Detailed Solutions

ESE-2019 Mains Test Series

Civil Engineering Test No : 15

Section A

Q.1 (a) Solution:

Given side of square plate, $L = 1\text{ m}$

$\therefore$ Area, $A = L^2 = 1\text{ m}^2$

$$\bar{x} = \frac{L}{\sqrt{2}} + 0.5$$

$$= 1.207\text{ m}$$

Hydrostatic force on plate, $F_x = \rho g A\bar{x}$

$$= 1000 \times 9.81 \times 1 \times 1.207$$

$$= 11840.67\text{ N}$$

Depth of centre of pressure, $\bar{h} = \bar{x} + \dfrac{I_{CG}\sin^2\theta}{A\bar{x}}$

where, $\theta = 90°$

and $I_{CG} = \dfrac{L^4}{12} = \dfrac{1}{12} = 0.083\text{ m}^4$

$\therefore$ $\bar{h} = 1.207 + \dfrac{0.083}{1 \times 1.207} = 1.276$ m from free surface of water

Q.1 (b) Solution:

Point rainfall data values are recorded through raingauges. The mean depth of precipitation is a significant data extensively used for Hydrological studies. These methods are listed below.

1. **Arithmetical Mean Method:** It is rarely used method. This method is used where there is little variations in rainfall data.

$$\bar{P} = \frac{1}{N}\sum_{i=1}^{N} P_i \qquad \{N = \text{No. of raingauge rainfall data}\}$$

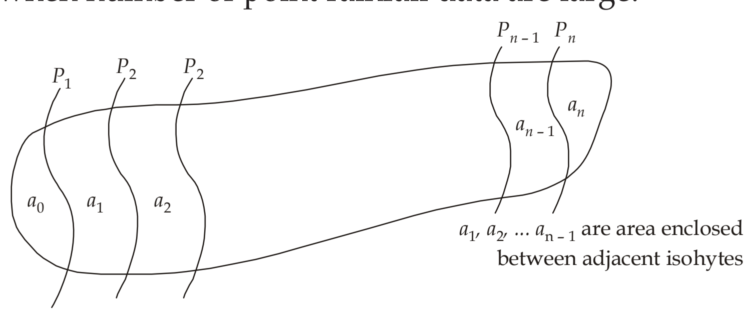
2. **Thiessen Mean Method:** According to this method, rainfall recorded at each station is given a weightage on basis of an area closest to that station. The position of rainguage stations in marked on the catchment map drawn to scale. These positions are joined to form a network of triangles. Perpendicular bisectors are then drawn for each side of triangle. These bisectors from a polygon around each station which is called *Thiessen Polygon*.

Let $A_1, A_2 \,.....\, A_i$ = area of Thiessen Polygons

$$A = \sum_{i=1}^{N} A_i \text{ catchment area}$$

$$\bar{P} = \frac{\sum_{i=1}^{N} P_i A_i}{A} \qquad \frac{A_i}{A} = \text{Weightage factor}$$

3. **Isohyetal Method:** Most accurate method and is performed over other two. Everything is drawn on scale and on the map, isohyets are drawn of various values. Also used when number of point rainfall data are large.



$$\bar{P} = \frac{P_1 a_0 + \left(\frac{P_1 + P_2}{2}\right) + a_2\left(\frac{P_2 + P_3}{2}\right) + \,.....\, + a_{n-1}\left(\frac{P_{n-1} + P_n}{2}\right) + P_n a_n}{A(\text{Total catchment area})}$$

Q.1 (c) Solution:

Advantages:

1. They are quite economical in operation.
2. They do not require skilled attention during operation.
3. There are no moving parts and so there is no mechanical wear and tear.
4. They require only preliminary treatments.
5. There is no difficulty in sludge removal. Also the sludge volume and weight are less.
6. The results obtained are good, with 60 to 70% removal of solids and 30 to 40% removal of BOD.

Disadvantages:

1. Because of greater depth, cost of construction is higher.
2. Unsuitable for acidic influents.
3. They give offensive odours, when improperly operated.
4. They have tendency to foam or boil, due to which the sludge particles may enter the sedimentation zone through the slot, thus affecting the quality of the effluent.

Q.1 (d) Solution:

(i)
$$\text{SVI} = \frac{180 \text{ ml}}{2.5 \text{ gm}} = 72 \text{ ml/gm}$$

(ii)
$$\text{SS concentration in return sludge} = \frac{10^6}{SVI} = \frac{10^6}{72} = 13888 \text{ mg}/l$$

(iii)
$$\text{Return sludge ratio} = \frac{Q_r}{Q_0} = \frac{X_t}{\frac{10^6}{\text{SVI}} - X_t}$$

$$= \frac{2500}{\frac{10^6}{72} - 2500} = 0.219 \simeq 0.22$$

Q.1 (e) Solution:

The monthly consumptive use (in cm) is calculated by Blaney-Criddle formula i.e.

$$C_u = k \times \frac{p}{40} \times (1.8t + 32)$$

where k is consumptive use coefficient, p is monthly per cent of annual day light hours and t is mean monthly temperature in °C

If $\quad \frac{p}{40}(1.8t + 32) = f$

then $\quad C_u = kf$

$\therefore$ Seasonal consumptive use, $C_u = k\Sigma f$

The calculations are tabulated below:

Month	t(°C)	p(%)	R_e(cm)	$f = \frac{p}{40}(1.8\,t + 32)$
November	18	7.20	2.60	11.6
December	15	7.15	2.80	10.5
January	13.5	7.30	3.50	10.3
February	14.5	7.10	2.00	10.3
			ΣR_e = 10.9 cm	**Σf = 42.7 cm**

$\therefore$ Seasonal consumptive use or evapotranspiration,

$$C_u = k \times \Sigma f = 0.8 \times 42.7 = 34.16 \text{ cm}$$

$\therefore$ Consumptive irrigation requirement = $C_u - \Sigma R_e$ = 34.16 – 10.9 = 23.26 cm

$\therefore$ Field irrigation requirement = $\frac{CIR}{\eta_a} = \frac{23.26}{0.65} = 35.8$ cm

Q.2 (a) Solution:

(i)

$$\text{Total runoff (volume)} = 1.2 \times 10^6 \text{ m}^3$$

$$A\text{, catchment area} = 3000 \text{ hectares}$$

$$= 3000 \times 10^4 \text{ m}^2$$

$$= 30 \text{ km}^2$$

$$\therefore \quad R = \text{Total runoff depth} = \frac{\text{Total runoff volume}}{\text{Catchment area}}$$

$$= \left(\frac{1.2 \times 10^6}{30 \times 10^6}\right) = 0.04 \text{ m} = 40 \text{ mm}$$

$$P = \text{Total rainfall} = 90 \text{ mm (from table)}$$

$$\therefore \quad \text{Total infiltration} = (P - R) = (90 - 40) = 50 \text{ mm}$$

$\therefore$ W-index $= \left(\dfrac{P-R}{t_r}\right) = \left(\dfrac{90-40}{14}\right) = \dfrac{50}{14} = 3.57$ mm/hr

Since ϕ-index has to be somewhat more than W-index, it is concluded that ϕ-index would be little more than 3.57 mm/hr, thereby causing infiltration in every 2 hour interval, somewhat more than 2 × 3.57 = 7.14 mm.

The incremental rainfalls in different 2 hour interval are worked out as:

Time from start of storm (hr)	**Accumulated rainfall in (mm)**	**Incremental rainfall during each interval in (mm)**
0	0	-
2	6	$\boxed{6}$
4	17	11
6	57	40
8	70	13
10	81	11
12	87	$\boxed{6}$
14	90	$\boxed{3}$
		$\Sigma P = 90$ mm

From the above table, it is concluded that values obtain in Col. (3) that no excess rainfall has occurred in 3 out of 7 intervals (each of 2 hours), when rainfall is lesser than the average infiltration. Hence, excess rain must have fallen only in 4 intervals i.e., 4 × 2h = 8 hr.

Hence, $t_e = 8$ hours

$$\phi\text{-index} = \frac{\text{Total infiltration during the period of excess rainfall}}{\text{Period of rainfall excess}}$$

$$= \frac{\text{Total infiltration} - \text{Infiltration during the period when no excess rain occurs}}{t_e}$$

$$= \frac{(50-6-6-3)}{8} = \frac{35}{8} \text{ mm/hr}$$

$\Rightarrow$ ϕ-index $= 4.375$ mm/hour

(ii)

1. **Field Irrigation requirement (FIR):** It is defined as the amount of water required to meet the net irrigation requirements plus the amount of water lost as surface runoff and through deep percolation.

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

η_a =Factor accounting for the loss of irrigation water by surface runoff and through deep percolation.

2. **Water application efficiecy (η_a):** It is defined as the ratio of the quantity of water stored in the root zone of the plants to the quantity of water delivered to the field.

$$\eta_a = \frac{W_s}{W_f} \times 100$$

3. **Water Storage Efficiency (η_s):** It is defined as the ratio of the quantity of water stored in the root zone during irrigation to the quantity of water needed to bring the moisture content of the soil to the field capacity. Thus if W_s is the quantity of water stored in the root zone during irrigation and W_n is the quantity of water needed to bring the moisture content of the soil to the field capacity.
(i.e., W_n = Field capacity – Available moisture in the soil prior to irrigation), then

$$\eta_s = \frac{W_s}{W_n} \times 100$$

4. **Water Distribution Efficiency (η_d):** Water distribution efficiency evaluates the degree to which water is uniformly distributed throughout the root zone during irrigation and hence it is also known as uniformity coefficient. The water distribution efficiency provides a measure for comparing different methods of irrigation.

   It is determined from the following expression in which y is the average numerical deviation in depth of water stored from the average depth of water d stored in the root zone during irrigation.

$$\eta_d = \left[1 - \frac{y}{d}\right] \times 100$$

Q.2 (b) Solution:

(i)

Some practical applications of hydraulic jump are given below:

1. It is used as energy dissipator at downstream end of dam, sluice gate etc.
2. It is used to increase the water depth in irrigation canal to divert water to side canal or field.
3. It is used for aeration in water distribution system.
4. It is also used for mixing chemicals, in many of water distribution systems.
5. It is also used to increase water depth in apron to counteract uplift pressure.
6. It can also be used in recreational activities like kayaking or canoeing.

