = 12 + 2\,h$

Hydraulic radius, $R = \frac{A}{P} = \frac{b\,h}{b + 2\,h} = \frac{12\,h}{12 + 2\,h}$

We know, $Q = \frac{1.49}{n} A\,R^{2/3}\,S^{1/2}$

$$A\,R^{2/3} = \frac{n\,Q}{1.49\sqrt{S}}$$

$$A\,R^{2/3} = \frac{0.015 \times 600}{1.49 \times \sqrt{0.00125}}$$

$$(12\,h)^{5/3}\left(\frac{1}{12 + 2\,h}\right)^{2/3} = 170.845$$

Solving this equation by trial and error, $h = 5.6$ ft

Question: The bottom width of a trapezoidal channel is 9 m and depth is 5 m. The side slope of the channel is 1:2, manning's coefficient n=0.04 and is laid at a slope of 1/500. What is the flow rate of the channel? (DSCC – 2019)

Solution:

Area, $A = (b + s\,h)\,h = (9 + 2 \times 5)\,5 = 95\ m^2$

Wetted perimeter, $P = b + 2\,s\,h = 3 + 2 \times 2 \times 5 = 29$ m

Hydraulic radius, $R = A/P = 95/29 = 3.27$ m

Discharge, $Q = \frac{1}{n} A\,R^{2/3}\,S^{1/2} = \frac{1}{0.04} \times 95 \times (3.27)^{2/3}(0.002)^{1/2} = 229.9\ m^3/s$

Question: A rectangular channel width is 20 feet, Manning's co-efficient 0.021, longitudinal slope 0.0015 and discharge 300 cfs. Find normal depth and velocity. (BPDB – 2018)

Solution:

Area, $A = b\,h = 12\,h$

Wetted perimeter, $P = b + 2\,h = 12 + 2\,h$

Hydraulic radius, $R = \frac{A}{P} = \frac{b\,h}{b + 2\,h} = \frac{12\,h}{12 + 2\,h}$

We know, $Q = \frac{1.49}{n} A\,R^{2/3}\,S^{1/2}$

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$$A R^{2/3} = \frac{n Q}{1.49 \sqrt{S}}$$

$$A R^{2/3} = \frac{0.021 \times 300}{1.49 \times \sqrt{0.00115}}$$

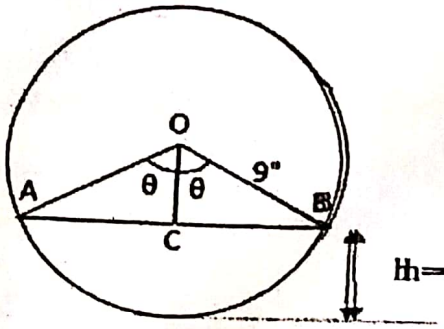
$$(20 h)^{5/3} \left(\frac{1}{20 + 2 h}\right)^{2/3} = 1109.177$$

Solving this equation by trial and error, $h = 3.308$ ft

$$\text{Velocity, } V = \frac{Q}{A} = \frac{Q}{B h} = \frac{3000}{220 \times 3.308} = 4.437 \text{ ft/s}$$

Question: An 18" diameter sewer pipe laid on a grade of 0.00115 and manning's coefficient is 0.013. Find the velocity and discharge if the depth of flow is 4.55". (BUET M.Sc – 2019)

Solution:



$$\cos\theta = \frac{4.5}{9}; \theta = 60^0$$

$$\angle AOB = 2\theta = 120^0$$

$$BC = 4.5 \tan\theta = 7.8"$$

$$\text{Area, } A = \frac{\pi}{4}\left(\frac{18}{12}\right)^2 \times \frac{120}{360} - 2 \times \frac{1}{2} \times \frac{4.55}{12} \times \frac{7.8}{12} = 0.3346 \text{ ft}^2$$

$$\text{Perimeter, } P = \frac{\pi \times 18}{12} \times \frac{120}{360} = 1.57 \text{ ft}$$

$$\text{Hydraulic radius, } R = \frac{A}{P} = \frac{0.3346}{0.222} = 0.222 \text{ ft}$$

$$V = \frac{1.49}{n} R^{2/3} S^{1/2} = \frac{1.49}{0.013}(0.222)^{2/3}(0.00115)^{1/2} = 5.11 \text{ fps}$$

$$Q = A V = 0.346 \times 5.1 = 1.766 \text{ ft}^3/\text{s}$$

Question: A rectangular channel is 3 m width and 1.5 m depth. Determine the rate of flow if manning's coefficient is, n = 0.014 and bed slope of the channel is, S = 0.0001

(BUET M.Sc – 2019)

Solution:

$$\text{Area, } A = b h = 3 \times 1.5 = 4.5 \text{ m}^2$$

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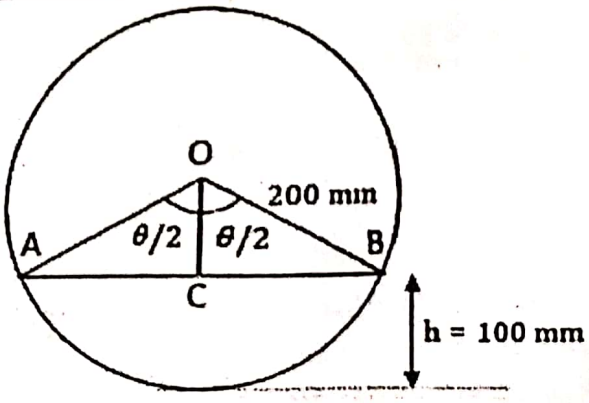
Wetted perimeter, $P = b + 2h = 3 + 2 \times 1.5 = 6$ m

Hydraulic radius, $R = A/P = 4.5/6 = 0.75$ m

Discharge, $Q = \frac{1}{n} A R^{2/3} S^{1/2} = \frac{1}{0.013} \times 4.5 \times (0.75)^{2/3} (0.001)^{1/2} = 8.39$ m^3/s

Question: A sewer of 400 mm diameter is laid on a grade of 0.015 and manning's coefficient is 0.013. Find the velocity and discharge if the depth of flow is 100 mm. (SGFL – 2020)

Solution:



$\cos \theta/2 = \frac{100}{200}; \theta/2 = 60^0$

$\angle AOB = \theta = 2 \times \theta/2 = 120^0 = 2.094$ rad

$BC = 100 \tan \theta/2 = 173$ mm

Area, $A = \frac{\pi}{4} \left(\frac{400}{1000}\right)^2 \times \frac{120}{360} - 2 \times \frac{1}{2} \times \frac{100}{1000} \times \frac{173}{1000} = 0.0245$ m^2

Or, $A = \frac{1}{8}(\theta - \sin \theta) D^2 = \frac{1}{8}(2.094 - \sin 120^0) \left(\frac{400}{100}\right)^2 = 0.025$ m^2

Perimeter, $P = \frac{\pi \times 400}{1000} \times \frac{120}{360} = 0.418$ m

Hydraulic radius, $R = \frac{A}{P} = \frac{0.025}{0.418} = 0.059$ m

Or, $R = \frac{1}{4}\left(1 - \frac{\sin \theta}{\theta}\right) D = \frac{1}{4}\left(1 - \frac{\sin 120^0}{2.094}\right) \times 0.4 = 0.06$ m.

$V = \frac{1.49}{n} R^{2/3} S^{1/2} = \frac{1}{0.013} (0.06)^{2/3} (0.0015)^{1/2} = 1.444$ m/s

$Q = A V = 0.025 \times 1.44 = 0.036$ m^3/s

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Question: A 15" sewer with $n = 0.015$ is laid on a slope of 0.018. What is the discharge and velocity of the sewer when flowing (i) half full and (ii) When depth of flow is 5". (34th BCS)

Solution:

When sewer is half full

Area, $A = \frac{\pi r^2}{2} = \frac{\pi}{2}\left(\frac{7.5}{12}\right)^2 = 0.6613 \text{ ft}^2$

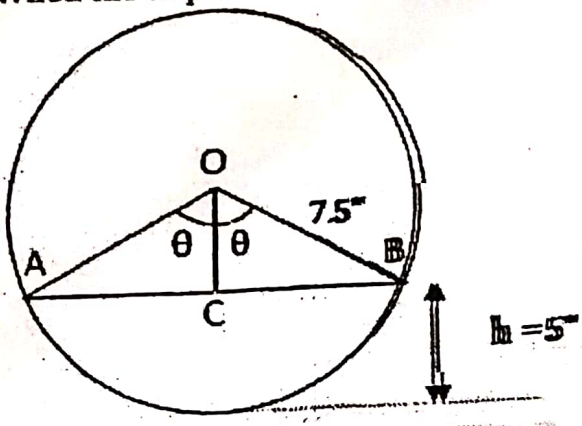
Perimeter, $P = \frac{\pi D}{2} = \frac{\pi \times 15}{2 \times 12} = 1.9635 \text{ ft}$

Hydraulic radius, $R = A/P = 0.6613/1.9635 = 0.3372 \text{ ft}$

Velocity, $V = \frac{1.49}{n} R^{2/3} S^{1/2} = \frac{1.49}{0.015}(0.3372)^{2/3}(0.0018)^{1/2} = 6.6100 \text{ fps}$

Discharge, $Q = AV = 0.6613 \times 6.6100 = 3.783 \text{ cfs}$

When the depth of flow is 5"



$\cos\theta = \frac{5}{7.5}; \theta = 48.18^0$

$\angle AOB = 2\theta = 96.36^0$

$BC = 5 \tan\theta = 5.58"$

Area, $A = \frac{\pi}{4}\left(\frac{15}{12}\right)^2 \times \frac{96.36}{360} - \frac{2 \times 11}{22} \times \frac{225}{144} \times \frac{5.58}{12} = 0.2332 \text{ ft}^2$

Perimeter, $P = \frac{\pi \times 15}{12} \times \frac{96.36}{360} = 1.005 \text{ ft}$

Hydraulic radius, $R = \frac{A}{P} = \frac{0.232}{1.05} = 0.22 \text{ ft}$

$V = \frac{1.49}{n} R^{2/3} S^{1/2} = \frac{1.49}{0.015}((0.22)^{2/3}((0.018)^{1/2} = 4.83 \text{ fps}$

$Q = AV = 0.232 \times 4.83 = 1.12 \text{ cfs}$

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Question: A rectangular channel is 10 ft width and 3 ft depth. Determine the velocity & rate of flow if manning's coefficient is, n = 0.015 and bed slope of the channel is, S = 0.003. (Combined Bank – 2020)

Solution:

Area, $A = b\,h = 10 \times 3 = 30\ \text{ft}^2$
Wetted perimeter, $P = b + 2\,h = 10 + 2 \times 3 = 16\ \text{ft}$
Hydraulic radius, $R = A/P = 30/16 = 1.875\ \text{ft}$

Velocity, $V = \frac{1}{n} R^{2/3} S^{1/2} = \frac{1}{0.015} \times (1.875)^{2/3} (0.003)^{1/2} = 5.56\ \text{ft/s}$

Discharge, $Q = A\,V = 30 \times 5.56 = 16.68\ \text{ft}^3/\text{s}$

Question: A rectangular channel of 4 m depth and 9 m width has a side slope of 0.0001, Calculate the shear velocity. (BUET M. Sc – 2018)

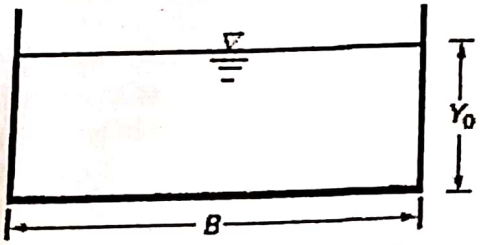
Solution:

$y = 4\ \text{m}, b = 6\ \text{m}$
Area, $A = b\,y = 6 \times 4 = 24\ \text{m}^2$
Perimeter, $P = b + 2\,y = 6 + 2 \times 4 = 14\ \text{m}$
Hydraulic radius, $R = A/P = 24/14 = 1.71\ \text{m}$
Shear velocity, $u = \sqrt{g\,R\,S_0} = \sqrt{9.81 \times 1.71 \times 0.0001} = 0.041\ \text{m/s}$

Wide Rectangular Channel

Channels with large bed-widths as compared to their respective depths are known as wide rectangular channels. In these channels, the hydraulic radius approximates to the depth of flow. Considering a unit width of a wide rectangular channel,
$A = y_0, R = y_0$ and $B = 1.0$
$\frac{Q}{B} = q =$ discharge per unit width $= \frac{1}{n} y_0^{5/3} S_0^{1/2}$

$$y_0 = \left(\frac{q\,n}{\sqrt{S_0}}\right)^{3/5}$$



This approximation of a wide rectangular channel is found applicable to rectangular channels with $y_0/B < 0.02$.

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Question: A wide channel has $n = 0.026$ and $S_o = 0.0028$. Calculate the normal water depth and velocity of flow if the channel discharges $30 m^3/s$. (BHDB — 2016)

Solution:

For wide channel, $A = h, R = h$

We know, $Q = \frac{1}{n} A R^{2/3} S^{1/2}$

$$A R^{2/3} = \frac{n Q}{\sqrt{S}}$$

$$h \times h^{2/3} = \frac{n Q}{\sqrt{S}}$$

$$h = \left(\frac{n Q}{\sqrt{S}}\right)^{3/5} = \left(\frac{0.026 \times 30}{\sqrt{0.0028}}\right)^{3/5} = 5.02 m$$

$$\text{Velocity, } V = \frac{Q}{A} = \frac{30}{5.02} = 5.977 m/s$$

Question: A wide channel has $n = 0.025$ and $S_o = 0.0025$. Calculate the normal water depth and velocity of flow if the channel discharges $3 m^3/s$. (SGEL — 2020, BGCB — 2020, IPEDC — 2020)

Solution:

For wide channel, $A = h, R = h$

We know, $Q = \frac{1}{n} A R^{2/3} S^{1/2}$

$$A R^{2/3} = \frac{n Q}{\sqrt{S}}$$

$$h \times h^{2/3} = \frac{n Q}{\sqrt{S}}$$

$$h = \left(\frac{n Q}{\sqrt{S}}\right)^{3/5} = \left(\frac{0.025 \times 3}{\sqrt{0.0025}}\right)^{3/5} = 1.27 m$$

$$\text{Velocity, } V = \frac{Q}{A} = \frac{3}{1.27} = 2.36 m/s$$

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Energy–Depth Relationships

The specific-energy head, E, of a flowing liquid is defined as the energy head with respect to a datum plane.

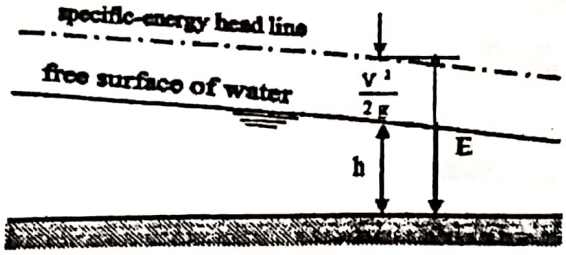
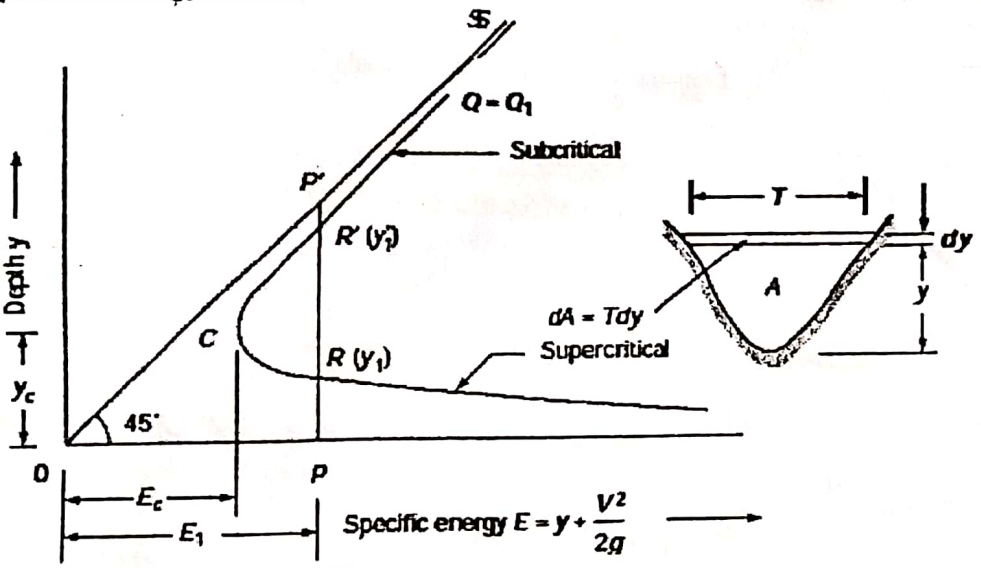


Fig. Specific-energy head of a flowing liquid

Specific energy, $E = y + \frac{V^2}{2g}$

Specific energy curve:



The depths of flow can be either PR = y_1 or PR' = y_1'

These two possible depths having the same specific energy are known as alternate depths. P'R' or P'R represents the velocity head. For a certain value of Q, $E = E_c$, the two depths will merge with each other. No value for y can be obtained when $E < E_c$, denoting that the flow under the given conditions is not possible in this region. The condition of minimum specific energy is known as the critical-flow condition and the corresponding depth y_c is known as the critical depth.

