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Detailed Solutions

ESE-2024 Mains Test Series

Civil Engineering Test No : 4

Section A : Design of Concrete and Masonry Structures

Q.1 (a) Solution:

We design shear reinforcement, neglecting the flange of T-beam.

1. Loads:

$$\text{Dead load of flange} = 0.15 \times 2.5 \times 1.0 \times 25 = 9.375 \text{ kN/m}$$

$$\text{Dead load of web} = 0.35 \times (0.8 - 0.15) \times 1 \times 25 = 5.6875 \text{ kN/m}$$

$$\therefore \quad \text{Total dead load, } w_d = 9.375 + 5.6875 = 15.0625 \text{ kN/m} \simeq 15.1 \text{ kN/m}$$

$$\therefore \quad \text{Live load, } w_L = 52 \text{ kN/m}$$

$$\therefore \quad \text{Total load, } w_T = w_d + w_L = 15.1 + 52 = 67.1 \text{ kN/m}$$

$$\therefore \quad \text{Factored load, } w_u = 1.5(w_T) = 1.5(67.1) = 100.65 \text{ kN/m}$$

Now, factored shear force,

$$V_u = \frac{w_u(L_{\text{clear}})}{2}$$

$$= \frac{(100.65)(10)}{2} = 503.25 \text{ kN}$$

2. Nominal shear stress, $\tau_v = \dfrac{V_u}{b_w d} = \dfrac{503.25 \times 10^3}{350 \times 700}$

$$= 2.054 \text{ MPa} < \tau_{c\,\max} \left(= 0.625\sqrt{f_{ck}} = 0.625\sqrt{30} = 3.5 \text{ MPa}\right)$$ Hence OK

3. Percentage of tensile reinforcement (p_t%)

$$p_t\% = \frac{A_{st}}{b_w \cdot d} \times 100$$

$$= \frac{8 \times \frac{\pi}{4} \times 25^2}{350 \times 700} \times 100 = 1.60\%$$

Now from the table given in question, we can calculate τ_c.

$p_t\%$	τ_c
1.50	0.76
1.60	τ_c
1.75	0.80

$$\frac{0.80 - 0.76}{1.75 - 1.50} = \frac{\tau_c - 0.76}{1.60 - 1.50}$$

$\Rightarrow$ $\tau_c = 0.776\ \text{N/mm}^2$

4. Shear force resisted by concrete, $V_c = \tau_c \cdot b_w \cdot d$

$$= \frac{0.776 \times 350 \times 700}{10^3}\ \text{kN}$$

$$= 190.12\ \text{kN}$$

5. Shear force to be resisted by stirrups, $V_s = V_u - V_c$

$$= 503.25 - 190.12 = 313.13\ \text{kN}$$

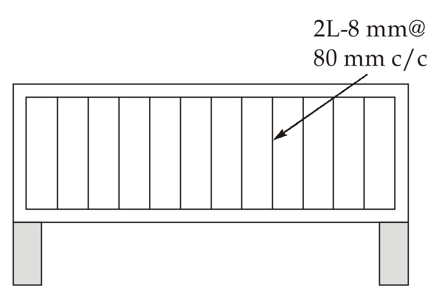
6. Spacing (S_v) of 2 legged – 8 mm shear stirrups is given as

$$S_V = \frac{0.87 f_y A_{sv} \cdot d}{V_s}$$

$$= \frac{0.87 \times 415 \times 2 \times \frac{\pi}{4} \times 8^2 \times 700}{313.13 \times 10^3}$$

$$= 81.14\ \text{mm} \simeq 80\ \text{mm (say)}$$

Hence, provide 2 legged – 8 mm bars @ 80 mm c/c spacing.



Q.1 (b) Solution:

Assumptions in LSM as per IS 456 : 2000:

1. A plane section before bending remains plane after bending.

2. Maximum compressive strain in bending compression in concrete is 0.0035.
3. Tensile strength of concrete is ignored.
4. The compressive strength of concrete shall be taken as $0.67 f_{ck}$ and partial factor of safety of 1.50 shall be applied in addition to this.

$$\text{Design stress in concrete} = \frac{0.67 f_{ck}}{1.5} = 0.45 f_{ck}$$

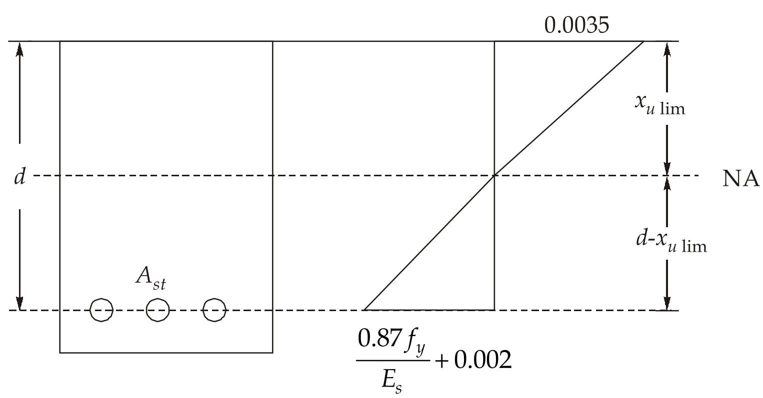
5. Stresses in steel are taken from stress-strain curves. Partial factor of safety of steel is taken as 1.15.
6. Maximum strain in tension steel at failure should not be less than $\frac{0.87 f_y}{E_s} + 0.002$.

For a limiting section:

(i) Strain value in concrete = 0.0035

(ii) Strain value in steel = $\frac{0.87 f_y}{E_s} + 0.002$

Now, strain values are proportional as per above assumption.



Now,
$$\frac{x_{u\,lim}}{0.0035} = \frac{d - x_{u\,lim}}{\frac{0.87 f_y}{E_s} + 0.002}$$

$$\Rightarrow \quad \frac{\frac{0.87 f_y}{E_s} + 0.002}{0.0035} = \frac{d}{x_{u\,lim}} - 1$$

$$\Rightarrow \quad \frac{d}{x_{u\,lim}} = \frac{\frac{0.87 f_y}{E_s} + 0.002}{0.0035} + 1$$

$$\Rightarrow \quad \frac{d}{x_{u\,\text{lim}}} = \frac{\frac{0.87 f_y}{E_s} + 0.002 + 0.0035}{0.0035}$$

$$\Rightarrow \quad x_{u,\,\text{lim}} = \left[\frac{0.0035}{\frac{0.87 f_y}{E_s} + 0.002 + 0.0035}\right] d$$

Now for Fe415 and $E_s = 2 \times 10^5$ N/mm^2

$$x_{u,\,\text{lim}} = \left[\frac{0.0035}{\frac{0.87 \times 415}{2 \times 10^5} + 0.002 + 0.0035}\right] d$$

$$= 0.479\, d \simeq 0.48\, d$$

Hence proved.

Q.1 (c) Solution:

Given data:

Size of beam, B = 300 mm; D = 600 mm

Effective depth, d = D – $E.C.$ = 600 – 50 = 550 mm

Area of steel in compression, A_{sc} = 800 mm^2

Area of steel in tension, A_{st} = 2160 mm^2

f_{ck} = 25 MPa

f_y = 415 MPa

f_{sc} = 350 MPa

Depth of neutral axis is given as

$$x_u = \frac{0.87 f_y A_{st} - (f_{sc} - 0.45 f_{ck}) A_{sc}}{0.36 f_{ck} \cdot B}$$

$$= \frac{0.87 \times 415 \times 2160 - (350 - 0.45 \times 25)800}{0.36 \times 25 \times 300}$$

$$= 188.47 \text{ mm}$$

Check, for limiting depth of neutral axis,

For Fe415,

$$x_{u,\,\text{lim}} = 0.48\ \text{d}$$

$$= 0.48 \times 550 = 264 \text{ mm}$$

Here, $x_{u,\,\lim} > x_u$

So, beam section is under reinforced.

Now ultimate moment of resistance, M_{uR} is given as

$$M_{uR} = 0.36 f_{ck} \cdot B \cdot x_u (d - 0.42 x_u) + (f_{sc} - 0.45 f_{ck}) A_{sc} (d - d')$$

$$= [0.36 \times 25 \times 300 \times 188.47(550 - 0.42 \times 188.47) + (350 - 0.45 \times 25)800 \times (550 - 50)] \times 10^{-6}$$

$$= 375.1 \text{ kNm}$$

Now, working moment of resistance,

$$\text{MOR} = \frac{M_{uR}}{1.5}$$

$$= \frac{375.1}{1.5} = 250.067 \text{ kN-m}$$

$$\simeq 250 \text{ kN-m}$$

Q.1 (d) Solution:

There are mainly two system of prestressing i.e. pre-tensioning and post-tensioning system.

(i) Pre-tensioning system: In pre-tensioning system, the tendons are first prestressed/tensioned between the rigid anchor blocks.

High strength concrete is used for this purpose. When concrete attains the sufficient strength, then the pressure from the jack is released. Due to elasticity of tendons, they tend to come to its original state thereby inducing compressive stress in concrete. Thus, the stress gets transferred from tendons to concrete through bond.

For the production of pre-tensioned elements on a large scale, HOYER system is adopted. In the Hoyer system of pre-tensioning, the tendons are stretched between the two bulk heads several hundred meters apart so that a number of pre-tensioned units can be casted on the same group of pre-tensioned tendons. The tension is applied through hydraulic jacks.

(ii) Post-tensioning system: In this system, the concrete units are casted first and provision of ducts incorporating grooves to house the tendons is provided. When concrete attains the required strength, the high tensile wires are tensioned by means of jack. The force is transferred to concrete by end anchorages and when the prestressing tendon is curved, then this force transfer is achieved through radial pressure between the duct and the cable. The space between the tendon and the duct is grouted after the tensioning process. Most of the patented and commercially available post-tensioning systems are based on the following principles:

1. Wedge action which produces a friction grip on the wire.
2. Direct bearing from rivet or bolt heads formed at the end of the wires.
3. Looping the wires around the concrete.

Q.1 (e) Solution:

Given: Bending moment, $BM = 285$ kN-m

$$f_{ck} = 25 \text{ N/mm}^2$$

$$f_y = 415 \text{ N/mm}^2$$

$$\text{E.C.} = 50 \text{ mm}$$

$$\text{Effective depth, } d = 150 + 350 + 150 - 50 = 600 \text{ mm}$$

Now assuming depth of neutral axis,

$$x_u < 150 \text{ mm}$$

$$\text{Factored } BM,\ BM_u = 1.5 \times 285 = 427.5 \text{ kN-m}$$

Now, equating BM_u and MOR_u of section, we get,

$$BM_u = 0.36 f_{ck} \cdot B_f \cdot x_u (d - 0.42 x_u)$$

$$\Rightarrow \quad 427.5 \times 10^6 = 0.36 \times 25 \times 600 \times x_u(600 - 0.42 x_u)$$

$$\Rightarrow \quad x_u = 147 \text{ mm} < 150 \text{ mm} \quad \text{(OK)}$$

Hence our assumption is correct.

Check limiting depth of neutral axis,

$$x_{u,\lim} = k_o \cdot d$$

$$= 0.48 \times 600 \quad \text{[For Fe 415]}$$

$$= 288 \text{ mm} > x_u \quad \text{Hence, OK}$$

∴ Beam section is under-reinforced.

Now,area of steel is calculated as

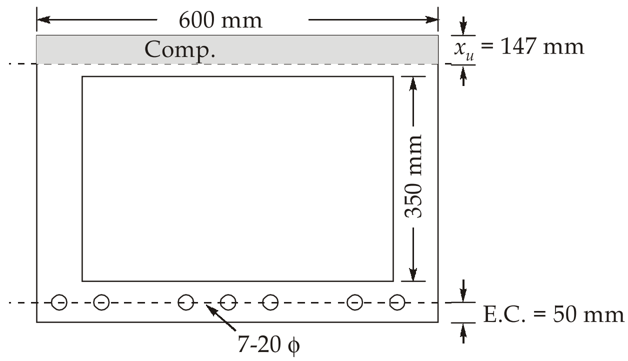
$$(BM)_u = 0.87 f_y A_{st} (d - 0.42 x_u)$$

$$\Rightarrow \quad 427.5 \times 10^6 = 0.87 \times 415 A_{st}(600 - 0.42 \times 147)$$

$$\Rightarrow \quad A_{st} = 2199.77 \text{ mm}^2$$

Provide bars of 20 mm diameter.

$$\therefore \text{ Number of 20 mm diameter bars} = \frac{2199.77}{\frac{\pi}{4} \times 20^2} = 7$$



Q.2 (a) Solution:

(i)

1. Assume short column.

Unsupported length, $L_o = 5$ m

$\therefore$ Effective length, $L_{eff.} = 0.65 \times 5 = 3.25$ m (Both ends are fixed)

Slenderness ratio, $\dfrac{L_{eff}}{D} \leq 12$ (For short column)

$$\Rightarrow \quad D \geq \frac{L_{eff}}{12}$$

$$\Rightarrow \quad D \geq \frac{3250}{12}$$

$$D \geq 270.83 \text{ mm}$$

2. $e_{\min} \leq 0.05$ D

$$\Rightarrow \quad \frac{L_0}{500} + \frac{D}{30} \leq 0.05D$$

$$\Rightarrow \quad \frac{5000}{500} \leq \left(0.05D - \frac{D}{30}\right)$$

$$\Rightarrow \quad D \geq 600 \text{ mm}$$

Hence, adopt diameter of circular column as 600 mm.

