MADE EASY

Leading Institute for ESE, GATE & PSUs

Detailed Solutions

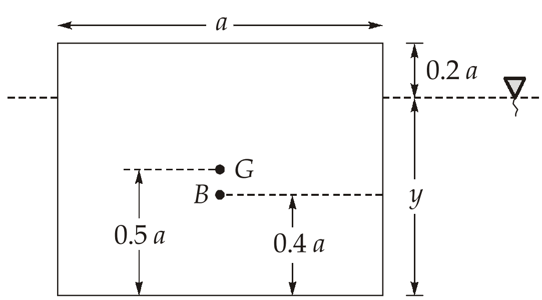
# ESE-2025 Mains Test Series

# Civil Engineering Test No : 8

## Section - A

**Q.1 (a) Solution:**

**(i)**




For equilibrium:

$$W_{cube} = \text{Weight of water displaced.}$$

$$\Rightarrow \quad 0.8 \times 10^3\, g \times a^3 = 10^3 \times g \times a^2\, y$$

$$\Rightarrow \quad y = 0.8a$$

and $\quad BG = 0.5\,a - 0.4\,a = 0.1\,a$

Now, meta centric height is given by

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

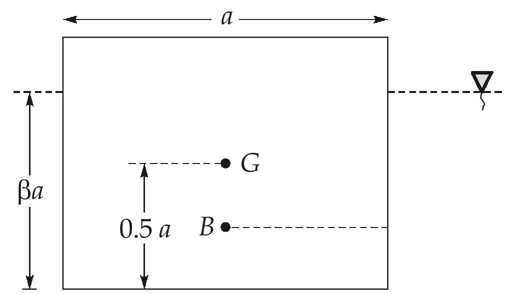
$$\Rightarrow \quad GM = \frac{a^4/12}{\left(0.8a \times a^2\right)} - 0.1\,a$$

$$\Rightarrow \quad GM = 4.167 \times 10^{-3}\,a > 0$$

$\because$ Meta centric height is positive so cube is in stable

**(ii)**

Let β is the specific gravity of cube for which cube will float




$$BG = 0.5a - \frac{\beta a}{2}$$

$$\Rightarrow \quad BG = \frac{a}{2}(1-\beta)$$

Now, meta centric height is given by,

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

$$\Rightarrow \quad GM = \frac{\left(a^4/12\right)}{(\beta a)\times a^2} - \frac{a}{2}(1-\beta)$$

$$\Rightarrow \quad GM = \frac{a}{12\beta} - \frac{a}{2}(1-\beta)$$

For cube to be stable

$$GM > 0$$

$$\Rightarrow \quad \frac{a}{12\beta} - \frac{a}{2}(1-\beta) > 0$$

$$\Rightarrow \quad 6\beta^2 - 6\beta + 1 > 0$$

$$\beta > 0.789 \quad \text{and} \quad \beta < 0.211$$

∴ Cube is stable for specific gravity range

$$\beta = (0, 0.211) \cup (0.789, 1.2)$$

**Q.1 (b) Solution:**

**(i)**

Given, $\phi\text{-index} = 3.9 \text{ cm/hr}$

$$\text{Runoff } R = [(9.5 - 3.9) + (6 - 3.9) + (4.5 - 3.9) + (8 - 3.9) + (5.5 - 3.9) + (5.5 - 3.9)] \times \frac{15}{60}$$

$$\Rightarrow \quad R = (5.6 + 2.1 + 0.60 + 4.1 + 1.60 + 1.60) \times \frac{1}{4}$$

$$\Rightarrow \quad R = 3.9 \text{ cm}$$

Total precipitation, $P = (3.3 + 3 + 9.5 + 6 + 4.5 + 8 + 2 + 5.5 + 5.5 + 2.5) \times \dfrac{15}{60}$

$$= \frac{49.8}{4} = 12.45 \text{ cm}$$

$$\text{W-index} = \frac{P-R}{t} = \frac{12.45-3.9}{2.5} = \frac{8.55}{2.50} = 3.42 \text{ cm/hr}$$

(ii)

**Advantages**

1. Simultaneous removal of gases and particulates.
2. Effective performance over a wide loading range.
3. Equipment occupies only a moderate amount of space compared to dry collector such as bag houses.
4. Hazards of explosive dust air mixtures are reduced.
5. Indifference to the temperature and moisture content of gas.
6. Corrosive gases may be neutralised by proper choice of scrubbing liquid.

**Disadvantages**

- Relatively high energy cost
- Problem of wet sludge disposal
- Corrosion problems.
- Visible wet plume, reduction in bouyany
- Very small particles (submicron sizes) may not be captured.

**Q.1 (c) Solution:**

Given: $A_1 = 1.8 \text{ m}^2; A_2 = 13.5\text{m}^2; V_1 = 12.5 \text{ m/s}$

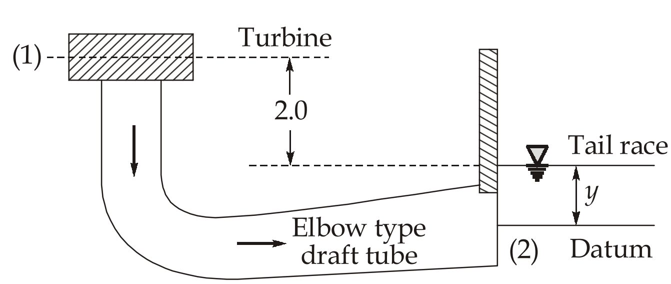
(i) Frictional loss in the draft tube,

$$H_L = 0.1\frac{V^2}{2g} = 0.1\times\frac{(12.5)^2}{2\times 9.81} = 0.7964 \text{ m}$$

$\because$ $Q = A_1V_1 = A_2V_2$

$\Rightarrow$ $1.8 \times 12.5 = 13.5\ V_2$

$\Rightarrow$ $V_2 = 1.667 \text{ m/s}$




**Draft tube set up**

Applying Bernoulli's equation between (1) and (2)

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

$$\Rightarrow \quad \frac{P_1}{\rho g}+\frac{(12.5)^2}{2\times 9.81}+(Z+y) = (0+y)+\frac{(1.667)^2}{2\times 9.81}+0+0.7964$$

$$\Rightarrow \quad \frac{P_1}{\rho g} = 0+\frac{(1.667)^2}{2\times 9.81}+0.7964-\frac{(12.5)^2}{2\times 9.81}-2$$

$$\Rightarrow \quad \frac{P_1}{\rho g} = -\,9.026 \text{ m (gauge)}$$ **Ans. (i)**

(ii) Power wasted to the tail race,

$$P_L = \rho g Q \times \frac{V_2^2}{2g}$$

$$\Rightarrow \quad P_L = 1000 \times 9.81 \times (1.8 \times 12.5) \times \frac{(1.667)^2}{2\times 9.81}$$

$$P_L = 31.26 \text{ kW}$$ **Ans. (ii)**

(iii) Efficiency of the draft tube

$$\eta_d = 1-\frac{H_L}{\left(\frac{V_1^2}{2g}-\frac{V_2^2}{2g}\right)}$$

$$\Rightarrow \quad \eta_d = 1-\frac{0.7964}{\frac{(12.5)^2}{2\times 9.81}-\frac{(1.667)^2}{2\times 9.81}} = 0.8982 \text{ or } 89.82\%$$ **Ans.(iii)**

**Q.1 (d) Solution:**

Given:

Flow rate $Q = 5 \text{ MLD} = 5 \times 10^6 \text{ lt/day}$

Recirculation Ratio, $R = 1.5$

Recirculation factor $F = \frac{1+R}{(1+0.1R)^2}$

$$\Rightarrow \quad F = \frac{1+1.5}{(1+0.1\times 1.5)^2} = 1.89$$

$$BOD_i = 260 \text{ mg/lt}$$

$\because$ 30% BOD removed in primary clarifier.

$$\text{Final effluent BOD desired} = 45 \text{ mg/lt}$$

$$\text{Total BOD in raw sewage} = 260 \times 5 \times 10^6 \times 10^{-6} \text{ kg/day}$$
$$= 1300 \text{ kg/day}$$

BOD left in the sewage entering per day in filter unit.

$$= (1 - 0.30) \times 1300 = 910 \text{ kg/day}$$

$$\text{Total BOD left in effluent per day} = 5 \times 10^6 \times 45 \times 10^{-6}$$
$$= 225 \text{ kg/day}$$

$\therefore$ Total BOD removed by the filter = 910 – 225 = 685 kg/day

Efficiency of filter

$$\eta\% = \frac{BOD \text{ removed}}{(BOD)} = \frac{685}{910}\times 100 = 75.274\%$$

$$\therefore \qquad \eta = 75.274 = \frac{100}{1+0.44\sqrt{\dfrac{910}{V\times 1.89}}}$$

$$\Rightarrow \qquad V = 863.907 \text{ m}^3$$

Assuming the depth of the filter as 1.2 m

$$\text{Surface area,} \qquad A = \frac{863.907}{1.2} = 719.92 \text{ m}^2$$

$$\Rightarrow \qquad \frac{\pi}{4}D^2 = 719.92$$

$$\Rightarrow \qquad D = 30.276 \text{ m} \simeq 31 \text{ m (say).}$$

Hence use a high rate trickling filter with 31 diameter and 1.2 m deep filter media with recirculation ratio 1.5.

**Q.1 (e) Solution:**

1. Length $L = 30$ m, $B = 14.0$ m, $H = 4.3$ m

   Flow $Q = 0.0796 \text{ m}^3/\text{sec.}$

   Soluble B.O.D. $Y_0 = 130 \text{ mg}/l$

   M.L.S.S. $= X_t = 2100 \text{ mg}/l$

   M.L.V.S.S. $= 1500 \text{ mg}/l$

   30 minute settled sludge volume = 230 ml/$l$

   Return sludge, $X_u = 9100 \text{ mg}/l$

**Aeration Period**

$$\text{Volume of tank} = 30 \times 14 \times 4.3 = 1806 \text{ m}^3$$

$$\text{Capacity of tank} = \text{Discharge} \times \text{Aeration period}$$

$$\Rightarrow \quad \text{Aeration Period} = \frac{1806}{0.0796} = 22688.44 \text{ sec} \simeq 6.3 \text{ hours}$$

2. $$\text{F/M Ratio} = \frac{QY_0}{\text{Volume} \times X_t} = \left[\frac{0.0796 \times 24 \times 3600 \times 130}{1806 \times 2100}\right]$$

$$= 0.2357 \text{ kg BOD per day/kg of MLSS.}$$

3. $$\text{Sludge volume index (SVI)} = \frac{\text{m}l\text{. of settled volume of sludge}}{\text{1gm. of dry weight of solid}}$$

Now 2100 mg settling sludge gives 230 ml. volume

$$\therefore \quad \text{1 gm, setting will give } \frac{230}{2.1} \text{ m}l\text{/gm} = 109.52 \text{ m}l\text{/gm}$$

$$\Rightarrow \quad SVI = 109.52 \text{ m}l\text{/gm}$$

4. $$\text{Return sludge percentage} = \frac{Q_R}{Q} = \frac{X_t}{X_u - X_t} = \frac{2100 \times 100}{9100 - 2100} = 30\%$$

**Q.2 (a) Solution:**

**(i)**

Given:

$$K_R = 0.4/\text{day}$$

$$\text{Velocity of river } V = 0.85 \text{ m/s}$$

$$DO_{\text{sat}} = (10 + 0.8) = 10.8 \text{ mg/}l$$

Time required to travel 48.3 km down stream is given by

$$t = \frac{\text{Distance down stream}}{\text{Velocity of flow in stream}}$$

$$\Rightarrow \quad t = \frac{48.3 \times 10^3}{0.85} = 56823.53 \text{ sec}$$

$$\Rightarrow \quad t = \frac{56823.53}{86400} = 0.6577 \text{ day}$$

Dissolved oxygen deficit after time $t$ is given by

$$D_t = \frac{K_D L}{K_R - K_D}\left[10^{-K_D t} - 10^{-K_R t}\right] + D_o \times 10^{-K_R t}$$

$$\Rightarrow \qquad D_t = \frac{0.2 \times 20}{0.4 - 0.2}\left(10^{-0.2 \times 0.6577} - 10^{-0.4 \times 0.6577}\right) + \left(0.8 \times 10^{-0.4 \times 0.6577}\right)$$

$$\Rightarrow \qquad D_t = 20 \times 0.193 + 0.44$$

$$\Rightarrow \qquad D_t = 4.3 \text{ mg}/l$$

$$\text{DO at 48.3 km down stream} = (10 + 0.8 - 4.3) \text{ mg}/l$$

$$= 6.5 \text{ mg}/l$$

**(ii)**

Sewage sickness occurs when sewage is continuously applied to agricultural land, causing soil pores to become clogged over time. This blocks air circulation, stops further absorption of sewage, and leads to anaerobic conditions with foul odors. Such a condition is known as sewage sickness.