Specific energy, $E = y + \frac{V^2}{2g}$

At critical depth, the specific energy is minimum. Thus differentiating the equation with respect to y (keeping Q constant) and equating to zero,

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$$\frac{dE}{dy} = 1 - \frac{Q^2}{g A^3}\frac{dA}{dy} = 0$$

$\frac{dA}{dy} = T$ = Top width of the channel at the water surface

Designating the critical flow conditions by the suffix 'c'

$$\frac{Q^2 T_c}{g A_c^3} = 1$$

$$\frac{Q^2}{g} = \frac{T_c}{A_c^3}$$

Question: Draw a typical non-dimensional specific energy curve for triangular channel section for a given discharge. Write its application. (BWDB – 2014)

Solution:

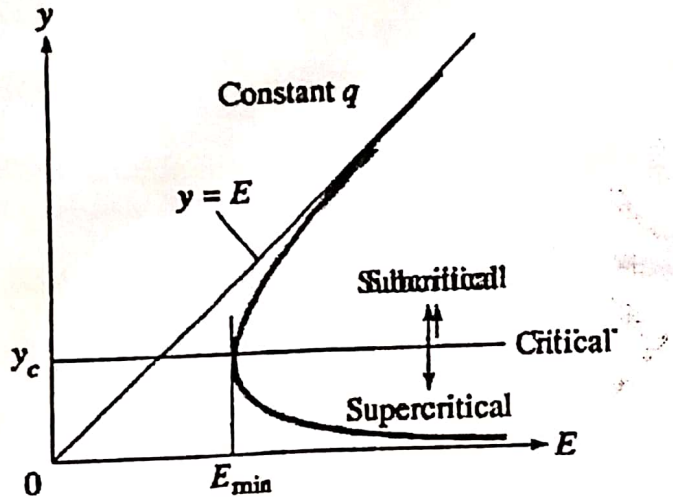


Fig: Specific Energy Curve

Calculation of the critical depth

Rectangular section: For a rectangular section, $A = B\,y$ and $T = B$

Hence, $\frac{Q^2 T_c}{g A_c^3} = \frac{V_c^2}{g\,y_c} = 1$

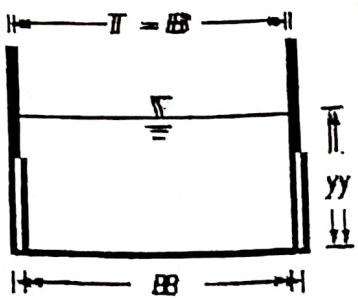
$$\frac{V_c^2}{2g} = \frac{1}{2}\,y_c$$

Specific energy at critical depth, $E_{cc} = y_c + \frac{V_c^2}{2g} = \frac{3}{2}y_c$

If q = discharge per unit width $= Q/B$

$$\frac{q^2}{g} = y_c^3$$

$$y_c = \left(\frac{q^2}{g}\right)^{1/3}$$



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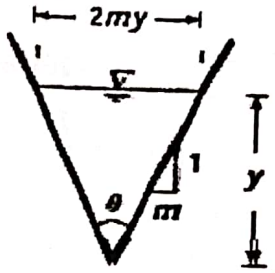
Trapezoidal channel: For a trapezoidal channel having bottom width of B and side slopes of m horizontal : 1 vertical

Area, $A = (B + m\,y)\,y$

Top width, $T = (B + 2\,m\,y)$

At the critical flow, $\frac{Q^2}{g} = \frac{A_c^3}{T_c} = \frac{(B + m\,y_c)^3\,y_c^3}{(B + 2\,m\,y_c)}$

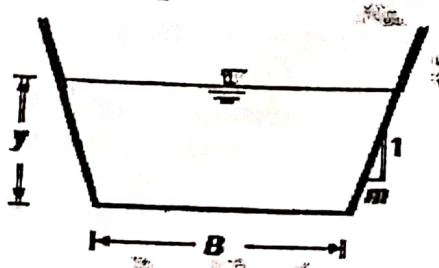


Triangular channel: For a triangular channel having a side slope of m horizontal : 1 vertical

$A = m\,y^2$ and $T = 2\,m\,y$

$$\frac{Q^2}{g} = \frac{A_c^3}{T_c} = \frac{m^3\,y_c^6}{2\,m\,y_c} = \frac{m^2\,y_c^5}{2}$$

Hence, $y_c = \left(\frac{2\,Q^2}{g\,m^2}\right)^{1/5}$



Specific energy at critical depth, $E_c = y_c + \frac{V_c^2}{2\,g} = y_c + \frac{Q^2}{2\,g\,A_c^2} = y_c + \frac{m^2\,y_c^5}{4\,m^2\,y_c^4} = 1.25\,y_c$

Question: A 2.5-m wide rectangular channel has a specific energy of 1.50 m when carrying a discharge of 6.48 m3/s. Calculate the alternate depths and corresponding Froude numbers.

Solution:

$$E = y + \frac{V^2}{2\,g} = y + \frac{Q^2}{2\,g\,B^2\,y^2} = y + \frac{6.48^2}{2 \times 9.81 \times 2.5^2\,y^2} = y + \frac{0.34243}{y^2}$$

Solving this equation by trial and error, the alternate depths y_1 and y_2 are obtained as $y_1 = 1.296$ m and $y_1 = 0.625$ m.

Froude number, $F = \frac{V}{\sqrt{g\,y}} = \frac{6.48}{2.5\,y\sqrt{9.81\,y}} = \frac{0.82756}{y^{3/2}}$

At $y_1 = 1.296$ m, $F_1 = 0.561$ and $y_2 = 1.296$ m, $F_2 = 0.561$

The depth $y_1 = 1.296$ m is in the subcritical flow region and the depth $y_2 = 1.296$ m is in the supercritical flow region.

Question: Calculate the critical depth and the corresponding specific energy for a discharge of 5.0 m³/s in the following channels:

(a) Rectangular channel, B = 2.0 m

(b) Triangular channel, m = 0.5

Solution:

(a) Rectangular channel

$q = \frac{Q}{B} = \frac{5.0}{2.0} = 2.5\ \text{m}^3/\text{s/m}$

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$$y_c = \left(\frac{q^2}{g}\right)^{1/3} = \left(\frac{2.5^2}{9.81}\right)^{1/3} = 0.860 \text{ m}$$

Since for a rectangular channel $\frac{E_c}{y_c} = 1.5, E_c = 1.290$ m

(b) Traingular channel

$$y_c = \left(\frac{2\,Q^2}{g\,m^2}\right)^{1/5} = \left(\frac{2 \text{ x } 5^2}{9.81 \text{ x } 0.5^2}\right)^{1/5} = 1.828 \text{ m}$$

for a triangular channel $\frac{E_c}{y_c} = 1.25, E_c = 2.284$ m

Question: A trapezoidal channel with a bed width of 4.0 m and side slopes of 1.5 H: I V carries a certain discharge. (a) Based on observations, if the critical depth of the flow is estimated as 1.70 m, calculate the discharge in the channel. (b) If this discharge is observed to be flowing at a depth of 2.50 m in a reach, estimate the Froude number of the flow in that reach.

Solution:

(a) At critical depth

Area, $A_c = (B + m\,y_c)\,y_c = (4 + 1.5 \text{ x } 1.7)\,1.7 = 11.135 \text{ m}^2$

Top width, $T_c = (B + 2\,m\,y_c) = (4 + 2 \text{ x } 1.5 \text{ x } 1.7) = 9.10\ m$

At the critical flow, $\frac{Q^2}{g} = \frac{A_c^3}{T_c} = \frac{11.135^3}{9.10} = 151.715$

Discharge, $Q = 38.759 \text{ m}^3/s$

(b) When the depth of flow, $y = 2.50$ m

Area, $A = (B + m\,y)\,y = (4 + 1.5 \text{ x } 2.5)\,2.5 = 19.375 \text{ m}^2$

Top width, $T = (B + 2\,m\,y) = (4 + 2 \text{ x } 1.5 \text{ x } 2.50) = 11.5$ m

$$\frac{A}{T} = \frac{19.375}{11.50} = 1.685 \text{ m}$$

$$V = \frac{Q}{A} = \frac{38.579}{19.375} = 1.991 \text{ m/s}$$

Froude number, $F = \frac{V}{\sqrt{g\left(\frac{A}{T}\right)}} = \frac{1.9931}{\sqrt{9.81 \text{ x } 1.6685}} = 0.4910$

Question: Calculate the bottom width of a channel required to carry a discharge of 15.0 m^3/s as a critical flow at a depth of 1.2 m, if the channel section is (a) rectangular, and (b) trapezoidal with side slope 1.5 horizontal: 1 vertical.

Solution:

(a) Rectangular section

$$y_c = \left(\frac{q^2}{g}\right)^{1/3}$$

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$q = \sqrt{g\, y_c^3} = \sqrt{9.81 \times 1.2^3} = 4.117\ m^3/s/m$

B = bottom width $= \frac{15}{4.117} = 3.643$ m

(b) Trapezoidal section

Area, $A_c = (B + m\, y_c)\, y_c = (B + 1.5 \times 1.2)\, 1.2 = (B + 1.8)\, 1.2$

Top width, $T_c = (B + 2\, m\, y_c) = (B + 2 \times 1.5 \times 1.2) = (B + 3.6)$

At the critical flow, $\frac{Q^2}{g} = \frac{A_c^3}{T_c} = \frac{(B + m\, y_c)^3\, y_c^3}{(B + m\, y_c)}$

$\frac{15^2}{9.81} = \frac{(B + 1.8)^3\, 1.2^3}{(B + 3.6)}$

By trial and error, $B = 2.535$ m

Question: Calculate the normal and critical depth of rectangular wide channel having discharge per meter width = 3 m²/s, Manning's co-efficient = 0.025 and slope = 0.0005 (BADC — 2020)

Solution:

For wide channel, $A = h, R = h$

We know, $Q = \frac{1}{n} A\, R^{2/3}\, S^{1/2}$

$A\, R^{2/3} = \frac{n\, Q}{\sqrt{S}}$

$h \times h^{2/3} = \frac{n\, Q}{\sqrt{S}}$

Normal depth, $h = \left(\frac{n\, Q}{\sqrt{S}}\right)^{3/5} = \left(\frac{0.025 \times 3}{\sqrt{0.0005}}\right)^{3/5} = 2.06$ m

Critical depth, $y_c = \left(\frac{q^2}{g}\right)^{1/3} = \left(\frac{3^2}{9.81}\right)^{1/3} = 0.97$ m

Hydraulically efficient channel section

A channel section is considered as the most economical or most efficient when it passes a maximum discharge for given cross section area, resistance coefficient, and bottom slope. Velocity is maximum when the hydraulic radius is maximum. And if we take the area as constant, hydraulic radius is maximum if the wetted perimeter is minimum. A semicircular section is the best economical channel but due to difficulty in creating it, a trapezoidal section can be considered most efficient.

The conveyance of a channel section of a given area increases with a decrease in its perimeter. Hence a channel section having the minimum perimeter for a given area of flow provides the maximum value of the conveyance. With the slope, roughness coefficient and area of flow fixed, a minimum perimeter section will represent the hydraulically efficient section as it conveys the maximum discharge. This channel section is also called the ***best section***.

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Sl. No	Channel Shape	Area (A_m)	Wetted Perimeter (P_m)	Width (B_m)	Hydraulic Radius (R_m)	Top width (T_m)	$\frac{Q_m}{y_m^{8/3} S_0^{1/2}} = K_m$
1	Rectangle (Half square)	$2y_m^2$	$4y_m$	$2y_m$	$\frac{y_m}{2}$	$2y_m$	1.260
2	Trapezoidal (Half regular hexagon, $m = \frac{1}{\sqrt{3}}$)	$\sqrt{3}y_m^2$	$2\sqrt{3}y_m$	$\frac{2}{\sqrt{3}}y_m$	$\frac{y_m}{2}$	$\frac{4y_m}{\sqrt{3}}$	1.091
3	Circular (semi-circular)	$\frac{\pi}{2}y_m^2$	πy_m	$D = 2y_m$	$\frac{y_m}{2}$	$2y_m$	0.968
4	Triangle (Vertex angle = 90°)	y_m^2	$2\sqrt{3}y_m$	–	$\frac{y_m}{2\sqrt{2}}$	$2y_m$	0.500

Question: Calculate the normal depth of earthen channel flow rate = 10 m^3/s and channel slope 1cm/1km. Assume best hydraulic triangular section. **(BIWTA – 2019)**

Solution:

Considering vortex angle = 90° and $m = 0.0155$

$A = m\,y^2 = y^2$

For efficient section, $R = \frac{y}{2\sqrt{2}}$

Slope, $S = \frac{1\text{ cm}}{1\text{ km}} = \frac{1}{1000 \times 1000} = 10^{-5.5}$

Discharge, $Q = \frac{1}{n} A R^{2/3} S^{1/2}$

$10 = \frac{1}{0.015} y^2 \left(\frac{y}{2\sqrt{2}}\right)^{2/3} (10^{-5})^{1/2}$

$y = 5.51\text{ m}$

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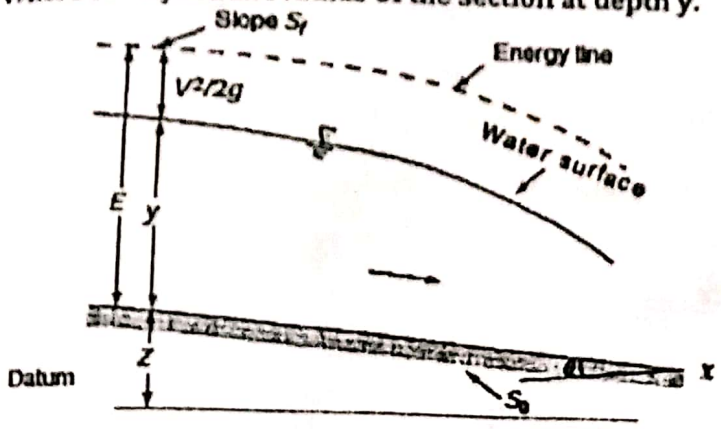
Gradually varied flow

A steady non-uniform flow in a prismatic channel with gradual changes in its water surface elevation is termed as gradually varied flow (GVF). The backwater produced by a dam or weir across a river and the drawdown produced at a sudden drop in a channel are few typical examples of GVF. In a GVF, the velocity varies along the channel and consequently the bed slope, water surface slope, and energy slope will all differ from each other. Regions of high curvature are excluded in the analysis of this flow.

The energy slope, $S_f = \frac{n^2 V^2}{R^{4/3}}$

Where R = hydraulic radius of the section at depth y.



Schematic sketch of GVF

$\frac{dH}{dx} = -S_f$

S_f is the downward slope of the energy grade line or friction slope. Since the total energy of the flow always decreases in the direction of motion, it is common to consider the slope of the decreasing energy line as positive.

$\frac{dZ}{dx} = -S_0$

S_0 is the bottom slope. It is common to consider the channel slope with bed elevations decreasing in the downstream direction as positive.

$\frac{dE}{dx} = S_0 - S_f$

The specific energy only changes if there is a difference between the geometric and friction slopes.

$\frac{dy}{dx} = \frac{S_0 - S_f}{1 - Fr^2} = \frac{S_0 - S_f}{1 - \frac{Q^2 T}{g A^3}}$

$\frac{dy}{dx}$ represents the water surface slope relative to the bottom of the channel.

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Gradually varied flow channel classification:

Sl. No	Channel category	Symbol	Characteristic condition	Remark
1	Mild slope	M	$y_n > y_c$	Subcritical flow at normal depth
2	Steep slope	S	$y_c > y_n$	Supercritical flow at normal depth
3	Critical slope	C	$y_c = y_n$	Critical flow at normal depth
4	Horizontal bed	H	$S_0 = 0$	Cannot sustain uniform flow
5	Adverse slope	A	$S_0 < 0$	Cannot sustain uniform flow

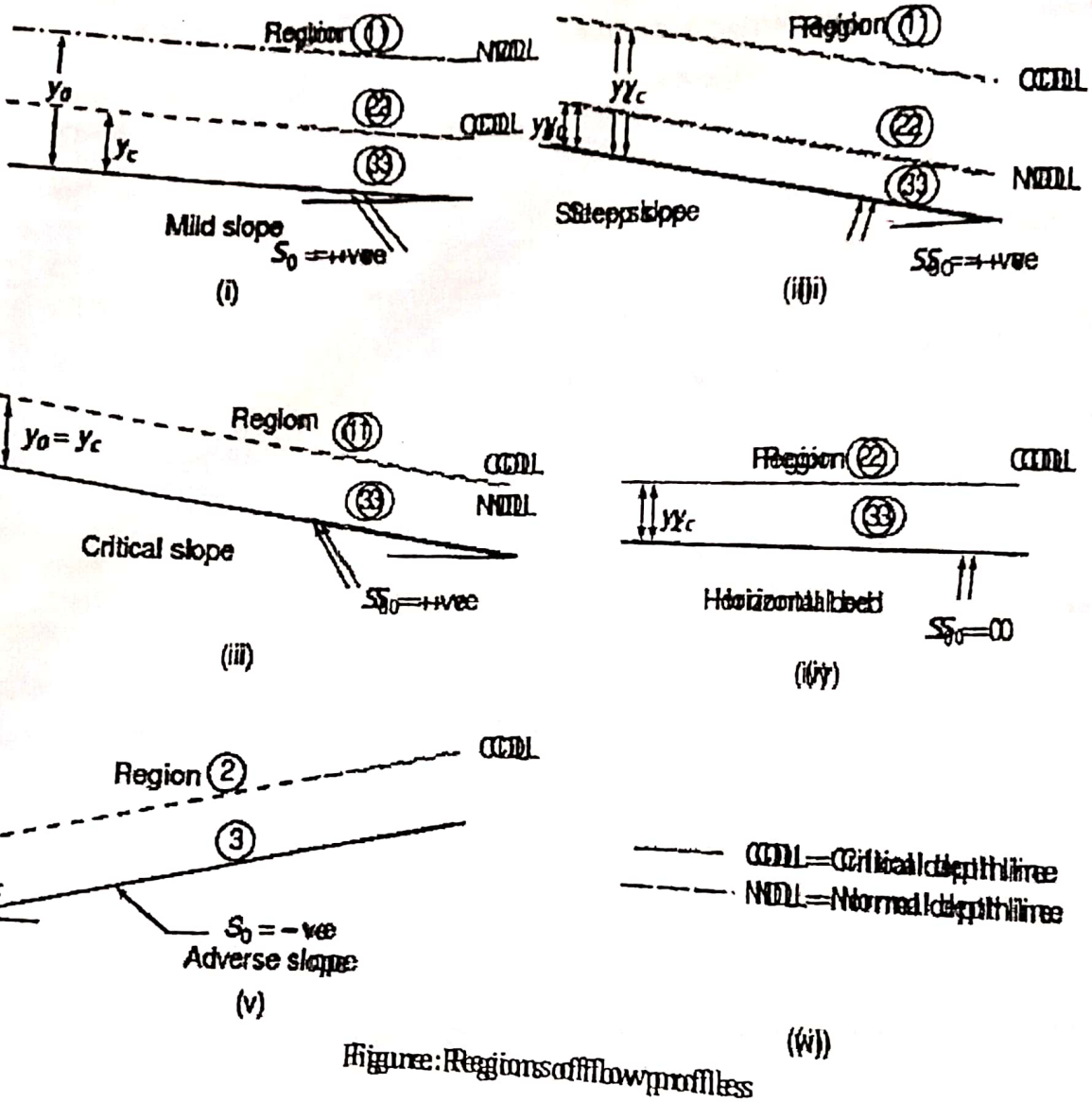

Figure: Regions of flow profiles

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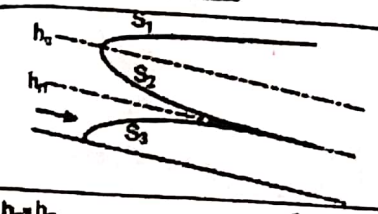
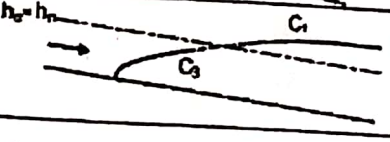
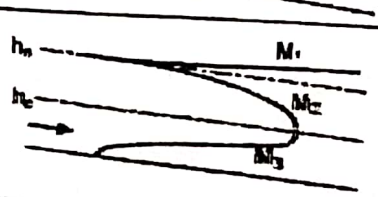
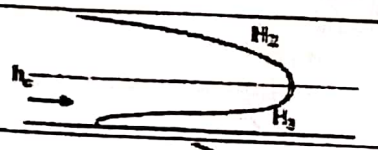
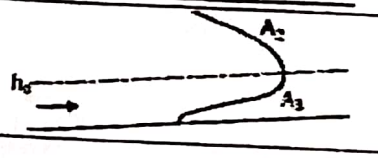
Type	Symbol	Definition	Sketches	Examples
STEEP (normal flow is supercritical)	S1	$h > h_c > h_n$		Hydraulic jump upstream with obstruction or reservoir controlling water level downstream.
	S2	$h_c > h > h_n$		Change to steeper slope.
	S3	$h_c > h_n > h$		Change to less steep slope.
CRITICAL (undesirable: unstable unsteady flow)	C1	$h > h_c = h_n$		
	C3	$h_c = h_n > h$		
MILD (normal flow is subcritical)	M1	$h > h_n > h_c$		Obstruction or reservoir controlling water level downstream.
	M2	$h_n > h > h_c$		Approach to free overfall.
	M3	$h_n > h_c > h$		Hydraulic jump downstream; change from steep to mild slope or downstream of sluice gate.
HORIZONTAL (limiting mild slope; $h_n \to \infty$)	H2	$h > h_c$		Approach to free overfall.
	H3	$h_c > h$		Hydraulic jump downstream; change from steep to horizontal or downstream of sluice gate.
ADVERSE (upslope)	A2	$h > h_c$		
	A3	$h_c > h$		