Now, $$e_{\min} = \frac{L_o}{500} + \frac{D}{30}$$

$$= \frac{5000}{500} + \frac{600}{30} = 30 \text{ mm}$$

$$e_{min} \geq 0.05D = 0.05 \times 600 = 30 \text{ mm (OK)}$$

Design of steel reinforcement:

$$P_u = 1.05[0.4 f_{ck} A_g + (0.67 f_y - 0.4 f_{ck})A_{sc}]$$

$$\Rightarrow \quad 1.5 \times 3600 \times 10^3 = 1.05\left[0.4 \times 30 \times \frac{\pi}{4} \times 600^2 + (0.67 \times 415 - 0.4 \times 30) A_{sc}\right]$$

$$\Rightarrow \quad A_{sc} = 6578 \text{ mm}^2$$

Provide 20 mm diameter bars.

$$\therefore \text{ Number of 20 mm diameter bars} = \frac{6578}{\frac{\pi}{4} \times 20^2} = 20.93 \simeq 21$$

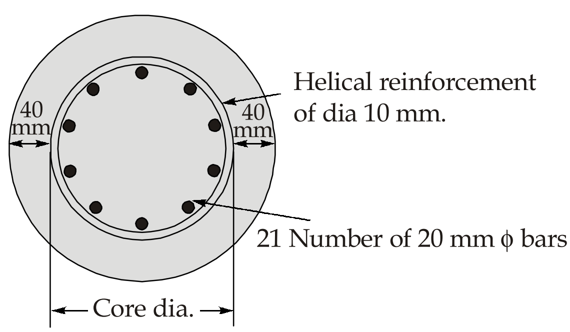
$$\therefore \quad p_c = \frac{A_{sc}}{\frac{\pi}{4} D^2} \times 100 = \frac{21 \times \frac{\pi}{4} \times 20^2}{\frac{\pi}{4} \times (600)^2} \times 100$$

$$= 2.33\% > 0.8\% \; (p_{c\,min})$$

$$< 4\% \qquad (p_{c\,max})$$

Design of helical reinforcement:

Let, the diameter of helical reinforcement is 10 mm.



$$\text{Gross diameter, } D = 600 \text{ mm}$$

$$\text{Diameter of core, } D_C = D - 2 \times 40 = 600 - 2 \times 40 = 520 \text{ mm}$$

Now $\quad D_h = D_C - \phi_h = 520 - 10 = 510$ mm

Now, $$\frac{V_h}{V_C} \geq \frac{0.36 f_{ck}}{f_y}\left(\frac{A_g}{A_c} - 1\right)$$

$$\Rightarrow \frac{\left(\frac{1000}{p}\right)(\pi D_h)\left(\frac{\pi}{4}\phi_h^2\right)}{1000\times\left(\frac{\pi}{4}D_c^2\right)} \geq \frac{0.36 f_{ck}}{f_y}\left(\frac{\frac{\pi}{4}D^2}{\frac{\pi}{4}D_c^2}-1\right)$$ where, p is pitch of helical reinforcement.

$$\Rightarrow \frac{\left(\frac{1000}{p}\times\pi\times 510\right)\left(\frac{\pi}{4}\times 10^2\right)}{1000\times\frac{\pi}{4}\times(520)^2} \geq \frac{0.36\times 30}{415}\times\left[\frac{\frac{\pi}{4}\times 600^2}{\frac{\pi}{4}\times 520^2}-1\right]$$

$\Rightarrow$ $p \leq 68.7125$ mm

$\therefore$ Provide pitch, $p = 65$ mm

Check conditions:

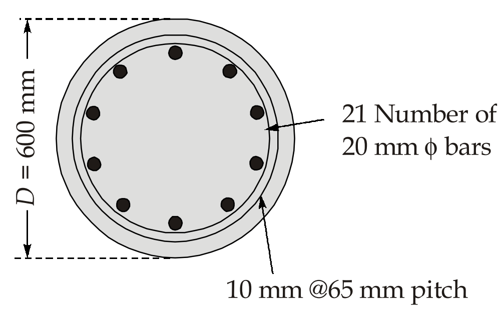
1. $p \ngtr 75$ mm OK

2. $p \ngtr \frac{D_c}{6} = \frac{520}{6} = 86.67$ mm OK

3. $p \geq 25$ mm OK

4. $p \geq 3\,\phi_h$
   ≥ 30 mm OK

Hence provide pitch, $p = 65$ mm



(ii)

Working stress method and limit state method are two different approaches used in structural design to ensure the safety and stability of structures. Differences between these two methods are tabulated below:

Aspects	WSM	LSM
Basis of design	Based on permissible stresses.	Based on limiting states of failure.
Safety factor	Uses factor of safety to ensure safety.	Uses load and resistance factors.
Assumptions	Assumes that materials behave elastically.	Accounts for material non-linearity and variability.
Design criterion	Considers only ultimate strength.	Considers both ultimate and serviceability limits.
Margin of safety	Typically provides a higher margin of safety.	Provides a more rational approach to safety.

Q.2 (b) Solution:

$$L_x = 6.5 \text{ m}, L_y = 8.5 \text{ m}$$

$$\therefore \quad r = \frac{L_y}{L_x} = \frac{8.5}{6.5} = 1.308 < 2 \quad [\therefore \text{ Two way slab}]$$

$$\text{Effective depth, } d = \frac{\text{Effective span}}{(A)\,\text{value} \times M_{ft}}$$

Assume $M_{ft} = 1.10$ for Fe500

$$\therefore \quad d = \frac{0.65 \times 10^3}{20 \times 1.1} = 295.45 \text{ mm}$$

Assume, $d \simeq 300$ mm

1. **Effective span**:

$$d = 300 \text{ mm}$$

$$\text{Width of supports} = 350 \text{ mm}$$

$$\therefore \quad L_{ex} = \min\begin{cases} L_{\text{clear},x} + d \\ L_{\text{clear},x} + w \end{cases} = \min\begin{cases} 6.50 + 0.3 = 6.8 \text{ m} \\ 6.50 + 0.35 = 6.85 \text{ m} \end{cases}$$

$$\Rightarrow \quad L_{ex} = 6.80 \text{ m}$$

Similarly, $L_{ey} = L_{\text{clear}, y} + d = 8.5 + 0.3 = 8.8$ m

$$\therefore \quad \text{Effective depth, } d = \frac{L_{ex}}{A \times M_{ft}}$$

$$= \frac{6800}{20 \times 1.1} = 309 \text{ mm} \simeq 300 \text{ mm (say)}$$

$$\text{Let effective cover} = 30 \text{ mm}$$

$$\therefore \quad \text{Overall depth of slab, } D = d + \text{Effective cover} = 300 + 30 = 330 \text{ mm}$$

2. Loads:

- Dead load of slab $= D \times \gamma$
  $= 0.33 \times 25 = 8.25 \text{ kN/m}^2$
- Dead load of flooring $= t \times \gamma_{flooring}$
  $= 0.08 \times 24 = 1.92 \text{ kN/m}^2$
- Live load $= 6 \text{ kN/m}^2$

Total load, $w_T = 8.25 + 1.92 + 6 = 16.17 \text{ kN/m}^2$

Factored load, $w_u = 1.5(w_T) = 1.5(16.17) = 24.255 \simeq 24.3 \text{ kN/m}^2$

3. Moment coefficients:

$$r = \frac{L_{ey}}{L_{ex}} = \frac{8.8}{6.8} = 1.294 \simeq 1.30$$

From the table

$$\alpha_x = 0.093$$

$$\alpha_y = 0.055$$

4. Bending moment

$$M_{ux(+)} = \alpha_x \cdot w_u \, (L_{ex})^2$$

$$= 0.093 \times 24.3 \times (6.8)^2 = 104.5 \text{ kN-m}$$

$$M_{uy(+)} = \alpha_y \cdot w_u \, (L_{ex})^2$$

$$= 0.055 \times 24.3 \times (6.8)^2 = 61.8 \text{ kN-m}$$

5. Effective depth, d

$$d \geq \sqrt{\frac{(M_u)_{max}}{R_0 \cdot f_{ck} \cdot B}}$$

$$\geq \sqrt{\frac{104.5 \times 10^6}{0.133 \times 30 \times 1000}}$$

$$\geq 161.83 \text{ mm}$$

But we provided effective depth (d) as 300 mm. Hence safe

Hence provide,

Overall depth, $D = 330$ mm

and effective depth in short direction, $d_x = 300$ mm

6. (a) Area of steel in shorter span, A_{stx} for $M_{ux} = 104.5$ kN-m

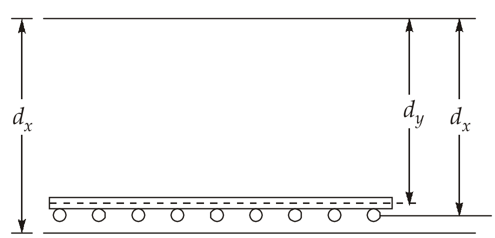
$$A_{stx} = \frac{0.5 f_{ck}}{f_y}\left[1-\sqrt{1-\frac{4.6 M_{ux}}{f_{ck} B d_x^2}}\right] B \cdot dx$$

$$= \frac{0.5\times 30}{500}\left[1-\sqrt{1-\frac{4.6\times 104.5\times 10^6}{30\times 1000\times 300^2}}\right]\times 1000\times 300$$

$$= 840 \text{ mm}^2$$

(b) Area of steel in longer span, A_{sty} for $M_{uy} = 61.8$ kN-m

Assume, $\phi_x = 10$ mm and $\phi_y = 10$ mm



$$\therefore \quad d_y = d_x - \frac{\phi_x}{2} - \frac{\phi_y}{2} = 300 - \frac{10}{2} - \frac{10}{2} = 290 \text{ mm}$$

$$\therefore \quad A_{sty} = \frac{0.5 f_{ck}}{f_y}\left[1-\sqrt{1-\frac{4.6 M_{uy}}{f_{ck} B d_y^2}}\right] B \cdot d_y$$

$$= \frac{0.5\times 30}{500}\left[1-\sqrt{1-\frac{4.6\times 61.8\times 10^6}{30\times 100\times 290^2}}\right] 1000\times 290$$

$$= 505 \text{ mm}^2$$

(c) Spacing of $(A_{st})_x$, S_x is given as

$$A_{st.x} = \left(\frac{1000}{S_x}\right)\times\frac{\pi}{4}\times(\phi_x)^2$$

$$\Rightarrow \quad 840 = \frac{1000}{S_x}\times\frac{\pi}{4}\times 10^2$$

$\Rightarrow \qquad S_x = 93.5 \text{ mm} \simeq 90 \text{ (say)}$

Say 90 mm

Hence provide 10 mmϕ @ 90 mm c/c spacing along shorter side.

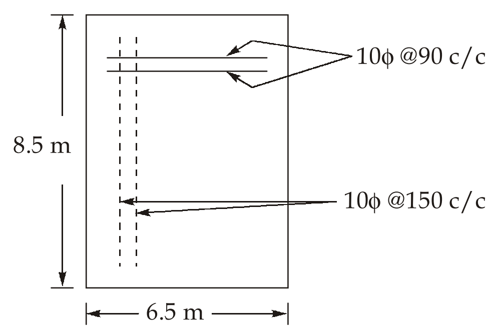
- Spacing of A_{sty}, S_y is given as

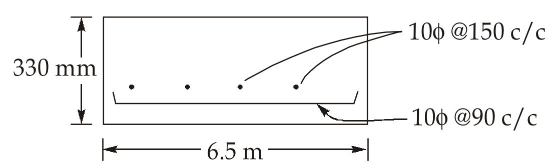
$$A_{sty} = \frac{1000}{S_y} \times \frac{\pi}{4} \times \phi_y^2$$

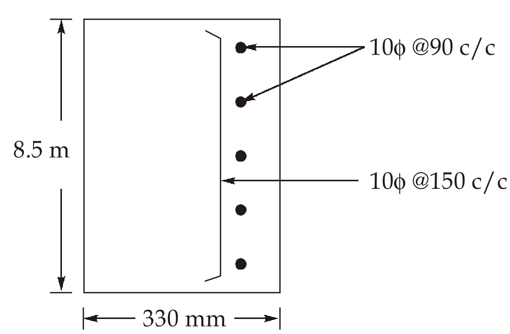
$$\Rightarrow \qquad 505 = \frac{1000}{S_y} \times \frac{\pi}{4} \times 10^2$$

$$\Rightarrow \qquad S_y = 155.52 \text{ mm} \simeq 150 \text{ mm (say)}$$

Hence provide 10 mm ϕ@ 150 mm c/c spacing along longer side.