**Methods to prevent sewage sickness:**

**Primary treatment:** Provide primary treatment to sewage to remove suspended solids.

**Intermittent sewage supply:** Apply sewage intermittently rather than continuously, based on soil nature.

**Crop rotation:** Practice crop rotation to allow different crops to absorb different nutrients.

**Sub-soil drainage:** Install sub-soil drainage systems to remove excess subsoil effluent.

**Deep ploughing:** Use deep ploughing by tractors to increase soil's absorption capacity.

**Surface soil management:** Periodically remove a thin layer of surface soil by scraping.

**(iii)**

The self-purification capacity of rivers depends on the following factors:

**Temperature:**

Higher temperatures reduce dissolved oxygen (D.O.) and increase the rate of chemical reactions, potentially leading to anaerobic conditions.

**Turbulence:**

Increased turbulence improves oxygen mixing in water, enhancing purification.

**Quantity and nature of organic matter:**

Some organic substances are easily decomposed, while others take longer time. The purification rate depends on the type and amount of organic matter present.

**Hydrography of the river stream:**

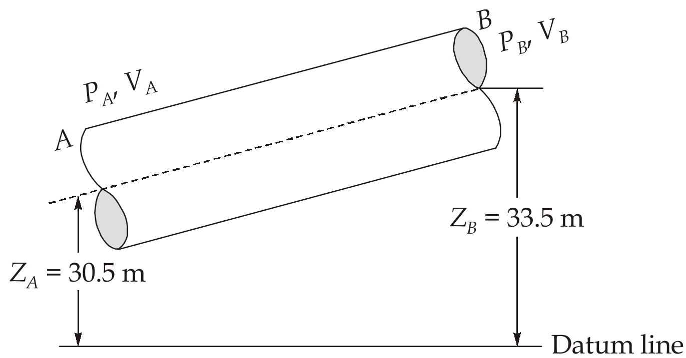
Rivers with higher flow velocity and larger cross-sections exhibit better turbulence and dilution of pollutants, supporting self-purification.

**Rate of re-aeration:**

The higher the re-aeration rate, the quicker the self-purification process occurs.

**Q.2 (b) Solution:**

**(i)**




Given, diameter of pipe, $D = 0.30$ m; velocity of water, $V = 24.4$ m/sec.

At point A, $P_A = 361 \text{ kN/m}^2$

$Z_A = 30.5$ m

$V_A = V = 24.4$ m/sec

$\therefore$ Total energy at $A$, $E_A = \dfrac{P_A}{\rho g} + \dfrac{V_A^2}{2g} + Z_A$

$$\Rightarrow \quad E_A = \frac{361 \times 10^3}{1000 \times 9.81} + \frac{(24.4)^2}{2g} + 30.50$$

$$\Rightarrow \quad E_A = 36.80 + 30.34 + 30.50 = 97.64 \text{ m}$$

At point $B$, $P_B = 288 \text{ kN/m}^2$

$Z_B = 33.5$ m

$V_B = V = 24.4$ m/s

$\therefore$ Total energy at $B$ = $E_B = \dfrac{P_B}{\rho g} + \dfrac{V_B^2}{2g} + Z_B$

$$\Rightarrow \quad E_B = \frac{288 \times 10^3}{1000 \times 9.81} + \frac{(24.4)^2}{2g} + 33.50$$

$$\Rightarrow \quad E_B = 29.36 + 30.34 + 33.50 = 93.20 \text{ m}$$

$\therefore$ Loss of head $= E_A - E_B$

$= (97.64 - 93.20) = 4.44$ m

$\because$ $E_A > E_B$, and thus flow is from point $A$ to $B$ in the pipe.

**(ii)**

Given horizontal rectangular channel of width $B = 4$ m

Discharge, $Q = 16 \text{ m}^3/\text{sec}$,

Initial depth, $y_1 = 0.50$ m

Whether jump occurs at depth $y_1$?

Froude's number, $$F_{r1} = \frac{V_1}{\sqrt{gy_1}}$$

$$V_1 = \frac{Q}{By_1} = \frac{16}{4 \times 0.50} = 8 \text{ m/s}$$

$$\therefore \quad F_{r1} = \frac{8.0}{\sqrt{9.81 \times 0.50}} = 3.61 > 1 \quad \text{(Super-critical flow)}$$

Hence, hydraulic jump forms,

Calculation of sequent depth:

$$\frac{y_2}{y_1} = \frac{1}{2}\left[-1 + \sqrt{1 + 8F_{r_1}^2}\right]$$

$$\Rightarrow \quad \frac{y_2}{y_1} = \frac{1}{2}\left[-1 + \sqrt{1 + 8(3.61)^2}\right]$$

$$\Rightarrow \quad y_2 = 2.315 \text{ m}$$

Energy loss in the jump,

$$\Delta E = \frac{(y_2 - y_1)^3}{4y_1y_2} = \frac{(2.315 - 0.50)^3}{4 \times 2.315 \times 0.50} = 1.291 \text{ m}$$

**Q.2 (c) Solution:**

(i)

$$\text{Demand of a particular time interval} = \left(\frac{Q_1 + Q_2}{2}\right) \times \text{time} = \left(\frac{Q_1 + Q_2}{2}\right) \times 3600 \times 2 \times 10^3$$

$$= 3.6\,(Q_1 + Q_2) \text{ million litres}$$

$$\text{Rate of pumping} = \frac{21.24}{24} \times 2 = 1.77 \text{ million litres per two hours}$$

| Time (hrs) | Demand (× $10^6$ litres) | Accumulated demand (× $10^6$ litres) | Accumulated Supply (× $10^6$ litres) | (Accumulated demand – Accumulated supply) (× $10^6$ litres) |
|---|---|---|---|---|
| 0 - 2 | 0.36 | 0.36 | 1.77 | – 1.41 |
| 2 - 4 | 0.9 | 1.26 | 3.54 | – 2.28 |
| 4 - 6 | 1.26 | 2.52 | 5.31 | – 2.79 (A) |
| 6 - 8 | 2.52 | 5.04 | 7.08 | – 2.04 |
| 8 - 10 | 3.96 | 9 | 8.85 | 0.15 |
| 10 - 12 | 3.6 | 12.6 | 10.62 | 1.98 |
| 12 - 14 | 2.52 | 15.12 | 12.39 | 2.73 (B) |
| 14 - 16 | 1.62 | 16.74 | 14.16 | 2.58 |
| 16 - 18 | 1.26 | 18 | 15.93 | 2.07 |
| 18 - 20 | 1.62 | 19.62 | 17.7 | 1.92 |
| 20 - 22 | 1.26 | 20.88 | 19.47 | 1.41 |
| 22 - 24 | 0.36 | 21.24 | 21.24 | 0 |

Balancing capacity of reservoir = $A + B$ = 2.79 + 2.73 = 5.52 million litres

**(ii)**

**Self cleansing velocity, and its importance:** The self cleansing velocity may be defined as the velocity at which the solid particles will remain in suspension, without settling at the bottom of sewer. Also it is that velocity at which even the scouring of the deposited particles of a given size will take place. It is not possible to maintain this self cleansing velocity throughout the day because of large fluctuations in sewage flow. During minimum flow of sewage, the velocity of flow is less than self cleansing velocity. Self-cleansing velocity must be maintained atleast once a day. The self cleansing velocity is given by,

$$V_S = \sqrt{\frac{8\beta}{f}(G_S - 1)gd_s}$$

where, $\beta$ = Characteristics of solids flowing in sewage in suspension

$f$ = Darcy Weisbach friction factor

$G_S$ = Specific gravity

$d_S$ = Diameter of solid particles

**Non-scouring velocity and its importance:** Though the minimum velocity in sewage flow should be equal to the self cleansing velocity so that particles do not settle and stick to the sewer invert, there is also some upper limit of velocity of flow so that interior surface of the sewer is not damaged due to wear. At higher velocity, the flow becomes turbulent, resulting in continuous abrasion of the interior surface of the sewer, by the suspended particles. Hence, maximum velocity of flow is also limited. The maximum velocity at which no such scouring action or abrasion takes place is non-scouring velocity.

Evidently such a velocity depends upon the material used for the construction of sewers.

Of the ceramic materials used in sewers, vitrified tiles and glazed bricks are more resistant to wear while burnt clay bricks and concrete are less resistant to wear.

For sewers is flat country, the design of sewers should be done in such a way that self cleansing velocity is obtained at maximum discharge.

**Q.3 (a) Solution:**

**(i)**

Given:

Design discharge, $Q = 45\text{m}^3/\text{s}$

Bed slope, $S = \dfrac{1}{5000} = 2\times 10^{-4}$

Manning's roughness coefficient,

$N = 0.0225$

Critical velocity ratio, $m = 1.05$

Initial depth, $D = 2.2\text{ m}$

Critical velocity, $V = 0.55\text{ m }D^{0.64} = 0.55 \times 1.05 \times 2.2^{0.64} = 0.957\text{ m/s}$

Area of flow in channel, $A = \dfrac{Q}{V} = \dfrac{45}{0.957} = 47.02\text{ m}^2$

Taking channel width as '$B$' and side slope as 1 V : 0.5 H

Area of flow, $A = (B + 0.5\,D)D = 47.02\text{ m}^2$

$\Rightarrow \quad (B + 0.5 \times 2.2) \times 2.2 = 47.02$

$\Rightarrow \quad B = 20.27\text{ m}$

Wetted perimeter, $P = B + 2D\sqrt{1+m^2} = 20.27 + 2\times 2.2\sqrt{1+0.5^2} = 25.2\text{ m}$

$\therefore$ Hydraulic radius, $R = \dfrac{A}{P} = \dfrac{47.02}{25.2} = 1.866\text{ m}$

Chezy's constant,
$$C = \frac{23 + \dfrac{1}{N} + \dfrac{0.00155}{S}}{1 + \left[23 + \dfrac{0.00155}{S}\right]\times \dfrac{N}{\sqrt{R}}}$$

$$\Rightarrow \qquad C = \frac{23 + \dfrac{1}{0.0225} + \dfrac{0.00155}{1} \times 5000}{1 + \left[23 + \dfrac{0.00155}{1} \times 5000\right] \times \dfrac{0.0225}{\sqrt{1.866}}} = 49.91$$

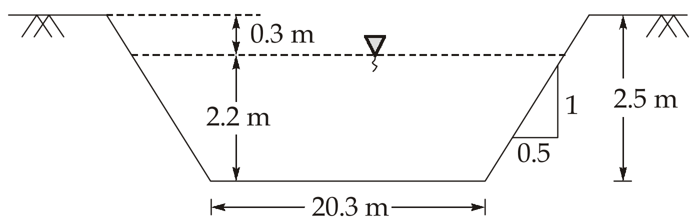
$$\therefore \qquad \text{Velocity, } V = C\sqrt{RS} = 49.91 \times \sqrt{1.866 \times 2 \times 10^{-4}}$$

$$= 0.964 \text{ m/s which is close to } 0.957 \text{ m/s}$$

Allowing for 300 mm as free board, overall depth of channel = 2.2 + 0.3 = 2.5 m

Thus channel width, $B$ = 20.27 m $\simeq$ 20.3 m

and depth of flow, $D$ = 2.2 m with channel side-slope of 1 V : 0.5 H and 300 mm free board




**Cross-section of canal**

**(ii)**

Given: 2-hour isolated rainfall and area of catchment = 445 km$^2$

Peak of flood hydrograph = 220 m$^3$/s

Base flow = 15 m$^3$/s

$\therefore$ Peak of direct runoff hydrograph = Peak of flood hydrograph – Base flow

= 220 – 15 = 205 m$^3$/s

Total depth of rainfall = 47 mm

Infiltration rate = 2.5 mm/hr

$\therefore$ Effective rainfall = Total rainfall – Infiltration rate × $t$

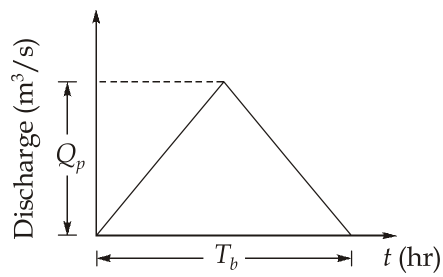
= 47 – 2.5 × 2 = 42 mm = 4.2 cm

$$\therefore \quad \text{Peak of unit hydrograph} = \frac{\text{Direct runoff}}{\text{Effective rainfall}} = \frac{205}{4.2} = 48.81 \text{ m}^3/\text{s}$$

Now, the unit hydrograph is triangular in shape.