Calculation of GVF by Direct-Step Method

This method is possibly the simplest and is suitable for use in prismatic channels. Consider the differential-energy equation of GVF.

$$\frac{dE}{dx} = S_0 - S_f$$

Writing this in the finite difference form, $\frac{\Delta E}{\Delta x} = S_0 - \bar{S}_f$

Where, $\bar{S}_f$ = average friction slope in the reach Δx

$$\Delta x = \frac{\Delta E}{S_0 - \bar{S}_f}$$

Between section 1 and 2, $(x_2 - x_1) = \Delta x = \frac{(E_2 - E_1)}{S_0 - \frac{1}{2}(S_{f1} + S_{f2})}$

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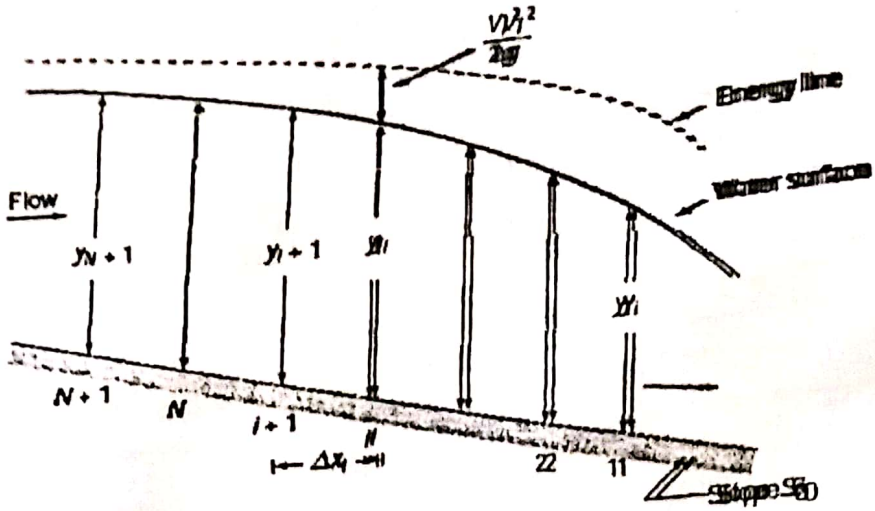
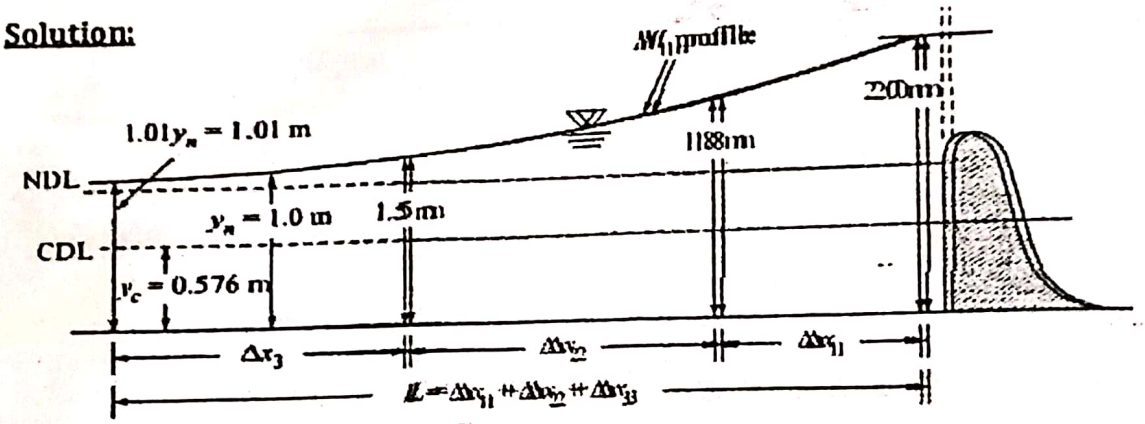


Question: A rectangular channel 8m wide has a bed slope of 0.0016 and $n = 0.0225$. Normal depth and critical depths are 1 m and 0.576 m. compute the length of back water created by a dam which backs up a depth 2 m immediately behind the dam in a gradually varied flows?
(BWDB – 2019)

Solution:



$y_1 = 2\text{ m}, y_n = 1\text{ m}, y_2 = 1.01\,y_n = 1 \times 1.01 = 1.01\text{ m}$

Area, $A = 8 \times 1 = 8\text{ m}^2$

Perimeter, $P = 8 + 2 \times 1 = 10\text{ m}$

$$V = \frac{1}{n} R^{2/3} S^{1/2} = \frac{1}{0.0025} \left(\frac{8}{10}\right)^{2/3} (0.0016)^{1/2} = 13.78\text{ m/s}$$

$$Q = AV = 8 \times 13.78 = 110.3\text{ m}^3/\text{s}$$

$$V_1 = \frac{Q}{A_1} = \frac{Q}{b\,y_1} = \frac{110.3}{8 \times 2} = 6.89\text{ m/s}$$

$$E_1 = y_1 + \frac{V_1^2}{2g} = 2 + \frac{6.89^2}{2 \times 9.81} = 4.41\text{ m}$$

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$$S_{f1} = \frac{V_1^2\, n^2}{R_1^{4/3}} = \frac{6.89^2 \times 0.0025^2}{\left(\frac{8 \times 2}{8 + 2 \times 2}\right)^{4/3}} = 0.000239$$

$$V_2 = \frac{Q}{A_2} = \frac{Q}{b\, y_2} = \frac{110.3}{8 \times 1.01} = 13.65 \text{ m/s}$$

$$E_2 = y_2 + \frac{V_2^2}{2\, g} = 1.01 + \frac{13.65^2}{2 \times 9.81} = 10.5 \text{ m}$$

$$S_{f2} = \frac{V_2^2\, n^2}{R_2^{4/3}} = \frac{10.5^2 \times 0.0025^2}{\left(\frac{8 \times 1.01}{8 + 2 \times 1.01}\right)^{4/3}} = 0.000918$$

$$\bar{S_f} = \frac{S_{f1} + S_{f2}}{2} = \frac{0.000239 + 0.000918}{2} = 0.00057$$

$$L = \frac{E_2 - E_1}{S_b - \bar{S_f}} = \frac{10.5 - 4.41}{0.0016 - 0.00057} = 5912.62 \text{ m} = 5.91 \text{ km}$$

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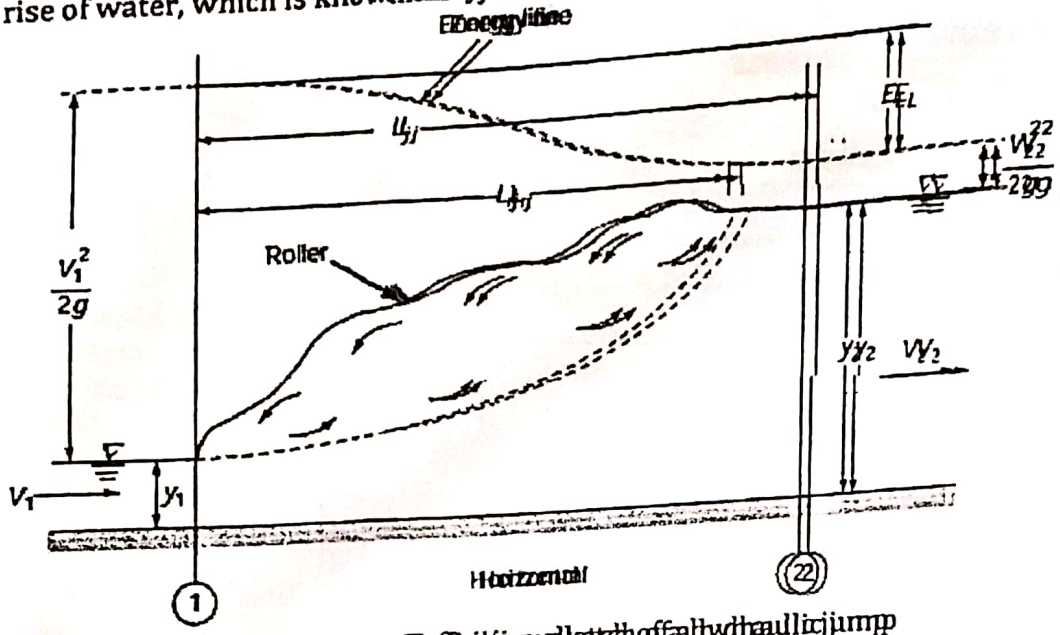
Rapidly Varied Flow

Question: Define Hydraulic jump. Describe the types of hydraulic jump. (BUET M.Sc – 2011)

Solution:

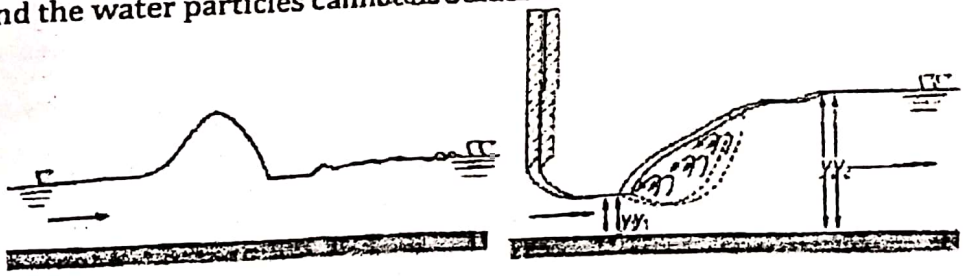
When a supercritical flow is made to change abruptly to subcritical flow, the result is an abrupt rise of water, which is known as hydraulic jump.



Definition sketch of a hydraulic jump

1. Undular Hydraulic Jump ($1 < Fr < 1.7$): Undular jump is irregular, not properly formed and there are certain turbulences in water particles.

2. Weak Jump ($1.7 < Fr < 2.5$): Weak jump takes place when the velocity in water is very less and the water particles cannot be stabilized and flows in various ways.



(a) Undular jump, $1.0 < F_1 \leq 1.7$

(b) Weak jump, $1.7 < F_1 \leq 2.5$

3. Oscillating Hydraulic Jump ($2.5 < Fr < 4.5$): Oscillating jump forms when an oscillating jet enter into super critical state and there the number of particles starts oscillating in clockwise or either anticlockwise direction, forming slighter tides or waves to the top surface.

4. Steady Hydraulic Jump ($4.5 < Fr < 9$): In steady jump, the ended surface is quite rough and the particles start to tend in one direction with heavy velocity and due to the effect of frictional losses are more in this type of jump.

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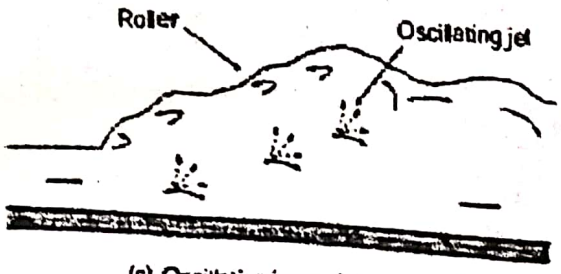
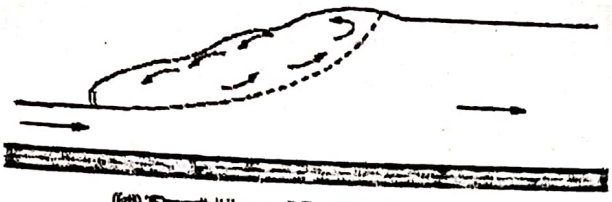
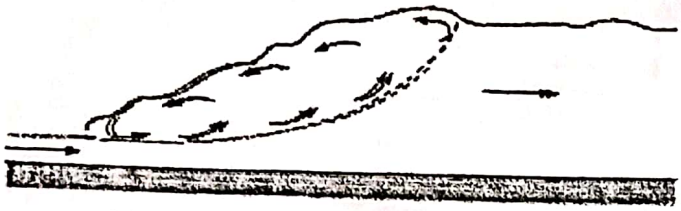


(c) Oscillating jump, $2.5 < Fr_1 \leq 4.5$



(d) 'Steady' jump, $4.5 < Fr_1 \leq 9.0$

5. Strong Hydraulic Jump($Fr > 9$): Strong jump is a perfect jump formed when frictional losses are more, air pressure division is equal and velocity is very high that losses take place. The water changes its state from super critical to subcritical in very shorter length when compared to all other types of hydraulic jumps, so this jump is highly preferred in dam structures.



(e) Strong or Choppy jump, $Fr_1 > 9.0$

Uses of hydraulic jump

1. To dissipate excessive energy.
2. To increase the water level on the downstream side.
3. To reduce the net uplift force by increasing the weight, i.e. due to increased depth.
4. To increase the discharge from a sluice gate by increasing the effective head causing flow.
5. To provide a control section.
6. For thorough mixing of chemicals in water.
7. For aeration of drinking water.
8. For removing air pockets in a pipe line.

The initial depth and the sequent depth

The depth before the jump is always less than the depth after the jump. The depth before the jump is called the initial depth h_1 and that after the jump is called the sequent depth h_2.

$$\frac{h_2}{h_1} = \frac{1}{2}\left(\sqrt{1 + 8\,Fr_1^2} - 1\right)$$

Energy loss due to hydraulic jump

Energy-head loss, ΔE_L, is due to the violent turbulent mixing and dissipation that occur within the jump itself. The loss of energy head due to the occurrence of the hydraulic jump is the difference between the specific energy heads at sections 1-2 and 2-2. Mathematically,

$$\Delta E = E_1 - E_2 = \left(h_1 + \frac{V_1^2}{2g}\right) - \left(h_2 + \frac{V_2^2}{2g}\right)$$

$$\Delta E_L = E_1 - E_2 = \frac{(h_2 - h_1)^3}{4\,h_1\,h_2}$$

The ratio $\frac{\Delta E_L}{E_1}$ is known as the relative energy – head loss.

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Efficiency

The ratio of the specific energy after the jump to that before the jump is defined as the efficiency of the jump. It can be shown that the efficiency is,

$$\frac{E_2}{E_1} = \frac{(8\,Fr_1^2 + 1)^{3/2} - 4\,Fr_1^2 + 1}{8\,Fr_1^2\,(2 + Fr_1^2)}$$

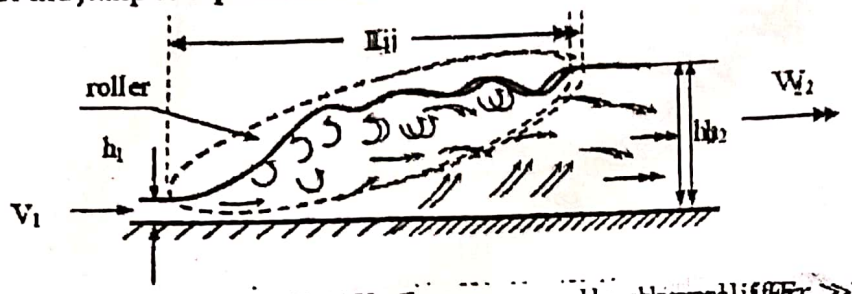
Height of hydraulic jump

The difference between the depths after and before the jump is the height of the jump.

$$h_j = h_2 - h_1$$

Length of jump

The length of the hydraulic jump may be defined as the distance measured from the front face of the jump to a point on the surface immediately downstream of the roller as indicated in Fig.



Pavolovski.s formula (1940), for a rectangular channel, if $Fr_1 \gg 1.0$:

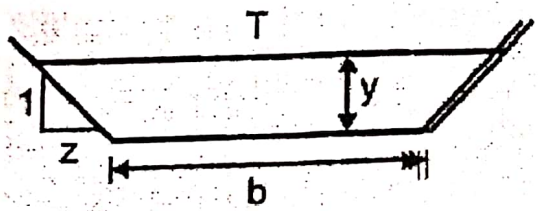
$$L_j = 2.5\,(1.9\,h_2 - h_1)$$

Dissipation of power in hydraulic jump

$$\Delta P = \rho\, g\, Q\, (E_1 - E_2)$$

Question: A trapezoidal channel of 1.15 m depth and 6 m bottom width with a flow velocity 1.00 m/s has a side slope of 1:2. Find out if the flow is sub-critical or super-critical? (BKB—2018)

Solution:



Top width of the channel, $T = b + 2\,s\,h = 6 + 2 \times 2 \times 1.15 = 12$ m

Area, $A = (b + s\,h)\,h = (6 + 2 \times 1.15) \times 1.15 = 13.5$ m^2

Hydraulic depth, $D = \frac{A}{T} = \frac{13.5}{12} = 1.125$ m

Froude number, $F_r = \frac{V}{\sqrt{g\,D}} = \frac{1.00}{\sqrt{9.81 \times 1.125}} = 0.301$

$F_r < 1$, Supercritical flow

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Question: A rectangular channel 6m wide and discharge 1200 l/s of water into a 6m wide apron with zero slope with a mean velocity of 16 m/s. What is the height of hydraulic jump? (BWDB – 2016)

Solution:

Hydraulic jump, $h_f = h_2 - h_1$

$$V_1 = \frac{Q}{b\,h_1}$$

$$h_1 = \frac{Q}{b\,V_1} = \frac{1200 \times 10^{-3}}{6 \times 16} = 0.0125$$

$$Fr_1 = \frac{V_1}{\sqrt{g\,h_1}} = \frac{16}{\sqrt{9.81 \times 0.0125}} = 45.69$$

$$\frac{h_2}{h_1} = \frac{1}{2}\left(\sqrt{1 + 8\,Fr_1^2} - 1\right)$$

$$h_2 = \frac{h_1}{2}\left(\sqrt{1 + 8\,Fr_1^2} - 1\right) = \frac{0.0125}{2}\left(\sqrt{1 + 8 \times 45.69^2} - 1\right) = 0.80$$

$$h_f = h_2 - h_1 = 0.80 - 0.0125 = 0.7875 \text{ m}.$$

Question: The sequent depth ratio of a hydraulic jump in a rectangular channel is 16.48. Find Froude number at the beginning of jump and type of jump. (BWDB – 2014)

Solution:

$$\frac{h_2}{h_1} = \frac{1}{2}\left(\sqrt{1 + 8\,Fr_1^2} - 1\right)$$

$$16.48 = \frac{1}{2}\left(\sqrt{1 + 8\,Fr_1^2} - 1\right)$$

$$1 + 8\,Fr_1^2 = 1153.28$$

$Fr_1 = 12$ which is greater than 9

So that jump is strong.