7. Check for shear:

(i) Shear force for shorter span, V_{u1},

$$V_{u1} = w_u \cdot L_x \left(\frac{r}{2+r}\right)$$

$$= 24.3 \times 6.5 \times \left(\frac{1.3}{2+1.3}\right) = 62.22 \text{ kN}$$

$$\therefore \quad \tau_{v1} = \frac{V_{u1}}{B \cdot d_x} = \frac{62.22 \times 10^3}{1000 \times 300} = 0.2074 \text{ MPa}$$

Hence, $\tau_{v1} < \tau_c(\text{min})$ (0.29 MPa) (Safe)

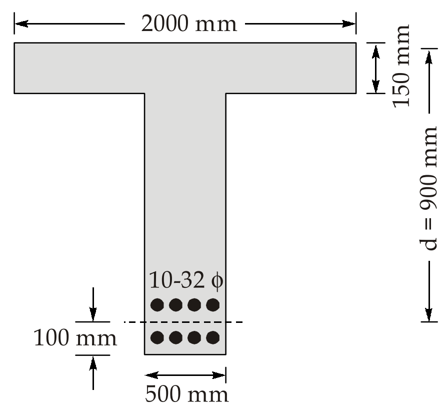
(ii) Shear force for longer span, V_{u2}

$$V_{u2} = \frac{w_u L_x}{3} = \frac{24.3 \times 6.5}{3} = 52.65 \text{ kN}$$

$$\therefore \quad \tau_{v2} = \frac{V_{u2}}{B\, d_y} = \frac{52.65 \times 10^3}{1000 \times 290} = 0.182 \text{ MPa}$$

Hence, $\tau_{u2} < \tau_{c\,\min}$ (0.29 MPa) (Safe)

Q.2 (c) Solution:



Effective width of isolated T beam, is given as

$$b_{\text{eff}} = \frac{l_o}{\frac{l_o}{B} + 4} + b_w$$

where $l_o = 0.7\, l_{\text{eff}} = 0.7 \times 15 = 10.5$ m

So,
$$b_{\text{eff.}} = \frac{10500}{\frac{10500}{2000} + 4} + 500$$

$$= 1635.135 \text{ mm} \simeq 1636 \text{ mm (say)} < b_f (= 2000 \text{ mm})$$

(i) Assume, ultimate depth of neutral axis, $x_u \leq d_f$

i.e $x_u \leq 150$ mm

But $$x_u = \frac{0.87 f_y \cdot A_{st}}{0.36 f_{ck}\, b_{eff.}}$$

$$= \frac{0.87 \times 500 \times 10 \times \frac{\pi}{4} \times 32^2}{0.36 \times 25 \times 1636}$$

$$= 237.6 \text{ mm} \nless 150 \text{ mm}$$

Hence our assumption is wrong.

(ii) Assume $x_u > \frac{7}{3} d_f$

i.e. $x_u > \frac{7}{3} \times 150 = 350$ mm

Now, $$x_u = \frac{0.87 f_y A_{st} - 0.45 f_{ck} \left(b_{eff} - b_w\right) d_f}{0.36 f_{ck} \cdot b_w}$$

$$\Rightarrow \quad x_u = \frac{0.87 \times 500 \times 10 \times \frac{\pi}{4} \times 32^2 - 0.45 \times 25 \times (1636 - 500) \times 150}{0.36 \times 25 \times 500}$$

$$\Rightarrow \quad x_u = 351.44 \text{ mm} > 350 \text{ mm}$$

∴ Our assumption is correct.

Now, ultimate moment of resistance, M_{uR} is given as

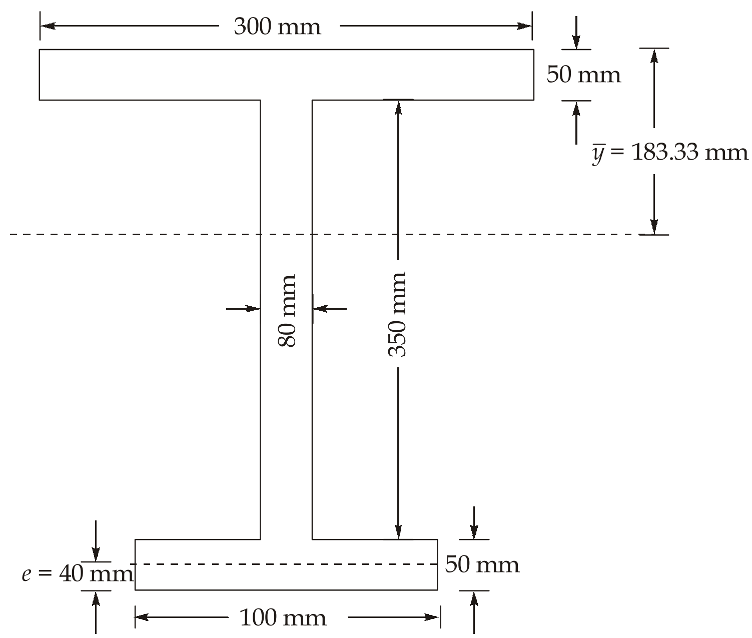
$$M_{uR} = 0.36 f_{ck} \cdot b_w \cdot x_u \left(d - 0.42 x_u\right) + 0.45 f_{ck} \left(b_{eff} - b_w\right) d_f \left(d - \frac{d_f}{2}\right)$$

$$= \begin{bmatrix} 0.36 \times 25 \times 500 \times 351.44 (900 - 0.42 \times 351.44) + 0.45 \\ \times 25 \times (1636 - 500) \times 150 \times \left(900 - \frac{150}{2}\right) \end{bmatrix} \times 10^{-6} \text{ kN-m}$$

$$= 2771.42 \text{ kN-m}$$

Q.3 (a) Solution:

Prestressing force, $P = 200$ kN



Gross area, $A = 300 \times 50 + 100 \times 50 + 350 \times 80 = 48000\ \text{mm}^2$

Let $\bar{y}$ = Distance of centroid of I-section from top

$$\Rightarrow \quad \bar{y} = \frac{300 \times 50 \times 25 + 350 \times 80\left(50 + \frac{350}{2}\right) + 100 \times 50 \times \left[400 + \frac{50}{2}\right]}{300 \times 50 + 350 \times 80 + 100 \times 50}$$

$= 183.33$ mm

Moment of inertia of *I*-section,

$$I = \frac{300 \times 50^3}{12} + 300 \times 50 \times (158.33)^2 + \frac{80 \times 350^3}{12} + 80 \times 350 \times (41.67)^2 + \frac{100 \times 50^3}{12} + 100 \times 50 \times (241.67)^2$$

$I = 10.067 \times 10^8\ \text{mm}^4$

$\therefore$ Eccentricity, $e = 450 - 40 - 183.33 = 226.67$ mm

Section modulus (Z):

$$Z_{top} = \frac{I}{y_{top}} = \frac{10.067 \times 10^8}{183.33} = 5.49 \times 10^6\ \text{mm}^3$$

$$Z_{bottom} = \frac{I}{y_{bottom}} = \frac{10.067 \times 10^8}{(450 - 183.33)} = 3.78 \times 10^6\ \text{mm}^3$$

- Self weight of beam, $w_d = A \times 1 \times \gamma_c = 48000 \times 10^{-6} \times 1 \times 25 = 1.2$ kN/m
- Moment due to self weight of beam, M_d is given as

$$M_d = \frac{w_d dl^2}{8} = \frac{1.2 \times 8^2}{8} = 9.6 \text{ kN-m}$$

- Moment due to imposed load, M_L is given as

$$M_L = \frac{w_L L^2}{8} = \frac{2 \times 8^2}{8} = 16 \text{ kN-m}$$

(i) Stresses due to prestress + self weight

(a) Stress at top, $\sigma_T = \frac{P}{A} - \frac{P.e.}{Z_{top}} + \frac{M_d}{Z_{top}}$

$$\Rightarrow \quad \sigma_T = \frac{200 \times 10^3}{48000} - \frac{200 \times 10^3 \times 226.67}{5.49 \times 10^6} + \frac{9.6 \times 10^6}{5.49 \times 10^6}$$

$$= 4.167 - 8.26 + 1.75$$

$$= -2.343 \text{ N/mm}^2$$

(b) Stress at bottom, $\sigma_b = \frac{P}{A} + \frac{Pe}{Z_{bottom}} - \frac{M_d}{Z_{bottom}}$

$$\Rightarrow \quad \sigma_b = \frac{200 \times 10^3}{48000} + \frac{200 \times 10^3 \times 226.67}{3.78 \times 10^6} - \frac{9.6 \times 10^6}{3.78 \times 10^6}$$

$$= 4.167 + 11.993 - 2.55 = 13.62 \text{ N/mm}^2$$

(ii) Stresses due to prestress + self weight + imposed load

(a) Stress at top, $\sigma_T = \frac{P}{A} - \frac{P.e}{Z_{top}} + \frac{M_d}{Z_{top}} + \frac{M_L}{Z_{top}}$

$$\Rightarrow \quad \sigma_T = \frac{200 \times 10^3}{48000} - \frac{200 \times 10^3 \times 226.67}{5.49 \times 10^6} + \frac{9.6 \times 10^6}{5.49 \times 10^6} + \frac{16 \times 10^6}{5.49 \times 10^6}$$

$$= 4.167 - 8.26 + 1.75 + 2.91 = 0.567 \text{ N/mm}^2$$

(b) Stress at bottom, $\sigma_b = \frac{P}{A} + \frac{Pe}{Z_{bottom}} - \frac{M_d}{Z_{bottom}} - \frac{M_L}{Z_{bottom}}$

$$\Rightarrow \quad \sigma_b = \frac{200 \times 10^3}{48000} + \frac{200 \times 10^3 \times 226.67}{3.78 \times 10^6} - \frac{9.6 \times 10^6}{3.78 \times 10^6} - \frac{16 \times 10^6}{3.78 \times 10^6}$$

$$= 4.167 + 11.993 - 2.54 - 4.23 = 9.39 \text{ N/mm}^2$$

Q.3 (b) Solution:

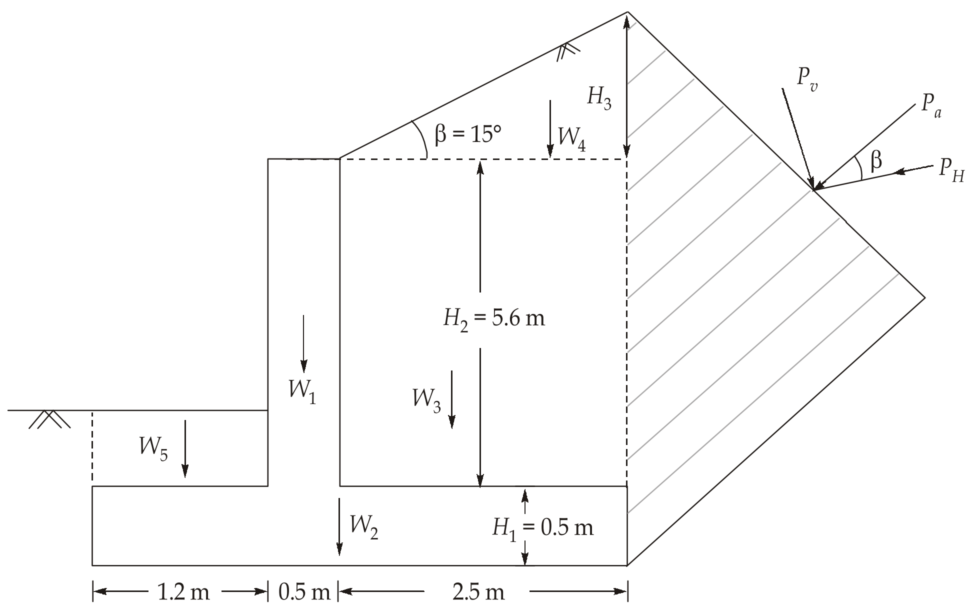
1. Active earth pressure

Active earth pressure coefficient:

$$k_a = \left[\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}\right]\cos\beta$$

$$= \left[\frac{\cos 15° - \sqrt{\cos^2 15° - \cos^2 32°}}{\cos 15° + \sqrt{\cos^2 15° - \cos^2 32°}}\right]\cos 15°$$

$$= 0.3405$$



$$\tan\beta = \frac{H_3}{2.5}$$

$\Rightarrow$ $$H_3 = \tan 15°(2.5)$$

$\Rightarrow$ $$H_3 = 0.6699 = 0.67 \text{ m}$$

$\therefore$ $$H = H_1 + H_2 + H_3 = 0.5 + 5.6 + 0.67 = 6.77 \text{ m}$$

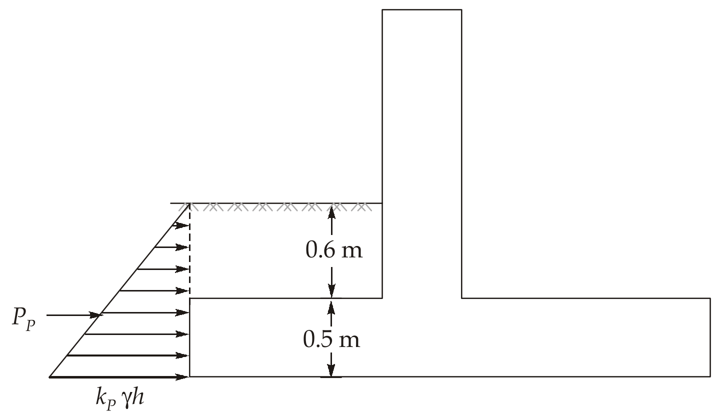
Total active earth pressure, $P_a = \frac{1}{2} k_a \cdot \gamma \cdot H^2$

$$= \frac{1}{2} \times 0.3405 \times 19 \times (6.77)^2 = 148.26 \text{ kN}$$

$\therefore$ $$P_H = P_a \cos\beta = 148.26 \cos 15° = 143.21 \text{ kN}$$

$$P_V = P_a \sin\beta = 148.26 \sin 15° = 38.37 \text{ kN}$$

2. Passive earth pressure



$$h = 0.5 + 0.6 = 1.1 \text{ m}$$

Coefficient of passive earth pressure, k_p

$$k_p = \frac{1+\sin\phi}{1-\sin\phi} = \frac{1+\sin 32^\circ}{1-\sin 32^\circ} = 3.255$$

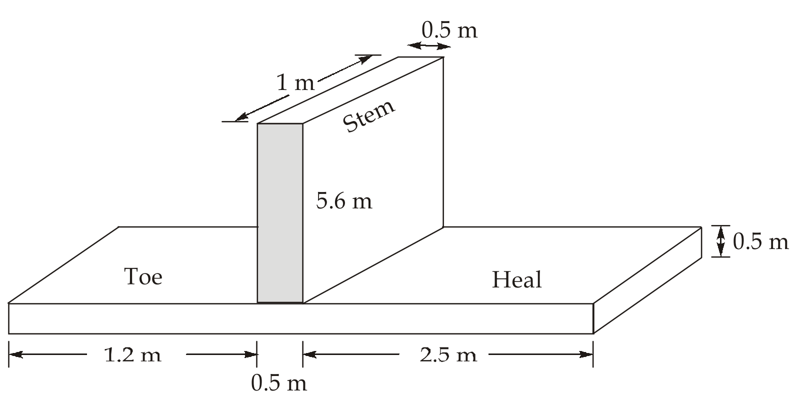
Total passive earth pressure

$$P_p = \frac{1}{2} k_p \cdot \gamma \cdot h^2$$

$$= \frac{1}{2} \times 3.255 \times 19 \times (1.1)^2 = 37.42 \text{ kN}$$

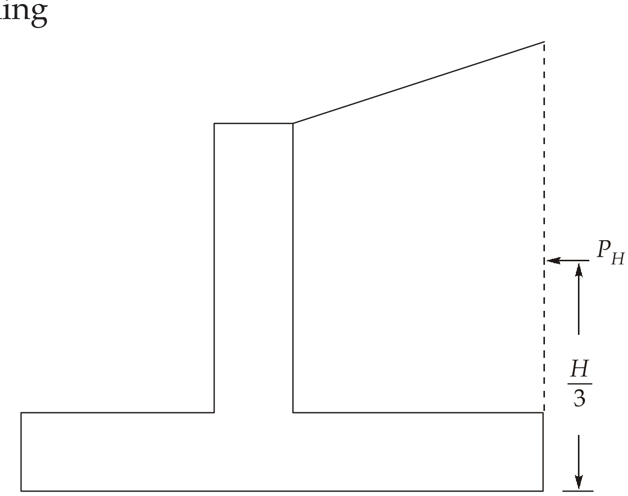
3. Load calculations

Moment of all forces about toe.



Force	Distance from toe	Moment about toe (Force × distance)
1. Stem $W_1 = 0.5 \times 5.6 \times 1 \times 25 = 70\,\text{kN}$	$x_1 = 1.2 + \frac{0.5}{2} = 1.45\,\text{m}$	70×1.45 $= 101.5\,\text{kN-m}$
2. Base $W_2 = 4.2 \times 0.5 \times 1 \times 25 = 52.5\,\text{kN}$	$x_2 = \frac{4.2}{2} = 2.1\,\text{m}$	52.5×2.1 $= 110.25\,\text{kN-m}$
3. Soil $W_3 = 2.5 \times 5.6 \times 1 \times 19 = 266\,\text{kN}$	$x_3 = 1.2 + 0.5 + \frac{2.5}{2} = 2.95$	266×2.95 $= 784.7\,\text{kN-m}$
4. $W_4 = \frac{1}{2} \times 2.5 \times 0.67 \times 1 \times 19 = 15.91\,\text{kN}$	$x_4 = 1.2 + 0.5 + \frac{2}{3} \times 2.5 = 3.37\,\text{m}$	15.91×3.36 $= 53.62\,\text{kN-m}$
5. $W_5 = 1.20 \times 0.6 \times 1 \times 19 = 13.68\,\text{kN}$	$x_5 = \frac{1.20}{2} = 0.6\,\text{m}$	13.68×0.6 $= 8.21\,\text{kN-m}$
6. $P_v = 38.37\,\text{kN}$	$x_6 = 1.2 + 0.5 + 2.5 = 4.2\,\text{m}$	39.72×4.2 $= 161.154\,\text{kN-m}$
$\Sigma W + P_v = 456.46\,\text{kN}$		$\Sigma Wx + P_v B$ $= 1219.434\,\text{kN-m}$

4. Check for overturning



$$\text{Overturning moment} = (P_H) \times \left(\frac{H}{3}\right) = (143.21) \times \frac{(6.77)}{3}$$

$$= 323.18\,\text{kN-m}$$

$$\text{Balancing moment} = \Sigma W_x + P_6$$

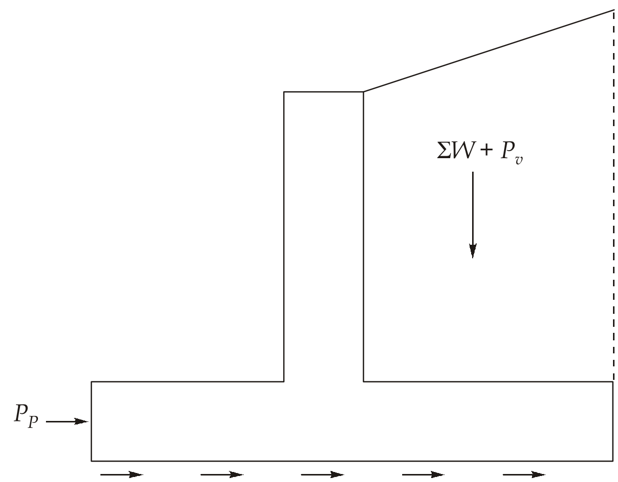
$$= 1219.434\,\text{kN-m}$$

$$\text{Factor of safety} = \frac{(0.9)(M)_{balancing}}{(M)_{overturning}}$$

$$= \frac{0.9 \times 1219.434}{323.18} = 3.4 > 1.4 \quad \text{(OK)}$$

Hence safe against overturning.

5. Check against sliding



$$\text{Friction force, } F_F = \mu.\, R$$

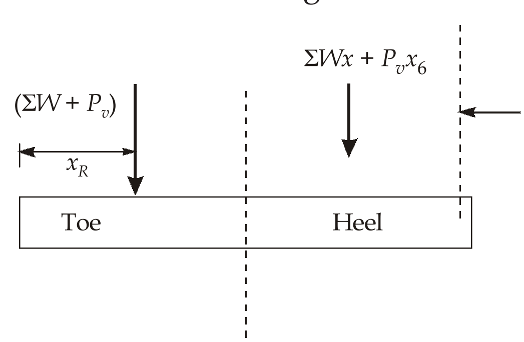
$$\text{Sliding force, } F_{sliding} = P_H = 143.21 \text{ kN}$$

$$\text{Balancing force, } F_{Balancing} = \mu(\Sigma W + P_v) = 0.6(456.46) = 273.876 \text{ kN}$$

$$\text{Factor of safety against sliding} = \frac{0.9 \times F_{Balancing}}{F_{sliding}} = \frac{0.9 \times 273.876}{143.21} = 1.72 > 1.4 \quad \text{(OK)}$$

Hence, safe against sliding.

6. Check for soil pressure below base of footing.



$$\Sigma M_{Toe} = 0$$

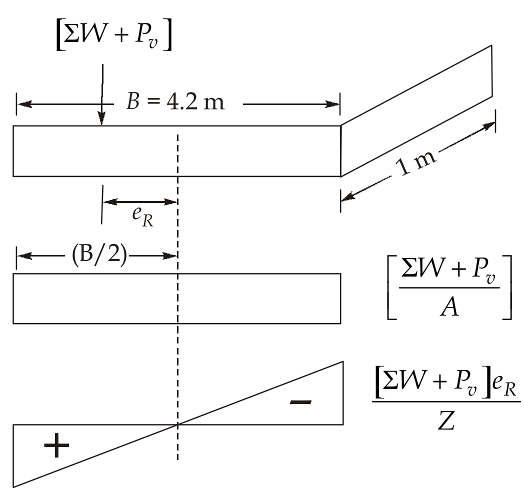
$$\Rightarrow \quad (\Sigma W + P_v)x_R = (\Sigma Wx + P_v x_6) - (P_H)\left(\frac{H}{3}\right)$$

$$x_R = \frac{[\Sigma Wx + P_v x_6] - \left[P_H \cdot \frac{H}{3}\right]}{\Sigma W + P_v}$$

$$= \frac{(1219.434) - (143.21) \times \frac{6.77}{3}}{456.46} = 1.963 \text{ m} = 1.944 \text{ m}$$

$$\text{Eccentricity, } e_R = \frac{B}{2} - x_R$$

$$= \frac{4.2}{2} - 1.963 = 0.137 \text{ m}$$



Maximum soil pressure

$$p_1 = \frac{\Sigma W + P_v}{A}\left[1 + \frac{6 \cdot e_R}{B}\right]$$

$$= \frac{(456.46)}{(1 \times 4.20)}\left[1 + \frac{6 \times 0.137}{4.20}\right]$$

$$= 129.95 \text{ kN/m}^2 < q_0 \ (= 300 \text{ kN/m}^2) \qquad \text{(OK)}$$

$$p_{min} = \frac{(456.46)}{(1 \times 4.20)}\left[1 - \frac{6 \times 0.137}{4.20}\right]$$

$$= 87.41 \text{ kN/m}^2 > 0 \text{ kN/m}^2 \qquad \text{Hence OK}$$

Q.3 (c) Solution:

- **High tensile steel**: In PSC structures, the initial tensile strain in steel is reduced after tensioning by as much as 15% to 20% due to various losses. This reduction in strain must be small compared to initial strain. Otherwise, most of the prestress will be lost. Hence the first principal requirements of a steel for use in prestressing is high ultimate tensile strength The second principal requirement is high ultimate elongation. High tensile steel have ultimate strength capacity as high as 2100 N/mm^2 and use of such steel will provide considerable amount of effective prestressing force even after losses in prestress.
- **High strength concrete**: Because of high tensile steel in prestressed concrete construction, the concrete has to be of good quality and of high strength. A good and well compacted dense concrete has less elastic strain, and has less shrinkage plastic flow, thus reducing the loss of prestress considerably. High quality concrete is also essential to bear high concentration of stresses under the end anchorages. It can be very useful from point of view of crack resistance of member. The durability of members will also be higher. IS code remommends a minimum cube strength of 40 MPa for pre-tensioned and 30 MPa for post-tensioned system.