7. It prevents the scouring on the downstream side of a dam structure.
8. It enables the efficient operation of flow measuring devices like flume, weirs, notch etc.
9. It can also be used to remove air from water supply and sewage lines to prevent air locking.
10. It is used to protect against damage due to cavitation, vibration, abrasion etc.

(ii)

For sudden reduction in discharge (due to closing of gate) a negative surge will move downstream.

Let, suffix 1 refers to flow conditions before the gate closure and suffix 2 after the passage of negative wave.

So, Velocity, $V_1 = \frac{20}{3 \times 1.8} = 3.704 \text{ m/s}$

Depth, $y_1 = 1.8 \text{ m}$

Discharge, $Q_1 = 20 \text{ m}^3/\text{s}$

After passage of wave,

Discharge, $Q_2 = 12 \text{ m}^3/\text{s}$

Discharge per unit width, $q_2 = \frac{12}{3} = 4 \text{ m}^2/\text{s}$

and, $q_2 = V_2 y_2$

$\Rightarrow$ $4 = V_2 y_2$ ...(i)

Also, for negative surge,

$$V_2 = V_1 + 2\sqrt{g y_2} - 2\sqrt{g y_1}$$

$$= 3.704 + 2\sqrt{9.81 \times y_2} - 2\sqrt{9.81 \times 1.8}$$

$$= 6.264\sqrt{y_2} - 4.70 \quad \text{...(ii)}$$

From equation (i) and (ii),

Solving by trial and error,

$y_2 = 1.43 \text{ m}$

$V_2 = 2.797 \text{ m/s}$

Height of surge, $H = 1.8 - 1.43 = 0.37 \text{ m}$

$$\text{Velocity of wave at crest, } C_1 = \sqrt{gy_1} + V_1$$

$$= \sqrt{9.81 \times 1.8} + 3.704 = 7.906 \text{ m/s}$$

$$\text{Velocity of wave at trough, } C_2 = \sqrt{gy_2} + V_2$$

$$= \sqrt{9.81 \times 1.43} + 2.797 = 6.542 \text{ m/s}$$

Q.2 (c) Solution:

(i)

Cavitation is defined as the phenomenon of formation of vapour bubbles of a flowing liquid in a region where the pressure of a liquid falls below its vapour pressure and sudden collapsing of these vapour bubbles in a region of higher pressure.

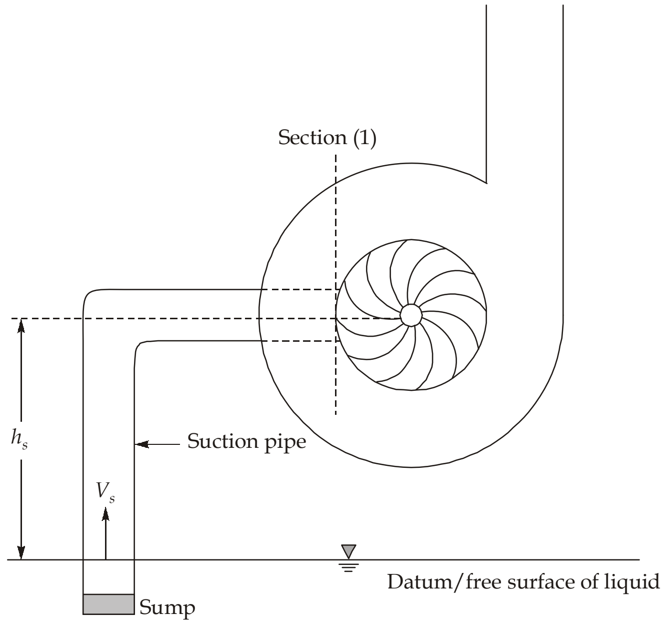
Effects: When the vapour bubbles collapse, a very high pressure is created in the space which otherwise was previously occupied by bubble.

- The metallic surfaces, above which these vapour bubbles collapse, is subjected to these high pressure, which causes pitting action on the surface. Thus cavities are formed on the surface.
- There is considerable noise and vibrations also in case of cavitation.
- Due to sudden collapse on the metallic surface, high pressure is produced and metallic surfaces are subjected to high local stresses. Thus surfaces get damaged.
- Efficiency of turbine decreases due to cavitation.

Available net positive suction head (NPSH) is very commonly used in pump industry. It is defined as the absolute pressure head at the inlet of the pump, minus the vapour pressure head (in absolute units) plus the velocity head.

$$\therefore \quad \text{NPSH} = \frac{p_1}{\rho g} - \frac{p_v}{\rho g} + \frac{V_s^2}{2g} \quad \text{...(i)}$$

Applying Bernoulli's principle at free surface of liquid and at section 1 in the pipe section just at the inlet of pump, we get



$$\frac{p_a}{\rho g}+\frac{V_a^2}{2g}+Z_a = \frac{p_1}{\rho g}+\frac{V_1^2}{2g}+Z_1+h_L$$

$$\frac{p_a}{\rho g} = \frac{p_1}{\rho g}+\frac{V_s^2}{2g}+h_s+h_{fs}$$

$$\frac{p_1}{\rho g} = \frac{p_a}{\rho g}-\left(\frac{V_s^2}{2g}+h_s+h_{fs}\right) \quad \text{...(ii)}$$

Using eq. (ii) in equation (i)

$$\text{NPSH} = \frac{p_a}{\rho g}-\left(\frac{V_s^2}{2g}+h_s+h_{fs}\right)-\frac{p_v}{\rho g}+\frac{V_s^2}{2g}$$

$$\Rightarrow \quad \text{NPSH} = \frac{p_a}{\rho g}-\frac{p_v}{\rho g}-h_s-h_{fs}$$

$$\Rightarrow \quad \text{NPSH} = \left[H_a-H_v-h_s-h_{fs}\right] = \text{Total suction head}$$

Thus NPSH may also be defined as the total head required to make the liquid flow through the suction pipe to the impeller of the pump.

Thoma's cavitation factor is used to indicate whether cavitation will occur in pumps. The value of Thoma's cavitation factor for pump is given by:

$$\sigma = \frac{(H_{atm}-H_v)-H_s-H_{fs}}{H_m}$$

$$\Rightarrow \quad \sigma = \frac{NPSH}{H_m} \quad \text{...(iii)}$$

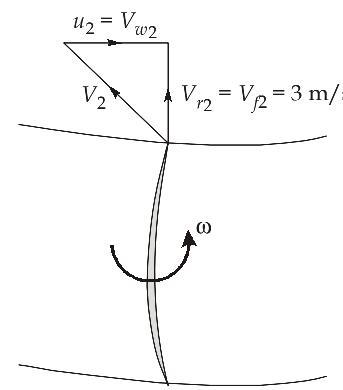
If the value of σ [calculated from equation (iii)] is less than the critical value σ_c, then cavitation will occur in the pumps.

σ_c depends on specific speed of the pump, N_s.

The following empirical relation may also be used to calculate σ_c :

$$\sigma_c = 1.03 \times 10^{-3} (N_s)^{4/3}$$

(ii)



Given data:

Outside diameter of impeller,

$$D_2 = 35 \text{ cm} = 0.35 \text{ m}$$

$$\text{Speed, } N = 1000 \text{ rpm}$$

$$\text{Width of outlet, } B_2 = 7 \text{ cm} = 0.07 \text{ m}$$

$$\text{Velocity of flow, } V_{f_2} = 3 \text{ m/sec}$$

$$\text{Velocity in suction pipe, } V_s = 2.5 \text{ m/s}$$

$$\text{Velocity in delivery pipe, } V_d = 1.5 \text{ m/s}$$

Now, tangential velocity of impeller at outlet,

$$u_2 = \frac{\pi D_2 N}{60} = \frac{\pi \times 0.35 \times 1000}{60} = 18.326 \text{ m/sec}$$

As the vanes are radial at exit,

$$V_{w_2} = u_2 = 18.326 \text{ m/s}$$

As we know, the head developed will be given by,

$$H_m = \frac{V_{w_2} u_2}{g} \eta_{mano}$$

Assuming losses of the pmp to be zero, head developed will be

$$H_m = \frac{V_{w_2} u_2}{g} = \frac{u_2^2}{g}$$

$$= \frac{(18.326)^2}{9.81} = 34.235 \text{ m}$$

Now, $$H_m = h_s + h_d + h_{fs} + h_{fd} + \frac{V_d^2}{2g}$$

where, $$h_s + h_d = H_s$$

which is known as the static head which is the net vertical height through which the liquid is lifted by the pump.

$\because$ Frictional losses are to be neglected

$$\therefore \quad H_s = H_m - \frac{V_d^2}{2g}$$

$$= 34.235 - \frac{1.5^2}{2 \times 9.81} = 34.12 \text{ m}$$

The power of the pump is given by,

$$H_p = \rho Q g H_s(\text{Watt}) = QgH_s(\text{kW}) = \frac{QgH_s}{0.746}(HP)$$

where, $Q = \pi B_1 D_i V_{f_1} = \pi \times 0.35 \times 0.07 \times 3 = 0.231 \text{ m}^3/\text{s}$

1 Horse power = 745.7 Watt $\simeq$ 0.746 kW

$$\therefore \quad \text{Horse power of pump, } H_p = \frac{0.231 \times 34.12 \times 9.81}{0.746} = 103.604 \text{ hp}$$

Q.3 (a) Solution:

(i)

Self cleansing velocity is given as

$$V_s = \sqrt{\frac{8k}{f}(G_S - 1) \times gd_s}$$

$$= \sqrt{\frac{8 \times 0.06}{0.02}(2.66 - 1) \times 9.81 \times 0.001} = 0.625 \text{ m/sec}$$

Since sewer runs half full $\therefore \frac{d}{D} = \frac{1}{2}$

$$\therefore \quad a = \frac{1}{2} \times \frac{\pi}{4} \times D^2 = \frac{\pi}{8}D^2$$

and $$p = \frac{\pi D}{2}$$

$$\therefore \quad r = \frac{a}{p} = \frac{\frac{\pi}{8}D^2}{\frac{\pi D}{2}} = \frac{D}{4} = \frac{0.6}{4} = 0.15 \text{ m}$$