Question: A rectangular channel is 3 m wide and depth of water 4 m flows at a velocity of 2 m/s. In the downstream a smooth upward step in the channel bed to produce a depth 1.5 m. Calculate the change in the water level. (BWDB – 2019)

Solution:

Here, $B = 3$ m, $y_1 = 4$ m, $y_2 = 1.5$ m, $V_1 = 2$ m/s

$$F_{r1} = \frac{V_1}{\sqrt{g\,y_1}} = \frac{2}{\sqrt{9.81 \times 4}} = 0.31 < 1 \text{ (Sub-critical flow)}$$

We know, $A_1\,V_1 = A_2\,V_2$

$3 \times 4 \times 2 = 3 \times 1.5 \times V_2$

$V_2 = 5.333$ m/s

Specific energy at upstream, $E_1 = y_1 + \frac{V_1^2}{2g} = 4 + \frac{2^2}{2 \times 9.81} = 4.20$ m

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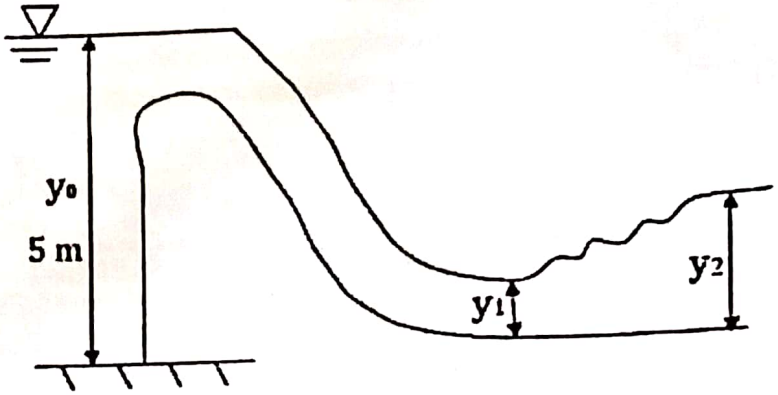
Specific energy at upstream, $E_2 = y_2 + \frac{V_2^2}{2g} = 1.5 + \frac{5.333^2}{2 \times 9.81} = 2.95$ m

Height of upward step, $\Delta Z = E_1 - E_2 = 4.20 - 2.95 = 1.25$ m

Change in water level, $\Delta h = y_1 - y_2 - \Delta Z = 4 - 1.5 - 1.25 = 1.25$ m

Question: A spillway discharge 2.5 m³/s/m width. For occurring hydraulic jump, what should be the value of Y_2. Neglect energy loss. (BWDB – 2018)



Solution:

By energy equation,

$$y_0 = y_1 + \frac{V_1^2}{2g} = y_1 + \frac{q^2}{2g y_1^2}$$

$$5 = y_1 + \frac{2.5^2}{2 \times 9.8 \times y_1^2}$$

$$19.62 y_1^2 - 98.1 y_1^2 + 6.25 = 0$$

By trial and error, $y_1 = 0.259$ m

Now, $V_1 = \frac{q}{y_1} = \frac{2.5}{0.259} = 9.65$ m/s

$$Fr_1 = \frac{V_1}{\sqrt{g y_1}} = \frac{9.65}{\sqrt{9.81 \times 0.259}} = 6.05$$

$$\frac{y_2}{y_1} = \frac{1}{2}\left(\sqrt{1 + 8 Fr_1^2} - 1\right)$$

$$y_2 = \frac{y_1}{2}\left(\sqrt{1 + 8 Fr_1^2} - 1\right) = \frac{0.259}{2}\left(\sqrt{1 + 8 \times 6.05^2} - 1\right) = 2.09 \text{ m}$$

Question: In a hydraulic jump occurring in a rectangular channel of 3.0 m, width, the discharge is 7.8 m³/s and the depth before the jump is 0.28 m. Estimate (i) sequent depth, and (ii) the energy loss in the jump.

Solution:

$$V_1 = \frac{Q}{A} = \frac{7.8}{3 \times 0.28} = 9.286 \text{ m/s}$$

$$Fr_1 = \frac{V_1}{\sqrt{g y_1}} = \frac{9.286}{\sqrt{9.81 \times 0.28}} = 5.603$$

Sequent depth ratio, $\frac{y_2}{y_1} = \frac{1}{2}\left(\sqrt{1 + 8 Fr_1^2} - 1\right) = \frac{1}{2}\left(\sqrt{1 + 8 \times 5.603^2} - 1\right) = 7.424$

Sequent depth $= y_2 = 0.28 \times 7.424 = 2.08$ m

Energy loss, $\Delta E_L = \frac{(y_2 - y_1)^3}{4 y_1 y_2} = \frac{(2.08 - 0.28)^3}{4 \times 0.28 \times 2.08} = 2.503$ m

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Question: A hydraulic jump takes place in a rectangular channel with sequent depths of 0.25 m and 1.50 m at the beginning and end of the jump respectively. Estimate the (i) discharge per unit width of the channel and (ii) energy loss.

Solution:

$$\frac{y_2}{y_1} = \frac{1}{2}\left(\sqrt{1 + 8\,Fr_1^2} - 1\right)$$

$$\frac{1.50}{0.25} = \frac{1}{2}\left(\sqrt{1 + 8\,Fr_1^2} - 1\right)$$

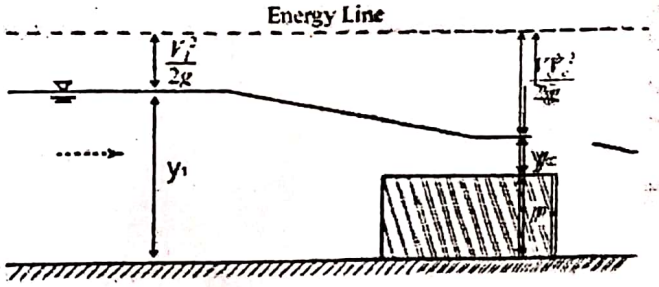
$$Fr_1 = 4.583$$

$$V_1 = Fr_1\sqrt{g\,y_1} = 4.583 \times \sqrt{9.81 \times 0.25} = 7.177 \text{ m/s}$$

Discharge per unit width $= q = V_1\,y_1 = 7.177 \times 0.25 = 1.794 \text{ m}^3/\text{s/m width}$

$$\text{Energy loss, } E_L = \frac{(y_2 - y_1)^3}{4\,y_1\,y_2} = \frac{(1.50 - 0.25)^3}{4 \times 0.25 \times 1.50} = 1.302 \text{ m}$$

Question: Determine the height of broad crested weir (P) from given data Width of flume = 25 cm, actual discharge = 7020.5 cm³/sec, neglect head loss. (BB – 2017)



Solution:

Given, $Q = 7020.5 \text{ cm}^3/\text{s}$, $B = 25$ cm, $y_1 = 17.33$ cm, $P = ?$

$$y_c = \sqrt[3]{\frac{\alpha\,Q^2}{g\,b^2}} = \sqrt[3]{\frac{1 \times 7020.5^2}{981 \times 25^2}} = 4.32 \text{ cm}$$

$$V_c = \frac{Q}{A_c} = \frac{7020.5}{25 \times 4.32} = 65.07 \text{ cm/s}$$

$$V_1 = \frac{Q}{A_1} = \frac{7020.5}{25 \times 17.33} = 16.20 \text{ cm/s}$$

$$y_1 + \frac{V_1^2}{2\,g} + Z_1 = y_c + \frac{V_c^2}{2\,g} + P$$

$$17.33 + \frac{16.20^2}{2 \times 9.81} + 0 = 4.32 + \frac{65.07^2}{2 \times 9.81} + P$$

$$P = 10.99 \text{ cm}$$

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Irrigation Engineering

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Irrigation Engineering

Irrigation: Irrigation may be defined as the process of artificially supplying water to soil for raising crops. It is a science of planning and designing an efficient, low-cost, economic irrigation system tailored to fit natural conditions.

Necessity of irrigation

Throughout the crop period adequate quantities of water is required near the root zone of the plants for their growth.

→ Insufficient rainfall
→ Uneven distribution of rainfall
→ Improvement of perennial crops
→ Development of agriculture in desert area

Benefits of irrigation:

The following are the important benefits of irrigation
→ Yield of crops
→ Protection from famine
→ Improvement of cash crops
→ Prosperity of farmers
→ Source of revenue
→ Navigation
→ Hydroelectric power generation
→ Water supply
→ General Communication
→ Development of fishery

Disadvantages of irrigation

→ Rising of water table: water-logging
→ Problem of water pollution (nitrates seepage into GW)
→ Formation of marshy land
→ Dampness in weather
→ Loss of valuable lands

Types of irrigation

Irrigation may broadly be classified into:
1. Surface irrigation.
2. Sub-surface irrigation.

Surface irrigation

In the surface methods of irrigation, water is applied directly to the soil surface from a channel located at the upper reach of the field. Water may be distributed to the crops in border strips, check basins or furrows. Two general requirements of prime importance to obtain high efficiency in

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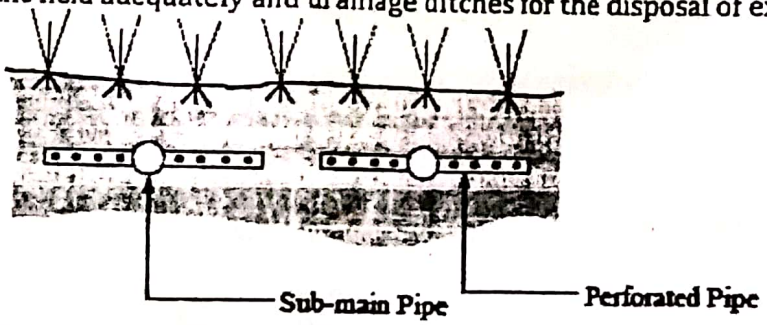
surface methods of irrigation are: distribution systems to provide adequate control of water to the fields and proper land preparation to permit uniform distribution of water over the field. Surface irrigation can be further classified into:

1. Flow irrigation
2. Lift irrigation

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

Subsurface irrigation

In sub-irrigation, water is applied below the ground surface by maintaining an artificial water table at some depth, depending upon the soil texture and the depth of the plant roots. Water reaches the plant roots through capillarity action. Water may be introduced through open ditches or underground through pipelines such as drains or mole drains. The depths of open ditches or trenches vary from 30 cm to 100 cm and they are spaced about 15 m to 30 m apart. The water application system consists of field supply channels, ditches or trenches suitably spaced to cover the field adequately and drainage ditches for the disposal of excess water.



Thechniques of water distribution in the farms

There are various way in which the irrigtion water can be applied to the fields. Their main classification is,

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

Border flooding: In this method the land is divided into a number of strips, seperated by low levees called borders. The land areas confined in each strip is of the order of 10 to 20 metres in width and 100 to 400 metres in length.

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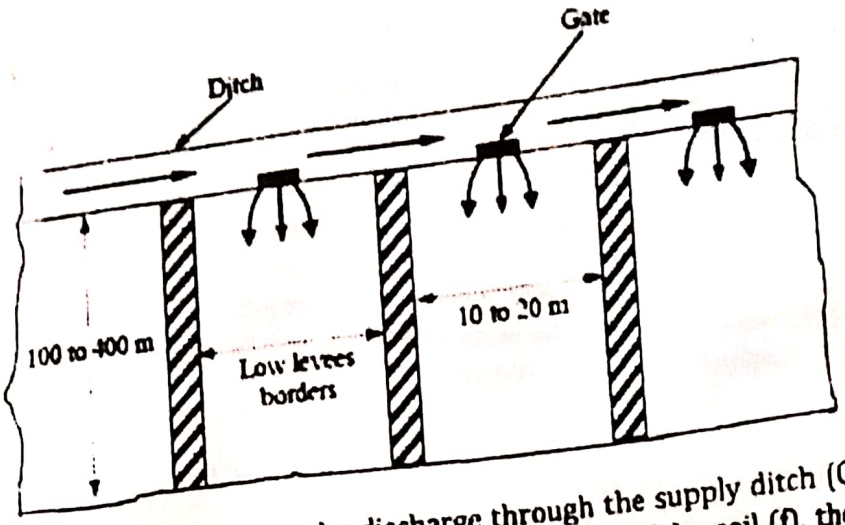

A relationship between the discharge through the supply ditch (Q), the average depth of water flowing over the strip (y), the rate of infiltration of the soil (f), the area of the land irrigated (A) and the approximate time required to cover the given area with water (t) is given by equation,

$$t = 2.3\,\frac{y}{f}\log_{10}\left(\frac{Q}{Q - fA}\right)$$

Q = discharge through the supply ditch

y = depth of water flowing over the strip

f = rate of infiltration of the soil

t = time required to cover the given area with water

A = the area of the land irrigated

Maximum area that can be irrigated,

$$A_{max} = \frac{Q}{f}$$

Question: Differentiate between Furrow irrigation with sprinkler irrigation. Which one is preferred in Bangladesh and Why? (30th BCS, 31th BCS, 40 BCS)

Solution:

Furrow irrigation: In the furrow irrigation system one half to one fifth of the surface is wetted and thus evaporation losses are very much reduced. In this method the irrigation water is supplied to the 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 of the crops. This method is suitable for the crops which are sown in rows. The crops are potato ground nut, tobacco, sugarcane etc.

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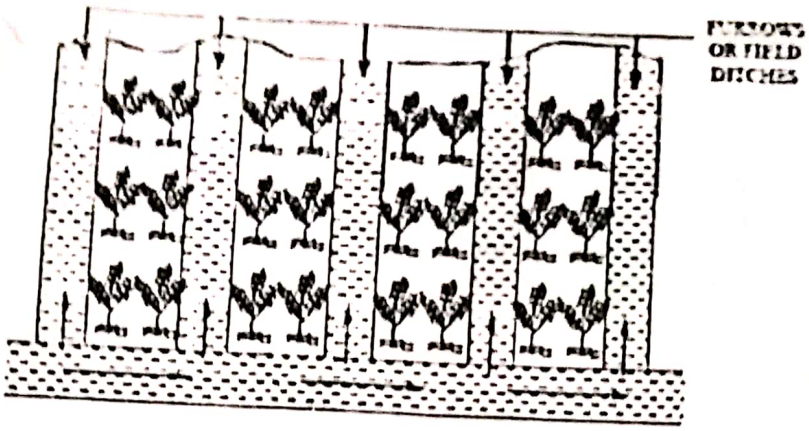
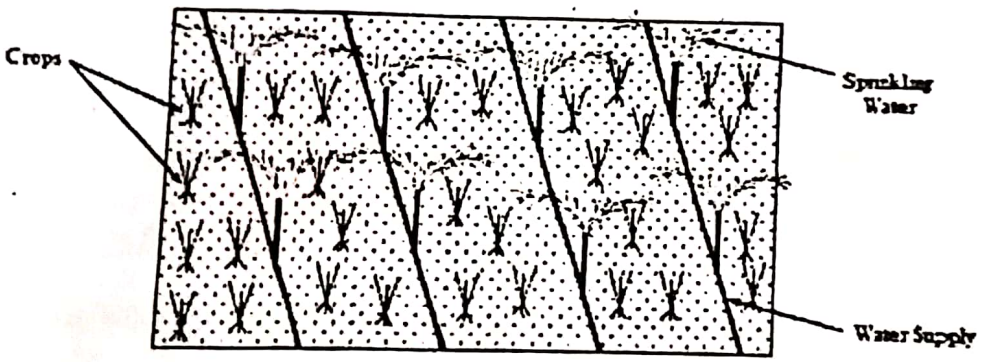


Sprinkler Irrigation: In this farm water application method water is applied to the soil in the form of spray through a network of pipes and pumps. It is a kind of an artificial rain which fulfills the normal requirement of uniform distribution of water. The greatest advantages of sprinkler irrigation is its adaptabilities to use under conditions where surface irrigation methods are not efficient. This method is kore useful where:

- The land cannot be prepared for surface methods
- Slopes are excessive.
- Topography is irregular.
- Soil is erosive
- Depth of soil is shallow over gravel or sand.



Furrow irrigation method is prefferd in bangladesh. Inspite of the numerous advantages which sprinkler irrigation method possesses over the other methods, it has not become popular in bangladesh for the simple reason that ours is poor and a developing nation. This method is not costly but requires a lot o technicalties. The coreect design and effiecient operation are very important for the success of this method. Special steps have to be taken for preventing entry of silt and debrics which are very harmful for the sprinkler equipment.

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Question: What are the advantages of sprinkler irrigation? Discuss its suitability in Bangladesh. (33th BCS)

Solution:

Sprinkler irrigation: In this farm water application method water is applied to the soil in the form of spray through a network of pipes and pumps. It is a kind of an artificial rain which fulfills the normal requirement of uniform distribution of water. The greatest advantages of sprinkler irrigation is its adaptabilities to use under conditions where surface irrigation methods are not efficient. This method is kore useful where:

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Inspite of the numerous advantages which sprinkler irrigation method possesses over the other methods, it has not become popular in bangladesh for the simple reason that ours is poor and a developing nation. This method is not costly but requires a lot o technicalties. The coreect design and effiecient operation are very important for the success of this method. Special steps have to be taken for preventing entry of silt and debrics which are very harmful for the sprinkler equipment.

Question: Determine the time required to irrigate a strip of land of 0.04 hectares in area from a tube well with a discharge of 0.02 cumec. The infiltration capacity of the soil may be taken as 5 cm/hr and the average depth of flow on the field as 10 cm. Also determine the maximum area that can be irrigated from this tube well.

Solution:

$A = 0.04$ hectares $= 0.04 \times 10000 = 400\ m^2$

$Q = 0.02$ cumecs $= 0.02\ m^3/s = 0.02 \times 3600 = 72\ m^3/hr$

$f = 5$ cm/hr $= 0.05$ m/hr, $y = 10$ cm $= 0.1$ m

$$t = 2.3 \frac{y}{f} \log_{10}\left(\frac{Q}{Q - fA}\right) = 2.3 \frac{0.1}{0.05} \log_{10}\left(\frac{72}{72 - 0.05 \times 400}\right) = 0.65 \text{ hour}$$

Maximum area that can be irrigated,

$$A_{max} = \frac{Q}{f} = \frac{72}{0.05} = 1440\ m^2 = 0.144 \text{ hectares}$$

Proportion of sodium ions to other cations: Most of the soils contain calcium and magnesium ions and small quantities of sodium ions. The percentage of the sodium ions generally less than 5% of the total exchangeable cations. If this percentage increase to about 10% or more, the aggregation of soil grains breaks down. The soil becomes less permeable and of poorer tilth. It starts crusting when dry and its pH increases towards that of an alkaline soil. High sodium soils are plastic, sticky when wet and are prone to form clods and the crust on drying.

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The proportion of sodium ions present in the soils is generally measured by a factor called sodium-absorption ratio (SAR) and represents the sodium hazards of water.

$$SAR = \frac{Na^{+}}{\sqrt{\frac{Ca^{++} + Mg^{++}}{2}}}$$

The concentration of the ions is expressed in equivalent per million (epm)

Type of water	Suitability
Low sodium water (S1) SAR: 0 - 10	Suitable for all types of crops and all types of soils except for those crops which are highly sensitive to sodium
Medium sodium water (S2) SAR: 10 - 18	Suitable for coarse textured of organic soil with good permeability. Relatively unsuitable in fine textured soils.
High sodium water (S3) SAR: 18 - 26	Harmful for almost all types of soils. Requires good drainage, high leaching, gypsum addition.
Very high sodium water (S4) SAR: above 26	Unsuitable for irrigation.

Type of water	Suitability for irrigation
Low salinity water (C1) Conductivity: 100 – 250 micro-mhos/cm at 25° C	Suitable for all types of crops and all kinds of soils. Permissible under normal irrigation practices except in soil of extremely low permeability.
Medium salinity water (C2) Conductivity: 250 – 270 micro-mhos/cm at 25° C	Can be used, if a moderate amount of leaching occurs. Normal salt tolerant plants can be grown without much salinity control.
High salinity water (C3) Conductivity: 750 – 2250 micro-mhos/cm at 25° C	Unsuitable for soil with restricted drainage. Only high salt tolerant plants can be grown.
Very high salinity water (C4) Conductivity: above 2250 micro-mhos/cm at 25° C	Unsuitable for irrigation.