Let, $T_b$ be the base period of UH (in hr)

$Q_p$ i.e. peak of unit hydrograph = 48.81 m$^3$/sec (as calculated above)




$$\text{Volume of runoff} = \text{Area under UH}$$

$$\Rightarrow \quad \text{Area of catchment} \times \text{ER (1 cm)} = \frac{1}{2} \times Q_P \left(\text{in m}^3/\text{sec}\right) \times T_b \left(\text{in hr}\right) \times 3600$$

$$\Rightarrow \quad 445 \times 10^6 \times \text{m}^2 \times 1 \times 10^{-2}\ \text{m} = \frac{1}{2} \times 48.81 \frac{\text{m}^3}{\text{sec}} \times T_b \left(\text{hr}\right) \times 3600$$

$$\Rightarrow \quad T_b = 50.65 \text{ hr}$$

$\therefore$ Base period of the unit hydrograph = 50.65 hr

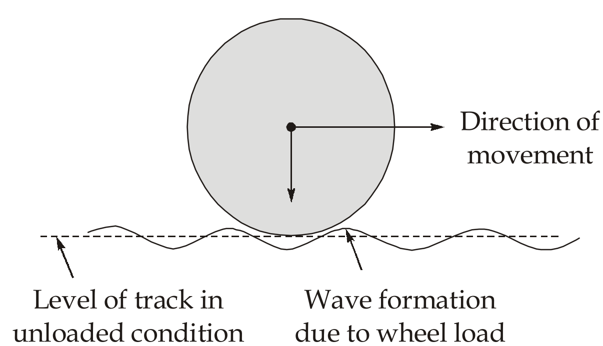
**Q.3 (b) Solution:**

**(i)**

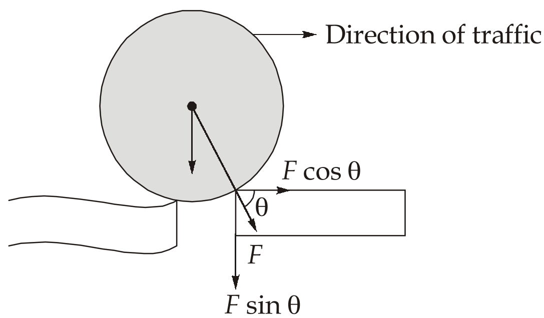
Creep is defined as longitudinal movement of the rail with respect to sleepers, due to movement of traffic over a period of time.

**Causes of creep:**

1. **Wave motion theory:** Due to wheel load, a wave in rails is created in the direction of traffic thereby resulting in creep.



2. **Percussion theory:** Due to impact of rail wheel on the joints, creep is observed. The horizontal component of reaction (i.e. $F\cos\theta$) causes creep and the vertical component of reaction (i.e. $F\sin\theta$) results in battering of rail end.




3. **Dragging Theory:** Due to frictional force between wheel and rail a drag force is generated opposite to movement of train, resulting in creep.
4. Due to braking action and acceleration of trains (skidding + slipping).

**Following are some of the avoidable causes of creep:**

- Rails are not properly fastened with sleepers.
- Loose packing of ballast around sleepers.
- Improper expansion gaps.
- Too light rail for heavy traffic
- Sharp gradients and sharp curves.
- Uneven spacing of sleepers etc.

**Preventive and remedial measures to avoid creep:**

- Rails should be firmly held to sleepers.
- Bearing load fastenings should always be slightly more than the ballast resistance.
- Ballast should be properly packed especially around sleepers and shoulders.
- Use of creep anchors at adequate interval.
- When creep becomes excessive (> 150 mm) then it causes maintenance problems. The same should be adjusted by pulling back manually or mechanically. During pulling back operation, survey of expansion joint, gaps and rail to rail joint with sleepers should be carried out properly.

**(ii)**

Given:

Field capacity, $F = 25\%$

$\Rightarrow$ Permanant wilting point, $\phi = 10\%$

Dry unit weight of soil, $\rho_s = 1.5 \dfrac{\text{gm}}{\text{cm}^3}$

Depth of root zone, $d$ = 60 cm

Moisture storage capacity of the soil in the root zone depth is given by

$$= \frac{\gamma_s}{\gamma_w} \times d[F - \phi] = \frac{\rho_s}{\rho_w} \times d[F - \phi] = \frac{1.5}{1} \times 60 \times \left[\frac{25}{100} - \frac{10}{100}\right] = 13.5 \text{ cm}$$

Now, when the moisture content falls to 15%, the deficiency of water depth created

$$= \frac{\rho_s}{\rho_w} \times d \times [F - \text{Fall in moisture content}] = \frac{1.5}{1} \times 60 \times \left[\frac{25}{100} - \frac{15}{100}\right] = 9 \text{ cm}$$

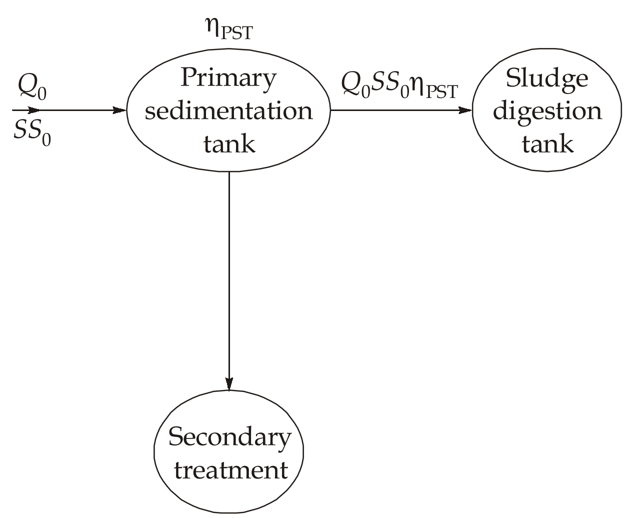
Hence, 9 cm depth of water is the net irrigation requirement.

$$\therefore \quad \text{Water application frequency} = \frac{\text{Net irrigation requirement}}{\text{Water depth required in the field}}$$

$$\Rightarrow \quad \frac{75}{100} = \frac{9}{\text{Water depth required in the field}}$$

$$\Rightarrow \quad \text{Water depth required in the field} = \frac{9 \times 100}{75} = 12 \text{ cm}$$

**Q.3 (c) Solution:**




**Volume of raw sludge:**

Weight of total solids coming to digestion tank

$$= Q_0 SS_0 \, \eta_{PST}$$

$$= 15 \times 10^6 \times 215 \times 10^{-6} \times 0.6$$

$$= 1935 \text{kg}$$

Weight of nonvolatile solids = 600kg (given)

Weight of volatile solids = 1935 – 600 = 1335 kg

$$V_{NVS} = \frac{W_{NVS}}{G_{NVS} \times \rho_w} = \frac{600}{2.45 \times 1000} = 0.2449 \text{ m}^3$$

$$V_{VS} = \frac{W_{VS}}{G_{VS} \times \rho_w} = \frac{1335}{1.03 \times 1000} = 1.2961 \text{ m}^3$$

∴ Total volume of solids in raw sludge = $V_{NVS} + V_{VS}$ = 0.2449 + 1.2961 = 1.541 m$^3$

Moisture content of raw sludge = 90% (given)

∴ Volume of water in raw sludge $= \dfrac{W_w}{\rho_w} = \dfrac{\frac{90}{10}(W_{\text{solid}})}{\rho_w} = \dfrac{\frac{90}{10}(1937)}{1000} = 17.433 \text{ m}^3$

∴ Total volume of raw sludge = 17.433 + 1.541

$V_1$ = 18.974 m$^3$

**Volume of digested sludge:**

Given that 30% of volatile solids are getting digested and 100% of nonvolatile solids will get digested.

∴ Weight of volatile solids (digested) = 1335 × 0.30 = 400.5 kg

∴ Weight of nonvolatile solids (digested) = 600kg

∴ Volume of solid digested sludge

$$= V_{VS} + V_{NVS}$$

$$= \frac{W_{VS}}{G_{VS}\rho_w} + \frac{W_{NVS}}{G_{NVS}\rho_w}$$

$$= \frac{400.5}{1.03 \times 100} + \frac{600}{2.45 \times 1000} = 0.6337 \text{ m}^3$$

∵ Moisture content of digested sludge = 80%

∴ Solids content of digested sludge = 20%

∴ Volume of water in digested sludge

$$\frac{W_w}{\rho_w} = \frac{\frac{80}{20}(400.5 + 600)}{1000} = 4.002 \text{ m}^3$$

∴ Volume of digested sludge = 0.6337 + 4.002

= 4.6357 m$^3$

∴ Volume of digestion tank, $V = \left[V_1 - \frac{2}{3}(V_1 - V_2)\right]t_d$

$$= \left[18.974 - \frac{2}{3}(18.974 - 4.6357)\right] \times 30 = 282.454 \text{ m}^3$$

We know, $$V = \frac{\pi}{4} D^2 H$$

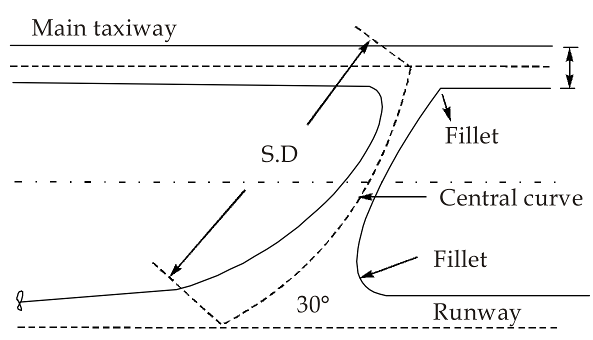
$$\Rightarrow \quad 282.454 = \frac{\pi}{4} \times D^2 \times 6$$

$$\Rightarrow \quad D = 7.74 \text{ m}$$

**4. (a) Solution:**

**(i)**

- **High speed exit taxiway:** The function of exit taxiway is to minimise runway occupancy time of landed aircraft. The average runway occupancy time of landed aircraft frequently determines the capacity of the runway system and airport as a whole. When the angle of turn off is of the order of 30 degrees, the term high speed exit taxiway or 'rapid exit taxiway' is often used to denote its design for higher turnoff speeds of aircraft. Typical exit taxiway configuration is shown below.