Now, $$v = \frac{1}{n} r^{2/3} s^{1/2}$$

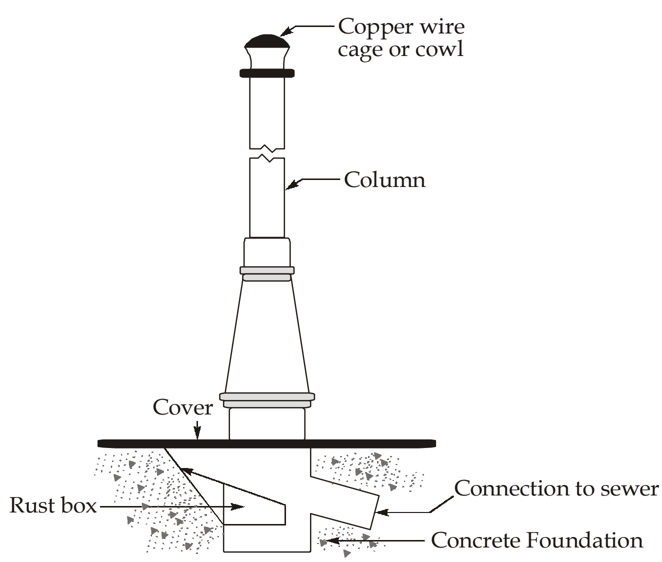
$$\Rightarrow \quad 0.625 = \frac{1}{0.012}(0.15)^{2/3} s^{1/2}$$

$$\Rightarrow \qquad s = \frac{1}{1416.9} \simeq \frac{1}{1417}$$

(ii)

Methods of ventilation : Ventilation may be done by adopting any one or combination of the following methods:

1. **Man holes with gratings :** The man hole covers are provided with perforations or gratings through which sewer gases escape. However, this method can not be adopted in residential locality since it will spread foul smell and will cause air pollution. It is adopted only in isolated localities. Another disadvantage of this method is that it permits road dust, large quantities of storm water and other debris.
2. **Ventilating columns or shafts :** Ventilation columns or shafts are provided at the upper end of every branch sewer and also at every location where there is change in size of sewers, apart from their regular placement at the interval of about 200 to 300 m along the sewer lines.



Ventilating column consists of a vertical shaft made by joining cast iron or steel pipe lengths. A foundation block is provided at the bottom end to keep it steady in the vertical position. At the top, copper wire dome or cowl prevent blockage by nesting birds. Foul gases escape from the cowl. The internal diameter of the column is preferably kept one third the diameter of the sewer served by it. The joints of pipes forming the column should be made air tight. The height of the shaft should be sufficient to effectively discharge the foul gases and to be clear of the flat roof of the nearby buildings. The location of the shaft should be such that it obtains sunshine for the major portion of the day.

3. **Proper design and construction of sewer :** Sewer should be so designed that they run half or two third full. The space above the flow level will provide ventilation. Also, the flow velocity should be self-cleansing. This can be achieved by laying the sewers at proper gradients.
4. **Proper house drainage system :** House vent and soil pipes may be used with advantage to ventilate house drain and the lateral sewers into which they drain, particularly where interceptors are not provided on the sewers connecting houses and buildings.
5. **Unobstructed outlets :** Unobstructed outlets provide partial ventilation to storm water drains and sewers.
6. **Use of mechanical devices :** Forced draft is provided by exhaust fans to expel out foul gases from sewers.

Q.3 (b) Solution:

Average sewage flow = 6.5 MLD

Total mass of suspended solids = 6.5 × 250 = 1625 kg per day

Mass of solids removed in primary clarifier = 0.58 × 1625 = 942.5 kg per day

Moisture content of influent sludge = 96%

∴ Solids content in influent sludge = 100 – 96 = 4%

So 942.5 kg of dry sludge will make = $\frac{100}{4} \times 942.5 = 23562.5$ kg of wet sludge per day

Given, specific gravity of primary wet sludge = 1.03

Volume of primary sludge produced per day = $\frac{23562.5}{1.03 \times 1000} = 22.88 \text{ m}^3$

Initial volatile solids content in sludge = 70%

Mass of volatile solids in sludge = $\frac{70}{100} \times 942.5 = 659.75$ kg per day

∴ Mass of non-volatile solids in sludge = 942.5 – 659.75 = 282.75 kg

Volatile solids destroyed@65% = $\frac{65}{100} \times 659.75 = 428.84$ kg per day

∴ Mass of non-digested volatile solids = $\frac{35}{100} \times 659.75 = 230.91$ kg per day

$\therefore$ Mass of solids in digested sludge = 282.75 + 230.91 = 513.66 kg per day

Digested sludge solids concentration = 8%

So, 513.66 kg of dry solids will make = $\frac{100}{8} \times 513.66 = 6420.75$ kg of digested sludge

Volume of digested sludge produced = $\frac{6420.75}{1.04 \times 1000} = 6.174 \text{ m}^3/\text{day}$

Mean cell residence time = 15 days

So, capacity of digestion tank,

$$V = \left[V_1 - \frac{2}{3}(V_1 - V_2)\right] \times t$$

$$= \left[22.88 - \frac{2}{3}[22.88 - 6.174]\right] \times 15$$

$$= 176.14 \text{ m}^3$$

Ans.

Q.3 (c) Solution:

Given data: $h_L = 20$ m; $d = 300$ mm;

$L = 2000$ m, $f = 0.04$

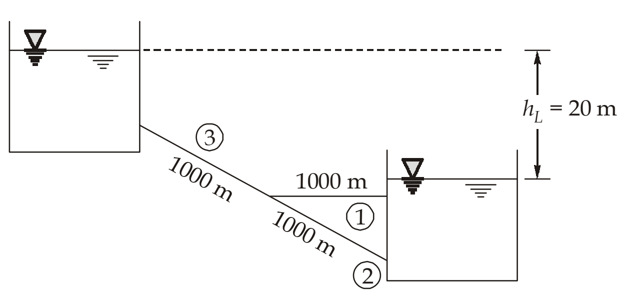
The difference in levels of the two reservoir will be the total head loss. This head loss may be calculated by Darcy Weisbach's formula as :

$$h_L = \frac{fLQ^2}{12.1d^5}$$

$$\Rightarrow \quad 20 = \frac{0.04 \times 2000 \times Q^2}{12.1 \times (0.3)^5}$$

$$\Rightarrow \quad Q = 0.0857 \text{ m}^3/\text{s}$$

Now, when a pipe with same diameter is attached parallel to the existing pipeline to the last 1000 m, the arrangement will be as shown below.



Let the discharge in the pipe 1, 2 and 3 be Q_1, Q_2 and Q_3 respectively.

$\therefore$ $Q_3 = Q_1 + Q_2$ [From continuity equation]

$h_{L_1} = h_{L_2}$ [Condition for parallel pipe connection]

$$\frac{f L_1 Q_1^2}{12.1 d_1^5} = \frac{f L_2 Q_2^2}{12.1 d_2^5}$$

[where $L_1 = L_2 = 1000$ m and, $d_1 = d_2 = 300$ mm]

$\Rightarrow$ $Q_1 = Q_2$

$\therefore$ $Q_3 = Q_1 + Q_1 = 2Q_1$

$\Rightarrow$ $Q_1 = \dfrac{Q_3}{2}$

Also, $Q_1 = Q_2 = \dfrac{Q_3}{2}$

$\therefore$ Total head loss will be

$$h_L = h_{L_3} + h_{L_1}$$

$$\Rightarrow \quad 20 = \frac{fL_3 Q_3^2}{12.1\, d^5} + \frac{fL_1 Q_1^2}{12.1\, {d_1}^5}$$

$$\Rightarrow \quad 20 = \frac{0.04 \times 1000 \times Q_3^2}{12.1 \times (0.3)^5} + \frac{0.04 \times 1000 \times Q_3^2}{4 \times 12.1 \times (0.3)^5}$$

$$\Rightarrow \quad 20 = \frac{0.04 \times 1000 \times Q_3^2}{12.1 \times (0.3)^5}\left[1 + \frac{1}{4}\right]$$

$$\Rightarrow \quad Q_3 = 0.1084 \text{ m}^3/\text{s}$$

It is evident from the value of Q_3 that the discharge has increased. Thus pipes are laid in parallel to increase the discharge.

$$\text{Percentage increase in discharge} = \left(\frac{Q_3 - Q}{Q}\right) \times 100$$

$$= \frac{0.1084 - 0.0857}{0.0857} \times 100$$

$$= 26.49\% \simeq 26.5\%$$

Q.4 (a) Solution:

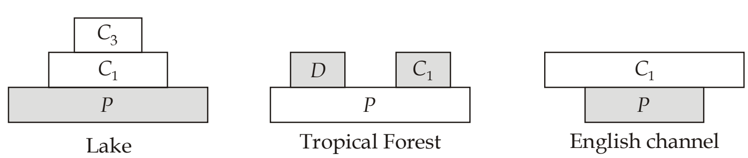
(i)

Ecological Pyramid : The interaction of the food chain phenomena (i.e. energy loss at each transfer) and the metabolism relationship results in communities having a definite trophic structure. The graphical representation of the trophic structure and also trophic function is called ecological pyramid. In ecological pyramid, the first or producer level forms the base and successive levels form the tiers which make up the apex. In simple words, if we arrange the organisms in a food chain according to trophic levels, they will form a pyramid. The ecological pyramids are of three types:

1. The Pyramid of Numbers
2. The Pyramid of Biomass
3. The Pyramid of Energy

1. **The pyramid of Numbers :** In this, the number of individual organisms at different trophic levels in an ecosystem are depicted. The total number of individual organisms at producer level (first trophic level) from the base of the numbers pyramid and the population of primary consumers, secondary, tertiary consumers and so on forms the successive tiers of the pyramid of numbers. The length of the bar, at different levels (tiers), represents the numbers (population) of organism at that particular trophic level, by using a convenient scale.

2. **The Pyramid of Biomass :** It is based on the total dry weight, calorific value, or any other measure of the total amount of living material. In this, the total amount of living material (say in gms dry wt. per sq. m) in organisms at different levels is depicted.



3. **The Pyramid of Energy :** In this, the rate of energy flow or productivity at successive trophic levels is shown.

   where, D = Detrivores, C_3 = Tertiary consumers, C_2 = Secondary consumers, C_1 = Primary consumers and P = Producers.