Question: What do you mean by Electric Conductivity of water in irrigation? (37th BCS)

Solution:

The electrical conductivity of an aqueous solution is its ability to conduct an electrical current. The electrical conductivity is an important parameter used to estimate the level of dissolved salts in water and soil. Salts include mineral plant nutrients that occur naturally in soil and water, fertilizers applied and also other dissolved minerals that are not plant nutrients and might even be harmful to plants.

Distilled water does not conduct electricity. When salts dissolve in water, they dissociate to into ions – positively charged ions, called cations, and negatively charged ions, called anions. This gives the water the ability to conduct electricity.

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A common parameter used to describe the total amount of the dissolved salts in water is the TDS (Total Dissolved Solids). TDS is the sum of the concentrations of all the dissolved ions, measured in milligrams per liter or ppm (parts per million), where 1 milligram/liter = 1 ppm. The electrical conductivity is, therefore, related to the TDS and is used as a measure of total amount of dissolved salts. Generally speaking, the higher the concentration of dissolved salts the higher the electrical conductivity.

Question: What is the classification of irrigation water having the following characteristics? Concentration of Na, Ca and Mg are 22, 3 and 1.5 milli-equivalents per litre respectively and the electrical conductivity is 200 micro mhos per cm at 25^0 C? What problems might arise in using this water on fine textured soils? What remedies do you suggest to overcome this trouble?

Solution:

$$SAR = \frac{Na^+}{\sqrt{\frac{Ca^{++} + Mg^{++}}{2}}} = \frac{22}{\sqrt{\frac{3+1.5}{2}}} = 14.67$$

SAR is between 10 to 18, it is classified as medium sodium water ($S2$)

Electrical conductivity is 200 micro mhos per cm at 25^0

So the water is low salinity water ($C1$)

In fine textured soils, the medium sodium water ($S2$) may create the following problems,

→ Soil becomes less permeable

→ It starts crusting when dry.

→ It becomes plastic and sticky when wet

→ Its pH increases towards that of alkaline water

Gypsum $CaSO_4$ addition either to soil or to water is suggested to overcome sodium hazards posed by the given water.

Water requirements of a crop means the total quantity and the way in which a crop requires water from the time it is sown to the time it is harvested.

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

Duty and delta: The total quantity of water required by the crop for its full growth is called duty. Duty is expressed as hectare-metre. Total depth of water (in cm) required by a crop to come to maturity is called delta (Δ).

Root zone depth: Root zone depth is the maximum depth of soil strata in which the crop spreads its root system and derives water from the soil.

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

Solution:

Question: Water is required at an interval of 10 days for a period of 120 days. It evidently means that 12 no. of watering are required and each time 10 cm depth of water is required.

Total depth of water required, $\Delta = 12 \times 10 = 120$ cm

Relation between duty and delta:

Total depth of water applied on a land, $\Delta = \frac{8.64\,B}{D}$ metres

Total depth of water applied on a land, $\Delta = \frac{864\,B}{D}$ cm

Δ is in cm, B is in days and D is duty in hectares/cumec

Question: Find the delta for a crop when its duty is 864 hectares/cumec on the field, the base period of this crop is 120 days.

Solution:

$$\Delta\,(\text{cm}) = \frac{864\,B}{D} = \frac{864 \times 120}{864} = 120\text{ cm}$$

Question: A crop requires a total depth of 92 cm of water for a base period of 120 days, find the duty of water.

Solution:

$$\Delta\,(\text{cm}) = \frac{864\,B}{D}$$

$$D = \frac{864\,B}{\Delta} = \frac{864 \times 120}{92} = 1127\text{ hectares/cumec}$$

Crop Season: From the agricultural point of view, the year can be divided into two principal cropping seasons:

Rabi → Starts from 1st October and ends on 31st March
→ Kharif crops are also called "summer crops"
Example: Rabi crops are rice, bajra, jowar, maize, cotton, tobacco, groundnut etc.

Kharif → Starts from 1st April and ends on 30th September
→ Rabi crops are "winter crops"
Example: Kharif crops are wheat, barley, gram, linseed, mustard, potatoes etc.

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Question: An irrigation canal has gross commanded area of 80,000 hectares out of which 85% is culturable irrigable. The intensity of irrigation for Kharif season is 30% and for Rabi season 60%. Find the discharge required at the head of the canal if the duty at its head is 800 hectares/cumecs for Kharif season and 1700 hectares cumec for Rabi season.

Solution:

Culturable irrigable area = 80000 x 0.85 = 68000 hectares
Area under Kharif season = 68000 x 0.30 = 20400 hectares
Area under Rabi season = 68000 x 0.60 = 40800 hectares

Water required at the head of the canal to irrigate the land under Kharif season,

$$\text{Discharge} = \frac{\text{Area}}{\text{Duty}} = \frac{20400}{800} = 25.5 \text{ cumecs}$$

Water required at the head of the canal to irrigate the land under Rabi season,

$$\text{Discharge} = \frac{\text{Area}}{\text{Duty}} = \frac{40800}{1700} = 24 \text{ cumecs}$$

Thus the water requirement during Kharif is more than that is Rabi. Hence the canal must be designed to carry a discharge of 25.5 cumecs.

Question: Define Duty and Delta. Show the relationship between Duty and Delta. (BBAD – 2018)

Solution:

Duty: Number of hectares of land that can be irrigated by 1 cumec of water throughout the base period of a crop. Denoted by D. Unit is in hectare/area.

Delta: It's the total depth of water in cm required by a crop to come to its maturity is called delta. Denoted by Δ. Unit is in meter.

Relation between Delta and Duty:

$$\text{Total depth of water applied, } \Delta = \frac{8640\,B}{D} \text{ mm} = \frac{864\,B}{D} \text{ cm} = \frac{8.64\,B}{D} \text{ m}$$

Question: A canal commands an irrigation area of 400 ha. The duty of water on the field during the peak period is 220 ha/cumec. Determine the design discharge of the canal at the off-take if the water loss in canal is 30%. (BWDB – 2016)

Solution:

Given, $A = 400$ ha, $D = 220$ ha/cumec, $f = 1 - 0.3 = 0.7$

We know, $Q = \frac{A}{D\,f} = \frac{400}{220 \times 0.7} = 2.59 \text{ m}^3/\text{s}$

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Question: A water course has culturable commanded area of 2600 hectares out of which the intensities of irrigation for perennial sugar-cane and rice crops are 20% and 40% respectively. The duty for these crops at the head of water course are 750 hectares/cumec and 1800 hectares/cumec respectively. Find the discharge required at the head of water course if the peak demand is 120% of the average requirement.

Solution:

Area under sugar-cane = 2600 x 0.2 = 520 hectares

Area under rice = 2600 x 0.4 = 1040 hectares

Water required for sugar cane = $\frac{520}{750}$ = 0.694 cumec

Water required for rice = $\frac{1040}{1800}$ = 0.578 cumec

Since sugar-cane is perennial crop, it will require water throughout the year.

Hence the water course must carry a total discharge of (0.694 + 0.578) = 1.272 cumecs.

The design discharge to meet the peak demand will be = 1.272 x 1.2 = 1.53 cumecs

Question: The left branch canal carrying a discharge pf 20 cumecs has culturable commanded area of 20000 hectares. The intensity of Rabi crop is 80 percent and the base period is 120 days. The right branch canal carrying discharge of 8 cumecs has culturable commanded area of 12000 hectares, intensity of irrigation of Rabi crop is 50 percent and the base period is 120 days. Compare the efficiencies of the two canal systems.

Solution:

For the left canal:

Area under Rabi crop = 20000 x 0.8 = 16000 hectares

Discharge = 20 cumecs

Duty = $\frac{16000}{20}$ = 800 hectares/cumec

For the right canal:

Area under Rabi crop = 12000 x 0.5 = 6000 hectares

Discharge = 8 cumecs

Duty = $\frac{6000}{8}$ = 750 hectares/cumec

Since the left canal system has higher duty, it is more efficient.

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Question: A water course has a culturable commanded area of 1200 hectares. The intensity of irrigation for crop A is 40% and for B is 35%, both the crops being Rabi crops. Crop A has a kor period of 20 days and crop B has kor period of 15 days. Calculate the discharge of the water course if the kor depth for crop A is 10 cm and for B it is 16 cm. (BWDB – 2020)

Solution:

(a) For crop A

Area under irrigation = 1200 x 0.40 = 480 hectares

Kor period = b = 20 days

Kor depth, δ = 10 cm = 0.1 m

$$\text{Duty} = \frac{8.64\,b}{\delta} = \frac{8.64 \times 20}{0.1} = 1728 \text{ hectares/cumec}$$

$$\text{Discharged required} = \frac{\text{Area under irrigation}}{\text{Outlet factor}} = \frac{480}{1728} = 0.278 \text{ cumec}$$

(b) For crop B

Area under irrigation = 1200 x 0.35 = 48 = 420 hectares

Kor period = b = 15 days

Kor depth, δ = 16 cm = 0.16 m

$$\text{Duty} = \frac{8.64\,b}{\delta} = \frac{8.64 \times 15}{0.16} = 810 \text{ hectares/cumec}$$

$$\text{Discharged required} = \frac{\text{Area under irrigation}}{\text{Outlet factor}} = \frac{420}{810} = 0.519 \text{ cumec}$$

Design discharge of water course = 0.278 + 0.519 = 0.797 cumec

Question: Determine the discharge of the channel is to be designed for irrigating 5000 hectares kharif crop and 5000 hectares Rabi crop. The water requirement for Kharif crop and Rabi crop are 2.5 inch and 1.5 inch respectively. The base period for Kharif and Rabi are 3 weeks and 4 weeks respectively. (36th BCS)

Solution:

Discharge for kharif crop

Area under irrigation = 5000 hectares

Kor period = b = 3 weeks = 21 days

Kor depth, δ = 2.5 inch = 2.5 x 0.0254 = 0.0635 m

$$\text{Duty} = \frac{8.64\,b}{\delta} = \frac{8.64 \times 21}{0.0635} = 2857.32 \text{ hectares/cumec}$$

$$\text{Discharge required} = \frac{\text{Area under irrigation}}{\text{Duty}} = \frac{5000}{2857.32} = 1.74 \text{ cumec}$$

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Discharge for rabi crop

Area under irrigation = 5000 hectares

Kor period = b = 4 weeks = 28 days

Kor depth, δ = 1.5 inch = 1.5 x 0.0254 = 0.0381 m

$$\text{Duty} = \frac{8.64\,b}{\delta} = \frac{8.64 \times 28}{0.0381} = 6349.60 \text{ hectares/cumec}$$

$$\text{Discharge required} = \frac{\text{Area under irrigation}}{\text{Duty}} = \frac{5000}{6349.60} = 0.78 \text{ cumec}$$

So, the channel is to be designed for the maximum discharge of 1.74 cumec, because this discharge capacity of the channel will be able to supply water to both the seasons.

Kharif Rabi ratio or crop ratio: The area to be irrigated for rabi crop is generally more than that for the kharif crop. This ratio of proposed area to be irrigated in kharif season to that in the rabi season called kharif rabi ratio.

Paleo irrigation: Sometimes in the initial stages before the drop is sown the land is very dry. This particularly happens at the time of sowing of Rabi crops because of hot September. When the soil may be too dry to be sown easily. In such a case, the soil is moistened with water so as to help in sowing of the crops. This is known as Paleo irrigation.

Kor watering: The first watering which is given to a crop, when the crop is a few centimetres high is called kor watering. The optimum depth of kor watering for different crops are different. For example the optimum depth of kor watering for rice is 10 cm.

Optimum utilization of irrigation water: If a crop is sown and produced under absolutely identical conditions using different amounts of water depths, the yield is found to vary. The yield increases with water, reaches a certain maximum value and then falls down. The quantity of water at which the yield is maximum is called the optimum water depth.

Irrigation efficiencies: Efficiency is the ratio of the water output to the water input and usually expressed as percentage. Input minus output is nothing but losses and hence if losses are more output is less and therefore efficiency is less.

Efficiency of water conveyance: It is the ratio of the water delivered into the fields from the outlet point of the channel to the water entering into the channel at its starting point. It takes into account the conveyance or transit losses.

$$\eta_c = \frac{W_f}{W_r} \times 100$$

η_c = water conveyance efficiency

W_f = water delivered to the farm or irrigation plot

W_r = water supplied or diverted from the river or reservoir

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Water application efficiency: The water application efficiency is the ratio of the quantity of water stored into the root zone of the crops to the quantity of water delivered to the field.

$$\eta_a = \frac{W_s}{W_f} \times 100$$

η_a = water application efficiency

W_s = water stored in the root zone during irrigation

W_f = water delivered to the farm

The common sources of loss of irrigation water during water application are surface run off R_f from the farm and deep percolation D_f below the farm root zone soil.

$$W_f = W_s + R_f + D_f$$

$$\eta_a = \frac{W_f - (R_f + D_f)}{W_f} \times 100$$

Water use efficiency: It is the ratio of water beneficially used including leaching water to the quantity of water delivered and is determined from the following expression,

$$\eta_u = \frac{W_u}{W_d} \times 100$$

η_u = water use efficiency

W_u = water used beneficially or consumptively

W_d = water delivered

Water storage efficiency: It is the ratio of the water stored in the root zone during irrigation to the water needed in the root zone prior to irrigation.

$$\eta_s = \frac{W_s}{W_n} \times 100$$

η_s = water storage efficiency

W_s = water stored in the root zone during irrigation

W_n = water needed in the root zone prior to irrigation = Field capacity – Available moisture

Water distribution efficiency:

$$\eta_d = \left(1 - \frac{d}{D}\right)$$

η_d = water distribution efficiency

D = mean depth of water stored during irrigation.

d = average of the absolute values of deviations from the mean

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Question: The depths of penetrations along the length of a boarder strip at points 30 meters apart were probed. Their observed values are 2.0, 1.9, 1.8, 1.6 and 1.5 meters. Compute the water distribution efficiency.

Solution:

$$\text{Mean depth} = D = \frac{2 + 1.9 + 1.8 + 1.6 + 1.5}{5} = 1.76 \text{ metres}$$

Values of deviations from the mean are $(2 - 1.76) = 0.24, (1.9 - 1.76) = 0.14, (1.8 - 1.76) = 0.04, (1.6 - 1.76) = -0.16, (1.5 - 1.76) = -0.26$

The absolute values of these deviations from the mean are 0.24, 0.14, 0.04, 0.16, and 0.26

The average of these absolute values of deviations from the mean,

$$d = \frac{0.24 + 0.14 + 0.04 + 0.16 + 0.26}{5} = 0.168 \text{ metre}$$

The water distribution efficiency, $\eta_d = \left(1 - \frac{d}{D}\right) = \left(1 - \frac{0.168}{1.76}\right) = 0.905$

Question: 15 cumecs of water is delivered to a 34 hectare field for 4 hours. Soil proving after the irrigation indicates that 0.3 meter of water has been stored in the root zone. Compute the water application efficiency. (33th BCS)

Solution:

Water delivered to the farm, $W_f = 15 \times 4 \times 60 \times 60 = 216000 \text{ m}^3$

water stored in the root zone during irrigation, $W_s = 34 \times 10000 \times 0.3 = 102000 \text{ m}^3$

Water application efficiency, $\eta_a = \frac{W_s}{W_f} \times 100 = \frac{102000}{216000} \times 100 = 47.22\%$

Question: A stream of 130 liters per second was diverted from a canal and 100 liters per second were delivered to the field. An area of 1.6 hectares was irrigated in 8 hours. The effective depth of root zone was 1.7 m. The runoff loss in the field was 420 m3. Available moisture holding capacity of the soil is 20 cm per meter depth of soil. It is required to determine the, (a) water application efficiency, (b) water storage efficiency. (BWDB – 2018)

Solution:

$$\text{(a) Water application efficiency} = \frac{\text{Water stored in the root zone during irrigation}}{\text{Water needed in the root zone prior to irrigation}}$$

Water supplied to field during 8 hours @ 100 liters per second $= 100 \times 8 \times 60 \times 60$ liters
$= 2.88 \times 10^6$ liters
$= 2.88 \times 10^6 / 10^3 \text{ m}^3 = 2880 \text{ m}^3$

Runoff loss in the field $= 420 \text{ m}^3$

The water stored in the root zone $= 2880 - 420 \text{ m}^3 = 2460 \text{ m}^3$

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Water application efficiency $(\eta_a) = \frac{2880}{2460} \times 100 = 85.4\%$

(b) Water storage efficiency $(\eta_s) = \frac{\text{Water needed in the root zone prior to irrigation}}{\text{Water stored in the root zone during irrigation}} \times 100$

Moisture holding capacity of soil = 20 cm per m length x 1.7 m height of root zone $= 34$ cm

Moisture already available in root zone at the time of start of irrigation $= \frac{50}{100} \times 34 = 17$ cm

Additional water required in root zone = 34 - 17 = 17 cm

Amount of water required in root zone = Depth x Plot area $= \frac{17}{100} \times (1.6 \times 10^4)\ m^3 = 2720\ m^3$

But actual water stored in root zone = 2460 m^3

Water storage efficiency $(\eta_s) = \frac{2460}{2720} \times 100 = 90\%$ (say)

Consumptive use or Evapotranspiration (C_u):

Consumptive use for a particular crop may be defined as the total amount of water used by the plant in transpiration and evaporation from adjacent soils or from plant leaves in any specified time.

Effective rainfall (R_e): Precipitation falling during the growing period of a crop that is available to meet the evapo-transpiration needs of the crop is called effective rainfall. It does not include precipitation lost through deep percolation below the root zone or the water lost as surface runoff.

Consumptive irrigation requirement (CIR): It is the amount of irrigation water required in order to meet the evapotranspiration needs of the crop during its full growth. It is therefore nothing but the consumptive use itself but exclusive of effective precipitation, stored soil moisture or ground water.

$CIR = C_u - R_e$

Net irrigation requirement (NIR): It is the amount of irrigation water required in order to meet the evapotranspiration need of the crop as well as other needs such as leaching.

$NIR = C_u - R_e$ + water lost as percolation in satisfying other needs such as leaching

Field irrigation requirement (FIR): Filed irrigation requirement is the amount of water required to meet net irrigation requirement plus the water lost in percolation in the field water courses, field channels and in field applications of water.

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

Gross irrigation requirement (GIR): Gross irrigation requirement is the sum of water required to satisfy the field irrigation requirement and the water lost as conveyance losses in distributaries upto the field.

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

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Estimation of consumptive use by Blaney-Criddle formula

Consumptive use, $C_u = \frac{k\,p}{40}(1.8\,t + 32)$

C_u = Monthly consumptive use in cm

k = Crop factor, determined by experiments for each crop

t = Mean monthly temperature in degree C

p = Monthly percent of annual day light hours that occur during the period

If $\frac{p}{40}(1.8\,t + 32)$ is represented by f, $C_u = k\,f$

$C_u = k\,\Sigma f$

Question: Define consumptive use of water. What are the factors affecting consumptive use of water? (30th BCS, 31th BCS, 38th BCS)

Solution:

Consumptive use for a particular crop may be defined as the total amount of water used by the plant in transpiration and evaporation from adjacent soils or from plant leaves in any specified time. Factors affecting consumptive use of water are given below,

- Temperature
- Humidity
- Wind Velocity
- Soil Topography
- Sunlight Hours
- Method of Irrigation
- Cropping Patterns
- Natural Ground Water Table
- Quality of water

Question: Wheat is to be grown at a certain place, the useful climantological conditions of which are tabulated below in table. Determine the evapotranspiration and consumptive irrigation requirement of wheat crop. Also determine the field irrigation requirement if the application efficiency is 80%. Make use of Blaney criidle equation and a crop factor equal to 0.8.