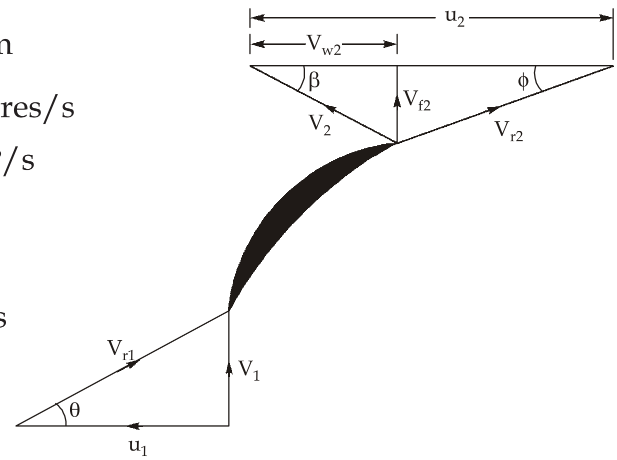

Plan of 30° exit taxiway

Location of exit taxiway depends upon the following factors:

- **Aircraft population:** Large and small aircrafts have different initial exit speeds.
- **Landing characteristics of aircraft**: This refers to approach and touch down speeds of landed aircraft. The touch down distance from runway threshold and runway occupancy time are determined by approach speed and touch down speed.

  Rate of deceleration which in turn depends upon pavement surface condition i.e., dry or wet pavement. ICAO recommended a deceleration rate of 1.25 m/sec². This average rate is comfortable to passengers and pilot when braking on wet pavement surface.
- **The number of exits:** A large airport may have as many as three angled exit taxiway direction, in addition to several 90° exit taxiways.

**(ii)**




$$\text{Diameter at outlet, } D_2 = 1.2 \text{ m}$$
$$\text{Speed, } N = 200 \text{ rpm}$$
$$\text{Discharge, } Q = 1880 \text{ litres/s}$$
$$= 1.88 \text{ m}^3\text{/s}$$
$$\text{Manometric head, } H_m = 6 \text{ m}$$
$$\text{Angle of vane at outlet, } \phi = 26°$$
$$\text{Velocity of flow at outlet, } V_{f2} = 2.5 \text{ m/s}$$
$$\text{Diameter at inlet, } D_1 = 0.6 \text{ m}$$

1. Manometric efficiency

$$\eta_{\text{man}} = \frac{gH_m}{V_{w2}u_2}$$

But $$u_2 = \frac{\pi D_2 N}{60} = \frac{\pi \times 1.2 \times 200}{60} = 12.57 \text{ m/s}$$

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

$\Rightarrow$ $$u_2 - V_{w2} = \frac{V_{f2}}{\tan\phi} = \frac{2.5}{\tan 26°} = 5.13 \text{ m/s}$$

$\Rightarrow$ $$V_{w2} = u_2 - 5.13 = 12.57 - 5.13 = 7.44 \text{ m/s}$$

Substituting these values in equation (i), we get,

$$\eta_{\text{man}} = \frac{9.81 \times 6.0}{7.44 \times 12.57} = 0.6294 \simeq 0.63 = 63\%$$

2. Least speed to start the pump:

Least speed to start the pump is given by equation,

$$\frac{u_2^2}{2g} - \frac{u_1^2}{2g} = H_m$$

where $u_2$ and $u_1$ are the tangential velocities of the vane at outlet and inlet respectively, corresponding to least speed of the pump.

But $$u_2 = \omega r_2 \text{ and } u_1 = \omega r_1$$

Substituting these values in above equation we get,

$$\frac{(\omega r_2)^2}{2g} - \frac{(\omega r_1)^2}{2g} = H_m = 6.0$$

$$\Rightarrow \quad \frac{\omega^2}{2g}\left[r_2^2 - r_1^2\right] = 6.0$$

But $\quad r_2 = \frac{D_2}{2} = \frac{1.2}{2} = 0.6\,\text{m}$ and $r_1 = \frac{D_1}{2} = \frac{0.6}{2} = 0.3\,\text{m}$

$$\therefore \quad \omega^2 = \frac{6.0 \times 2.0 \times 9.81}{0.36 - 0.09} = 436$$

$$\Rightarrow \quad \omega = \sqrt{436} = 20.88 = \frac{2\pi N}{60}$$

$$\therefore \quad N = \frac{60 \times 20.88}{2\pi} = 199.4$$

**Q.4 (b) Solution:**

**(i)**

$$\frac{\text{Gross air}}{\text{Cloth}} = \frac{\text{Gas flow rate}}{\text{Total fabric area}}$$

Given: Gas flow rate = 1,200,000 $m^3$/min

1. To calculate the total fabric area, calculate bag area using equation for area of cylinder.

$$\text{Area of bag, } A = \pi dh$$
$$= \pi \times 11 \times 30 = 1036.7\ \text{m}^2/\text{bag}$$
$$\text{Total number of bags} = 360 \times 20 = 7200$$
$$\therefore \quad \text{Total fabric area} = 7200 \times 1036.7$$
$$= 7.46 \times 10^6\ \text{m}^2$$

$$\therefore \quad \frac{\text{Air}}{\text{Cloth}}(\text{Gross}) = \frac{1200000\ \text{m}^3/\text{min}}{7.46\times 10^6\ \text{m}^2} = 0.161\ \text{m/min}$$

2. The net air to cloth ratio is calculated by subtracting out the compartments which are not in filtering service.

Total number of bags in service = 360 × 18 = 6480 bags

$$\therefore \quad \text{Total fabric area} = 6480 \times 1036.7 = 6.718 \times 10^6\ \text{m}^2$$

$$\therefore \quad \frac{\text{Air}}{\text{Cloth}}(\text{Net}) = \frac{1200000}{6.718 \times 10^6} = 0.179\ \text{m/min}$$

**(ii)**

1. **Sludge density index**: Sludge density index is used in a way similar to the sludge volume index to indicate the settleability of a sludge in a secondary clarifier or effluent. It is expressed as the weight in grams of one milliliter of sludge after settling for 30 minutes and is calculated as;

$$\text{SDI} = \frac{100}{SVI}$$

where, SDI = Sludge density index, (g/m*l*) SVI = Sludge volume index, (m*l*/g)

A sludge with good settling characteristics has SDI between 1.0 and 20. Whereas an SDI of 0.50 indicates a bulky or non settleable sludge.

2.
$$L_{\text{eq}} = 10\log_{10}\left(\frac{1}{T}\sum_{i=1}^{n}(10)^{L_i/10}\times t_i\right)$$

where $T = 1$ hour

$$L_{eq} = 10\log_{10}\left(\frac{1}{T}\left(10^{0.1L_1}\times\Delta t_1 + 10^{0.1L_2}\times\Delta t_2\right)\right)$$

where $\Delta_{t1} = 5$ min and $L_1 = 75$ dB

$\Delta_{t2} = 55$ min and $L_2 = 55$ dB

$$\therefore \quad L_{eq} = 10\log_{10}\left(\frac{1}{60}\left(10^{7.5}\times 5 + 10^{5.5}\times 55\right)\right) = 64.66 \text{ dB}$$

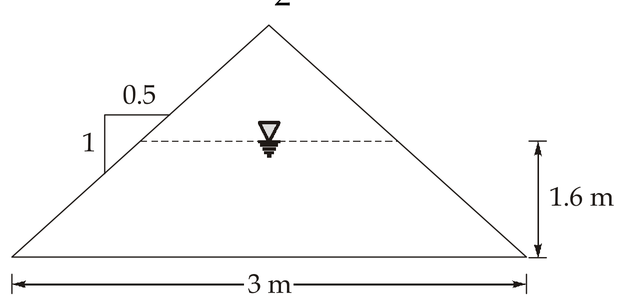
**Q.4 (c) Solution:**

**(i)**

Here, $m = -0.5$

Top width, $T_c = 3.0 - (2 \times 0.5 \times 1.6) = 1.40$ m

Flow area, $A_c = \dfrac{(3.0+1.4)}{2}\times 1.60 = 3.52 \text{ m}^2$



At critical flow, $\dfrac{Q^2}{g} = \dfrac{A_c^3}{T_c} = \dfrac{(3.52)^3}{1.40} = 31.153 \text{ m}^5$

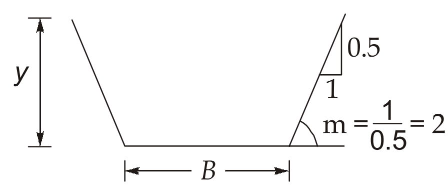
$\therefore$ Discharge, $Q = 17.48 \text{ m}^3/\text{s}$

$$\therefore \quad V_c = \frac{Q}{A_c} = \frac{17.48}{3.52} = 4.966 \text{ m/s}$$

$$\therefore \quad \frac{V_c^2}{2g} = \frac{(4.966)^2}{2\times 9.81} = 1.257 \text{ m}$$

$$\therefore \quad E_c = y_c + \frac{V_c^2}{2g} = 1.60 + 1.257 = 2.857 \text{ m}$$

**(ii)**




$B = 3$ m

$y = 1.2$ m (slope 0.5 V : 1H)

Side slope $= 1 \text{ H} : \dfrac{1}{2} \text{ V}$

$$P = B + 2y\sqrt{1+m^2} = B + 2y\sqrt{1+2^2}$$

$$\Rightarrow \quad P = B + 2\sqrt{5}y = 3 + 2\sqrt{5}\times 1.2 = 8.367 \text{ m}$$

$$A = (B + my)y = (3 + 2 \times 1.2) \times 1.2 = 6.48 \text{ m}^2$$

$S = \tan\theta$ (as $\theta$ is very small and thus $\sin\theta \simeq \tan\theta = S$)

$$\tau = \text{Shear stress} = \frac{(\gamma_w AL)\sin\theta}{\text{Wetted area}}$$

$$\Rightarrow \quad \tau = \frac{(\gamma_w AL)S}{PL} \Rightarrow \tau = \gamma_w RS$$

$n = 0.012$

For rigid boundary channel, $n = \dfrac{1}{24} d^{1/6}$ ($d$ in m) (Stickler's formula)

$$\Rightarrow \quad 0.012 = \frac{1}{24} d^{1/6}$$

$$\Rightarrow \quad d = 5.71 \times 10^{-4} \text{ m}$$

But $d \geq 10.8$ RS

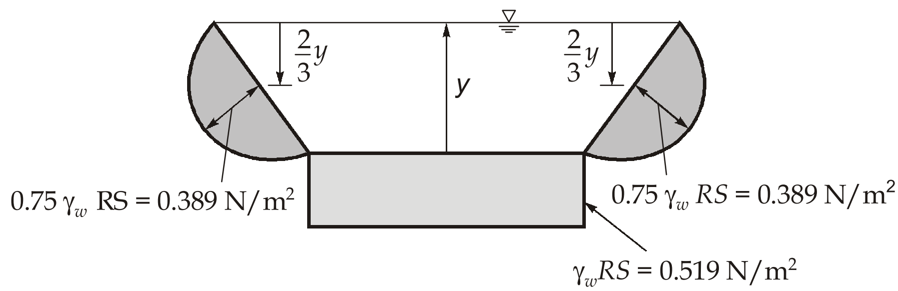
$$\Rightarrow \qquad 5.71 \times 10^{-4} = 10.8 \times \left(\frac{6.48}{8.367}\right) \times S$$

$$\Rightarrow \qquad S = 6.83 \times 10^{-5}$$

$$\tau_{\text{required at bed}} = \gamma_w RS = 9.81 \times 1000 \times \left[\frac{6.48}{8.367}\right] \times 6.83 \times 10^{-5}$$

$$\Rightarrow \qquad \tau = 0.519 \text{ N/m}^2$$

$$\tau_{av \text{ on sides}} = 0.75\, \gamma_w RS = 0.75 \times 0.519 = 0.389 \text{ N/m}^2$$




- Shear stress required to move a grain on side slopes is less than that shear stress required to move grain on canal bed.
- Shear stress will be zero at junction of bed and sides of section because it is a sharp joint

## Section - B

**Q.5 (a) Solution:**

Let '$t$' be the time in which half of ultimate settlement takes place.

**Given:** $H = 6$ m; $C_v = 4.92 \times 10^{-2}$ mm$^2$/sec

**Layer - 1:** $H_1 = 1.5$ m

**Layer - 2:** $H_2 = 4.5$m

Let $U_1$ and $U_2$ be the degree of consolidation of layer 1 and 2 respectively.