   The energy pyramid always takes a true upright pyramid shape, provided all source of food energy in the system are considered, because less food energy is available at the upper trophic level than is available to lower levels.

(ii)

1.

Time (min)	dB	Serial no. (m)	dB	Percent of time equalled or exceeded $=\left(\frac{m}{N+1}\right)100$
10	71	1	84	9.1
20	75	2	80	18.2
30	70	3	78	27.3
40	78	4	76	36.4
50	80	5	75	45.5
60	84	6	75	54.5
70	76	7	74	63.6
80	74	8	74	72.7
90	75	9	71	81.8
100	74	10	70	90.9

By interpolation

$$L_{60} = 74+\frac{75-74}{(63.6-54.5)}(63.6-60)$$

$$= 74.4 \text{ dB} \qquad \textbf{Ans.}$$

2.
$$L_{eq} = 10\log\left(\sum_{i=1}^{n} t_i 10^{L_i/10}\right)$$

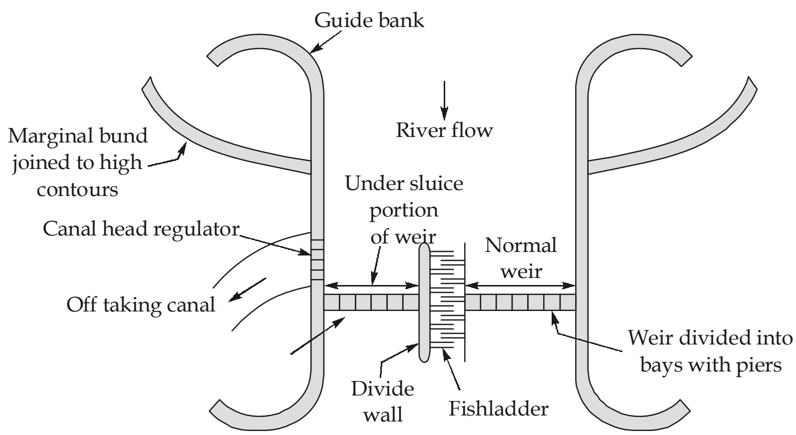
$$= 10\log\begin{bmatrix}10^{71/10}+10^{75/10}+10^{70/10}+10^{78/10}+10^{80/10}+10^{84/10} \\ +10^{76/10}+10^{74/10}+10^{75/10}+10^{74/10}\end{bmatrix}\times\frac{10}{100}$$

$$= 77.71 \text{ dB} \qquad \textbf{Ans.}$$

Q.4 (b) Solution:

(i)

The typical layout of a diversion headwork is given below:



Diversion head works consists of:

1. **Weir proper :** The weir is used to raise the level of water on the upstream side.
2. **Under sluice or scouring sluices:** A comparatively less turbulent pocket of water is created near canal head regulator by constructing under sluices portion of the weir. The divide wall separates the main weir portion from the under sluice portion of the weir. The crest of the under sluice portion of the weir is kept at a lower level than the crest of normal portion of the weir. Under sluices help in bypassing the excess supplies to the downstream side of the river. These also help in scouring and removing the deposited silt from the under sluiced pocket and hence are also called scouring sluices.
3. **Divide wall:** The divide wall is a masonry or concrete wall constructed at right angles to the axis of the weir and separates the weir proper from the under sluices. It helps in providing a comparatively less turbulent water pocket near the canal head regulator, resulting in deposition of silt in this pocket and thus to help in the entry of silt free water into the canal. Divide wall may keep the cross currents, if at all they are formed, away from the weir.
4. **River training works such as marginal bunds, guide banks, groynes etc:** River training works are required near the weir site in order to ensure a smooth and an axial flow of water, and thus, to prevent the river from out flanking the works due to a change in its course. The guide banks force the river into a restricted channel and thus ensuring a smooth and an almost axial flow near the weir site. Marginal

bunds are provided on the upstream side of the works in order to protect the area from submergence due to rise in HFL caused by afflux. These bunds are therefore, continued till they join contours higher than the new HFL.

5. **Fish ladder:** Large rivers are generally inhabited by several types of fish, many of which are migratory. Such migratory type of fishes are called anadromous fish which move from one part of the river to another part, according to season. Thus, a structure which enables the fishes to pass upstream is called a fish ladder. It is a device by which the flow energy can be dissipated in such a manner as to provide smooth flow at sufficiently low velocity.

6. **Canal head regulator:** A canal head regulator serves the following functions:
   (i) It regulates the supply of water entering into the canal.
   (ii) It controls the entry of silt in the canal.
   (iii)It prevents the river floods from entering into the canal.

7. **Silt regulation works:** The entry of silt into a canal which takes off from a headworks can be reduced by constructing certain special works, called silt control works.

   These silt control works can be of two types:

   1. **Silt Excluder :** These are constructed on the bed of river upstream of head regulator. The clear water enters the head regulator and the silted water enters the silt excluder. In this type of work, the silt is therefore removed from water before it enters into the canal.
   2. **Silt ejector :** These devices extract the silt from the canal water after the silted water has travelled a certain distance in the off-take canal. These works are therefore constructed on the bed of the canal, and a little distance downstream from the head regulator.

(ii)

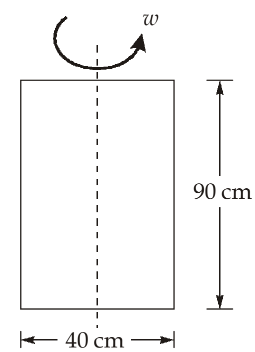
For 4 hour duration, excess rain fall, R = 3 cm

Time (h)	Ord. of 2-Hour UH (m^3/s) (i)	2-Hour UH lagged by 2 hr. (ii)	Ord. of 4 hr DRH (i) + (ii) = (iii)	Ord. of 4-Hour UH = (iii)/2	Ord. of 4-Hour = 4 hr UH × 3 (cm)	Base flow (m^3/s)	Ord. of 4 hr flood hydrograph = 4 hr DRH + Base flow
0	0	-	0	0	0	5	5
2	8	0	8	4	12	5	17
4	25	8	33	16.5	49.5	5	54.5
6	30	25	55	27.5	82.5	5	87.5
8	22	30	52	26	78	5	83
10	16	22	38	19	57	5	62
12	8	16	24	12	36	5	41
14	5	8	13	6.5	19.5	5	24.5
16	0	5	5	2.5	7.5	5	12.5

$\therefore$ Ordinate of resulting flood hydrograph = (3 × ordinate of 4 Hr UH + Base flow)

Q.4 (c) Solution:

(i)

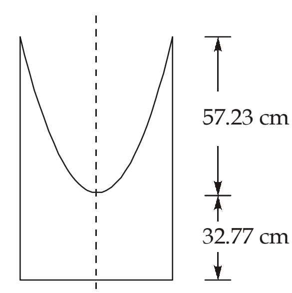


Given $N = 160$ rpm

$\therefore$ $$w = \frac{2\pi N}{60} = \frac{2\pi \times 160}{60} = 16.755 \text{ rad/s}$$

$$Z = \frac{w^2 r^2}{2g} = \frac{(16.755)^2 \times (0.2)^2}{2 \times 9.81}$$

$$= 0.5723 \text{ m} = 57.23 \text{ cm}$$



Hydrostatic pressure,

1. At bottom of cylinder at edge,

$$P_1 = \rho g h$$

$\Rightarrow$ $$\frac{P_1}{\rho g} = 0.9 \text{ m}$$

2. At centre bottom,

$$P_2 = \rho g\,(h - Z)$$

$$\frac{P_2}{\rho g} = 0.3277 \text{ m}$$

(ii)

Given : Free cylindrical vortex $\Rightarrow V \,\alpha\, \frac{1}{R};\ V = \frac{C}{R}$

at $R = 0.1$ m, $V = 10$ m/s $$10 = \frac{C}{0.1}$$

$$C = 1$$

$\Rightarrow$ $$VR = 1$$

Fundamental equation, $$dP = \frac{\rho \cdot v^2}{r} \cdot dr - \rho g \cdot dz = \frac{\rho \cdot (1/r)^2}{r} \cdot dr - \rho g dz$$

$$dP = \frac{\rho}{r^3} \cdot dr - \rho g dz$$

Integrating both sides $$\int_{P_1}^{P_2} dP = \int_1^2 \frac{\rho}{r^3} \cdot dr - \int_1^2 \rho g dz$$

$$P_2 - P_1 = -\frac{\rho}{2}\left[\frac{1}{r_2^2} - \frac{1}{r_1^2}\right] - \rho g(z_2 - z_1)$$

Here, $P_1 = 200 \times 10^3$ N/m^2, $r_1 = 0.1$ m

$r_2 = 0.2$ m, P_2 is to be found

$\rho = \rho_\omega = 10^3$ kg/m^3

and $z_2 - z_1 = 0$

$$\Rightarrow \quad P_2 - 200 \times 10^3 = -\frac{10^3}{2}\left[\frac{1}{0.2^2} - \frac{1}{0.1^2}\right]$$

$$\Rightarrow \quad P_2 = 237.5 \text{ kN/m}^2$$

Section B

Q.5 (a) Solution:

19.5 cm on the map was originally 20 cm,

$\therefore$ 1 cm^2 on the map was originally $\frac{20^2}{19.5^2}$ cm^2

$= 1.05194$ cm^2

$\therefore$ 125.5 cm^2 area on map was originally

$= 125.5 \times 1.05194$

$= 132.01847$ cm^2

Since scale of map was 1 cm = 40 m

$\Rightarrow$ 1 cm^2 = 1600 m^2

$\therefore$ Actual area on the ground = 1600×132.01847

$= 211229.552$ m^2

Since the chain was 0.05 m too long.

$$\therefore \quad \text{True area} = \frac{20.05^2}{20^2} \times \frac{211229.552}{10^4} = 21.23 \text{ hectares}$$

Q.5 (b) Solution:

Embankment given: $V = 1$ m^3; $\gamma_d = 18$ kN/m^3; $w = 15\%$

$$\therefore \quad \gamma_d = \frac{G \cdot \gamma_w}{1+e}$$

$$\Rightarrow \quad 18 = \frac{2.7 \times 10}{1+e}$$

$$\Rightarrow \quad e = 0.5$$

$\therefore$ Volume of solids, $$V_{s_1} = \frac{V}{1+e} = \frac{1}{1+0.5} = 0.67 \text{ m}^3$$