Month	Monthly temp. in degree C, average over the 5 years	Monthly % of day time hr. of the year computed from the sun-shine tables	Useful rainfall in cm, average over the last 5 years
November	18.0	7.20	1.7
December	15.0	7.15	1.42
January	13.5	7.30	3.01
February	14.5	7.10	2.25

Blaney – Criddle formula, $C_u = k\,\Sigma f$

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Where, $f = \frac{p}{40}(1.8t + 32)$

Month	t	p	R_e	f
November	18.0	7.20	1.7	11.6
December	15.0	7.15	1.42	10.5
January	13.5	7.30	3.01	10.3
February	14.5	7.10	2.25	10.3

$C_u = k \Sigma f = 0.8 \times 42.7 = 34.16$ cm

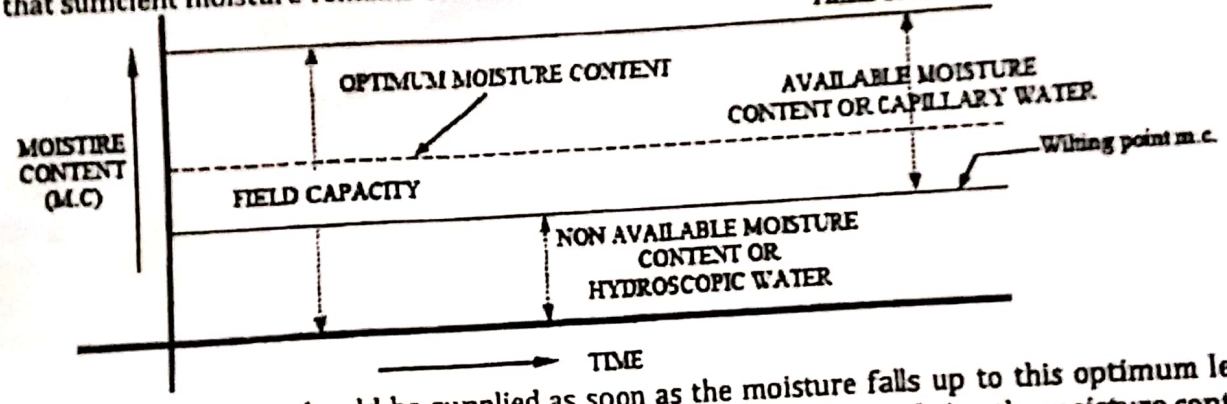
Hence, Consumptive use, $C_u = 34.16$

Consumptive irrigation requirement $= C_u - R_e = 24.34$ cm

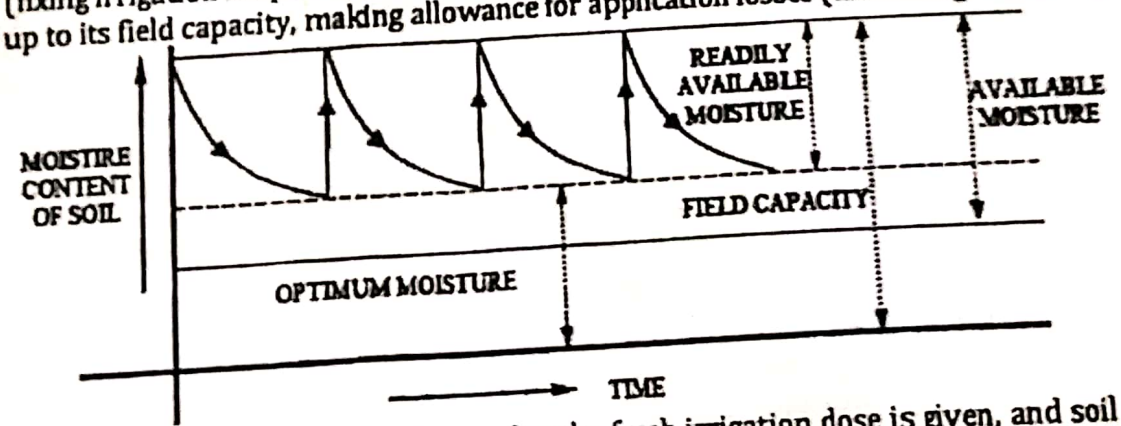
Field irrigation requirement $= \frac{CIR}{\eta_a} = \frac{24.34}{0.8} = 30.43$ cm

Estimating depth and frequency of irrigation on the basis of soil moisture regime concept

Water or soil moisture is consumed by plants through their roots. It, therefore, becomes necessary that sufficient moisture remains available in the soil from the surface to the root zone depth.



The irrigation water should be supplied as soon as the moisture falls up to this optimum level (fixing irrigation frequency) and its quantity should be just sufficient to bring the moisture content up to its field capacity, making allowance for application losses (thus fixing water depth)



Water will be utilized by the plants after the fresh irrigation dose is given, and soil moisture will start falling. It will again be recouped by a fresh dose of irrigation, as soon as the soil moisture reaches the optimum level.

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Permanent wilting point: It is that water content at which plant can no longer extract sufficient water for its growth, and wilts up. It is the point at which permanent wilting of plants take place.

Available moisture: It may be defined as the difference in water content of the soil between field capacity and permanent wilting point.

Readily available moisture: It is that portion of the available moisture which is most easily extracted by the plants, and is approximately 75 to 80 % of the available moisture.

Field capacity: Immediately after a rain or irrigation water application when all the gravity water has drained down to the water table, a certain amount of water is retained on the surfaces of soil grains by molecular attraction and by loose chemical bonds (adsorption). This water cannot be easily drained under the action of gravity and is called the field capacity. The field capacity is thus the water content of a soil after free drainage has taken place for a sufficient period. This period of free gravity drainage is generally taken as 2 to 5 days.

$$\text{Field capacity} = \frac{\text{Weight of water retained in a certain volume of soil}}{\text{Weight of the same volume of dry soil}} \times 100$$

$$F = \frac{\text{Weight of water retained in unit area of soil}}{\gamma_d\, d} \times 100$$

Weight of water retained in unit area of soil $= \gamma_d\, d\, F$ kN/m^2

$$\text{Volume of water stored in unit area of soil} = \frac{\gamma_d\, d\, F \text{ kN/m}^2}{\gamma_w \text{ kN/m}^3}$$

Hence, the depth of water stored in the root zone in filling the soil upto field capacity,

$$= \frac{\gamma_d\, d\, F}{\gamma_w} \text{ metres}$$

F = the field capacity m. c.

d = depth of root zone in m

γ_w = the unit weight of water

γ_d = the dry unit weight of soil

A part of this depth of water will be available for evapo-transpiration.

$$\text{Available moisture depth, } d_w = \frac{\gamma_d\, d}{\gamma_w} \text{ (Field capacity – Wilting coefficient)}$$

$$d_w = \frac{\gamma_d\, d}{\gamma_w}(F_c - w_c) = S\, d\, (F_c - w_c)$$

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Question: The field capacity of a certain soil is 20% and its apparent specific gravity is 1.6. Before applying irrigation water, a wet sample of soil was taken and its mass was found as 150 gm. The same sample weighed as 136 gm after oven drying. Determine the depth of water that must be applied to irrigate the soil to a depth of 0.9 m.

Solution:

Moisture content before irrigation $= \frac{150 - 136}{136} \times 100 = 10.29\%$

Depth of water required to bring the moisture up to field capacity,

$$\text{Depth of water required} = \frac{\gamma_d \, d \, (F_c - MC)}{\gamma_w}$$

$$\text{Depth of water required} = \frac{1.6 \times 0.9 \, (0.2 - 0.1029)}{1} = 0.14 \text{ m} = 140 \text{ mm}$$

Question: The field capacity and moisture content at the time of irrigation are 27% and 19% respectively. The apparent specific gravity is 1.3 and root zone depth is 100 cm. Determine the time required to irrigate 2 ha with a flow of 60 l/s if the water application losses are taken to be 20%. (BWDB – 2016, WARPO – 2017)

Solution:

$$\text{Depth of water stored in root zone} = \frac{\gamma_d \, d \, (F_c - MC)}{\gamma_w} = \frac{1.3 \times 1 \, (0.27 - 0.19)}{1} = 0.104 \text{ m}$$

$$\text{Field irrigation requirement} = \frac{0.104}{(100 - 20)\%} = 0.13 \text{ m}$$

Volume of water required $= 0.13 \times 2 \times 10^4 = 2600 \text{ m}^3$

$$\text{Time required} = \frac{2600 \times 10^3}{60} \text{ l/lps} = 43333.33 \text{ s} = 12.03 \text{ hr.}$$

Question: Determine the field capacity of a soil for the following data, depth of root zone = 1.8 m, existing moisture = 8%, dry density of soil = 1450 kg/m³, quantity of water applied to soil = 650 m³, water lost due to deep percolation and evaporation = 10%, area to be irrigated = 1000 m². (BWDB – 2019, BADC – 2020)

Solution:

Volume of total water applied $= 650 \text{ m}^3$

Loss of water = 10%

$$\text{Water used in soil} = 650 \times \frac{90}{100} = 585 \text{ m}^3$$

Weight of water used $= 585 \times 9.81 = 5738.85$ KN

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Total dry weight of soil $= \frac{1000 \times 1.8 \times 1450 \times 9.81}{1000} = 25604.1$ KN

% of water added $= \frac{5738.85}{25604.1} \times 100 = 22.4\ \%$

Field capacity $= 8 + 22.4 = 30.4\ \%$

Question: What is to be grown in field having a field capacity equal to 27% and the permanent wilting point is 13%. Find the storage capacity in 80 cm depth of soil, if the dry unit weight of the soil is 1.5 g/cc. If the irrigation water is to be supplied when the average soil moisture falls to 18%, find the water depth required to be supplied to field if the field application efficiency is 80%. What is the amount of water needed at the canal out let if the water lost in the water course and the field canal is 15% of the out let discharge?

Solution:

Maximum storage capacity $= \frac{\gamma_d\, d\,(F_c - w_c)}{\gamma_w} = \frac{1.5 \times 80\,(0.27 - 0.13)}{1} = 16.8\ cm$

Depth of irrigation water $= \frac{\gamma_d\, d\,(F_c - MC)}{\gamma_w} = \frac{1.5 \times 80\,(0.27 - 0.18)}{1} = 10.8\ cm$

Field irrigation requirement $= \frac{10.8}{0.8} = 13.5$ m

Amount of water needed at the canal outlet $= \frac{13.5}{0.85} = 15.9$ cm

Question: Wheat is to be grown in a field having a field capacity equal to 27% and the permanent wilting point is 13%. Find the storage capacity in 80 cm depth of the soil, if the dry unit weight of the soil is 14.72 kN/m^3. If irrigation water is to be supplied when the average soil moisture falls to 18%, find the water depth required to be supplied to the field if the field application efficiency is 80%. What is the amount of water needed at the canal outlet if the water lost in the water course and the field channel is 15% of the outlet discharge?

Solution:

Maximum storage capacity $= \frac{\gamma_d\, d\,(F_c - w_c)}{\gamma_w} = \frac{14.72 \times 0.8\,(0.27 - 0.13)}{9.81} = 0.168\ m = 16.8\ cm$

Since the moisture is allowed to vary between 27% and 18%, the deficiency created in this fall

Depth of irrigation water $= \frac{14.72 \times 0.8\,(0.27 - 0.18)}{9.81} = 0.108\ m = 10.8\ cm$

Hence, 10.8 cm depth of water is the net irrigation requirement.

Field irrigation requirement, $FIR = \frac{NIR}{\eta_a} = \frac{10.8}{0.8} = 13.5$ cm

Quantity of water needed at the canal outlet $= \frac{FIR}{\eta_c} = \frac{13.5}{0.85} = 15.88$ cm

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Question: A loam soil has field capacity of 25% and permanent wilting percentage 10%, the dry unit weight of soil is 14.72 kN/m³. If the depth of root zone is 0.75m, determine the storage capacity of the soil. Irrigation water is supplied when moisture content drops to 14%. If water application efficiency is 75%, determine the water depth required to be applied in the field. (38th BCS)

Solution:

$$\text{Maximum storage capacity} = \frac{\gamma_d\, d\,(F_c - w_c)}{\gamma_w} = \frac{14.72 \times 0.75\,(0.25 - 0.10)}{9.81} = 0.168\ \text{m} = 16.8\ \text{cm}$$

$$\text{Depth of irrigation water} = \frac{14.72 \times 0.75\,(0.25 - 0.14)}{9.81} = 0.123\ \text{m} = 12.3\ \text{cm}$$

Hence, 10.8 cm depth of water is the net irrigation requirement.

$$\text{Field irrigation requirement, } FIR = \frac{NIR}{\eta_a} = \frac{12.3}{0.75} = 16.40\ \text{cm}$$

Question: An irrigation field has following data: Field capacity = 25%, Permanent wilting point = 15%, Specific weight = 14.03 kN/m³, Root zone depth = 90 cm, Calculate the available moisture holding capacity of soil. (TGTDCL – 2021)

Solution:

$$\text{Maximum storage capacity} = \frac{\gamma_d\, d\,(F_c - w_c)}{\gamma_w}$$

$$\text{Maximum storage capacity} = \frac{14.03 \times 0.90\,(0.25 - 0.15)}{9.81} = 0.128\ \text{m}$$

Question: Compute the depth and frequency of irrigation required for a certain crop with the following data-

Root zone depth = 95 cm
Field capacity = 28%
Wilting Point = 14%
Apparent specific gravity of soil = 1.3
Consumptive use of water for the given crop = 15mm/day
Efficiency of irrigation = 70% (37th BCS)

Solution:

Available moisture = Field capacity – permanent wilting = 28 – 14 = 14%

Let us assume that the readily available moisture or the optimum soil moisture level is 80% of available moisture.

Readily available moisture = 0.80 x 14% = 11.2%

Optimum moisture = 28 – 11.2 = 16.8%

It means that the moisture will be filled by irrigation between 16.8% and 28%

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Depth of water stored in root zone between these two limits = $S_g\, d\, (F_c - OMC)$

$= 1.3 \times 0.95 \times (0.28 - 0.168) = 0.1383$ m $= 13.83$ cm

Hence, water available for evapo – transpiration = 13.83 cm

1.5 cm of water is utilised by the plant is in day

13.83 cm of water will be utilised by the plant in $= \frac{1 \times 13.83}{1.5} = 9.22$ days ; say 9 days

Hence, after 9 days water should be supplied to the given crop.

Question: A sandy loam soil holds water at 140 mm/m depth between field capacity and permanent wilting point. The root depth of the crop is 30 cm and the allowable depletion of water is 35%. The daily water use by the crop is 75 mm/day. The area to be irrigated is 60 ha and water can be diverted at 28 lps. The surface irrigation application efficiency is 40%. There are no rainfall and groundwater contribution. Determine: (i) Allowable depletion depth between irrigations (ii) Frequency of irrigation (iii) Net application depth of water (iv) Volume of water required (v) Time to irrigate 4 ha plot. (40th BCS)

Solution:

Moisture holding capacity of soil = 140 mm/m depth
Depth of root zone = 30 cm = 0.3 m
Moisture holding capacity of root zone = $140 \times 0.3 = 42$ mm = 4.2 cm
Allowable depletion = 35%

(*i*) Available moisture depth or allowable depletion depth between irrigations = $35\% \times 4.2$
$= 1.47$ cm

Daily use of water = consumptive use = 5 mm/day = 0.5 cm/day

(*ii*) Frequency of irrigation $= \frac{\text{Available moisture}}{\text{Moisture consumed per day}} = \frac{1.47}{0.5} = 2.94$ days, say 3 days

(*iii*) Net water depth to be applied while irrigating each time after 3 days,
Net irrigation requirement $= 3 \times 0.5 = 1.5$ cm

Field irrigation requirement $= \frac{\text{Net irrigation requirement}}{\text{Efficiency of irrigation}} = \frac{1.5}{0.4} = 3.75$ cm

(*iv*) Quantity of water required in the field = 3.75 cm of water depth

$= 3.75$ cm x area of field $= 3.75$ cm x 60 ha $= \frac{3.75}{100} \times (60 \times 10^4) = 22500\ m^3$

Hence, volume of water required to irrigate 60 ha area, each time at 3 days interval.

(*v*) Time to irrigate 4 ha when irrigation water is supplied @ 28 lps:
Volume of water required to irrigate 4 ha plot = 3.75 cm x 4 ha

$= \frac{3.75}{100} \times (4 \times 10^4) = 1500\ m^3$

Time during which 1500 of water can be supplied @28 lps $= \frac{1500 \times 10^3}{28 \times 60 \times 60} = 14.88$ hr

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Sediment transport and design of irrigation channels

Whenever water flows in a channel, it tries to scour its surface. Silt or gravel or even larger boulders are detached from the bed or sides of the channel. These detached particles are swept downstream by the moving water. This phenomenon is known as sediment transport.

Alluvial soil: The soil which is formed by transportation and deposition of silt through the agency of water over a course of time is called the alluvial soil.

Non-alluvial soil: Mountainous regions may go on disintegrating over a period of time, resulting in the formation of a rocky plain area, called non alluvial area.

Sediment load: The sediment in a canal is a burden to be done by the flowing water and is designated as sediment load.

Bed load: The sediment may move in water either as bed load or as suspended load. Bed load is that in which the sediment moves along the bed with occasional jumps into the channel. The suspended load is the one in which the material is maintained in suspension due to the turbulence of the flowing water.

Regime channels: A channel is said to be in a state of regime if the flow is such that 'silting and scouring' need no special attention. Such a state is not easily possible in rivers but in artificial channels, such a state can be obtained by properly designing the channel.

Shear or tractive force: When water flows in a channel, the pull of water produces a force that acts on the channel bed in the direction of flow. This force is known as the shear or tractive or drag force.

Average shear stress or tractive force, $\tau_0 = \gamma_w R S_0$

Shear stress velocity, $u = \sqrt{g R S_0}$

Where, R is the hydraulic depth and S_0 is the slope.

For wide channel, $u = \sqrt{g h S_0}$

Critical shear stress: A given particle will move only when the shear stress acting on it is greater than the resistance of the particle to movement. The magnitude of shear stress required to move a given particle is known as the critical shear stress τ_{cr}. The resistance of the particles to movement and thus its entrainment will vary depending on its size, its size relative to surrounding particles, how it is oriented and the degree to which it is embedded.

$\tau_c = 0.687\, d_a$

d_a = arithmatic average diameter of sediment in mm

τ_c = critical shear stress in N/m^2

When the unit tractive force caused by the flowing water (τ_0) exceeds the critical unit tractive force (τ_c); naturally sediment starts moving.