Now, $\Delta h_1 + \Delta h_2 = \Delta h$

$$\Rightarrow \qquad \frac{\Delta h_1}{H} + \frac{\Delta h_2}{H} = \frac{\Delta h}{H}$$

$$\frac{U_1 \times H_1}{H} + \frac{U_2 \times H_2}{H} = U$$

At time '$t$', $U = 0.5$

$$\therefore \qquad \frac{U_1 \times 1.5}{6} + \frac{U_2 \times 4.5}{6} = 0.5$$

$$\Rightarrow \qquad U_1 + 3\,U_2 = 2 \qquad \text{...(i)}$$

Drainage path length of layer – 1, $d_1 = \frac{H_1}{2}$

Drainage path length of layer – 2, $d_2 = \frac{H_2}{2}$

$$T_{V1} = \frac{C_v \cdot t}{d_1^2}$$

$$\Rightarrow \quad \frac{\pi}{4} \times U_1^2 = \frac{C_v \cdot t}{d_1^2}$$

$$\Rightarrow \quad t = \frac{\pi}{4} \times U_1^2 \times \left(\frac{H_1}{2}\right)^2 \times \frac{1}{C_v} = \frac{\pi}{16} \times \frac{U_1^2 \times H_1^2}{C_v}$$

Similarly,

$$t = \frac{\pi}{16} \times \frac{U_2^2 \times H_2^2}{C_v}$$

Equating above two equations,

$$\frac{\pi}{16} \times \frac{U_2^2 \times H_2^2}{C_v} = \frac{\pi}{16} \times \frac{U_1^2 \times H_1^2}{C_v}$$

$$\Rightarrow \quad \frac{U_1}{U_2} = \frac{H_2}{H_1} = \frac{4.5}{1.5} = 3$$

$$\therefore \quad U_1 = 3\,U_2 \qquad \text{...(ii)}$$

From equation (i) and (ii),

$$3\,U_2 + 3\,U_2 = 2$$

$$\Rightarrow \quad U_2 = \frac{1}{3}$$

$$\therefore \quad t = \frac{\pi}{16} \times \frac{(1/3)^2 \times (4.5 \times 10^3)^2}{4.92 \times 10^{-2}} \text{ sec.}$$

$$\Rightarrow \quad t = 103.928 \text{ days} \simeq 104 \text{ days}$$

$\therefore$ It would take approximately 104 days to attain half its ultimate settlement.

**Q.5 (b) Solution:**

As the soil is compressible, the reduced shear strength parameters and bearing capacity factors corresponding to local shear condition are used.

$$C_1 = \frac{2}{3}C'_{cu} = \frac{2}{3}\times 30 = 20 \text{ kN/m}^2$$

$$\tan\phi = \frac{2}{3}\tan\phi'_{cu}$$

$$\Rightarrow \quad \phi_1 = \tan^{-1}\left(\frac{2}{3}\tan\phi'_{cu}\right) = \tan^{-1}\left(\frac{2}{3}\tan 25°\right) = 17.3°$$

For $\phi_1 = 17.3°$, the bearing capacity factors are given as;

$$N'_C = 13.91$$

$$N'_\gamma = 4.02$$

$$N'_q = 5.17$$

$$\therefore \quad q_s = \frac{1}{F}\left[C_1 N'_C + q\left(N'_q - 1\right) + 0.5\gamma B N'_\gamma\right] + q$$

$$\Rightarrow \quad q_s = \frac{1}{3}\,[20 \times 13.91 + 18.3 \times 1.5(5.17 - 1) + 0.5 \times 18.3 \times 1 \times 4.02] + 18.3 \times 1.5$$

$$\Rightarrow \quad q_s = \frac{1}{3}[429.45] + 27.45$$

$$q_s = (143.15 + 27.45) = 170.6 \text{ kN/m}^2$$

$$\therefore \quad Q_s = q_s \times B = 170.6 \times 1 = 170.6 \text{ kN/m}$$

$\therefore$ The safe load that can be carried by the wall = 170.6 kN/m

**Q.5 (c) Solution:**

Assuming the pavement to consist of single layer of base course material only, the pavement thickness is given by:

$$T_b = \left\{\sqrt{\left(\frac{3PXY}{2\pi E_s \Delta}\right)^2 - a^2}\right\}\left(\frac{E_s}{E_b}\right)^{1/3}$$

$$= \left\{\sqrt{\left(\frac{3\times 4000\times 1.6}{2\pi\times 100\times 0.25}\right)^2 - 16^2}\right\}\left(\frac{100}{400}\right)^{1/3}$$

$$= 121.179 \times 0.6299 = 76.338 \text{ cm}$$

Now, let 7.5 cm bituminous concrete surface with $E_c$ = 1000 kg/cm$^2$ be equivalent to the thickness $t_b$ of base course.

$$\frac{t_b}{t_c} = \left(\frac{E_c}{E_b}\right)^{1/3}$$

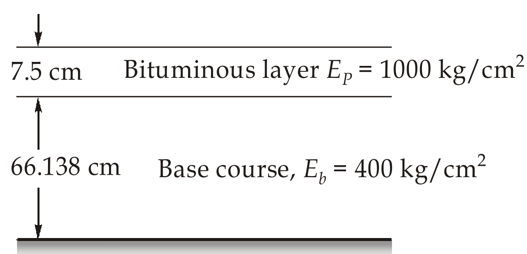
$$\Rightarrow \quad \frac{t_b}{7.5} = \left(\frac{1000}{400}\right)^{1/3}$$

$$\Rightarrow \quad t_b = 7.5 \times \left(\frac{1000}{400}\right)^{1/3} = 10.2 \text{ cm}$$

Therefore the required base course thickness

$$= 76.338 - 10.2 = 66.138 \text{ cm}$$

The pavement section consists of 66.138 cm thick WBM base course and 7.5 cm thick bituminous concrete surface course.




**Pavement Section**

**Q.5 (d) Solution:**

The latitude and departure of the lines are given by:

$$\text{Latitude, } L = l\cos\theta$$

$$\text{Departure, } D = l\sin\theta$$

| Line | Length (m) | Bearing | Latitude | Departure |
|---|---|---|---|---|
| *AB* | 217.50 | *S*59°45′*E* | –109.57 | 187.88 |
| *BC* | 300.00 | *N*62°30′*E* | 138.52 | 266.10 |
| *CD* | 375.00 | *N*37°36′*W* | 297.11 | –228.80 |
| *DE* | 280.00 | *N*24°42′*W* | 254.38 | –117.00 |

$$\text{Sum of latitudes, } \Sigma L = -109.57 + 138.52 + 297.11 + 254.38$$

$$= 580.44 \text{ m}$$

$$\text{Sum of departures, } \Sigma D = 187.88 + 266.10 - 228.0 - 117.00$$

$$= 108.18 \text{ m}$$

Let $L$ be the latitude of $EA$ and $D$ be the departure of $EA$, then

$$580.44 + L = 0$$

$$\Rightarrow \quad L = -580.44 \text{ m}$$

Similarly, $$D = -108.18 \text{ m}$$

$$\therefore \quad \text{Length of } EA = \sqrt{L^2 + D^2}$$

$$\Rightarrow \quad EA = \sqrt{(-580.44)^2 + (-108.18)^2} = 590.44 \text{ m}$$

$$\tan\theta = \frac{\Sigma D}{\Sigma L} = \frac{108.18}{580.44} = 0.1864$$

$$\Rightarrow \quad \theta = 10°33'26.72''$$

As both the latitude and departure of $EA$ are negative and thus the quadrant is $S$-$W$.

$$\text{Bearing of } EA = \text{S}10°33'26.72''\text{W}$$

**Q.5 (e) Solution:**

Sleeper density is defined as number of sleepers per rail length.

**It depends on the following factors:**

- Axle load
- Traffic volume
- Speed of train
- Type of section of rails
- Type of strength of sleepers
- Type and depth of ballast cushion
- Bearing capacity of the formation.

Sleeper density is expressed as $(n + x)$

Where, $n$ = Length of rail in meters

For BG, rail length, $L$ = 12.8 m (take $n$ = 13)

For MG, rail length, $L$ = 11.89 m (take $n$ = 12)

$x$ = Varies from 4 to 7 for IR network.

**Given data:** Type of track = BG

Length of track = 640 m

Sleeper density = $(n + 5)$

Length of a rail for BG = 12.8 m

Number of sleepers required = ?

Sleeper density of BG = $(n + 5)$

Length for rail BG = 12.8 m (Given)

So, take $n$ = 13

∴ Sleeper density for BG = (13 + 5) = 18 Sleepers for 12.8 m length

∴ Number of sleepers required for 640 m length

$$= \frac{18}{12.8} \times 640 = 900 \text{ sleepers}$$

**Q.6 (a) Solution:**

**(i)** The various geometric elements to be designed are:

Ruling minimum radius

Superelevation

Extra widening

Length of transition curve

SSD, ISD and set-back distance

Ruling minimum radius of curve for ruling design speed of 80 kmph:

$$R_{\text{ruling}} = \frac{V^2}{127(e+f)} = \frac{80^2}{127(0.07+0.15)}$$

$$= 229.06 \text{ nm say } 230 \text{ m}$$

Design value of superelevation:

$$e = \frac{V^2}{225R} = \frac{80^2}{225 \times 230} = 0.124 > 0.07$$

As the value is higher than the maximum sperelevation of 0.07, the value of $e$ is limited to 0.07. The curve should be safe for the full speed of 80 kmph as the ruling minimum radius has been adopted. However check for the transverse skid resistance developed as required.

$$f = \frac{V}{127R} - e = \frac{80^2}{127 \times 230} - 0.07 = 0.149 < 0.15$$

(Less than 0.15 and hence safe)

For lane pavement, i.e. $n$ = 2 and $l$ = 6 m. Extra widening of pavement required is,

$$W_e = \frac{nl^2}{2R} + \frac{V}{9.5\sqrt{R}} = \frac{2 \times 6^2}{2 \times 230} + \frac{80}{9.5\sqrt{230}}$$

$$= 0.157 + 0.555 = 0.712 \text{ m}$$

An extra width of 0.71 m and a total width of pavement $B$ = 7.71 m are provided.

Length of transition curve is designed by calculating the values based on (i) rate of change of centrifugal acceleration C (ii) rate of introduction of the amount of superelevation E and (iii) minimum length formula; the highest of these values is adopted at the design length of transition curve $L$,

$$C = \frac{80}{75+V} = \frac{80}{75+80} = 0.52$$

(This value is within the range 0.5 to 0.8, the value is acceptable for design).

$$\therefore \quad L_s = \frac{0.0215V^3}{CR} = \frac{0.0215 \times 80^3}{0.52 \times 230} = 92 \text{ m}$$

Total amount of superelevation E i.e., the raising of the outer edge of the pavement with respect to inner edge = $B \times e$ = 7.71 × 0.07 = 0.54 m. As the terrain is rolling.

$$\therefore \quad L_s = E \times N = 0.54 \times 150 = 81 \text{ m}$$

Minimum value of $L_s$ as per IRC is given by:

$$L_s = \frac{2.7V^2}{R} = \frac{2.7 \times 80^2}{230} = 75.1 \text{ m}$$

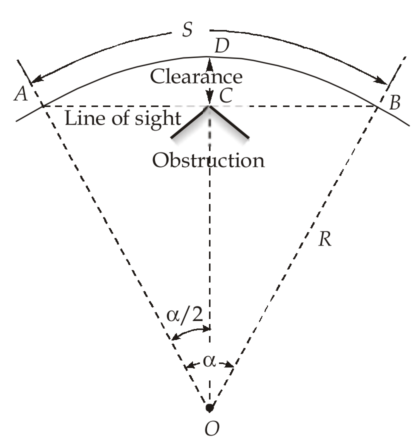
Adopting the highest of the three values, design length of transition curve = 92 m.