Q.4 (a) Solution:

Given: $R = 150$ mm; $T = 300$ mm

Width of landing beams, $W = 0.4$ m M20 and Fe415

1. Effective span:

$$L = 12 \times 300 + 400 = 4000 \text{ mm}$$

$$\text{Thickness of waist slab} = \frac{\text{Span}}{20} = \frac{4000}{20} = 200 \text{ mm}$$

2. Loads:

Dead load of slab (on slope),

$$W_s = 0.2 \times 1 \times 25 = 5 \text{ kN/m}$$

Dead load of slab on horizontal span,

$$W = \frac{W_s\sqrt{R^2 + T^2}}{T}$$

$$= \frac{5 \times \sqrt{(150)^2 + (300)^2}}{300}$$

$$= 5.59 \text{ kN/m} \simeq 5.60 \text{ kNm}$$

$$\text{Dead load of one step} = 0.5 \times 0.150 \times 0.300 \times 25$$

$$= 0.5625 \text{ kN/m}$$

$$\text{Load of steps per meter length of flight} = \frac{0.5625 \times 1000}{300} = 1.875 \text{ kN/m}$$

$$\text{Finished load} = 0.6 \text{ kN/m}$$

$$\text{Total dead load} = 5.60 + 1.875 + 0.6 = 8.075 \text{ kN/m}$$

$$\text{Live load} = 4 \text{ kN/m}$$

$$\text{Total service load} = 8.075 + 4$$

$$= 12.075 \text{ kN/m} ; 12.1 \text{ kNm (say)}$$

- Total ultimate load, $W_u = 1.5 \times 12.1 = 18.15$ kN/m

3. Bending moment

Maximum BM at centre of span,

$$M_u = \frac{w_u L^2}{8} = \frac{18.15 \times 4^2}{8} = 36.3 \text{ kN-m}$$

4. Check for depth of waist slab.

$$d = \sqrt{\frac{M_u}{R_{o} \cdot f_{ck} \cdot b}}$$

$$= \sqrt{\frac{36.3 \times 10^6}{0.138 \times 20 \times 1000}} = 114.68 \text{ mm}$$

Now assuming a clear cover of 20 mm and using 12 mm diameter bars,

$$\text{Overall depth of waist slab} = 114.68 + 20 + \frac{12}{2} = 140.68 \text{ mm} < 200 \text{ mm}$$

Hence, depth provided is greater than the required depth.

$$\therefore \quad d = 200 - 20 - 6 = 174 \text{ mm}$$

5. Area of steel,

$$A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck} B d^2}}\right] B.d$$

$$= \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 36.3 \times 10^6}{20 \times 1000 \times 174^2}}\right] \times 1000 \times 174$$

$$= 624.64 \text{ mm}^2$$

6. Spacing of 12 mm ϕ,

$$S = \left(\frac{1000}{625}\right)\times\frac{\pi}{4}\times 12^2 = 180.95 \text{ mm} \simeq 180 \text{ mm c/c (say)}$$

Hence provide 12 mm ϕ diameter bars @ 180 mm c/c spacing.

7. Distribution reinforcement/minimum area of steel.

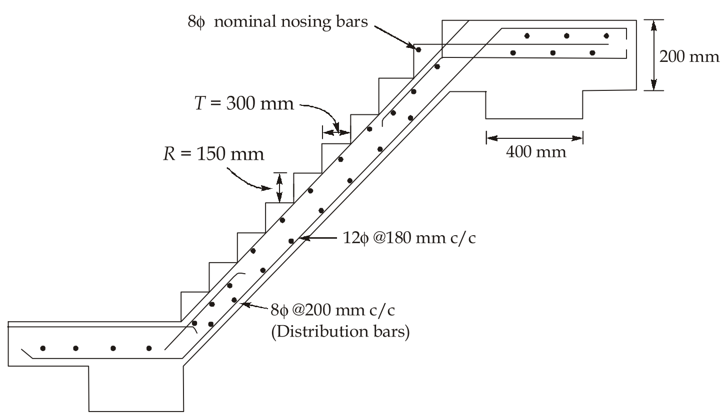
$$A_{st\ min} = \frac{0.12}{100}\times B\times D$$

$$= \frac{0.12}{100}\times 1000\times 200 = 240 \text{ mm}^2$$

$$\text{Spacing of 8 mm } \phi \text{ bars, } S = \left(\frac{1000}{240}\right)\times\frac{\pi}{4}\times 8^2 = 209.43 \text{ mm} \simeq 200 \text{ mm c/c (say)}$$

Hence, provide 8 mm ϕ bars @ 200 mm c/c spacing as distribution reinforcement.

The reinforcement details is as shown



Reinforcement details

Q.4 (b) Solution:

Cracking in concrete occurs due to the following reasons:

1. Flexural tensile stresses due to applied loadings.
2. Shear and torsion induce diagonal tension in concrete members.
3. Restraints against volume changes due to variations in temperatures.
4. Very high compressive stresses induce lateral tensile strains owing to Poisson's effect.
5. Direct tensile stress under the applied loads (like hoop tension in circular tanks).

Factors affecting crack widths: The following factors affect the width of cracks in reinforced concrete members:

1. Tensile stresses in the reinforcing bars.
2. Bond strength and tensile strength of concrete.
3. Depth of concrete member and the location of neutral axis.
4. Diameter and spacing of reinforcing bars.
5. Concrete cover.

(ii) Check for deflection:

For cantilever, minimum effective depth required,

$$d = \frac{\text{Span}}{7} = \frac{6.5 \times 1000}{7} = 928.57 \text{ mm} \simeq 930 \text{ m (say)}$$

But effective depth available,

$$d = 550 - 50 = 500 \text{ mm}$$

∴ Beam is not safe in deflection.

Check for lateral stability:

For lateral stability,

$$\text{Span} = \min \begin{cases} (i) \;\; 25B = 25 \times 250 = 6250 \text{ mm} \\ (ii) \dfrac{100B^2}{d} = \dfrac{100 \times 250^2}{500} = 12500 \text{ mm} \end{cases}$$

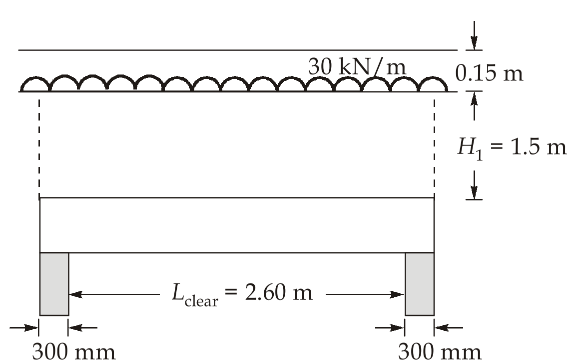
$$= 6250 \text{ mm}$$

But actual span 6500 mm > 6250 mm

∴ Beam fails in lateral stability.

∴ The given cantilever beam fails in both deflection and lateral stability.

Q.4 (c) Solution:



1. Effective span of lintel beam, L_{eff}:

Width of walls = 300 mm = 0.3 m

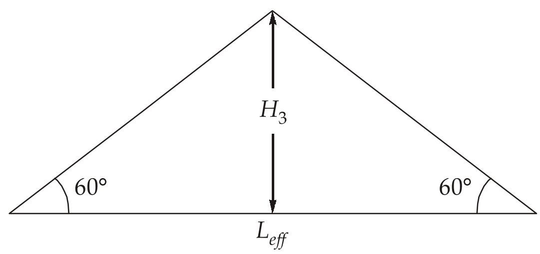
Assume, lintel width, B = 250 mm

and lintel depth, D = 300 mm

Effective depth of lintel, d = 300 – 50 = 250 mm = 0.25 m

$$L_{eff.} = \min \begin{cases} L_{clear} + w = 2.6 + 0.3 = 2.9\,\text{m} \\ L_{clear} + d = 2.6 + 0.25 = 2.85\,\text{m} \end{cases}$$

$$= 2.85 \text{ m}$$



Now,
$$\tan 60° = \frac{H_3}{\left(\frac{L_{eff}}{2}\right)}$$

$$H_3 = (\tan 60°)\left(\frac{2.85}{2}\right) = 2.468$$

$\therefore$ $1.25H_3 = 1.25(2.468) = 3.085 \text{ m} > 1.5 \text{ m}$

$\therefore$ $H_1 < 1.25(H_3)$

Hence triangle will not be formed above lintel and all load above the lintel need to be considered.

2. Loads on lintel:

Self weight of lintel = 0.25 × 0.3 × 1 × 25 = 1.875 kN/m

Weight of brick work = 0.25 × 1.5 × 1 × 20 = 7.5 kN/m

Slab load = 30 kN/m

$\therefore$ Total load, w_T = 1.875 + 7.5 + 30 = 39.375 kN/m

$\simeq$ 40 kN/m (say)

3. Design for flexure

a. Ultimate bending moment, BM_u is given as

$$BM_u = \frac{(1.5w_T)(L_{eff})^2}{8}$$

$$= \frac{(1.5\times40)(2.85)^2}{8} = 60.92 \text{ kN-m}$$

b. Effective depth required,

$$d_{\text{req.}} = \sqrt{\frac{BM_u}{R_{\sigma}\cdot f_{ck}B}} = \sqrt{\frac{60.92\times10^6}{0.133\times30\times250}}$$

$$= 247.13 \text{ mm} \simeq 248 \text{ mm}$$

But we have provided effective depth,

$$d = 250 \text{ mm,} \qquad \text{Hence OK}$$

c. Area of flexural steel required,

$$A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1-\sqrt{1-\frac{4.6BM_u}{f_{ck}B.d^2}}\right]B\ d$$

$$= \frac{0.5\times30}{500}\left[1-\sqrt{1-\frac{4.6\times60.92\times10^6}{30\times250\times250^2}}\right]250\times250$$

$$= 685.931 \text{ m}^2 \simeq 686 \text{ mm}^2$$

$$p_t = \frac{686}{250\times250}\times100 = 1.0976\%$$

$$p_{t\,\text{lim}} = 41.61\frac{f_{ck}}{f_y}.\frac{x_{u\,\text{lim}}}{d} = 41.61\times\frac{30}{500}\times0.46$$

$$= 1.148\% > p_t$$

∴ Singly reinforced lintel beam section is required.

Provide 2 – 20ϕ and 1 – 10ϕ mm.

$$p_{t\,\text{prov.}} = \frac{706.85}{250\times250}\times100 = 1.13\%$$

∴ $$A_{t\,\text{prov.}} = 2\times\frac{\pi}{4}\times20^2+1\times\frac{\pi}{4}\times10^2 = 706.85 \text{ mm}^2$$

4. Design of shear reinforcement:

Ultimate shear force, $V_u = \dfrac{w_u L_{\text{clear}}}{2} = \dfrac{(1.5\times40)\times2.6}{2} = 78 \text{ kN}$

Nominal shear stress, $\tau_v = \dfrac{V_u}{B.d} = \dfrac{78\times10^3}{250\times250}$

$$= 1.248 \text{ MPa} < \tau_{c\ max.} \left(= 0.625\sqrt{f_{ck}} = 0.625\sqrt{30} = 3.5 \text{ MPa}\right)$$

Now τ_c can be calculated from the table.

$$\frac{1.25-1.0}{0.71-0.66} = \frac{1.13-1.0}{\tau_c - 0.66}$$

$$\Rightarrow \quad \tau_c = 0.686 \text{ MPa}$$

$$V_c = \tau_c(Bd) = 0.686 \times (250 \times 250) \times 10^{-3} \text{ kN}$$

$$= 42.875 \text{ kN} \simeq 42.9 \text{ kN}$$

$$V_s = V_u - V_c = 78 - 42.9 = 35.1 \text{ kN}$$

7. Now, spacing of 2 legged 8 mm ϕ_1, shear stirrups

$$V_s = \frac{0.87 f_y A_{sv}\, d}{S_v} \qquad \left[f_y \not> 415\,\text{MPa}\right]$$

$$\Rightarrow \quad S_v = \frac{0.87 \times 415 \times 2 \times \frac{\pi}{4} \times 8^2 \times 250}{35.1 \times 10^3}$$

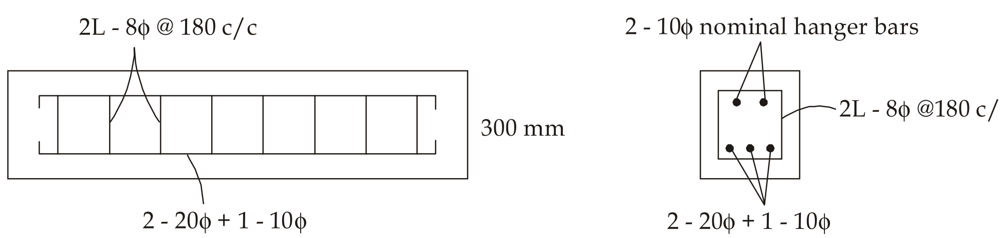
$$\Rightarrow \quad S_v = 258.52 \text{ mm} \simeq 250 \text{ mm}$$

Check for maximum spacing of stirrups:

$$\Rightarrow \quad S_{v\ maximum} = \min\begin{cases} 0.75d = 0.75 \times 250 = 187.5\,\text{mm} \\ 300\,\text{mm} \end{cases}$$

$$S_{v\ maximum} = 187.5 \text{ mm} \simeq 180 \text{ mm}$$

Hence, provide 2 legged – 8 mm ϕ @ 180 mm c/c in full span of the lintel.