Borrow pit: $\gamma = 17 \text{ kN/m}^3$; $w = 8\%$;

$$\Rightarrow \quad \gamma_d = \frac{\gamma}{1+w}$$

$$\Rightarrow \quad \gamma_d = \frac{17}{1+0.08} = 15.74 \text{ kN/m}^3$$

$$\Rightarrow \quad \frac{G \cdot \gamma_w}{1+e} = 15.74$$

$$\Rightarrow \quad \frac{2.7 \times 10}{1+e} = 15.74$$

$$\Rightarrow \quad e = 0.715$$

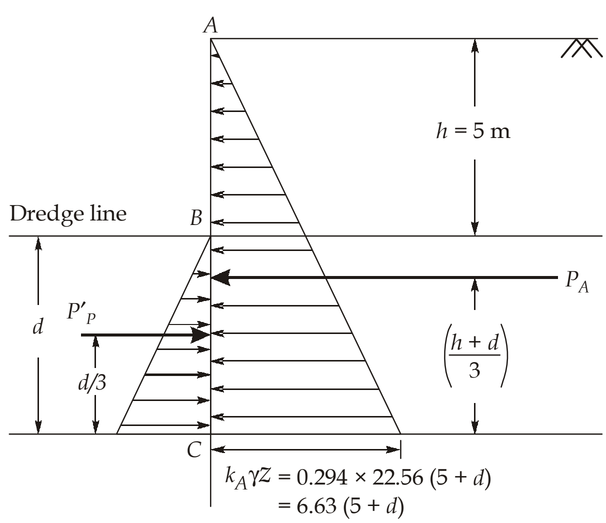
Let 'V' be the volume excavated from borrow pit for 1 m^3 of embankment.
But, volume of solids is same both in borrow pit in embankment,

$$\therefore \quad V_{s_1} = V_{s_2}$$

$$\Rightarrow \quad 0.67 = \frac{V}{(1+0.715)}$$

$$\Rightarrow \quad V = 1.1433 \text{ m}^3$$

Q.5 (c) Solution:



Given data : $h = 5$ m, $d = ?$, $\gamma = 2.3 \times 9.81 \text{ kN/m}^3 = 22.56 \text{ kN/m}^3$, $\phi = 33°$

$$k_A = \tan^2\left(45 - \frac{\phi}{2}\right) = \tan^2\left(45 - \frac{33}{2}\right) = 0.294$$

$$k_p = \tan^2\left(45 + \frac{\phi}{2}\right) = \tan^2\left(45 + \frac{33}{2}\right) = 3.4$$

$\therefore$ $$P_A = \frac{1}{2} \times 6.63(5+d)(5+d) = 3.315(5+d)^2$$

Now, the actual passive resistance developed on front side in the embedded portion is,

$$\Rightarrow \quad P'_P = \frac{2}{3} \times \text{Theoretical passive resistance}$$

$$\Rightarrow \quad P'_P = \frac{2}{3} \times P_P = \frac{2}{3}\left[\frac{1}{2} k_p \gamma d^2\right]$$

$$\Rightarrow \quad P'_P = \frac{2}{3} \times \frac{1}{2} \times 3.4 \times 22.56 d^2$$

$$\Rightarrow \quad P'_P = 25.568\, d^2 \simeq 25.57 d^2$$

Taking moments about point C

$$P_A\left(\frac{h+d}{3}\right) = P'_p \times \frac{d}{3}$$

$$\Rightarrow \quad 3.315(5+d)^2\left(\frac{5+d}{3}\right) = 25.57 d^2 \cdot \frac{d}{3}$$

$$\Rightarrow \quad (5+d)^3 = 7.71 d^3$$

$$\Rightarrow \quad 5 + d = 1.98 d$$

$$\Rightarrow \quad d = 5.102 \text{ m}$$

Increase this depth by 20%, so as to enable the development of passive resistance behind the wall. Hence, drive the sheet piles to a depth = 1.2 × 5.102 $\simeq$ 6.15 m.

Q.5 (d) Solution:

The factors influencing the choice of side-support system are as follows:

(a) Shape of cross-section of excavation: If the cross-section is circular then ribs with lagging or segmental lining is often used. This is not so for square or rectangular cross-section of excavation.

(b) Presence of groundwater : If ground water is present and water tight side support is needed, then only sheet piles, diaphragm walls, secant piles or segmental lining with water tight gaskets are used. Wooden lagging and soil nails are not that much effective.

(c) Temporary or permanent support : If support is to be permanent, then concrete is usually the preferred material. For temporary support, steel and wooden members are used since they can be reused.

(d) **Size of the excavation :** For small sized excavations, wooden members are used and for large excavations, steel sheet piles and concrete members are used. In narrow width excavations, struts are used for bracing whereas in large width excavations, rackers or tiebracks are used.

(e) **Depth of excavation :** For deep excavations, strong steel and concrete members are used.

(f) **Installation technique :** Driven supports are avoided close to buildings due to noise and vibrations. Diaphragm walls are avoided if trench excavation causes settlement of adjacent footing.

Q.5 (e) Solution:

$$L_{major} = 100 \text{ m}, L_{minor} = 20 \text{ m}$$

$$T_{major} = 12 \text{ sec}$$

$$L_{major} = \frac{gT^2}{2\pi} \tan h\,(kd); \qquad \text{where } k = \frac{2\pi}{L}$$

$$\Rightarrow \quad 100 = \frac{9.81 \times 12^2}{2\pi} \times \tan h\,(kd)$$

$$\Rightarrow \quad \tan h\,(kd) = 0.445 \qquad \text{...(i)}$$

But
$$\tan h\, x = \frac{e^x - e^{-x}}{e^x + e^{-x}}; \qquad x = kd$$

$$\frac{e^x - e^{-x}}{e^x + e^{-x}} = 0.445$$

On solving for x from (i)

$$x = 0.4784$$

$$\therefore \quad kd = 0.4784;$$

$$\therefore \quad \frac{2\pi}{L_{major}} d = 0.4784$$

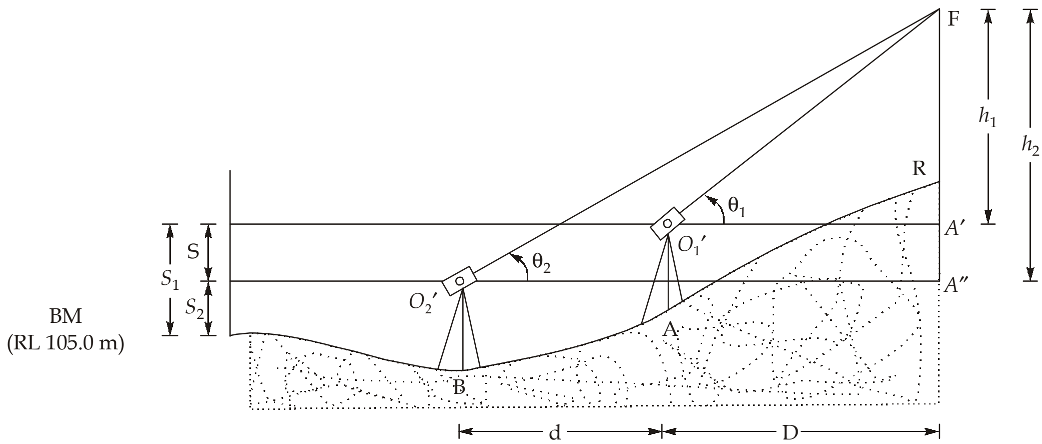
But
$$L_{major} = 100 \text{ m} \quad \text{(Given)}$$

$$\therefore \quad \frac{2\pi}{100} \times d = 0.4784$$

$$\therefore \quad d = 7.61 \text{ m}$$

Q.6 (a) Solution:

(i)



From figure, $h_2 - h_1 = S_1 - S_2 = S$ (say)

From triangle $\Delta O_1'A'F$, $h_1 = D \tan \theta_1$ ...(i)

and from $\Delta O_2'A''F$, $h_2 = (D + d) \tan \theta_2$ ...(ii)

Subtracting eq. (i) from eq.(ii), we get

$$h_2 - h_1 = (D + d) \tan \theta_2 - D \tan \theta_1$$

$$\Rightarrow \quad D = \frac{(d \tan \theta_2 - S)}{(\tan \theta_1 - \tan \theta_2)} \qquad [\text{As } h_2 - h_1 = S]$$

$$\Rightarrow \quad D = \frac{60 \tan 12°40' - (2.815 - 1.865)}{\tan 18°30' - \tan 12°40'} = 114.11 \text{ m}$$

Now, $h_1 = D \tan\theta_1 = 114.11 \tan 18°30'$

$= 38.18$ m

RL of top of transmission tower $=$ RL of BM $+ S_1 + h_1$

$= 105.0 + 2.815 + 38.18$

$= 145.995 \text{ m} \simeq 146 \text{ m}$

(ii)

Differential GPS : Differential GPS is a system in which differences between observed and computed coordinates or ranges called differential corrections, at a particular known point called as the reference station, are transmitted to users to improve the accuracy of the user's receiver position. In fact, DGPS is a way to make GPS even more accurate. It can yield measurements good to a couple of meters in moving applications, and even better in stationary situations. This improved accuracy has a very profound effect on the importance of GPS as a resource and it becomes a universal measurement system, capable of positioning things on a very precise scale. DGPS is mainly a navigation method in which there is a fixed reference station and coordinates of moving platform or rover are determined in real time.