**(ii)** Intermediate sight distance = 2 SSD

$$= 2\left[0.278Vt + \frac{V^2}{254f}\right] = 2\left[0.278 \times 80 \times 2.5 + \frac{80^2}{254 \times 0.35}\right]$$

$$= 2 \times 127.6 = 255 \text{ m}$$

Given that the length of circular curve is greater than the desired sight distance SD. The minimum clearance of set-back distance needed $m$ = CD and half the central angle $\alpha/2$ = angle AOD.



The distance $d$ between the centre line of the pavement and the centre line of the inside lane may be taken as one-fourth the width of pavement at the curve (being a two lane pavement) = 7.71/4 = 1.93 m.

$$\frac{\alpha}{2} = \frac{180S}{2\pi(R-d)} = \frac{180\times255}{2\pi(230-1.93)} = 32°$$

$$\therefore \quad \text{Set-back distance } m = R - (R-d)\cos\frac{\alpha}{2}$$

$$= 230 - (230 - 1.93)\cos 32° = 36.6 \text{ m}$$

Therefore the minimum set-back distance or clearance required to provide a clear vision for an ISD of 255 m is 36.6 m.

**Q.6 (b) Solution:**

**(i)**

$$\text{Chain of curve} = 30 \text{ m}$$

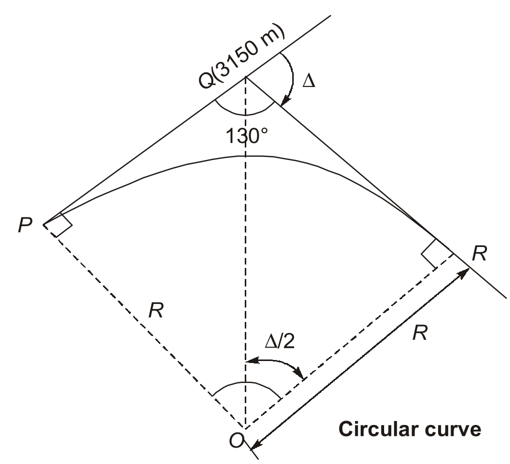
$$\text{Degree of curve, } D = 4°$$

$$\therefore \quad \text{Radius, } R = \frac{1720}{D}$$

$$\Rightarrow \quad R = \frac{1720}{4} = 430 \text{ m}$$

$$\Delta = 180 - 130$$

$$= 50°$$




**Tangent length** $PQ = QR = R\tan\left(\frac{\Delta}{2}\right)$

$$= 430\tan\left(\frac{50}{2}\right)$$

$$= 200.51 \text{ m}$$

**Length of curve** $L = \frac{2\pi R}{360}\times\Delta = \frac{2\pi\times430}{360}\times50° = 375.25 \text{ m}$

$$\text{Chainage of } P = \text{Chainage of } Q - PQ$$

$$= 3150 - 200.51 = 2949.49 \text{ m}$$

$$\text{Chainage of } R = \text{Chainage of } P + \text{Length of curve}$$

$$= 2949.49 + 375.25 = 3324.74 \text{ m}$$

**Intermediate chord lengths**

Length of 1st subchord, $C_1 = 99 \times 30 - 2949.49 = 20.51$ m

Length of last subchord, $C_n = 3324.74 - 110 \times 30 = 24.71$ m

$$\therefore \quad \text{No. of chords, } n = \frac{375.25 - 20.51 - 24.74}{30} + 2 = 13$$

**Offset from chords produced method**

$$1^{st} \text{ offset, } O_1 = \frac{C_1^2}{2R} = \frac{20.51^2}{2\times 430} = 0.49 \text{ m}$$

$$2^{nd} \text{ offset, } O_2 = \frac{C_1C_2 + C_2^2}{2R} = \frac{20.51\times 30 + 30^2}{2\times 430} = 1.76 \text{ m}$$

$3^{rd}$ to $12^{th}$ offset,

$$O_3 = O_4 = \text{........}O_{12} = \frac{C^2}{R} = \frac{30^2}{430} \qquad \{C_2 = C_3 = C_4 = \ldots = C_{12} = 30 \text{ m}\}$$

$$= 2.09 \text{ m}$$

**$13^{th}$ offset** $$O_{13} = \frac{C_{12}C_{13} + C_{13}^2}{2R} = \frac{30\times 24.74 + 24.74^2}{2\times 430} = 1.57 \text{ m}$$

**(ii)**

**Tacheometer:** It is a transit theodolite fitted with stadia diaphragm. The stadia diaphragm consists of two stadia hairs at equal distances one above and the other below the horizontal hair of the cross-hairs.

1. The value of multiplying constant and additive constant should be 100 and 0 respectively.
2. The telescope must/should be fitted with an anallactic lens.
3. The magnifications of the telescope should be 20-80 diameters.
4. The magnifying powers of the eyepiece is kept high.

**Methods of tacheometry:**

1. **Stadia methods:** In a tacheometer, the various wires, in addition to the cross-wires on the diaphragm are known as stadia wires and the vertical distance between the stadia wires is termed as stadia interval. The method derives its name from the fact that the observations are made with respect to these wires.

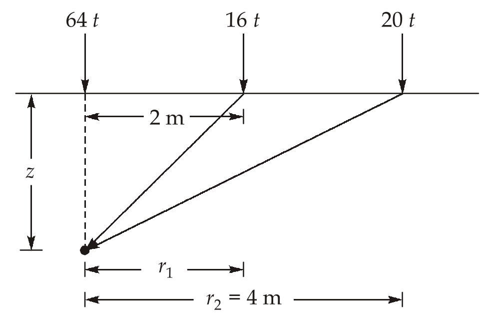
   When the parallactic angle $\alpha$ defined by means of stadia wires is kept fixed and the staff intercept is varied e.g. $AB$ and $A'B'$, the method is called fixed hair method. A tacheometer and a staff is used to make the observations.
2. **Tangential method:** In this method of techeometry, observations are made for vertical angles and staff intercepts are obtained by the cross-wires only. Stadiawires are not used at all. This is similar to trigonometrical levelling.
3. **Range finding:** This method is used to find the horizontal distance and direction of line without going to far end of the line. The instrument is known as range finder. A fixed base is used to compute the ranges.

**Q.6 (c) Solution:**

**(i)**

For given loading,

Depth, $z = 1.5$ m




1. Vertical stress due to 64 $t$

$$\frac{r}{z} = 0, \quad I_B = 0.4775$$

$$\sigma_{v1} = \frac{Q}{z^2} I_B = \frac{64}{(1.5)^2} \times 0.4775$$

$$= 13.58 \text{ t/m}^2$$

2. Vertical stress due to 20 t

As $$\frac{r}{z} = \frac{4}{1.5} = 2.67$$

$$I_B = 0.0025$$

$$\sigma_{v2} = \frac{Q}{z^2} I_B = \frac{20}{(1.5)^2} \times 0.0025 = 0.022 \text{ t/m}^2$$

3. For vertical stress due to 16 t

As $$\frac{r}{z} = \frac{2}{1.5} = 1.33$$

$$I_B = 0.0374$$

$$\sigma_{v3} = \frac{Q}{z^2} I_B = \frac{16}{(1.5)^2} \times 0.0374 = 0.27 \text{ t/m}^2$$

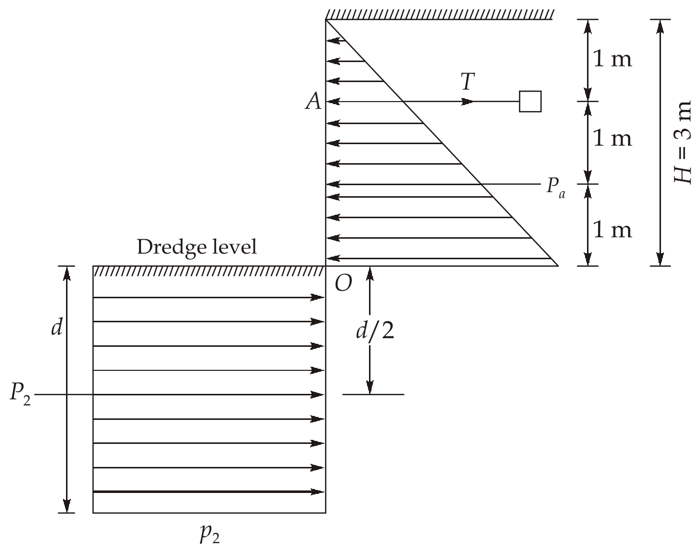
Total vertical stress, $\sigma_v = \sigma_{v1} + \sigma_{v2} + \sigma_{v3}$

$$= 13.58 + 0.022 + 0.27$$

$$= 13.872 \text{ t/m}^2$$

**(ii)**

For given data




For soil above the dredge line i.e., sand

Unit weight, $\gamma = 19 \text{ kN/m}^3$

Frictional angle, $\phi = 30°$

Coefficient of active earth pressure,

$$k_a = \tan^2\left(45° - \frac{\phi}{2}\right) = \tan^2\left(45° - \frac{30°}{2}\right) = \frac{1}{3}$$

Active earth pressure, $$P_a = \frac{1}{2}\gamma k_a H^2 \times 1 = \frac{1}{2} \times 19 \times \frac{1}{3} \times 3^2 \times 1$$

$$= 28.5 \text{ kN (for 1 m length)}$$

Point of action of $P_a = \frac{2}{3} \times 3 = 2$ m (from top)

Below dredge line,

Passive earth pressure, $p_2 = 4C - \gamma h = 4 \times 20 - 19 \times 3 = 23 \text{ kN/m}^2$

Assuming depth below dredge line as '$d$'

Taking moments about anchor, $$M_A = 28.5 \times 1 - 23 \times d \times \left(2 + \frac{d}{2}\right) = 0$$

$\Rightarrow$ $28.5 - 11.5\, d\,(4 + d) = 0$

$\Rightarrow$ $d = 0.545$ m

Depth of sheet pile wall $= 3 + 0.545 = 3.545$ m

Force in anchor, $T = 28.5 - 23 \times 0.545 = 15.965$ kN

**Q.7 (a) Solution:**

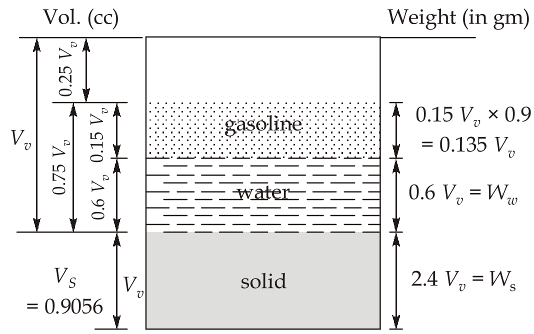
**(i)**

Given, unit weight of soil solid ($\gamma_s$) = 26 kN/m$^3$

Water content of soil ($w$) = 25% = 0.25

Specific gravity of gasoline ($G_g$) = 0.9

$$\gamma_w = 9.81 \text{ kN/m}^3$$




$\therefore$ Specific gravity of soil solid ($G_s$) = $\dfrac{\gamma_s}{\gamma_w}$

$$= \frac{26}{9.81} = 2.65$$

Also, $V_w + V_{\text{gasoline}} = 0.75\ V_v$

$\Rightarrow$ $V_w + 0.25\ V_w = 0.75\ V_v$ {Given that $V_{\text{gasoline}}$ = 25% of volume of water}

$\Rightarrow$ $1.25\ V_w = 0.75\ V_v$

$\Rightarrow$ $$V_w = \frac{0.75\ V_v}{1.25} = 0.6\ V_v$$

Now, weight of soil solids (in gm) = $\dfrac{W_w}{w} = \dfrac{0.6V_v}{0.25} = 2.4\ V_v$

Now, volume of soil solids (in cc)

$$V_s = \frac{W_s}{\gamma_s} = \frac{2.4\ V_v}{2.65 \times 1} = 0.9057\ V_v$$

Now, Void ratio, ($e$) = $\dfrac{V_v}{V_s} = \dfrac{V_v}{0.9057\ V_v} = 1.104$

Porosity, ($n$) = $\dfrac{e}{1+e} = \dfrac{1.104}{1+1.104} = 0.525 \simeq 52.5\%$