Section B : SOM - 1 + Highway Engineering - 2 + Surveying and Geology-2

Q.5 (a) Solution:

Design traffic for buses

Gross weight = 16000 kg

Standard axle load = 8160 kg

Planning and construction period, x = 1.5 years

Number of vehicles per day, P_1 = 250

Traffic growth rate, r_1 = 5%

Now,

Number of buses at end of construction period,

$$A_1 = P_1(1 + r_1)^x$$

$$= 250\left(1+\frac{5}{100}\right)^{1.5} = 268.98 \text{ vehicles}$$

Now, vehicle damage factor, $VDF_1 = \left(\frac{16000}{8160}\right)^4 = 14.78$

Lane distribution factor, LDF_1 = 0.5

Now, design CSA (in msa) value of bus, CSA_1 is given by

$$CSA_1 = \frac{365\left[A_1\left(1+\frac{r_1}{100}\right)^n - 1\right]\times VDF_1 \times LDF_1}{\frac{r_1}{100}\times 10^6}$$

$$= \frac{365\times\left[268.98\left(\frac{1+5}{100}\right)^{15} - 1\right]\times 14.78\times 0.5}{\frac{5}{100}\times 10^6}$$

$$= 15.655 \text{ msa}$$

Design traffic for truck,

Gross weight = 22000 kg

Standard axle load = 14968 kg

x = 1.5 years

P_2 = 1200

$$A_2 = P_2(1+r_2)^x$$

$$= 1200\left(1+\frac{8}{100}\right)^{1.5}$$

$$= 1346.84 \text{ trucks}$$

Now, vehicle damage factor, $VDF_2 = \left(\frac{22000}{14968}\right)^4 = 4.67$

Lane distribution factor, $LDF_2 = 0.5$

Now, design CSA value (in msa) of truck, CSA_2 is given by

$$CSA_2 = \frac{365\left\{A_2\left(1+\frac{r_2}{100}\right)^n - 1\right\} \times VDF_2 \times LDF_2}{\frac{r_2}{100}\times 10^6}$$

$$= \frac{365\left\{1346.84\left(1+\frac{8}{100}\right)^{15} - 1\right\} \times 4.67 \times 0.5}{\frac{8}{100}\times 10^6}$$

$$= 31.16 \text{ msa}$$

So, design CSA values $= CSA_1 + CSA_2$

$= 46.81$ msa

Now, from the table given in question.

For N = 46.81 msa, by interpolation.

- Thickness of BC wearing course = 40 mm
- Thickness of DBM binder course, t

$$\frac{50-40}{115-105} = \frac{46.81-40}{t-105}$$

$\Rightarrow$ $t = 111.8$ mm

Thickness of WMM base course = 250 mm

Thickness of GSB sub-base course = 200 mm

Overall thickness of flexible pavement = 40 + 111.8 + 250 + 200

$= 601.8 \text{ mm} \simeq 602 \text{ mm}$

Q.5 (b) Solution:

Importance of Engineering Geology:

- The selection of sites for various engineering structures is affected by presence of faults and folds, as they affect the safety of facilities, and pose a significant risk. The presence of faults or breaks, reduces the resistance of the rocks and jeoparadize the facilities provided by making the rocks liable to collapse.
- The folds are also a problem, since the existence of structures concave folds it with a great ability to collect ground water, causing water problems at large. So this justifies the importance of geological studies and their role in avoiding collapse of structures and buildings. These studies help to avoid the high costs that may be borne by the concerned authorities in the event they set up these projects in the areas of joints and faults and folds.

Geological Hazards:

- A **geologic hazard** is one of several types of adverse geologic conditions capable of causing damage or loss of property and life.

Geologic Event	Associated Hazards
Earthquake	A. Ground shaking B. Surface faulting C. Landslides and liquefaction 1. Rock avalanches 2. Rapid soil flows 3. Rock falls D. Tsunamis
Volcanic Eruption	A. Tephra falls and ballistic projectiles B. Pyroclastic phenomena C. Lahars (mud flows) and floods D. Lava flows E. Poisonous gases eruption

Q.5 (c) Solution:

Here, maximum BM will be at A,

$$M = P \times (150 - 55)$$

$$= 40 \times 10^3 \times 95 = 3.8 \times 10^6 \text{ N-mm}$$

Now, maximum bending stress,

$$\sigma = \frac{M}{I} y_{max} = \frac{M}{Z}$$

where Z is section modulus of cross section which is circular.

For circular cross-section, $$Z = \frac{\pi D^3}{32}$$

$$\therefore \quad \sigma = \frac{32M}{\pi D^3}$$

$$\Rightarrow \quad D^3 = \frac{32 \times 3.8 \times 10^6}{\pi \times 30}$$

$$\Rightarrow \quad D = 108.86 \text{ mm}$$

Q.5 (d) Solution:

Three major groups of rocks are defined: igneous, sedimentary, and metamorphic. The scientific study of rocks is called petrology, which is an essential component of geology.

1. **Igneous Rocks:** Igneous rocks are formed by the solidification of magma, a silicate liquid generated by partial melting of the upper mantle or the lower crust. Different environments of formation, and the cooling rates associated with these, create very different textures and define the two major groupings within igneous rocks:
   - Volcanic rocks (Extrusive Igneous Rocks): Volcanic rocks form when magma rises to the surface and erupts, either as lava or pyroclastic material. The rate of cooling of the magma is rapid, and crystal growth is inhibited. Volcanic rocks are characteristically fine-grained. Volcanic rocks often exhibit structures caused by their eruption, e.g. flow banding (formed by shearing of the lava as it flows), and vesicles (open cavities that represent escaped gasses).
   - Plutonic rocks (Intrusive Igneous Rocks): Plutonic rocks form when magma cools within the Earth's crust. The rate of cooling of the magma is slow, allowing large crystals to grow. Plutonic rocks are characteristically coarse-grained.
2. **Sedimentary Rocks:** Sedimentary rocks are the product of the erosion of existing rocks. Eroded material accumulates as sediment, either in the sea or on land, and is then buried, compacted and cemented to produce sedimentary rock (a process known as diagenesis).

   There are two major groupings of sedimentary rocks:
   - **Clastic sedimentary rocks:** The fragments of pre-existing rocks or minerals that make up a sedimentary rock are called clasts. Sedimentary rocks made up of clasts are called clastic (clastic indicates that particles have been broken and transported). Clast shape, or the degree of rounding of clasts, is important in differentiating some sedimentary rocks. Clasts vary in shape from rounded to angular, depending on the distance they have been transported and / or the

environment of deposition, e.g. rounded clasts are generally the product of long transportation distances and / or deposition in high energy environments (beaches, rivers).

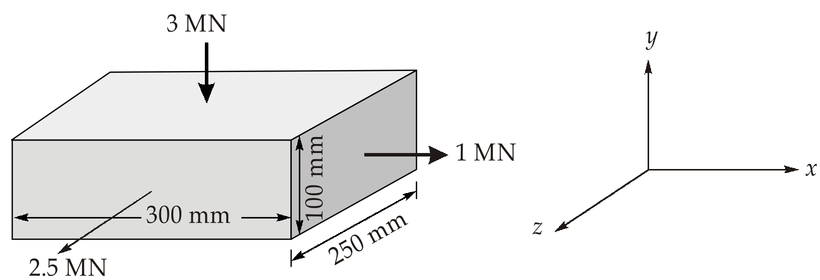
- **Non-clastic sedimentary rocks:** These sedimentary rocks occur when minerals/ mineraloids are precipitated directly from water, or are concentrated by organic matter / life. Components have not been transported prior to deposition. No clasts are present.

3. **Metamorphic Rocks:** Metamorphism is the alteration of pre-existing rocks in the solid state due to changes in temperature and pressure. Under increasing temperature and / or pressure existing minerals become unstable and break down to form new minerals. In the case of regional metamorphism the rocks are subjected to tectonic forces which provide the necessary mechanisms for metamorphism. Products include schist and slate. Contact metamorphism involves metamorphosis through heating by an intruding plutonic body. Hornfels is the result of this type of metamorphism.

   Metamorphic textures: The two distinctive metamorphic textures are:

   - **Foliation:** This represents a distinct plane of weakness in the rock. Foliation is caused by the re-alignment of minerals when they are subjected to high pressure and temperature. Individual minerals align themselves perpendicular to the stress field such that their long axes are in the direction of these planes (which may look like the cleavage planes of minerals). Usually, a series of foliation planes can be seen parallel to each other in the rock. Well developed foliation is characteristic of most metamorphic rocks. Metamorphic rocks often break easily along foliation planes.
   - **Granular:** This describes a metamorphic rock consisting of interlocking equant crystals (granules), almost entirely of one mineral. A granular texture is developed if a rock's chemical composition is close to that of a particular mineral. This mineral will crystallize if the rock is subjected to high pressure and temperature. A granular texture is characteristic of some metamorphic rocks.

Q.5 (e) Solution:



$$\text{Stress in } x\text{-direction}, \sigma_x = \frac{1\times10^6}{250\times100} = 40 \text{ N/mm}^2$$

$$\text{Stress in } y\text{-direction}, \sigma_y = \frac{-3\times10^6}{300\times250} = -40 \text{ N/mm}^2$$

$$\text{Stress in } z\text{-direction}, \sigma_z = \frac{2.5\times10^6}{300\times100} = 83.33 \text{ N/mm}^2$$

Now volumetric strain,

$$\epsilon_v = \left(\frac{1-2\mu}{E}\right)\left(\sigma_x+\sigma_y+\sigma_z\right)$$

$$= \left[\frac{1-2(0.25)}{2\times10^5}\right]\times(40-40+83.33)$$

$$= 2.08 \times 10^{-4}$$

So, volumetric change,

$$\Delta V = \varepsilon_V \times V$$

$$= 2.08 \times 10^{-4} \times 300 \times 250 \times 100 = 1560 \text{ mm}^3$$

Let, the stress in z-direction is σ_z, so that volume change, ε_v is zero.

Now,
$$\varepsilon_v = \frac{\sigma_x+\sigma_y+\sigma_z}{E}(1-2\mu)$$

For ε_v to be zero,

$$\sigma_x + \sigma_y + \sigma_z = 0$$

$$\Rightarrow \quad \sigma_z = -\sigma_x - \sigma_y$$

$$= -40 - (-40) = 0$$

So, 2.5 MN of compressive load should be applied in z-direction, so that volume change is zero.

Q.6 (a) Solution:

(i)

- **Traffic forecast**: The roadway facilities are to be designed and developed not by taking into account just the present traffic, but it is essential to consider the projected traffic for the desired design period. The prediction of future traffic is of greater significance for planning the roadway geometric, with particular reference to widths of formation and road land or the right of way.