Principle of DGPS :

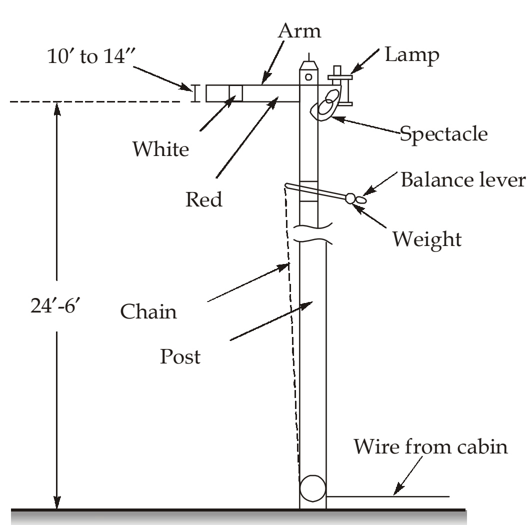
DGPS is based on the concept that bias error in position of the location is similar to those for all locations in a given local area (say within 100 km). Unfortunately, a user can not just figure out the error once and use it to correct all the measurements made for the rest of the day, because the satellite errors are continuously changing. A user needs to have two receivers working simultaneously to do the job. The reference receiver stays put and continuously monitors the errors and then transmits or records corrections for those errors so that the second receiver (the one that is out roving around doing positioning work) can apply these corrections to its measurements, either as it is making them or some time later. Thus, by having a reference receiver at a fixed location, the user can time up the accuracy of a roving receiver, or for a whole fleet of roving receivers.

Q.6 (b) Solution:

(i)

1. Semaphore Signals

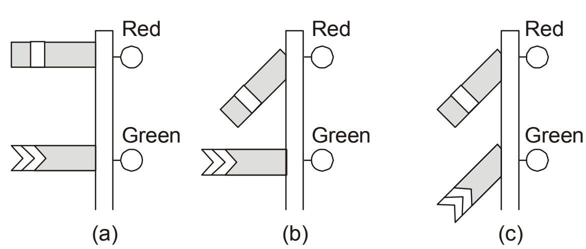
- It consists of a movable arm pivoted on a horizontal pin near the top of a post, spectacle holding two coloured glasses, lamp for night indication, crank rod, cam, lever and counter weights, signal post, chain and pulley, wire to cabin.
- The arm is 4 to $5\frac{1}{2}$ ft. long ,9 to 12 inch wide at the inner edge and 10 to 14 inch wide at the outer edge.



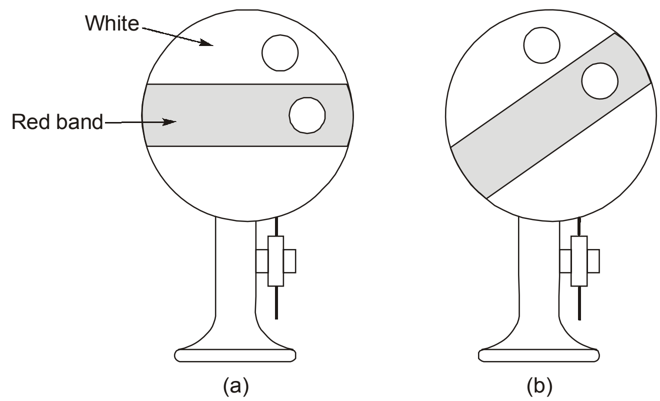
- On the side facing the driver, it is painted red with a white vertical band.
- The other side is painted white with a black vertical band.
- When the arm is in horizontal position or shows red light at night, it indicates stop.
- When the arm is inclined at 45°, it indicates proceed with the corresponding light as green.
- A semaphore signal at the entry to the station is combined with a warner signal.
- The arm of a warner signal is fish tailed and its white band is in the form of a V in unison with the outer edge.
- The warner is painted yellow and exhibits yellow or amber colour at night instead of red.
- It is placed on the same post of semaphore signal (about 6 to 7 ft. below the semaphore signal)
- When both the signals are horizontal, they indicate "Stop". This means neither approaching section is clear, nor next block section is clear.
- When semaphore is lowered and the warner is horizontal, they indicate "proceed with caution".

  This means section is clear upto the station but the next block section is not clear.
- When both the semaphore and warner are lowered, they indicate "proceed with confidence".

  This means the station section and also the next block section are clear.



2. Disc Signals (Shunting Signals)

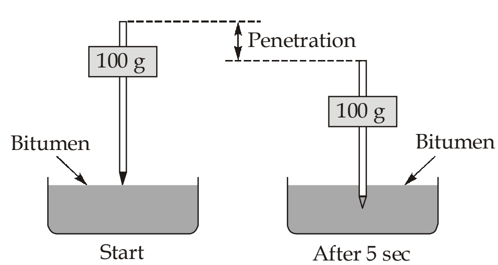


- It is used in station yards for shunting operation.
- Also known as shunting signals or miniature semaphore signals.
- They consists of circular discs painted white with a red band along its diagonal.
- The disc can revolve in a vertical plane by pulling of a lever by hand.
- When the red band is horizontal or shows red light at night, its indicates "Stop" and when inclined at 45° or show green light at night, it indicates "Proceed".
- The centre of the disc is about 2 feet above the ground level.

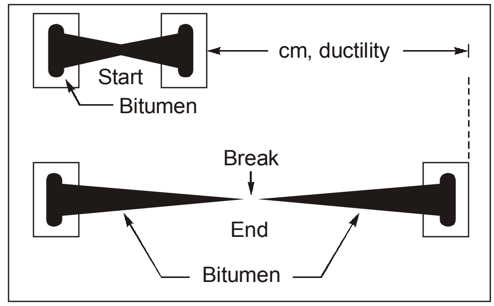
(ii)

Tests of Bitumen

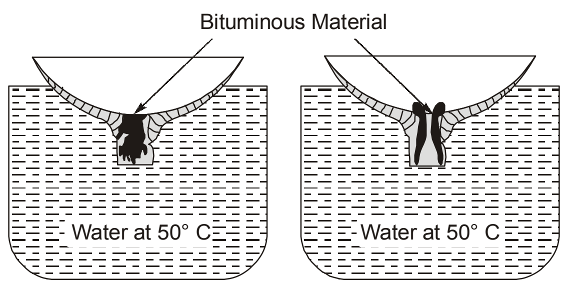
1. **Penetration Test:** Hardness of bitumen is obtained by the penetration test. It measures the distance upto which a standard blunt pointed needle will vertically penetrate a sample of bitumen material at 27°C, the load being 100g and time of application of load being 5 seconds.



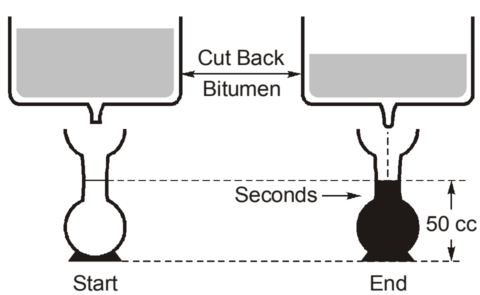
2. **Ductility Test:** Bitumen should be sufficiently ductile and capable of being stretched without breaking. Ductility is measured as the distance in cm to which a standard briquette of size 10 × 10 mm can be stretched before the thread breaks at a standard temperature of 27°C and the rate of elongation is 50 mm per minute.



3. **Float Test:** This test is used to measure the consistency of bitumen for which penetration and viscosity test cannot be used. Higher the float value, the stiffer is the bitumen.



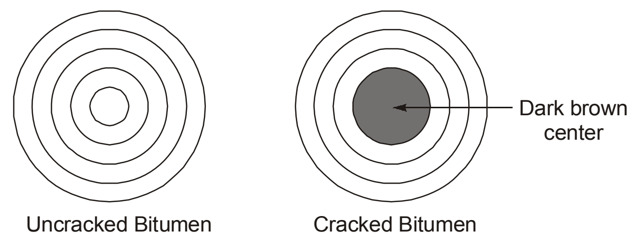
4. **Viscosity Test:** Viscosity is the inverse of fluidity and it is a measure of resistance to flow. The viscosity of liquid bitumen is measured by efflux viscometer. In this test, viscosity is measured by time taken in seconds by 50 m*l* of bitumen to flow from a container through a specified orifice of size 10 mm under standard test conditions and temperature of 25°C.



5. **Solubility Test:** This test is used to measure the quantity of impurity present in the bitumen. Pure bitumen is soluble in carbon disulphide (CS_2) and carbon tetrachloride (CCl_4) while the impurities are insoluble.
6. **Softening Point Test:** The softening point is the temperature at which the substance attains a particular degree of softening under specified condition of test. Apparatus used to determine softening point of bitumen is RING AND BALL assembly. A

steel ball is placed over the bitumen sample and then liquid is heated at the rate of 5°C per minute. The temperature at which the softened bitumen touches the metal placed at a particular specified distance below the ring is taken as softening point of bitumen.

7. **Spot Test:** It is used to detect whether the bitumen is cracked or not. In this test 2g of bitumen is dissolved in 10 m*l* of naptha. A drop of this solution is taken on a filter paper. If the strain of the spot on the paper is uniform in colour, the bitumen is accepted as uncracked but if the spots form dark brown or black circle in the centre, the bitumen is considered to be over heated or cracked.



8. **Loss on Heating Test:** When bitumen is heated, it loses the volatiles and gets hardened. This test is conducted by an accelerated heat test. 50 g of sample is heated at a temperature of 163°C for 5 hours in a special oven designed for this test. Not more than 1% loss in weight is desirable. Lesser the loss on heating, the better is the bitumen.

9. **Water Content Test:** It is desirable that the bitumen contains minimum water content to prevent foaming of the bitumen when it is heated above the boiling point of water. The water content in bitumen is determined by mixing a known weight of bitumen sample in pure petroleum distillate which is free from water, heating and distilling off the water. The weight of water condensed and collected is expressed as percentage by weight of original sample. The maximum water content in bitumen should not exceed 0.2 percent by weight.