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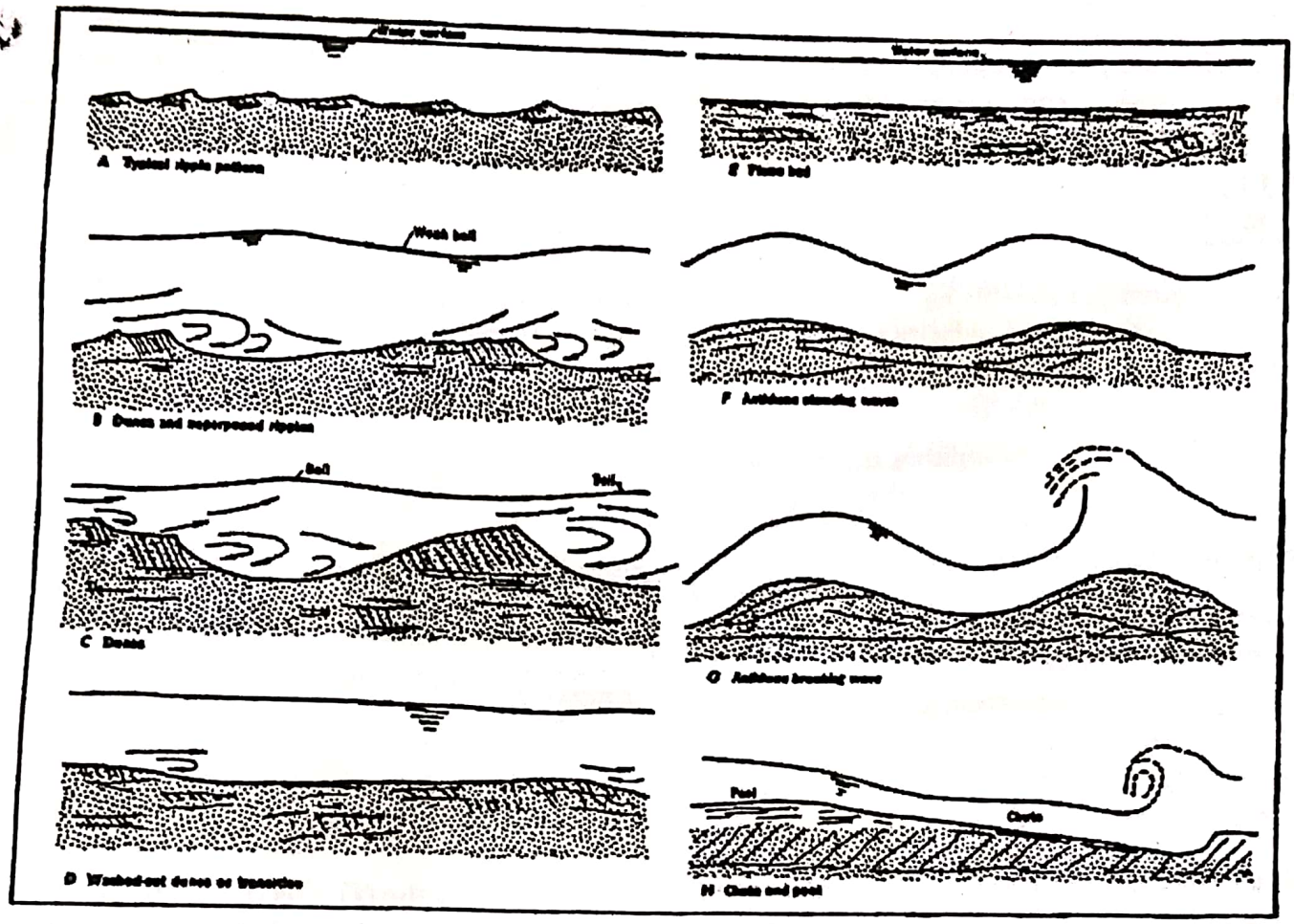

Figure: Forms of bed roughness on alluvial channel

Stress balance

For a fluid to begin transporting sediment that is currently at rest on a surface, the bed shear stress τ_b exerted by the fluid must exceed the critical shear stress τ_{cr} for the initiation of motion of grains at the bed. This basic criterion for the initiation of motion can be written as:

$$\tau_b = \tau_{cr}$$

bed shear stress, $\tau_b = \gamma R S$

This is typically represented by a comparison between a dimensionless shear stress τ_b^* and a dimensionless critical shear stress τ_{cr}^*. The nondimensionalization is in order to compare the driving forces of particle motion (shear stress) to the resisting forces that would make it stationary (particle density and size). This dimensionless shear stress τ^* is called the Shields parameter and is defined as:

$$\tau^* = \frac{\tau}{(\gamma_s - \gamma_w)\, d} = \frac{\tau}{(\rho_s - \rho)\, g\, d}$$

And the new equation to solve becomes:

$$\tau_b^* = \tau_{cr}^*$$

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The Shields diagram empirically shows how the dimensionless critical shear stress (the dimensionless shear stress required for the initiation of motion) is a function of a particular form of the particle Reynolds number.

$\tau_b^* = f\, R_e$

$R_e = \dfrac{u_{*c}\, d}{v}$

The specific particle Reynolds number of interest is called the boundary Reynolds number and it is formed by replacing the velocity term in the Particle Reynolds number by the shear velocity u_* which is a way of rewriting shear stress in terms of velocity.

Shear velocity at incipient condition, $u_{*c} = \sqrt{\dfrac{\tau_{cr}}{\rho}}$

Dimensionless critical shear stress, $\tau_{*c} = \dfrac{\tau_{cr}}{(\gamma_s - \gamma_w)\, d} = \dfrac{\tau_{cr}}{(G_s - 1)\, \gamma_w\, d}$

v = kinematic viscosity of water
τ_{cr} = critical shear stress
γ_s = unit weight of the sediment particle
γ_w = unit weight of water
g = acceleration due to gravity
d = size of the particle of interest

Question: The value of dimensionless effective stress is 0.045 N/m². Find the (i) effective velocity (ii) value of incipient depth of a wide river channel with the bed sediment size of 0.2 mm with longitudinal river bed slope of 0.0001. Specific gravity is 2.65. (BWDB – 2016)

Solution:

Dimensional critical shear stress, $\tau_{*c} = \dfrac{\tau_{cr}}{(\gamma_s - \gamma_w)\, d}$

Critical shear stress, $\tau_{cr} = \tau_{*c}\, (\gamma_s - \gamma_w)\, d = \tau_{*c}\, (G_s - 1)\, \gamma_w\, d$

$= 0.045\, (2.65 - 1)\, \text{x}\, 9810\, \text{x}\, \dfrac{0.2}{1000} = 0.145\ \text{N/m}^2$

Shear velocity at incipient depth,

$U_{*c}^2 = \dfrac{\tau_{cr}}{\rho} = \dfrac{\tau_{*c}\, (G_s - 1)\, \gamma_w\, d\, g}{\gamma_w} = \tau_{*c}\, (G_s - 1)\, d\, g$

$= 0.045\, (2.65 - 1)\, \dfrac{0.2}{1000}\, \text{x}\, 9.81 = 1.45\, \text{x}\, 10^{-4}$

$U_{*c} = 0.012$ m/s

For Incipient depth, $\tau_{cr} = \gamma\, D\, S_0$

$D = \dfrac{\tau_{cr}}{\gamma\, D_0} = \dfrac{0.145}{9810\, \text{x}\, 0.0001} = 0.147$ m

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Mittal and swamee has worked out a general relation between τ_c and d which gives results within +5% of the values given by shields curve, for all values of d. The relation for water and soil of $S_s = 2.65$, is

$$\tau_c(\mathrm{N/m^2}) = 0.155 + \frac{0.409\, d_{mm}^2}{\sqrt{1 + 0.177\, d_{mm}^2}}$$

Question: Water flows at a depth of 0.6 m in a wide stream having a bed slope of 1 in 2500. The median diameter of the sand bed is 1.00 mm. Determine whether the soil grains are stationary or moving and comment as to whether the stream bed is scouring or non-scouring.

Solution:

$$\tau_c(\mathrm{N/m^2}) = 0.155 + \frac{0.409\, d_{mm}^2}{\sqrt{1 + 0.177\, d_{mm}^2}} = 0.155 + \frac{0.409 \times 1}{\sqrt{1 + 0.177 \times 1}} = 0.53\ \mathrm{N/m^2}$$

$$\tau_0 = \gamma_w R S = 9.81 \times 0.6 \times \frac{1}{2500} = 2.35 \times 10^{-3}\ \mathrm{KN/m^2}$$

$\tau_0 = 2.35\ \mathrm{N/m^2}$ which is more than $0.53\ \mathrm{N/m^2}$

Since $\tau_0 > \tau_c$, the soil grains will not be stationary and the scouring and sediment transport will occur.

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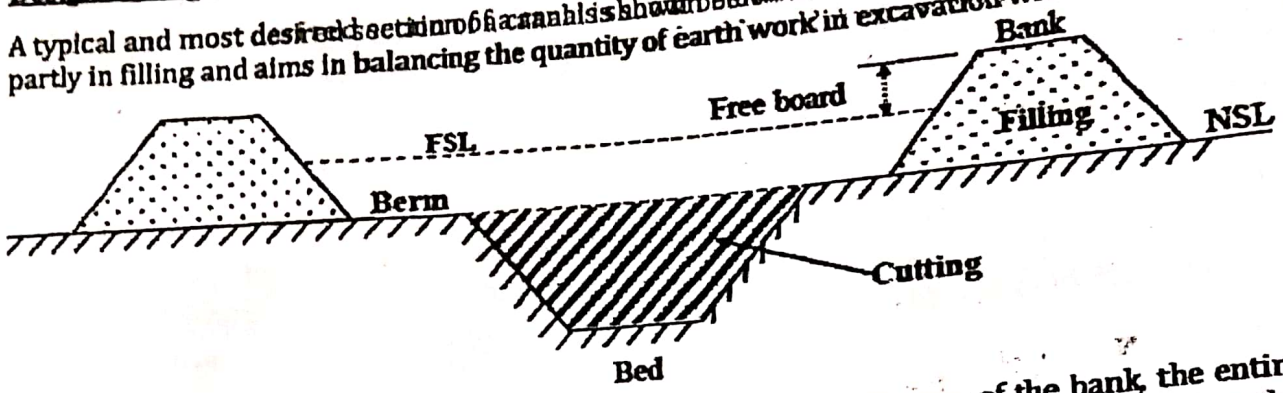
Design of Irrigation Channel

A typical and most desirable section of a canal is shown below. This section is partly in cutting and partly in filling and aims in balancing the quantity of earth work in excavation with that in filling.



Note: Sometimes, when the natural surface level (NSL) is above the top of the bank, the entire canal section will have to be in cutting, and it shall be called 'canal in cutting'. Similarly, when the NSL is lower than the bed level of the canal, the entire canal section will have to be built in filling, and it is called 'canal in filling' or 'canal in banking'.

Components of cross-section:

- Side slopes
- Berms
- Freeboard
- Banks
- Service roads
- Back Berm or Counter Berms
- Spoil Banks
- Borrow Pits

The success of the irrigation system depends on the design of the network of canals. The canals may be excavated through the difference types of soils such as alluvial soil, non-alluvial soil etc. The design discharge of an irrigation channel depends up on the irrigated area of crops in different seasons and the water requirement of crops. The design of the canal is mainly governed by the quantity of silt in the water and the type of boundary surface of the channel. Depending up on these factors, the irrigation channels can be broadly classified in to the following types.

1. Non-alluvial channels
2. Rigid boundary channels
3. Alluvial channels

1. The non- alluvial channels are excavated in non-alluvial soils such as loam, clay, boulder etc. Generally there is no silt problem in these channels and they are relatively stable.

2. In rigid boundary channels, the surface of the channel is lined. The quantity of silt transported by such channels remains more or less the same as that has entered the channel at the head. In such channels, relatively high velocity of flow is usually permitted which does not allow the silt to get deposited. Therefore for these channels, the problem of silt usually does not exist.

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3. The alluvial channels are excavated in alluvial soils, such as silt. In the case of alluvial channels the quantity of silt may vary from section to section along reach. The silt content may increase due to scouring of bed & sides of the channel. The silt content may due to silting at some sections. If the velocity is high, scouring will occur, but when the velocity is low, silting occurs. Both these phenomena result in modifying the cross section of the channel.

If the bed and sides are scoured, the cross-section increases and the full supply level falls, which results in the decrease of command. On the other hand, if so silting occurs, the cross-section decreases and the discharge capacity decreases, which also results in a decrease of command, such channels should be designed for a non-scouring and non-silting velocity, called critical velocity.

Design of Non-Alluvial Channels

In non- alluvial channels the water is clear and therefore, no silting problem occurs. Non-alluvial soils relatively are clay, hard loam or soils formed as a result of disintegration of rocks. Non-alluvial channels are considered stable if there is no silt problem in such channels. These channels are usually designed based on the **maximum permissible velocity** which the channel boundary surface can resist without scouring. The **permissible velocity** is defined as the mean velocity at or below which the channel bottom and sides are not eroded.

Manning Equation

A resistance formula proposed by Robert Manning, an Irish engineer, for uniform flow in open channels is,

$$V = \frac{1}{n} R^{2/3} S_0^{1/2}$$

V = Velocity of flow in the channel,
N = Manning's roughness Coefficient
R = Hydraulic radius of the channel
S = Longitudinal or hydraulic slope of a channel

S.No	Type of surface	Manning's n
1	Earth channels, clean straight & uniform	0.016 to 0.020
2	Earth channels, clean but weathered	0.018 to 0.025
3	Earth channels, with grass & weeds	0.022 to 0.033
4	Channels in gravels with stones	0.03 to 0.035
5	Channels in rock, smooth & uniform	0.025 to 0.040
6	Channels in rock, rough	0.035 to 0.050

Table: Manning's Coefficient

Chezy equation

Introduced by the French engineer Antoine Chezy in 1768 while designing a canal for the water supply system of Paris

$$V = C\sqrt{R S_0}$$

C = Chezy coefficient

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$$C = \sqrt{\frac{8g}{f}} = \frac{1}{n} R^{1/6}$$

Or, $C = \frac{87}{1 + \frac{K}{\sqrt{R}}}$; K = Basin's coefficient

Design of Lined Canals /Rigid Boundary Channel

A lined canal is a rigid boundary channel. It can with stand much higher velocity as compared to an unlined, non-alluvial channel or alluvial channel. The design is similar to the design of non-alluvial channels discussed in the preceding section. However, the maximum permissible velocity is relatively high. The design of lined canal is usually done by Manning's formula. The value of Manning's Coefficient n depends up on the type of lining. The higher values are for relatively rough surface and the lower, for smooth surface.

S.No	Type of lining	Manning's n
1	Cement concrete lining	0.013 - 0.022
2	Brick lining	0.014 - 0.017
3	Asphalt lining	0.013 - 0.016
4	Wood planed clean	0.011 - 0.013
5	Concrete lined excavated rock	0.017 - 0.027

Table: Manning's Coefficient n for lined canals

In most cases lined canals are designed as most economical sections. For most economical section, the hydraulic radius (R) should be maximum. Theoretically, a semi-circular section is the best section for as open channel. However, it is not practicable to adopt this section. For practical consideration a channel of trapezoidal or triangular section is usually selected. The corners of these sections are rounded to increase the hydraulic radius.

Design of Alluvial Channels

An alluvial channel is defined as a channel in which the flow transports sediment having the same characteristics as that of the material in the channel bottom. In the case of alluvial channels, the channel surface consists of alluvial soil which can be easily scoured. Moreover, the velocity is low which encourages silting. Therefore, in an alluvial channel, both scouring and silting may occur if the channel is not properly designed. The quantity of silt transported by water in alluvial channel varies from section to section due to scouring of bed & sides as well as due to silting (or deposition). If the velocity is too high, scouring may occur. On the hand, fi the velocity is too low, silting may occur.

The command of an irrigation channel decreases if the scouring occurs because the full supply level falls. The discharge capacity is decreased if the silting occurs because the reduction in cross-sectional area. Therefore, the alluvial channel should be designed such that neither scouring nor silting occurs. The velocity at which this condition occurs is called the critical velocity. Such an alluvial channel is called a stable channel. A stable channel is one in which banks and bed are not scoured and also in which no silting occurs. Even if there is some minor scouring and silting, the bed and banks of a stable channel remain more or less unaltered over a long period of time.

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Two approaches have been used for the design of stable alluvial channels:
(1) Regime theory
(2) Tractive force method

The tractive force approach is more rational, since it utilizes the laws governing sediment transport and resistance to flow. The regime theory is purely empirical in nature.

Regime Theory

Lacey defined a regime channel as a channel carrying a constant discharge under uniform flow in an unlimited incoherent alluvium having the same characteristics as that transported without changing the bottom slope, shape, or size of the cross section over a period of time. The regime theory is purely empirical in nature and was developed based on observations on a number of irrigation canals in the Indo-Pakistan subcontinent. Since the sediment concentration in these canals is usually less than 500 ppm by weight, the regime theory should be assumed to be applicable to channels carrying similar concentration of sediment load.

Generally speaking, the design of alluvial canals using regime theory depends on the investigations made on sediment/silt load of channels. Several investigators have studied the problem and suggested various theories. These are known as silt theories. The following two theories are extensively used for the design of canals in alluvial soils.

1. Kennedy's silt theory
2. Lacey's silt theory.

Kennedy's Theory

Gerard Kennedy, from his observation concluded that the silt supporting power in a channel cross-section is mainly dependent up on the generation of eddies from the bottom width of the channel section, rising to the surface. These eddies are generated due to the friction of the flowing water with the channel bottom surface. The vertical component of the eddies try to move the sediment up, while the weight of the sediment tries to bring it down, thus keeping the sediment in suspension. Thus he added if the velocity is sufficient to generate these eddies, so as to keep the sediment just in suspension, silting will be avoided. Then Kennedy stated that a channel is said to be in a state of "regime" if there is neither silting nor scouring in the channel. Based on that, he defined the critical velocity (V_0) in channel as the mean velocity, which will keep the channel free from silting or scouring & related it the depth of flow by equation.

$V_0 = 0.55\, m\, y^{0.64}$
V_0 = Critical velocity in channel in m/sec
y = depth of water in meters
m= critical velocity ratio (C.V.R)

Kutter's Formula

$$\text{Actual mean velocity, } V = \left[\frac{\frac{1}{n} + \left(23 + \frac{0.0015}{S}\right)}{1 + \left(23 + \frac{0.0015}{S}\right)\frac{n}{\sqrt{R}}}\right]\sqrt{RS}$$

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Lacey's Theory

Lacey carried out an extensive investigation on the design of stable channels in alluviums. On the basis of this research work, he found many drawbacks in Kennedy's Theory and he put forward his new theory.

Lacey's regime Channels: It is stated by Kennedy that a channel is said to be in a state of "regime" if there is neither silting nor scouring in the channel. But Lacey came out with the statement that even a channel showing no silting and no scouring may actually not be in regime. He, therefore, differentiated b/n three regime conditions,

(i) True regime (ii) Initial regime (iii) Final regime

According to him, a channel which is under initial regime is not a channel in regime (through outwardly it appears to be in regime as there is no silting or scouring) and hence, regime theory is not applicable to such channels. His theory is therefore applicable only to those channels which are either in true regime or in final regime.

True Regime: A Channel shall be in regime, if there is neither silting nor scouring. For this condition to be satisfied, the silt load entering the channel must be carried through by the channel section. Moreover, there can be only one channel section and one bed slope at which a channel carrying a give discharge and a particular quantum and type of silt would be in regime. Hence, an artificially constructed channel having a certain fixed section & a certain fixed slope can behave in regime only if the following conditions are satisfied. An artificial channel section having a certain fixed section and bed slope can behave in regime only if,

1. Discharge is constant
2. Flow is uniform
3. Silt charge is constant, i.e., the amount of silt is constant
4. Silt grade is constant, i.e., the size & type of silt is always the same
5. Channel is flowing through a material which can be scoured as easily as it can be deposited (such soil known as incoherent alluvium) and is of the same grade as is transported.

Hence the designed channel shall be in "true regime". If the above conditions are satisfied. But in practice all these conditions can never be satisfied. Therefore, artificial channels can never be in "true regime", they can either be in initial regime or final regime.

Initial Regime: A channel is in the initial regime when the bed slope gets affected due to silting and scouring. The lacey's regime theory is not valid for the initial regime.