- **Importance of traffic forecast**: The economic as well as the financial viability of a highway project depends on the accuracy of the predicted traffic. Most of the projects of national highway development programme are being undertaken under build, operate and transfer (BOT) basis. Some of the state highways development projects are also being taken up in similar mode. Therefore the concessionaire is more concerned about the expected traffic and the toll viability for the project.
- **Factors influencing traffic growth**: The traffic growth depends on the following factors:
  - (a) Economic factors such as (i) gross national product (GNP) or gross domestic product (GDP)
    - (i) agricultural output.
    - (ii) industrial output.
  - (b) Demographic factors such as
    - (i) population
    - (ii) proportion of rural and urban population

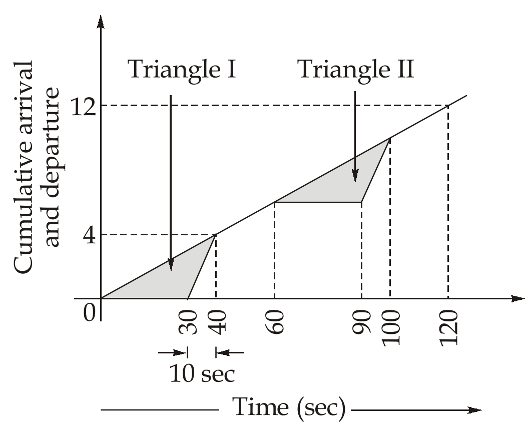
(ii)

1. Between 0 to 120 sec:

Given, q (from 0 -120 sec)= 360 vph

Number of vehicles that arrive in 120 sec

$$= \frac{360}{3600} \times 120 = 12$$



$$\text{Saturation flow rate} = 1440 \text{ vph}$$

$$\therefore \quad 360(t + 30) = 1440 \times t$$

$$\Rightarrow \quad t + 30 = 4t$$

$$\Rightarrow \quad t = 10 \text{ sec}$$

$$\text{Average delay} = \frac{\text{Area of triangle I or II}}{\text{Number of arrivals in a cycle}}$$

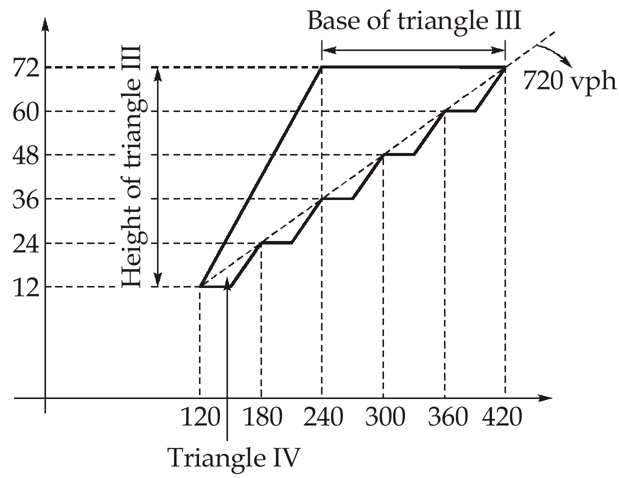
$$\text{Area of triangle I or II} = \frac{1}{2} \times 30 \times 4 = 60$$

Number of arrivals in a cycle

$$= 360 \times \frac{60}{3600} = 6$$

$$\therefore \text{ Average delay between 0 - 120 sec} = \frac{60}{6} = 10 \text{ sec}$$

2. Between 120 - 420 sec:



Given, q (from 120 - 240 sec) = 1800 vph

q (from 240 - 420 sec) = 0 vph

The maximum number of vehicles that can cross the intersection from a given approach per unit time

$$= \frac{30}{60} \times 1440 = 720 \text{ vph}$$

Number of vehicles that arrive between 120 - 240 sec

$$= \frac{1800}{3600} \times 120 = 60 \text{ vehicles}$$

Total number of vehicles after 240 sec = 60 + 12 = 72 vehicle

$$\text{Average delay} = \frac{\text{Area of triangle-III} + 5 \times \text{Area of triangle-IV}}{\text{Number of arrivals from 120 - 420 sec}}$$

$$= \frac{\left(\frac{1}{2} \times 180 \times 60\right) + \left(5 \times \frac{1}{2} \times 30 \times 12\right)}{60} = 105 \text{ sec}$$

3. Between 0 - 420 sec:

The average delay to all the vehicles between 0 - 420 sec can be obtained by dividing the total delay (faced by all vehicles) by the total number of vehicles.

$$\therefore \text{Average delay} = \frac{\left(\frac{1}{2} \times 30 \times 4\right) \times 2 + \left(\frac{1}{2} \times 180 \times 60\right) + \left(5 \times \frac{1}{2} \times 30 \times 12\right)}{72} = 89.17 \text{ sec}$$

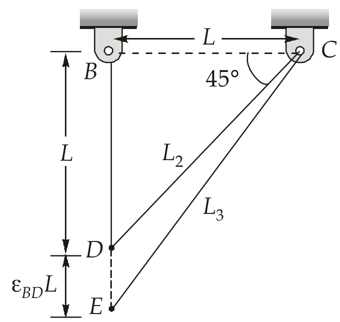
Q.6 (b) Solution:

(i)

Given: $E_{Al} = 70 \times 10^3$ MPa

Strain in BD, $\varepsilon_{BD} = 0.0814$

From geometry,



Let original length of BD be L.

$$\text{Elongation in } BD,\ DE = \in_{BD} \times L = 0.0814\text{ L}$$

$$\text{Distance, } BC = L\cot 45°$$

$$= L$$

Original length of bar CD,

$$L_2 = L\text{cosec}45°$$

$$= 1.414\text{L}$$

Now, elongated length of bar $BD = L + 0.0814$ L

$$= 1.0814\text{ L}$$

$\therefore$ Length CE i.e. $L_3 = \sqrt{L^2 + (1.0814L)^2} = 1.473$ L

$\therefore$ Elongation in bar CD,

$$\delta_{CD} = L_3 - L_2$$

$$= 1.473L - 1.414L = 0.059L$$

$\therefore$ Strain in bar CD, $\varepsilon_{CD} = \dfrac{\delta}{L_2} = \dfrac{0.059}{1.414} = 0.0417$

Stress in bar CD, $\sigma = E_{Al} \times \in$

$$= 70 \times 10^3 \times 0.0417$$

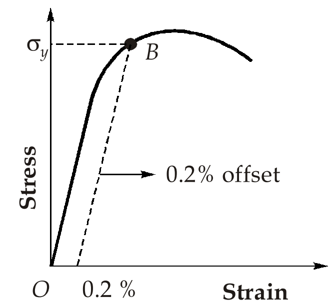
$$= 2919 \text{ MPa} \simeq 2.92 \text{ GPa} \quad \text{(Tensile)}$$

Due to symmetrical arrangement, stress in outer bar AD will also be 2.92 GPa (Tensile).

(ii)

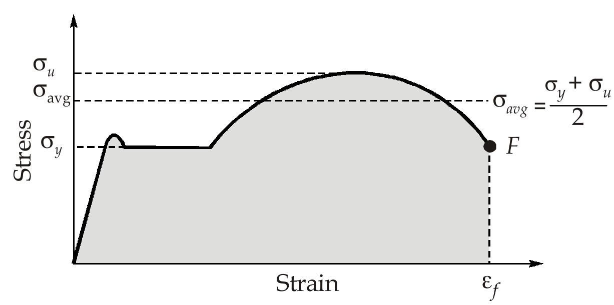
1. **Proof Stress:** Some of the ductile metals like Aluminium (Al), Copper (Cu) and Silver (Ag) do not show clear yield point in tension test. Therefore, their yield stress (σ_y) is not clearly known. For such metals, design stress is calculated by offset method.

   An offset of permanent plastic strain equal to 0.2% is generally is marked on x-axis and a straight line is drawn which is parallel to initial portion of stress-strain curve. The point of intersection of stress-strain curve with this straight line is called proof point and the corresponding stress at that point is called ***proof stress***.



Proof stress

2. **Modulus of Toughness :** It is the property of material which enables it to absorb energy without fracture. This property is very desirable in case of cyclic loading or shock loading. If a material is tough, then it has the ability to store large strain energy before fracture. **Modulus of toughness** is total strain energy per unit volume of material upto fracture stage. It is equal to total area under stress-strain curve upto fracture.



Area under stress strain upto Fracture

$$\text{Modulus of toughness} = \left(\frac{\sigma_y + \sigma_u}{2}\right) \times \epsilon_f$$

where, σ_y = yield tensile strength, σ_u = ultimate tensile strength and ϵ_f = strain at fracture point.

The modulus of toughness depends upon ultimate tensile strength and strain at failure (fracture strain). Hence, the material which is very ductile will exhibit a higher modulus of toughness as is the case with mild steel.

Q.6 (c) Solution:

(i)

Given: $d_1 = 8.8$ cm

$d_2 = 8.85$ cm, $d_3 = 9.1$ cm

For nylon, $E = 2.7$ GPa and $\mu = 0.40$

For space between nylon bar and steel tube to be closed, diameter of nylon bar is to be increased by $(d_2 - d_1)$ by application of compressive force P.

∴ Lateral strain,

$$\in_l = \frac{\Delta d_1}{d_1} = \frac{d_2 - d_1}{d_1} = \frac{8.85 - 8.80}{8.80} = 0.00568$$

Axial strain,

$$\in = \frac{-\in_l}{\mu}$$

$$= \frac{-0.00568}{0.40} = -0.0142$$

∴ Axial stress, $\sigma = E \cdot \in$ [Assuming Hooke's law is valid]

$$= 2.7 \times (-0.0142)$$

$$= -0.03834 \text{ GPa} = -38.34 \text{ N/mm}^2$$

Here. (–ve) sign implies that stress is compressive.

Now, Axial force, $P = \sigma A$

$$= 38.34 \times \frac{\pi}{4} \times (8.8 \times 10)^2 = 233.19 \times 10^3 \text{ N}$$

$$= 233.19 \text{ kN}$$

(ii)

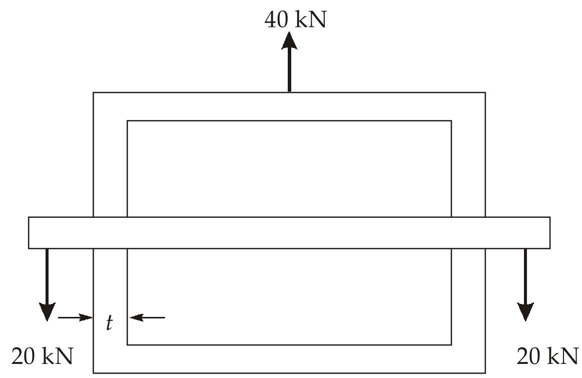
Let the reaction developed at A be R (downwards)

Using equilibrium equation,

$$\Sigma M_B = 0$$

$\Rightarrow$ $R(L) - 20(2L) = 0$

$\Rightarrow$ $R = 40$ kN



where, t is thickness of the beam section.

Now, average shear stress in pin,

$$\tau_{avg} = \frac{40\times10^3}{2\times\left(\frac{\pi}{4}\times(20)^2\right)} = 63.66 \text{ MPa}$$

Average bearing stress on pin,

$$\sigma_b = \frac{40\times10^3}{2\times(dt)} = \frac{40\times10^3}{2\times20\times10} = 100 \text{ MPa}$$

Q.7 (a) Solution:

(i)

The area covered on ground by one photograph,

$$a = \left\{(1-P_L)\times\frac{l}{S}\right\}\times\left\{(1-P_s)\times\frac{w}{S}\right\}$$

Given size of photograph,

$$l \times w = 250 \text{ mm} \times 250 \text{ mm}$$

$$\text{Scale} = \frac{1}{10,000}$$

Longitudinal overlap, P_L = 65%

Side overlap, P_s = 35%

$$a = \left\{(1-0.65)\times\frac{0.25}{\left(\frac{1}{10,000}\right)}\right\}\times\left\{(1-0.35)\times\frac{0.25}{\left(\frac{1}{10,000}\right)}\right\}$$

$$= 1421875 \text{ m}^2 = 1.42 \text{ km}^2$$

Area to be covered, $A = 750\ \text{km}^2$

Hence, number of photographs required,

$$N = \frac{A}{a} = \frac{750}{1.42} = 528.17 \simeq 529$$

(ii)

1. **Various Laws of Weight:**

   (a) The weight of the weighted arithmetic mean of a number of observations is the sum of individual weights of the quantity.

   (b) Weight of algebraic sum of two or more quantities is equal to reciprocal of sum of reciprocal of individual weights.

   (c) When a quantity of a given weight is multiplied by a factor, then the weight of resultant quantity is given by dividing the weight of the quantity by the square of the factor.

   (d) If a quantity is divided by a factor, then the weight of the resultant quantity is obtained by multiplying the given weight by the square of that factor.

   (e) If a quantity is multiplied by its own weight, then the weight of the resultant quantity is equal to the reciprocal of the weight of that quantity.

   (f) The weight of a quantity remains unchanged when all the signs of the terms of equation are changed.

   (g) The weight of an equation remains unchanged when it is added or subtracted from a constant.

2. **Types of errors: The errors are of following types:**

Gross Errors or Mistakes

- These mistakes occur on the part of survey personnel due to lack of experience or carelessness.
- Mistakes, if not detected, can lead to erroneous results thereby making the whole survey as faulty. Adequate check measurements are thus made to detect this type of error.