Q.6 (c) Solution:

(i)

Soil fails under local shear,

$\therefore$ Mobilised parameters c_m and ϕ_m should be used

$$c_m = \frac{2}{3}\times c = \frac{2}{3}\times 36 = 24 \text{ kN/m}^2$$

$$\tan\phi_m = \frac{2}{3}\tan\phi$$

$$\Rightarrow \quad \phi_m = \tan^{-1}\left(\frac{2}{3}\tan 20°\right) = 13.6°$$

$$\therefore \quad N_c = 11, N_q = 4, N_\gamma = 1.6$$

$$D_f = 1 \text{ m}, \quad B = 1.5 \text{ m}$$

Ultimate bearing capacity, $q_u = c_m N_c + qN_q + 0.5B\gamma N_\gamma$

$$\Rightarrow \quad q_u = (24 \times 11) + (17.6 \times 1 \times 4) + (0.5 \times 1.5 \times 17.6 \times 1.6)$$

$$\Rightarrow \quad q_u = 355.52 \text{ kN/m}^2$$

$$\therefore \quad q_{nu} = q_u - \gamma D_f = 355.52 - (17.6 \times 1) = 337.92 \text{ kN/m}^2$$

$$\therefore \quad q_{ns} = \frac{q_{nu}}{F} = \frac{337.92}{3} = 112.64 \text{ kN/m}^2$$

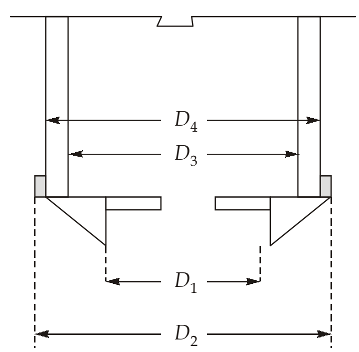
$\therefore$ Safe bearing capacity, $q_s = q_{ns} + \gamma D_f$

$$q_s = 112.64 + (17.6 \times 1) = 130.24 \text{ kN/m}^2$$

$\therefore$ Safe bearing capacity per metre length of footing,

$$Q_s = q_s \times B = 130.24 \times 1.5 = 195.36 \text{ kN/m}$$

(ii)



Given: $D_1 = 69$ mm; $D_2 = 73$ mm; $D_3 = 70$ mm; $D_4 = 72$ mm

(i) Inside clearance, $$C_i = \frac{D_3 - D_1}{D_1} \times 100 = \frac{70-69}{69} \times 100 = 1.4493\%$$

(ii) outside clearance, $$C_o = \frac{D_2 - D_4}{D_4} \times 100 = \frac{73-72}{72} \times 100 = 1.3889\%$$

(iii) Area ratio, $$A_R = \frac{A_2 - A_1}{A_1} \times 100 = \frac{\left(\frac{\pi}{4}D_2^2\right) - \left(\frac{\pi}{4}D_1^2\right)}{\frac{\pi}{4}D_1^2} \times 100$$

$$= \frac{D_2^2 - D_1^2}{D_1^2} \times 100 = \frac{73^2 - 69^2}{69^2} \times 100 = 11.9303\ \%$$

As per IS Code, area ratio should be less than 10% but in no case it should exceed 20%.

Q.7 (a) Solution:

(i) The worst-case scenario occurs when the ground water rises to the surface.

$\therefore$ Uplift pressure, $p_w = h\gamma_w = (5 + 1)9.81 = 58.86$ kPa

Uplift force, P_{up} at the base of the culvert = Bp_w

$\therefore$ $= 5 \times 58.86 = 294.3$ kN/meter length

Assume a wall thickness of t m

$\therefore$ Weight per unit length of box culvert, $W = [(5 \times 6) - (5 - 2t)(6 - 2t)] \times 24$

$$\therefore \quad \text{FOS} = \frac{W}{P_{up}}$$

$$\Rightarrow \quad W = P_{up} \times \text{FOS}$$

$$\Rightarrow \quad 24\,[(5 \times 6) - (5 - 2t)(6 - 2t)] = 294.3 \times 1.2$$

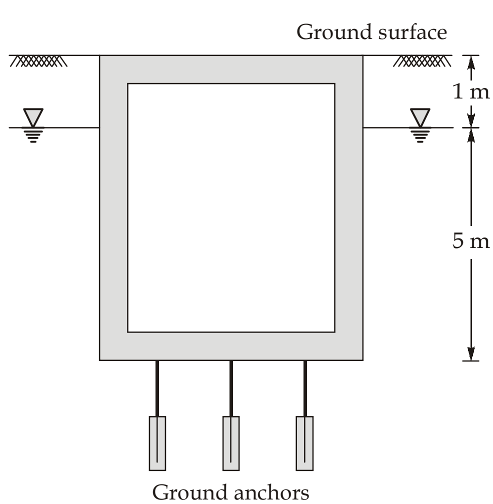
$$\Rightarrow \quad 4t^2 - 22t + 14.715 = 0$$

$$\therefore \quad t = 4.72 \text{ m}, 0.779 \text{ m}$$

$$\because \quad t = 4.72 \text{ m not possible being too large}$$

$$\text{So} \quad t = 0.779 \text{ m} = 779 \text{ mm} \simeq 780 \text{ mm (say)}$$

(ii) One potential method to prevent uplift is to use ground anchors, as shown below. The anchors must support the difference between the uplift force and the net downward resistance (weight plus side shear resistance of soil) with sufficient factor of safety (> 1.2).

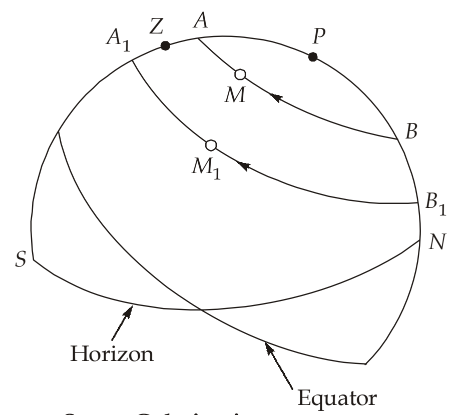


Q.7 (b) Solution:

(i)

Let θ be lattitude of the observer

$$\delta = \text{Declination of the star}$$



Star at Culmination

Let M be the star having A and B as its upper and lower transits.

At upper transits, zenith distance

$$ZA = ZP - AP$$
$$= (90 - \theta) - (90 - \delta) = \delta - \theta$$

$\therefore$ Altitude of star $= 90° - \text{Zenith distance}$

$$= 90 - (\delta - \theta)$$

But according to question,

$$70°20' = 90 - (\delta - \theta)$$
$$\delta - \theta = 19°40' \quad \text{...(i)}$$

At lower transits, the zenith distance of star,

$$ZB = ZP + PB$$
$$= (90 - \theta) + (90 - \delta) = 180 - (\theta + \delta)$$

$\therefore$ Altitude of star $= 90° - \text{Zenith distance}$

$$= \theta + \delta - 90°$$

But $\theta + \delta - 90° = 20°40'$

$\Rightarrow$ $\theta + \delta = 110°40'$ ...(ii)

Solving (i) and (ii), we get

$$\delta = 65°10'$$
$$\theta = 45°30'$$

(ii)

Fundamental lines of a theodolite: The fundamental lines of a theodolite are the vertical axis, axis of plate levels, the line of collimation, the horizontal axis and the bubble line of altitude. The following relationships are desired:

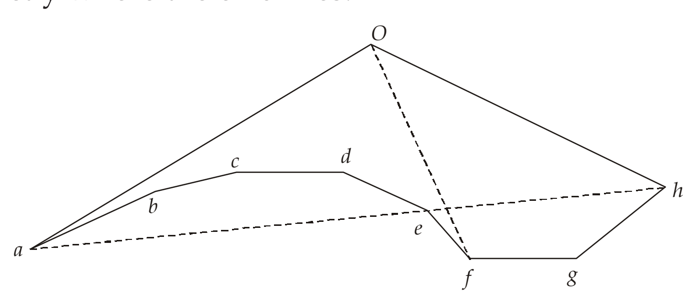
1. The axis of the plate levels must be at perpendicular to the vertical axis.
2. The line of collimation must be at right angles to the horizontal axis.
3. The horizontal axis must be perpendicular to the vertical axis.
4. The axis of the telescope level must be parallel to the line of collimation.

Errors: The following errors arises if above mentioned relationships are not maintained:

1. **Vertical axis error (α) :** Axis not vertical in an observations either from imperfect plate level adjustment or settlement of instrument.
2. **Lateral collimation error (β) :** Line of collimation not perpendicular to the horizontal axis.
3. **Horizontal axis error (γ) :** Horizontal axis not perpendicular to the vertical axis.
4. **Vertical collimation error (δ) :** Line of altitude bubble not parallel to the line of collimation when the verniers of vertical circle read zero.

Checks for an open traverse: In an open traverse, method to check the measurements as a whole are not available. Though linear measurements cannot be determined, the angular errors may be found by indirect method as follows:

1. **Well defined point:**
   (i) A well-defined point such as O is chosen and bearing of the object O is measured from different stations say a, f, h etc.
   (ii) Knowing the coordinates of O and h, the bearing of line hO can be calculated. The traverse from a to h can be checked by comparing observed and computed bearing of hO.
   (iii) The limitation of the method is that if any discrepancy is found, it is not possible to say exactly where the error lies.



2. **Cutoff lines:** By the cutoff lines, the different parts of the traverse can be checked as the work progresses.

Q.7 (c) Solution:

(i)

Area of longitudinal reinforcement per metre length of slab is given by

$$A_s = \frac{fbhw}{100s_s}$$

where, A_s = Area of longitudinal steel (cm^2)

f = Coefficient of friction = 1.5

b = Distance between joint and nearest free edge (m) = $\frac{7.2}{2} = 3.6$ m

h = Thickness of slab (in cm) = 22 cm

w = Unit weight of concrete (in kg/m^3) = 2500 kg/m^3

s_s = Allowable working stress in tension in steel (in kg/cm^2) = 1400 kg/cm^2

s_b = Allowable bond stress in concrete (in kg/cm^2) = 24.6 kg/cm^2

$$\therefore \quad A_s = \frac{1.5\times 3.6\times 22\times 2500}{100\times 1400} = 2.12 \text{ cm}^2$$

Assuming the diameter of the tie bar as 1 cm, number of tie bars required per metre will be

$$n = \frac{2.12}{\frac{\pi}{4}\times 1^2} = 2.699$$

$$\therefore \quad \text{Spacing of the tie bars} = \frac{1000}{2.699} = 370.51 \text{ mm} \simeq 370 \text{ mm}$$

Alternatively,

$$\text{Spacing of 1 cm diameter tie bars} = \frac{1000\times \frac{\pi}{4}\times 1^2}{2.12}$$

$$= 370.47 \simeq 370 \text{ mm} \quad \text{(say)}$$

$$\text{Now, length of the tie bar,} \quad L_t = \frac{ds_s}{2s_b}$$

$$\Rightarrow \quad L_t = \frac{1\times 1400}{2\times 24.6} = 28.5 \text{ cm say } 30 \text{ cm}$$

$\therefore$ Provide 1 cm diameter bars of length 30 cm at a spacing of 370 mm c/c.