Mass of gasoline = $\rho_{\text{gasoline}}$ × Volume of gasoline

$= (\rho_w \times G_g) \times 0.15\ V_v$

$= (1 \times 0.9) \times 0.15\ V_v = 0.135\ V_v$ (gm)

$\therefore$ Total density, ($\rho_b$) = $\dfrac{M_{\text{total}}}{V_{\text{total}}} = \dfrac{0.135V_v + 0.6V_v + 2.4V_v}{0.9057V_v + V_v} = 1.645$ gm/cc

Dry density ($\rho_d$) = $\dfrac{M_{\text{solid}}}{V_{\text{total}}} = \dfrac{2.4V_v}{0.9057\ V_v + V_v} = 1.26$ gm/cc

**Note:** To determine saturated unit weight.

$$\gamma_{sat} = \frac{M_{sat}}{V_{total}} = \frac{2.4\,V_v + 0.6\,V_v + 0.135\,V_v + \text{Weight of water having volume } 0.25\,V_v}{V_v + 0.9056\,V_v}$$

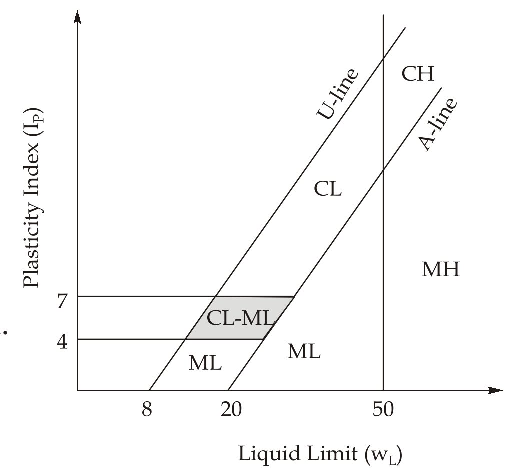
$$= \frac{2.4\,V_v + 0.6\,V_v + 0.135\,V_v + 0.25\,V_v}{1.9057\,V_v} = 1.776 \text{ gm/cc}$$

**(ii)**

For classification of fine grained soils, plasticity index v/s liquid limit graph i.e., plasticity chart is used. In this two major lines viz. *A*-line and *U*-line are used.

A-line $\Rightarrow$ $I_P = 0.73\,(W_L - 20)$

U-Line $\Rightarrow$ $I_P = 0.9\,(W_L - 8)$




(i) For soil, below *A*-line are silt or organic soil.

(ii) Soils between *A*-line and *U*-line are clay.

(iii) Soil above *U*-line does not exist.

(iv) For $I_P$ between 4 to 7%, above *A*-line is mixture of clay and silt.

(v) Soils below 50% liquid limit are called lean soils and soils above 50% liquid limit are called fat soils.

**Q.7 (b) Solution:**

**(i)** Given:

Maximum capacity = 2000 veh/hr

Average length of vehicle = 3.5 m

Traffic volume = 1200 veh/hr

$$q_{max} = \frac{K_j V_f}{4}$$

$$K_j = \frac{1000}{3.5} = 285.714 \text{ veh/km}$$

$$\therefore \quad 2000 = \frac{285.714 \times V_f}{4}$$

$$\Rightarrow \quad V_f = 28 \text{ km/hr}$$

Given:Linear relationship between flow speed and traffic density

$\therefore \quad V = V_f\left(1-\frac{K}{K_j}\right)$

Given: $q = 1200$ veh per hr

But $q = kV$

$\therefore \quad 1200 = K \times V_f\left(1-\frac{K}{K_j}\right)$

$\therefore \quad 1200 = K \times 28\left(1-\frac{K}{285.714}\right)$

$\Rightarrow \quad 1200 = 28K - \frac{28K^2}{285.714}$

$\Rightarrow \quad 42.857 = K - \frac{K^2}{285.714}$

$12244.845 = 285.714\,K - K^2$

$\Rightarrow K^2 - 285.714K + 12244.845 = 0$

$K = 233.208$ veh/km and $52.506$ veh/km

For $K = 233.208$ veh/km

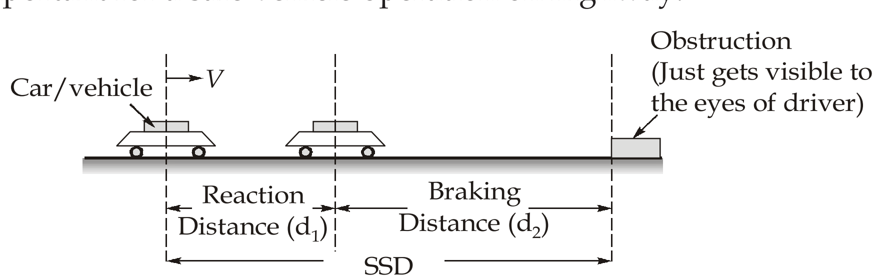
$$q = V_f\left(1-\frac{K}{K_j}\right)K = 28\left(1-\frac{233.208}{285.714}\right)233.208$$

$$= 1199.99 \simeq 1200 \text{ veh/hr}$$

For $K = 52.506$, $\quad q = 28\left(1-\frac{52.506}{285.714}\right)52.506 = 1199.99 \simeq 1200$ veh/hr

$\therefore \quad q = 1200$ veh/hr

**(ii) SSD:** The safe and efficient operation of vehicles on roads depend among other factors, on the road length at which an obstruction, if any becomes visible to the driver in the direction of travel. In other words, the ability to see ahead or visibility is very important for a safe vehicle operation on highway.




SSD = Reaction distance (or lag distance) + braking distance

Let $V$ = Design speed in (m/s)

$t$ = Reaction time = 2.5 seconds

$f$ = Coefficient of friction

$d_1$ = Lag distance

$d_2$ = Braking distance

**Note:** No acceleration during lag time. Assume $V$ (design speed) of vehicle from the time it starts comprehending the danger, lag distance = $V \times t$

Now,

Velocity after covering the lag distance,

$$u_1 = V$$

Final velocity, $u_2 = 0$ (As the vehicle stops)

Applying $u_2^2 - u_1^2 = 2a\,d_2$

Here, $a = fg$ (where $a$ = retardation)

$\therefore$ $0^2 - V^2 = 2(-fg) \times d_2$

$$\Rightarrow \quad d_2 = \frac{V^2}{2fg}$$

$$\therefore \quad SSD = d_1 + d_2$$

$$\Rightarrow \quad SSD = Vt + \frac{V^2}{2gf}$$

Calculation of HSD and ISD,

$$HSD = SSD = Vt + \frac{V^2}{2gf}$$

$$\Rightarrow \quad HSD = \left(80 \times \frac{5}{18}\right) \times 2.5 + \frac{\left(80 \times \frac{5}{18}\right)^2}{2 \times 9.81 \times 0.36}$$

$$\Rightarrow \quad HSD = 55.56 + 69.92 = 125.48 \text{ m} \simeq 126 \text{ m}$$

$$ISD = 2 \times SSD = 2 \times 125.48 = 250.96 \text{ m} \simeq 251\text{m}$$

**Q.7 (c) Solution:**

**(i)**

The table in question indicates that the hottest month of the year is the month of June.

Therefore, monthly mean of the maximum daily temperatures.

$$T_m = 50°C$$

Monthly mean of the average daily temperature for the hottest month,

$$T_a = 40°C$$

$$\text{Airport reference temperature} = T_a + \left(\frac{T_m - T_a}{3}\right)$$

$$= 40 + \left(\frac{50-40}{3}\right) = 43.33°C$$

Since, the runway is at the mean sea level, no correction in length is required for elevation.

The correction for the temperature is necessary as the airport reference temperature is above the standard atmospheric temperature at MSL i.e., 15°C. The rise of temperature is,

$$\Delta T = 43.33°C - 15°C$$

$$= 28.33°C$$

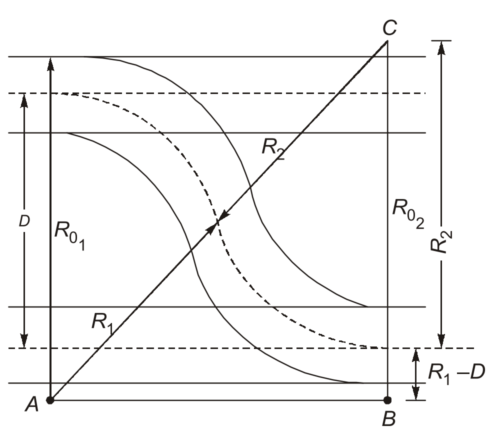
$$\therefore \quad \text{Temperature correction} = \frac{1000}{100} \times 28.33 = 283.3 \text{ m}$$

$$\therefore \quad \text{Corrected runway length} = 1000 + 283.3 = 1283.3 \text{ m}$$

No correction is required for gradient since, the runway is assumed as level in length direction. Therefore the actual length of runway to be provided = 1283.3 m $\simeq$ 1285 m.

**(ii)**

Given : BG, $G$ = 1.75 m, $N$ = 8.5, $D$ = 4.5 m, $R$ = 450 m




$$R_1 = R_{0_1} = 1.5G + 2GN^2$$

$$R_1 = R_{0_1} - \frac{G}{2} = R - \frac{G}{2} \qquad [\text{let } R_{0_1} = R]$$

$$R_{0_2} = 1.5G + 2GN^2$$

$$R_2 = R_{0_2} - \frac{G}{2} = R - \frac{G}{2} \qquad [\text{let } R_{0_2} = R]$$

$$AC = R_1 + R_2 = 2R - G$$

$$BC = R_2 + R_1 - D = 2R - G - D$$

$\therefore$ $$AB = \sqrt{AC^2 - BC^2}$$

$\Rightarrow$ $$AB = \sqrt{(2R-G)^2 - (2R-G-D)^2}$$

$\Rightarrow$ $$AB = \sqrt{(2\times450-1.75)^2 - (2\times450-1.75-4.5)^2}$$

$\Rightarrow$ $$AB = 89.80 \text{ m}$$

So, overall length of crossing, $l = AB = 89.80$ m

Intermediate length of cross-over

$$s = l - 4GN$$

$\Rightarrow$ $$s = 89.80 - 4 \times 1.75 \times 8.5$$

$\Rightarrow$ $$s = 30.3 \text{ m}$$

**Q.8 (a) Solution:**

The ultimate load carrying capacity ($Q_f$) is given by

$$Q_f = Q_b + Q_s$$

where $$Q_b = q_b A_b$$

and $$Q_s = f_s A_s$$

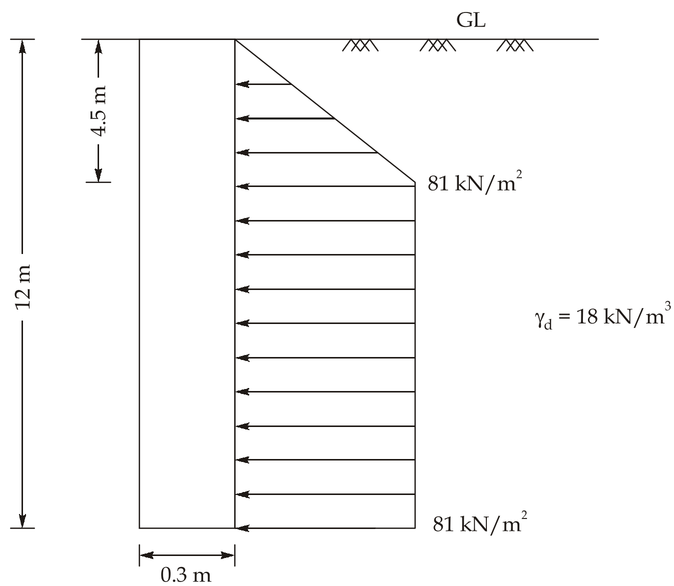
$\therefore$ $$Q_f = q_b A_b + f_s A_s$$

1. Calculation of $Q_b = q_b A_b$

For sandy soils

$$q_b = \sigma' N_q = \gamma' D_f N_q$$

The value of $\sigma'$ or $D_f$ in the above equation should be limited to the critical depth values, which may be taken as $= 15d = 15 \times 0.3$ m $= 4.5$ m.