Final Regime: When the parameters like slope, depth, etc. are varying as per the flow rate and grade of silt then it is said to be in the final regime.

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Question: Why the irrigation channel is design as regime channel? Justify your answer. (33th BCS)

Solution:

A stream channel is said to be in regime if it is transporting water and sediment in equilibrium such that there is neither scour of the channel bed and banks nor sediment deposition in the channel.

We also know that water moving with a given velocity and a certain depth can carry is suspension, only a certain amount of silt of a certain nature. If water of a given velocity and depth is not fully charged with silt (that it can carry is suspension) it will scour the bed and sides of the channel, till it is fully charged with silt. Hence, if the velocity of flow in the channel is more, the bed and the banks are likely to be eroded and similarly if the velocity is less the silt which was formerly carried is suspension is likely to be dropped. Silting and scouring in channels is not very uncommon and is understandable but must be avoided by proper designs. Scouring lowers the full supply level and causes loss of command. It may also cause breaching of canal banks of a channel as the channel section gets reduced by siltation thereby reducing the discharging capacity of the channel.

Therefore, while thinking to design a properly functioning channel, one must think to design such a channel is which neither silting nor scouring takes place. Such channels are known as stable channels or regime channels.

Question: What is meant by "regime of a river"? Compare briefly silt theories of Kennedy and Lacey. (30th BCS)

Solution:

A river regime is the annual variation in the discharge or flow of a river at a particular point, and is usually measured in cumecs. The variability in river's discharge throughout the course of a year occurs due to precipitation, temperature, evapo-transpiration, and drainage basin characteristics.

Kennedy's theory	Lacey's theory
Silt is kept in suspension by the vertical components of eddies. Kennedy assumed that silt is kept in suspension by eddies generated only at the bed and D is the relevant parameter.	Silt is kept in suspension by the vertical components of eddies. Lacey advocated that such eddies generated throughout the perimeter of the channel and considered the hydraulic radius R as the relevant parameter.
It recommended the use of kutter's equation.	It gave its own flow equation.
Introduced the critical velocity ratio (m) to account for silt grade.	Introduced silt factor and gave an equation for its determination from the average particle size.
Bed slope was selected on the basis of the available ground slope or is obtained from wood's normal table.	Gave its own equation for bed slope.
Design of channel is not unique. It depends upon bed slope S and B/D ratio.	Design is unique.
Tedious and involves trial and error method.	Simple and straight forward.
Applicable only irrigation channels.	Applied to natural channels (rivers) which are quasi-regime during floods.

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Design procedure for Lacey's theory

(1) Calculate the velocity from equation, $V = \left(\frac{Q f^2}{140}\right)^{1/6}$

Silt factor, $f = 1.76\sqrt{d_{mm}}$

V is in m/s, Q is in cumec, d_{mm} = Average particle size in mm

(2) Calculate hydraulic mean depth, $R = \frac{5}{2}\left(\frac{V^2}{f}\right)$

V is in m/s, R is in m

(3) Compute area of channel section, $A = \frac{Q}{V}$

(4) Compute wetted perimeter, $P = 4.75\sqrt{Q}$

P is in m, Q is in m^3/s

(5) Calculate the bed slope, $S = \left[\frac{f^{5/3}}{3340\,Q^{1/6}}\right]$

f is the factor, Q is the discharge in cumec

For side slope of an irrigation channel $\frac{1}{2}:1$

Area, $A = B\,D + \frac{D^2}{2}$

Perimeter, $P = B + D\sqrt{5}$

Hence, $D = \frac{P - \sqrt{P^2 - 6.944A}}{3.472}$ and $B = P - \sqrt{5}\,D$

Lacey's regime width and scour depth for alluvial rivers

For wide streams or rivers, wetted perimeter P approximately equals the river width.

The regime width, $W = 4.75\sqrt{Q}$

For such streams, Lacey has also defined the regime scour depth as,

Lacey's normal regime scour depth, $R_r' = 0.473\left(\frac{Q}{f}\right)^{1/3}$

The above scour depth equation will be applicable only when the river width equals the regime width $4.75\sqrt{Q}$. For any other value of active river width, the normal scour depth is given by,

Lacey's normal scour depth, $R' = 1.35\left(\frac{q^2}{f}\right)^{1/3}$

q = discharge intensity per unit width of stream $= \frac{Q}{L}$, L = actual river width

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Question: Calculate scour depth of river having width 500 m. Unit discharge 50 m^3/s /m. $d_{so} = 0.1$ mm. design discharge 80% of bank full discharge. (BWDB – 2020)

Solution:

Design discharge, $q = 50 \times 0.8 = 40\ m^3/s/m$

$f = 1.76\sqrt{d_{mm}} = 1.76\sqrt{0.1} = 0.55$

Lacey's normal scour depth, $R' = 1.35\left(\frac{q^2}{f}\right)^{1/3}$

$R' = 1.35\left(\frac{40^2}{0.55}\right)^{1/3} = 19.27\ m$

Question: The river Ganga flows bank full of rate is 2500 m^3/sec. find (*i*) Hydraulic Geometry (*ii*) Scour Depth. Bed material size is 0.15 mm. (BWDB – 2016)

Solution:

$f = 1.76\sqrt{d_{mm}} = 1.76\sqrt{0.15} = 0.68$

$V = \left(\frac{Q\,f^2}{140}\right)^{1/6} = \left(\frac{2500 \times 0.68^2}{140}\right)^{1/6} = 1.42\ m/s$

$A = \frac{Q}{V} = \frac{2500}{1.42} = 1760.56\ m^2$

$R = \frac{5}{2} \times \frac{V^2}{f} = \frac{5}{2} \times \frac{1.42^2}{0.68} = 7.41\ m$

$P = 4.75\sqrt{Q} = 4.75\sqrt{2500} = 237.5\ m$

Bed slope, $S = \frac{f^{5/3}}{3340\ Q^{1/6}} = \frac{0.68^{5/3}}{3340 \times 2500^{1/6}} = 0.000043$

Lacey's regime scour depth, $R_r' = 0.473\left(\frac{Q}{f}\right)^{1/3} = 0.473\left(\frac{2500}{0.68}\right)^{1/3} = 7.3\ m$

Question: Design stable alluvial channel using Lacey's theory. The channel is to carry 10 m^3/s through 1 mm sand. (BB AD – 2018)

Solution:

Here, $Q = 10\ m^3/s, d = 1\ mm$

$f = 1.76\sqrt{d_{mm}} = 1.76\sqrt{1} = 1.76$

$V = \left(\frac{Q\,f^2}{140}\right)^{1/6} = \left(\frac{10 \times 1.76^2}{140}\right)^{1/6} = 0.77\ m/s$

$A = \frac{Q}{V} = \frac{10}{0.77} = 12.99\ m^2$

$P = 4.75\sqrt{Q} = 4.75\sqrt{10} = 15.02\ m$

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For a trapezoidal channel with $\frac{1}{2}H:1V$ side slope

$$D = \frac{P - \sqrt{P^2 - 6.944A}}{3.472} = \frac{15.02 - \sqrt{15.02^2 - 6.944 \times 12.99}}{3.472} = 0.97 \text{ m}$$

$$B = P - \sqrt{5}\,D = 15.02 - \sqrt{5} \times 0.97 = 12.85 \text{ m}$$

$$\text{Bed slope, } S = \frac{f^{5/3}}{3340\,Q^{1/6}} = \frac{1.76^{5/3}}{3340 \times 10^{1/6}} = 0.000523$$

Hence, the channel has a bed width $B = 12.85$ m and a depth of 0.97 m.

Use a bed slope of 1 in 1910.

Question: Design an irrigation channel of discharge 11270 cusec. The bed material size is 0.014 cm, using lacey's theory design the channel. (SGFL – 2020, PGCB – 2020, DPDC – 2020)

Solution:

For irrigation channel side slope is usually $\frac{1}{2}:1$

$Q = 11270$ cusec $= 319.26$ cumec

$$f = 1.76\sqrt{d_{mm}} = 1.76\sqrt{0.014 \times 10} = 0.66$$

$$V = \left(\frac{Q\,f^2}{140}\right)^{1/6} = \left(\frac{319.26 \times 0.66^2}{140}\right)^{1/6} = 1 \text{ m/s}$$

$$A = \frac{Q}{V} = \frac{319.26}{1} = 319.26 \text{ m}^2$$

$$R = \frac{5}{2} \times \frac{V^2}{f} = \frac{5}{2} \times \frac{1^2}{0.66} = 3.8 \text{ m}$$

$$P = 4.75\sqrt{Q} = 4.75\sqrt{319.26} = 84.87 \text{ m}$$

$$D = \frac{P - \sqrt{P^2 - 6.944A}}{3.472} = \frac{84.87 - \sqrt{84.87^2 - 6.944 \times 319.26}}{3.472} = 3.8 \text{ m}$$

$$B = P - \sqrt{5}\,D = 84.87 - \sqrt{5} \times 3.8 = 76.4 \text{ m}$$

$$\text{Bed slope, } S = \frac{f^{5/3}}{3340\,Q^{1/6}} = \frac{0.66^{5/3}}{3340 \times 319.26^{1/6}} = 0.000057$$

Hence, the channel has a bed width $B = 76.4$ m and a depth of 3.8 m.

The longitudinal slope $S = 1/17545$

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Question: Design a regime Channel to carry a discharge of 50 cumec. Assume silt factor as 1.0. Use Lacey's theory for design. Assume reasonable value for missing data. (30th BCS)

Solution:

$Q = 50$ cumecs, $f = 1$

$$V = \left(\frac{Q f^2}{140}\right)^{1/6} = \left(\frac{50 \times 1^2}{140}\right)^{1/6} = 0.84 \text{ m/s}$$

$$A = \frac{Q}{V} = \frac{50}{0.84} = 59.52 \text{ m}^2$$

$$R = \frac{5}{2} \times \frac{V^2}{f} = \frac{5}{2} \times \frac{0.84^2}{1} = 1.764 \text{ m}$$

$$P = 4.75\sqrt{Q} = 4.75\sqrt{50} = 33.58 \text{ m}$$

For a trapezoidal channel with $\frac{1}{2}H:1V$ side slope

$$D = \frac{P - \sqrt{P^2 - 6.944A}}{3.472} = \frac{33.58 - \sqrt{33.58^2 - 6.944 \times 59.52}}{3.472} = 1.97 \text{ m}$$

$$B = P - \sqrt{5}D = 33.58 - \sqrt{5} \times 1.97 = 29.17 \text{ m}$$

$$\text{Bed slope}, S = \frac{f^{5/3}}{3340\, Q^{1/6}} = \frac{1^{5/3}}{3340 \times 50^{1/6}} = 0.000155$$

Hence, the channel has a bed width $B = 29.17$ m and a depth of 1.97 m.

Use a bed slope of 1 in 6410.

Question: Design a regime channel for a discharge of 60 cumec. Assume silt factor = 1.1. Assume any reasonable value of missing data. (34th BCS)

Solution:

$Q = 60$ cumecs, $f = 1.1$

$$V = \left(\frac{Q f^2}{140}\right)^{1/6} = \left(\frac{60 \times 1.1^2}{140}\right)^{1/6} = 0.89 \text{ m/s}$$

$$A = \frac{Q}{V} = \frac{60}{0.89} = 67.41 \text{ m}^2$$

$$R = \frac{5}{2} \times \frac{V^2}{f} = \frac{5}{2} \times \frac{0.89^2}{1.1} = 1.8 \text{ m}$$

$$P = 4.75\sqrt{Q} = 4.75\sqrt{60} = 36.79 \text{ m}$$

For a trapezoidal channel with $\frac{1}{2}H:1V$ side slope

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$$D = \frac{P - \sqrt{P^2 - 6.944\,A}}{3.472} = \frac{36.79 - \sqrt{36.79^2 - 6.944 \times 67.41}}{3.472} = 2.02 \text{ m}$$

$$B = P - \sqrt{5}\,D = 36.79 - \sqrt{5} \times 2.02 = 32.27 \text{ m}$$

$$\text{Bed slope, } S = \frac{f^{5/3}}{3340\,Q^{1/6}} = \frac{1.1^{5/3}}{3340 \times 60^{1/6}} = 0.0001602$$

Hence, the channel has a bed width $B = 32.27$ m and a depth of 2.02 m.

Use a bed slope of 1 in 6242.

Drainage system: While designing a canal irrigation network, it is sometimes desirable to provide suitable drainage system for removing the excess irrigation water.

Drainage coefficient: The rate at which the water is removed by a drain is called the drainage coefficient. It is expressed as the depth of water in cm or metres, to be removed in 24 hours from the drainage area. The drainage coefficient largely depends upon the rainfall but varies with the type of soil, type of crop and degree of surface drainage etc. Its recommended value is 1% of the average annual rainfall to be removed per day.

Drainage area: The area actually drained by the tile drain system is called its drainage area. Sometimes, the surface water is also to be removed by the tiles. In that case, the watershed area will be drainage area, even though it may not be entirely tiled.

Question: Why circular section is commonly used in sewer? (37th BCS)

Solution:

Sewers of circular section are most commonly used. A circular section gives the least perimeter for a given area, and therefore has the maximum hydraulic mean depth for running full and half full conditions. It is therefore the most efficient section at these flow conditions. It is the most economical section since it requires minimum quantity of material for its construction and it's easy to construct. The inner surface is smooth hence the flow of sewage is uniform and there is no chance of deposition of suspended particles.

Question: Determine the size of a circular tile drain, draining 6 hectares of a drainage area if the drainage coefficient is 1.5 cm and grade is 4%. Assume rugosity coefficient for the tile material is 0.013. (34th BCS)

Solution:

1.5 cm drainage coefficient means that 1.5 cm of water from an area of 6 hectares is entering the tiles per day.

$$\text{Volume of water passing the drain in 1 day} = \left(\frac{1.5}{100} \times 6 \times 10^4\right) = 900 \text{ m}^3/\text{day}$$

$$\text{Volume of water passing the drain in 1 second, } Q = \left(\frac{900}{24 \times 3600}\right) = \frac{1}{96} \text{ m}^3/\text{s}$$

For a circular drain of diameter D,

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$A = \frac{\pi D^2}{4}, P = \pi D \;\&\; R = \frac{A}{P} = \frac{D}{4}$

Discharge, $Q = \frac{1}{n} A R^{2/3} S^{1/2}$

$$\frac{1}{96} = \frac{1}{0.013} \times \left(\frac{\pi D^2}{4}\right) \times \left(\frac{D}{4}\right)^{2/3} \left(\frac{4}{100}\right)^{1/2}$$

$D^{8/3} = 2.18 \times 10^{-3}$

$D = 0.100 \text{ m} = 10 \text{ cm}$

Question: A drain of 15 cm diameter is laid at the end of a channel. The slope is 0.3% and manning's coefficient is 0.014. Find the minimum drainage area in hector where value of drainage coefficient 15 cm/day. (CPGCBL – 2018)

Solution:

Area, $A = \frac{\pi D^2}{4} = \frac{3.1416 \times 15^2}{4} = 176.715 \text{ cm}^2$

Perimeter, $P = \pi D = 3.1416 \times 15 = 47.124 \text{ cm}$

Hydraulic radius, $R = A/P = 176.715/47.124 = 0.266 \text{ cm}$

Discharge, $Q = \frac{1}{n} A R^{2/3} S^{1/2} = \frac{1}{0.014} \times 176.715 \times (0.266)^{2/3} (0.003)^{1/2} = 24.63 \text{ m}^3/\text{day}$

$A = \frac{Q}{C} = \frac{24.63}{0.15} = 164.2 \text{ m}^2 = 0.01642 \text{ hector}$

Question: Design drain size at the outlet of a 6 ha of drainage system of DC is 1 cm (1 cm/day) and the tile grade is 0.3%. Assume the rugosity co-efficient for the drain material as 0.011. (WARPO – 2017)

Solution:

$Q = \frac{1 \times 6 \times 10^4}{24 \times 3600 \times 100} = \frac{1}{144} \text{ m}^3/\text{s}$

We know, $Q = \frac{1}{n} A R^{2/3} S^{1/2}$

$$\frac{1}{144} = \frac{1}{0.011} \times \frac{\pi D^2}{4} \times \left(\frac{D}{4}\right)^{2/3} \times \left(\frac{0.3}{100}\right)^{1/2}$$

$D = 0.132 \text{ m} = 132 \text{ mm}$ diameter pipe

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Question: What are the causes of water-logging? (33th BCS)

Solution:

When the water table rises to such heights that the soil pores in the root zone become saturated, thus displacing the air, the land is said to be water logged. Water logging is happened when the soil is so filled or soaked with water that caused the roots of the plant to rot. Water logging is 100% when water table rises to the surface.

Causes of water logging:

- Inadequate surface drainage
- Over irrigation
- Obstruction of nature drainage
- Poor irrigation management
- Irregular or flat topography
- Seepage from canal system and higher irrigated areas.
- Excessive rain
- Submergence due to floods
- Impervious Obstruction

Question: What is meant by water logging? Explain principal causes and effects of water logging in an irrigation canal. (34th BCS)

Solution:

When water table reaches upto or near to ground level then such a land is called as water logged. Due to water logging productivity of land gets affected. Water logging can be also referred as saturation of soil with excess of water.

Causes of waterlogging

- Over and Intensive Irrigation
- Seepage of water from the adjoining high lands
- Seepage of water through the canals
- Impervious Obstruction
- Inadequate natural drainage
- Inadequate surface drainage
- Excessive rain
- Submergence due to floods
- Irregular or flat topography

Effects of water logging:

- Inhabiting activity of soil bacteria
- Decrease in available capillary water
- Fall in soil temperature
- Defective air circulation
- Delay in cultivation operations
- Growth of wild flora
- Adverse effect on community health

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Question: What are the causes of water logging? Explain in brief the measures adopted for controlling water logging? (30th & 38th BCS)

Solution:

When the water table rises to such heights that the soil pores in the root zone become saturated, thus displacing the air, the land is said to be water logged. Water logging is happened when the soil is so filled or soaked with water that caused the roots of the plant to rot. Water logging is 100% when water table rises to the surface. However the process of water logging starts even when the water table is quite below the surface.

Causes of Waterlogging

- Over and Intensive Irrigation
- Seepage of water from the adjoining high lands
- Seepage of water through the canals
- Impervious Obstruction
- Inadequate natural drainage
- Inadequate surface drainage
- Excessive rain
- Submergence due to floods
- Irregular or flat topography

Measures adopted for controlling water logging

Lining of canals and water courses: Attempts should be made to reduce the seepage of water from the canals and water courses. This can be achieved by lining them. It is very effective method to control water-logging.

Reducing the intensity of irrigation: In areas where there is a possibility of water-logging, intensity of irrigation should be reduced. Only a small portion of irrigable land should receive canal water in one particular season. The remaining areas can receive water in the next season, by rotation.

By optimum use of water: It is a known fact that only a certain fixed amount of irrigation water gives best productivity. Less than that and more than that reduces the yield.

By providing intercepting drains: Intercepting drains along the canals should be constructed, wherever necessary. These drains can intercept and prevent the seeping canal water from reaching the area likely to be water logged.

By provision of an efficient drainage system: An efficient drainage system should be provided in order to drain away the storm water and the excess irrigation water.

By improving the natural drainage of the area: To reduce the percolation, the water should not be allowed to stand for a longer period. This can be achieved by removing bushes, forests, etc. and improving the slopes of the natural drainage lines.

By adopting consumptive use of surface and subsurface water: The introduction of lift irrigation to utilize ground water helps in lowering the water table in a canal irrigated area, where water table tends to go up.

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