(ii)

Speed range (m / s)	Average speed (V_i) (m / s)	Volume of flow (q_i)	$V_i q_i$	$\frac{q_i}{V_i}$
6 – 10	8	1	8	0.125
11 – 15	13	3	39	0.231
16 – 20	18	0	0	0
21 – 25	23	6	138	0.261
		$\Sigma q_i = 10$	$\Sigma V_i q_i = 185$	$\Sigma \frac{q_i}{V_i} = 0.617$

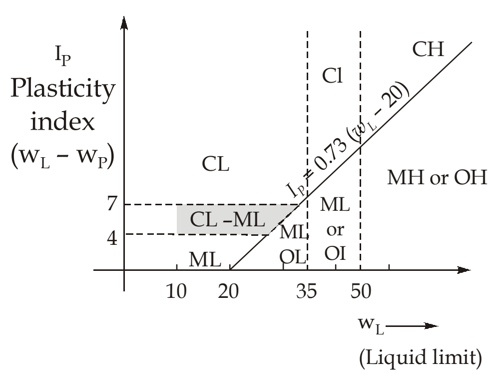
$$\text{Space mean speed} = \frac{\Sigma q_i}{\Sigma\left(\frac{q_i}{V_i}\right)} = \frac{10}{0.617} = 16.21 \text{ m/s}$$

$$\text{Time mean speed} = \frac{\Sigma q_i V_i}{\Sigma q_i} = \frac{185}{10} = 18.5 \text{ m/s}$$

Q.8 (a) Solution:

(i)

Equations of *A*-line and *U*-line for plasticity chart:



Plasticity Chart

***A* - line** $\rightarrow I_P = 0.73\ (w_L - 20)$

***U* - line** $\rightarrow I_P = 0.9\ (w_L - 8)$

- Plasticity chart is used to classify fine grained soils.
  $L \rightarrow$ Low compressible/Low plastic $\rightarrow w_L < 35\%$
  $I \rightarrow$ Medium compressible/Medium plastic $\rightarrow w_L = 35$ to 50%
  $H \rightarrow$ High compressible/High plastic $\rightarrow w_L > 50\%$
  $C \rightarrow$ clay $M \rightarrow$ silt $O \rightarrow$ Organic soil.
- Plasticity index of soil is calculated and let it be $(I_P)_{soil}$.
- $(I_P)_{soil} > (I_P)_{A\text{-line}}$, then soil is clay else it is silt or organic soil.

- *U*-line represents upper boundary above which no result should lie. If results are found to be above *U*-line then test should be repeated.

(ii)

$$e = \frac{n}{1-n} = \frac{0.3}{1-0.3} = 0.428$$

$$i_c = \frac{G-1}{1+e} = \frac{2.6-1}{1+0.428} = 1.12$$

$$i = \frac{2\,m}{1.5\,m} = 1.33$$

$$\text{F.O.S} = \frac{i_c}{i} = \frac{1.12}{1.33} = 0.84$$

Now, for F.O.S = 2

$$i = \frac{i_c}{\text{F.O.S}} = \frac{1.12}{2} = 0.56$$

for $i = \dfrac{h}{L}$

$$L = \frac{2}{0.56} = 3.57\text{ m}$$

Gravel layer required = 3.57 – 1.5 = 2.07 m

Q.8 (b) Solution:

(i)

True difference in levels between A and B = $\dfrac{(3.810-2.165)+(2.355-0.910)}{2} = 1.545$ m

Error due to curvature = $0.07849\, d^2$ where d is in km

= $0.07879\,(1.53)^2 = 0.1837$ m

∴ When the level is at A, corrected staff reading on $B = 3.810 - (c_c - c_r) + c_1$

where c_c = correction due to curvature = 0.1837 m

c_r = correction due to refraction

c_1 = correction due to collimation

$$= -\left(\frac{-0.004}{100}\right)\times 1530 = 0.0612\text{ m}$$

Corrected staff reading on B = $3.810 - (0.1837 - c_r) + 0.0612$

= $3.6875 + c_r$

∴ True difference in levels between A and B,

$$= (3.6875 + c_r - 2.165)$$
$$= 1.5225 + c_r$$
$$\therefore \quad 1.5225 + c_r = 1.545$$
$$\Rightarrow \quad c_r = 0.0225 \text{ m}$$
$\therefore$ Correction due to refraction = 0.0225 m

(ii)

Objectives of triangulation surveys: The triangulation surveys are carried out :

(i) To establish accurate control for plane and geodetic surveys of large areas, by terrestrial methods.

(ii) To establish accurate control for photogrammetric surveys of the large areas.

(iii)To assist in the determination of the size and shape of the earth by making observations for latitude, longitude and gravity.

(iv)To determine accurate locations of points in engineering works such as fixing of centreline and abutments of long bridges over large rivers. etc.

Criteria for selection of layouts of triangles: The criteria while deciding and selecting a suitable layout of triangles are as follows:

1. Triangles should preferrably be equilateral.
2. Braced quadrilaterals should preferrably be equilateral.
3. Centered polygons are regular.
4. The arrangements should be such that computations can be done through two or more independent routes.
5. The arrangements should be such that at least one route and preferrably two routes from well conditioned triangles.
6. No angle of the figure opposite to the known side should be small.
7. The sides of figures should be of comparable lengths. Very long and very short lengths should be avoided.
8. Angles of similar triangles should not be less than 45° and in case of quadrilaterals no angle should be less than 30°.

Well conditioned triangles:

- The triangles of such a shape, in which any error in angular measurement has minimum effect upon the computed lengths is known as well-conditioned triangles.
- The best shape of an a well-conditioned triangle is isosceles triangle is that triangle, whose base angles are 56°14′ each. However, from practical considerations, an equilateral triangle may be treated as well condition triangle. In actual practice, the triangles having an angle less than 30° or more than 120° should not be considered.

Strength of figure: The strength of figure is a factor to be considered in establishing a triangulation system to maintain the computations within the desired degree of precision. It also plays an important role in deciding layout of a triangulation system.

Q.8 (c) Solution:

(i)

1. **Initial Compression:** If soil is partially saturated, then immediately after the application of load, the volume decreases due to expulsion of air as well as due to compression of pore air which is called initial compression. At the end of initial compression of soil, it becomes fully saturated if load is sufficiently large. The result of initial compression is the immediate settlement which is usually determined by using elastic theory, even though the deformation itself is not truly elastic. Computation of immediate settlement has to be made in the design of shallow foundations.
2. **Primary Consolidation:** After the initial compression, soil is fully saturated and further decrease in volume occurs due to the expulsion of pore water and compression of pore water (water is incompressible, hence volume change due to compression of pore water is negligible). It is a time dependent phenomenon which depends upon permeability of soil and magnitude of load applied. The rate of flow is controlled by pore pressure, the permeability and compressibility of soil with the passage of time as the pore pressure dissipates, the rate of flow decreases and eventually, flow ceases altogether, leading to a condition of constant effective stress. This signifies the end of primary consolidation.
3. **Secondary Consolidation:** After the completion of primary settlement when pore water pressure ceases to zero at the top surface, and no decrease in volume may be expected hence forth. But in practice there is always a certain decrease in volume after primary consolidation after a long time which is called secondary consolidation. The actual cause for secondary consolidation is not well established but it may be attributed to plastic readjustment of soil solids. The secondary settlement in granular soils is insignificant but in highly plastic soils it is 10-20% of total consolidation.
4. **Primary Compression Ratio:** It is the ratio of primary compression to total compression and is denoted by r_p.

   $$\therefore \qquad r_p = \frac{\text{Primary compression}}{\text{Total compression}}$$

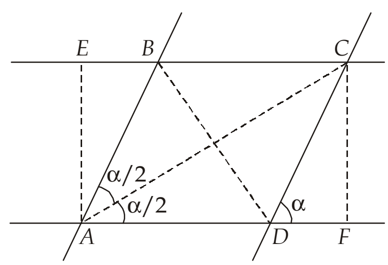
   Primary compression ratio and other ratios like initial compression ratio and secondary compression ratios are used while determining the coefficient of consolidation (C_v) in laboratory. The two methods namely "Casagrande logarithm of time fitting method" and "Taylor square root of time method" use these ratios effectively. In these methods, dial gauge readings which give compression of sample are plotted against logarithm of time or square root of time for various degree of consolidation. Thus

$$r_p = \frac{10}{9} \times \frac{(R_0 - R_{90})}{(R_i - R_f)} = \frac{R_0 - R_{100}}{R_i - R_f}$$

Where R_i = initial dial gauge reading
R_f = final dial gauge reading
R_0 = dial gauge reading at zero per cent consolidation
R_{90} = dial gauge reading at 90% consolidation
R_{100} = dial gauge reading at 100% consolidation

(ii)

Let α = Angle of crossing, N = Number of crossing, G = 1.676 m



$\because \quad \sin\alpha = \dfrac{FC}{DC}$

$\Rightarrow \quad \sin\alpha = \dfrac{G}{DC}$

$\Rightarrow \quad DC = \text{G cosec } \alpha$

$AB = \text{BC} = \text{CD} = \text{DA} = \text{G cosec } \alpha$

Also, $\quad \text{N} = \cot\alpha$

$\Rightarrow \quad \alpha = \cot^{-1}(12) = 4.764°$

(i) $\quad \text{AB} = \text{BC} = \text{CD} = \text{DA} = 1.676 \text{ cosec } (4.764°)$

$= 20.18 \text{ m}$

(ii) $\quad \text{EB} = \text{DF} = \text{G} \cot\alpha$

$= 1.676 \times 12 = 20.11 \text{ m}$

(iii)
$$BD = G\sec\frac{\alpha}{2}$$

$$= \frac{1.676}{\cos\frac{\alpha}{2}} = \frac{1.676}{\cos\left(\frac{4.764°}{2}\right)} = 1.677 \text{ m}$$

(iv)
$$AC = G \text{ cosec}\frac{\alpha}{2} = \frac{1.676}{\sin\left(\frac{4.764°}{2}\right)} = 40.33 \text{ m}$$

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