∴ Limiting vertical effective stress at 4.5 m depth

$$\sigma' = 18 \text{ kN/m}^3 \times 4.5 \text{ m} = 81 \text{ kN/m}^2$$

$$q_b = \sigma' N_q = 81 \times 137 = 11097 \text{ kN/m}^2$$

$$Q_b = q_b A_b = 11097 \times \frac{\pi}{4} \times (0.3)^2 = 784.4 \text{ kN}$$

2. Calculation of $Q_s = f_s A_s$

Skin friction resistance ($Q_s$) shall be calculated in two parts; i.e.,

(i) skin friction resistance over the upper 4.5 m pile length

(ii) skin friction resistance over the remaining 12 – 4.5 = 7.5 m pile length

Both these values are calculated below:

(i) Skin friction resistance over 4.5 m pile length

Average, $\sigma' = \sigma'_{avg} = \dfrac{0+81}{2} = 40.5 \text{ kN/m}^2$

∴ $f_s = K_0' \, \sigma'_{avg} \tan \delta$ ...(i)

where $K_0'$ for dense sand = 2.0 (given)

$$\delta = 0.75\, \phi° \text{ (assumed)}$$
$$= 0.75 \times 40° = 30°$$

∴ $f_s = 2 \times 40.5 \times \tan 30°$

$$= 46.77 \text{ kN/m}^2$$

Skin friction resistance $Q_{s_1}$ of upper 4.5 m pile depth = 46.77 × $A_s$

$$= 46.77 \times (\pi d) \times 4.5 \text{ m}$$
$$= [46.77 \times (\pi \times 0.3 \times 4.5)] \text{ kN}$$
$$= 198.34 \text{ kN}$$

(ii) Skin friction resistance over the remaining 7.5 m pile length

Here σ′ becomes constant = 81 kN/m²

$$\therefore \quad \sigma'_{(avg)} = 81 \text{ kN/m}^2$$
$$\therefore \quad f_s = K_0' \, \sigma'_{(avg)} \tan \delta$$
$$= 2 \times 81 \times \tan 30°$$
$$= 93.53 \text{ kN/m}^2 \quad \text{...(ii)}$$

Skin friction resistance of lower 7.5 m depth of pile

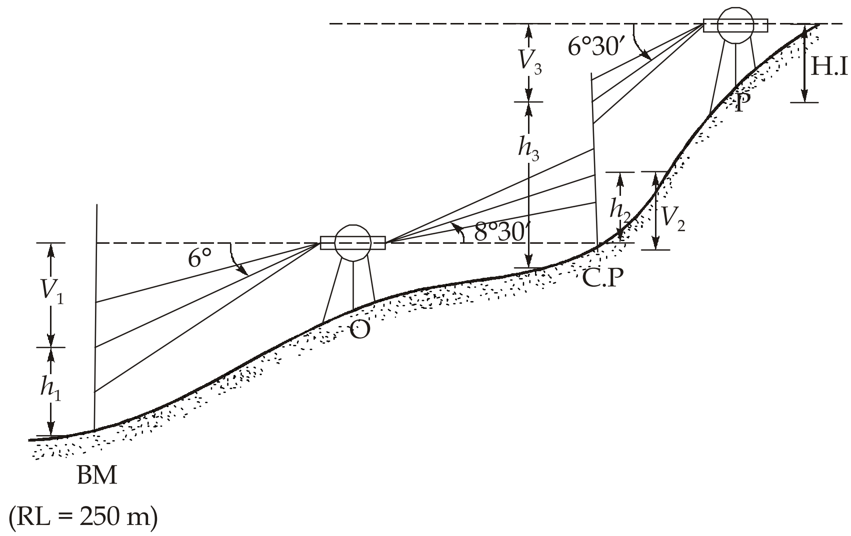
$$Q_{s2} = 93.53 \times A_s$$
$$= [93.53 \times \pi \times 0.3 \times 7.5] \text{ kN}$$
$$= 661.1 \text{ kN}$$

$$\text{Total} \quad Q_s = Q_{s_1} + Q_{s_2}$$
$$= \text{(i)} + \text{(ii)}$$
$$\therefore \quad Q_s = 198.36 + 661.1 = 859.46 \text{ kN}$$
$$\text{Now,} \quad Q_f = Q_b + Q_s$$
$$= 784.4 + 859.46 = 1643.86 \text{ kN}$$

Now safe or allowable capacity of pile

$$= \frac{Q_f}{FOS} = \frac{\text{Ultimate capacity}}{FOS}$$
$$= \frac{1643.86}{2.5} = 657.54 \text{ kN}$$

**Q.8 (b) Solution:**




Instrument at station $O$ and staff at BM

$$S_1 = 2.460 - 1.335 = 1.125 \text{ m}$$

$$\text{H.I.} = 1.45 \text{ m},\ k = 100,\ \theta_1 = 6°\ 00' \text{ (depression)}$$

$$V_1 = \frac{kS_1}{2}\sin 2\theta_1 + C\sin\theta_1$$

$$\Rightarrow \quad V_1 = \frac{100 \times 1.125}{2} \times \sin 12° + 0.0 \times \sin 6°$$

$$\Rightarrow \quad V_1 = 11.695 \text{ m}$$

$$\text{R.L. of line of sight} = \text{R.L. of B.M.} + h_1 + V_1$$

$$= 250.00 + 1.895 + 11.695 = 263.590 \text{ m}$$

Instrument at station $O$ and staff at C.P.

$$S_2 = 1.745 - 0.780 = 0.965 \text{ m}$$

$$\text{H.I.} = 1.45 \text{ m} \quad \theta_2 = 8°\ 30'$$

$$V_2 = \frac{kS_2 \sin 2\theta_2}{2} + C\sin\theta_2$$

$$\Rightarrow \quad V_2 = \frac{100 \times 0.965 \times \sin(2 \times 8°30')}{2} + 0$$

$$\Rightarrow \quad V_2 = 14.107 \text{ m}$$

$$\text{R.L. of C.P.} = \text{R.L. of line of sight} + V_2 - h_2$$

$$= 263.590 + 14.107 - 1.265 = 276.432 \text{ m}$$

Instrument at $P$ and staff at C.P.

$$S_3 = 2.075 - 1.155 = 0.920 \text{ m}, \quad \theta_3 = 6° 30' \quad \text{(depression)}$$

$$V_3 = \frac{kS_3 \sin 2\theta_3}{2} + C\sin\theta_3$$

$$\Rightarrow \quad V_3 = \frac{100 \times 0.920 \times \sin 13°}{2} + 0.0 \times \sin(6°30')$$

$$\Rightarrow \quad V_3 = 10.348 \text{ m}$$

$$\therefore \quad \text{R.L. of station } P = \text{R.L. of C.P.} + h_3 + V_3 - \text{H.I.}$$

$$= 276.432 + 1.615 + 10.348 - 1.45$$

$$= 286.945 \text{ m}$$

**Q.8 (c) Solution:**

**(i)**

**Various types of bituminous dense surfacing are as following:**

| | | | |
|---|---|---|---|
| (i) | Surface dressing | (ii) | Grouted or penetration macadam |
| (iii) | Built-up spray grout | (iv) | Bitumen bound Macadam |
| (v) | Bituminous carpet | (vi) | Bituminous concrete |

For bituminous concrete-surface course.

**Construction Steps:**

**(i) Preparation of the existing base course layer:** The existing surface is prepared by removing the pot holes or ruts if any. The irregularities are filled in with premix chippings at least a week before laying surface course. If the existing pavement is extremely wavy, a bituminous levelling course of adequate thickness is provided to lay a bituminous concrete surface course on a binder course instead of directly laying it on a WBM.

**(ii) Application of Tack Coat:** It is desirable to lay AC layer over a bituminous base or binder course. A tack coat of bitumen is applied at 6.0 to 7.5 kg per 10 $m^2$ area, this quantity may be increased to 7.5 to 10 kg for non-bituminous base.

**(iii) Preparation and placing of premix:** The premix is prepared in a hot mix plant of a required capacity with the desired quality control. The bitumen may be heated upto 150-177°C and the aggregate temperature should not differ by over 14°C from the binder temperature. The hot mixed material is collected from the mixer by the transporters, carried to the location and is spread by a mechanical paver at a temperature of 121 to 163°C. The camber and the thickness of the layer are accurately verified. The control of the temperatures during the mixing and the compaction are of great significance in the strength of the resulting pavement structure.

**(iv) Rolling:** The mix after it is placed on the base course, is thoroughly compacted by rolling at a speed not more than 5 km per hour. The initial or break down rolling is done by 8 to 12 tonnes roller and the intermediate rolling is done with a fixed wheel pneumatic roller of 15 to 30 tonnes having a tyre pressure of 7 kg per $cm^2$. The wheels of the roller are kept damp with water. The number of passes required depends on the thickness of the layer. In warm weather rolling on the next day helps to increase the density if the initial rolling was not adequate. The final rolling or finishing is done by 8 to 12 tonne tandem roller.

**(v) Quality control of bituminous concrete construction:** The routine checks are carried out at site to ensure the quality of the resulting pavement mixture and the pavement surface. Periodical checks are made for (a) aggregate grading (b) grade of bitumen (c) temperatures of aggregate (d) temperatures of paving mix during mixing and compaction. At least one sample for every 100 tonnes of the mix discharged by the hot mix plant is collected and tested for above requirements. Marshall tests are also conducted to check whether it is at least 95 % of the density obtained in the laboratory. The variation in thickness allowed is ± 6 mm per 4.5 m length of construction.

**(vi) Finished surface:** The AC surface should be checked by a 3.0 metre straight edge. The longitudinal undulations should not exceed 8.0 mm and the number of undulations higher than 6.0 mm should not exceed 10 in a length of 300 metre. The cross profile should not have undulations exceeding 4.0 mm.

**(ii)**

Using IRC method,

For Bus: Present traffic, $P = 500$ vpd

Rate of growth, $r = 2\%$

Design life, $n = 20$ years

Construction period, $x = 1.5$ years

Traffic after construction period,

$$A = P(1+r)^x$$

$$\Rightarrow \quad A = 500 \times [1 + 0.02]^{1.5}$$

$$\Rightarrow \quad A = 500 \times 1.02^{1.5} = 515.07 \text{ vpd}$$

For four lane single carriageway,

Lane distribution factor, $D = 0.40$

Vehicle damage factor for single axle/dual axle,

$$F = \left(\frac{L_0}{8.2}\right)^4$$

where $L_0$ is gross wheel load

$$\therefore \quad F = \left(\frac{16}{8.2}\right)^4 \simeq 14.5$$

$\therefore$ Cumulative number of standard axles,

$$\therefore \quad N_s = \left[\frac{365 ADF\left((1+r)^n - 1\right)}{r \times 10^6} \times LSF\right] \text{msa}$$

where msa is million standard axles.

Considering load safety factor (LSF) = 1

$$\therefore \quad N_s = \frac{365 \times 515.07 \times 0.4 \times 14.5 \times \left[(1+0.02)^{20} - 1\right]}{0.02 \times 10^6}$$

$$\simeq 26.5 \text{ msa}$$

Similarly for truck:

$$P = 2000 \text{ vpd}$$
$$r = 10\%$$
$$n = 20 \text{ years}$$
$$x = 1.5 \text{ years}$$
$$\therefore \quad A = P(1 + r)^x$$
$$= 2000 \times (1 + 0.1)^{1.5} = 2307.38 \text{ vpd}$$

Vehicle damage factor for single axle

$$F = \left(\frac{L_0}{15.1}\right)^4 = \left(\frac{20}{15.1}\right)^4 = 3.08$$

$$\therefore \quad N_s = \frac{365 \times 2307.38 \times 0.4 \times 3.08 \times \left[(1+0.1)^{20} - 1\right]}{0.1 \times 10^6}$$

$$= 59.43 \text{ msa}$$

$\therefore$ Design traffic for pavement = 26.5 + 59.43 = 85.93 msa

■■■■