Systematic or Cumulative Errors

- These errors are called as systematic because they always follow a definite pattern or a mathematical/physical law. These errors are of same magnitude and sign.

  For example: Measuring a length with a steel tape and error involved due to temperature. This is a systematic error because it follows the physical law of expansion of solids on increasing the temperature.

- This type of error makes the result either too large or too small.
- The systematic error can be computed and suitable corrections applied.

Accidental or Random Errors

- This type of error occurs due to human limitation in reading an observation.
- A good thing about accidental errors is that when a large number of observations are made, then they use to cancel out because there is equal probability of the error to be positive or negative. Thus this type of error is also called as **compensating error.**
- But compensating effect of accidental errors is not full proof and there always remains some accidental errors. This error cannot be eliminated altogether from the observations whatever precautions are taken but magnitude of this error is generally very small.
- Smaller the random error, more precise is the measurement. Thus random/accidental errors limit the level of precision while taking an observation.

Q.7 (b) Solution:

Given:

$$\in_x = 450 \times 10^{-6}$$

$$\in_y = 60 \times 10^{-6}$$

$$\gamma_{xy} = 400 \times 10^{-6}$$

(a) At an angle $\theta = 80°$,

$$\in'_x = \frac{\in_x + \in_y}{2} + \frac{\in_x - \in_y}{2}\cos 2\theta + \frac{\gamma_{xy}}{2}\sin 2\theta$$

$$= \left(\frac{450+60}{2}\right)\times 10^{-6} + \left[\left(\frac{450-60}{2}\right)\times \cos 160°\right]\times 10^{-6} + \left(\frac{400}{2}\times \sin 160°\right)\times 10^{-6}$$

$$= 140.16 \times 10^{-6}$$

$\because$ $$\in_x + \in_y = \in'_x + \in'_y$$

$\therefore$ $$\in'_y = \in_x + \in'_y - \in'_x$$

$$= (450 + 60 - 140.16) \times 10^{-6}$$

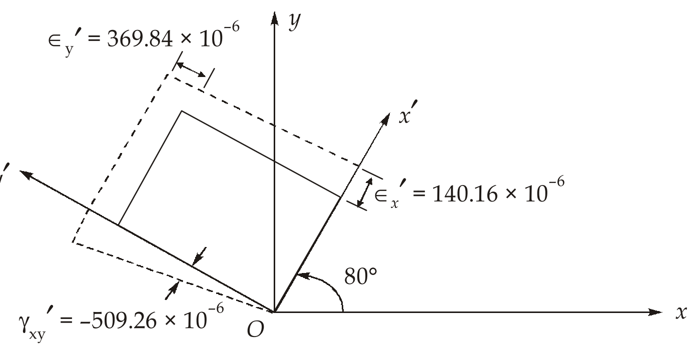
$$= 369.84 \times 10^{-6}$$

Also, $$\frac{\gamma'_{xy}}{2} = -\left(\frac{\in_x - \in_y}{2}\right)\sin 2\theta + \frac{\gamma_{xy}}{2}\cos 2\theta$$

$\Rightarrow$ $$\gamma'_1 = -\left(\frac{450-60}{2}\right)\times \sin 160°\times 10^{-6} + \left(\frac{400}{2}\right)\cos 160°\times 10^{-6}$$

$$\Rightarrow \qquad \gamma'_{xy} = -509.26 \times 10^6$$

Strain element,



(b) Principal strains,

$$\in_{1,2} = \frac{\in_x + \in_y}{2} \pm \sqrt{\left(\frac{\in_x - \in_y}{2}\right)^2 + \left(\frac{\gamma_{xy}}{2}\right)^2}$$

$$= \left[\frac{450+60}{2} \pm \sqrt{\left(\frac{450-60}{2}\right)^2 + \left(\frac{400}{2}\right)^2}\right] \times 10^{-6}$$

$$= [255 \pm 279.33] \times 10^{-6}$$

$$\therefore \qquad \in_1 = 534.33 \times 10^{-6}$$

$$\in_2 = -24.33 \times 10^{-6}$$

Location of principal planes,

$$\tan 2\theta_p = \frac{\gamma_{xy}}{\in_x - \in_y} = \frac{400 \times 10^{-6}}{(450-60)\times 10^{-6}} = 1.0256$$

$$\therefore \qquad \theta_p = \frac{45.72°}{2} \text{ and } \frac{225.72°}{2}$$

$$= 22.86° \text{ and } 112.86°$$

For $\theta_p = 22.86°$

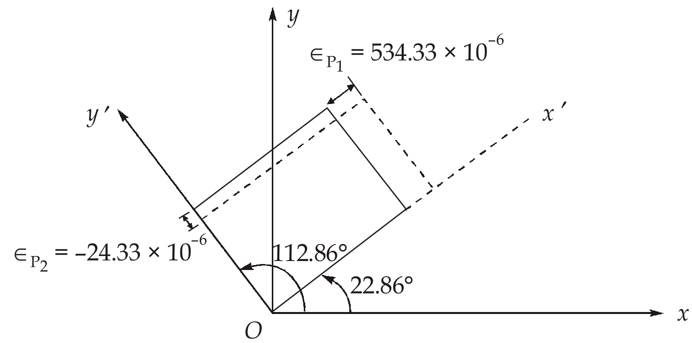
$$\in_{x1} = \frac{\in_x + \in_y}{2} + \frac{\in_x - \in_y}{2}\cos 2\theta + \frac{\gamma_{xy}}{2}\sin 2\theta$$

$$= \left[\frac{450+60}{2} + \frac{450-60}{2} \times \cos 45.72° + \frac{400}{2}\sin 45.72°\right]$$

$$= 534.33 \times 10^{-6}$$

For $\theta_{P1} = 22.86°, \quad \in_{P1} = 534.33 \times 10^{-6}$

and for $\theta_{P2} = 112.86°$, $\in_{P2} = -24.33 \times 10^{-6}$



Q.7 (c) Solution:

(i)

$$a = v_{12} = 37$$
$$b = v_{13} + v_{14} + v_{15} = 303 + 64 + 52 = 419$$
$$c = v_{32} + v_{42} + v_{52} = 122 + 54 + 132 = 309$$
$$d = v_{53} + v_{54} + v_{43} = 62 + 15 + 18 = 95$$

weaving ratio, p_{1-2} is given by

$$p_{1-2} = \frac{b+c}{a+b+c+d}$$
$$= \frac{419+308}{37+419+308+95} = 0.846$$

Now, practical capacity of rotary is given as

$$Q_p = \frac{280w\left(1+\frac{e}{w}\right)\left(1-\frac{p_{1-2}}{3}\right)}{\left(1+\frac{w}{L}\right)}$$

where, e = average width of carriage way,

$$= \frac{10+10}{2} = 10 \text{ m}$$

Width of weaving section, $w = e + 3.5 = 10 + 3.5 = 13.5$ m

Weaving length, L = 50 m

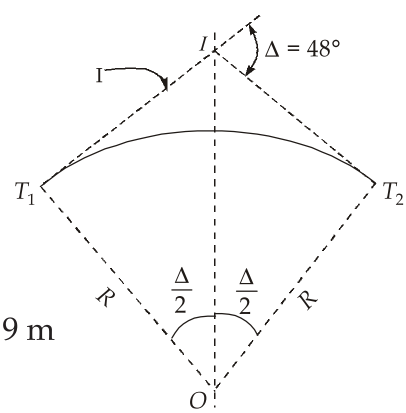
Now, $$Q_p = \frac{280 \times 13.5 \times \left(1+\frac{10}{13.5}\right)\left(1-\frac{0.846}{3}\right)}{\left(1+\frac{13.5}{50}\right)} = 3720 \text{ pcu/hr}$$

(ii)

General guidelines the designing traffic rotary are:

1. Rotaries are suitable when the traffic entering from all the four approaches are relatively equal.
2. A total volume of about 3000 vehicles per hour can be considered as the upper limiting case and a volume of 500 vehicles per hour is the lower limit.
3. A rotary is very beneficial when the proportion of the right turning traffic is very high, typically if it is more than 30 percent.
4. Rotaries are suitable when there are more than four approaches or if there are no separate lanes available for right turning traffic. Rotaries are ideally suited if the intersection geometry is complex.

Q.8 (a) Solution:



Given Deflection angle, $\Delta = 48°$

Radius of curve, $R = 380$ m

$$\text{Tangent length, } IT_1 = R\tan\frac{\Delta}{2}$$

$$= 380\times\tan\left(\frac{48°}{2}\right) = 169.19 \text{ m}$$

Chainage of intersection point, I

$$= (40\times30)+\left(60\times\frac{30}{100}\right) = 1218 \text{ m}$$

$$\text{Chainage of } T_1 = 1218 - 169.19 = 1048.81 \text{ m}$$

$$\text{Length of curve} = R\times\left(\Delta\times\frac{\pi}{180°}\right)$$

$$= 380\times\left(\frac{48°\times\pi}{180°}\right) = 318.35 \text{ m}$$

$$\text{Chainage of } T_2 = \text{Chainage of } T_1 + \text{Length of curve}$$

$$= 1048.81 + 318.35 = 1367.16 \text{ m}$$

Length of chords:

Length of first sub chord, $C_1 = 1050 - 1048.81 = 1.19$ m

Length of last sub chord, $C_n = 1367.16 - 1350 = 17.16$ m

In between first and last sub chord,

$$\text{Number of chords, } n = \frac{318.35 - C_1 - C_n}{30}$$

$$= \frac{318.35 - 1.19 - 17.16}{30} = 10$$

There are in total 10 + 2 chords i.e. 12 chords.

Calculation of offset:

$$O_1 = \frac{C_1^2}{2R} = \frac{1.19^2}{2 \times 380} = 0.0019 \text{ m}$$

$$O_2 = \frac{C_2(C_1 + C_2)}{2R} = \frac{30 \times (1.19 + 30)}{2 \times 380} = 1.231 \text{ m}$$

$$O_3 = \frac{C_3(C_2 + C_3)}{2R} = \frac{C^2}{R} = \frac{30^2}{380} = 2.368 \text{ m} \qquad [c_2 = c_3]$$

Now,

$$O_3 = O_4 = O_5 = O_6 = O_7 = O_8 = O_9 = O_{10} = O_{11} = O_3$$

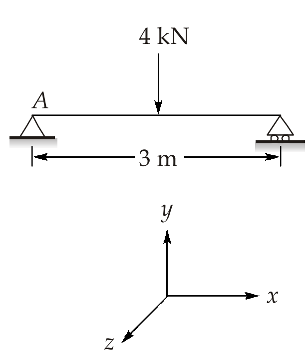
$$= 2.368 \text{ m}$$

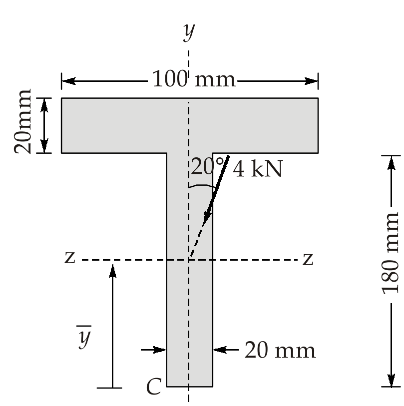
$$O_{12} = \frac{C_{12} \times (C_{11} + C_{12})}{2R}$$

$$= \frac{17.16 \times (30 + 17.16)}{2 \times 380} = 1.065 \text{ m}$$

Q.8 (b) Solution:

(i)





Maximum bending moment in z-direction,

$$M_z = \frac{(P\cos\theta)L}{4}$$

$$= \frac{(4\sin 20°)\times 3}{4} = 2.82 \text{ kN-m}$$

Maximum bending moment in y-direction,

$$M_y = \frac{(4\cos 20°)\times 3}{4} = 1.026 \text{ kN-m}$$

Location of neutral axis from bottom,

$$\bar{y} = \frac{100\times 20\times 190 + 20\times 180\times 90}{100\times 20 + 20\times 180}$$

$$= 125.71 \text{ mm}$$

Now, $$I_{zz} = \frac{100\times 20^3}{12} + 100\times 20\times(190-\bar{y})^2 + \frac{20\times 180^3}{12} + 20\times 180\times(\bar{y}-90)^2$$

$$= 66{,}666.67 + 8266408.2 + 9720000 + 4590734.76$$

$$= 2.26\times 10^7 \text{ mm}^4$$

$$I_{yy} = \frac{180\times 20^3}{12} + \frac{20\times 100^3}{12} = 1.79\times 10^6 \text{ mm}^4$$

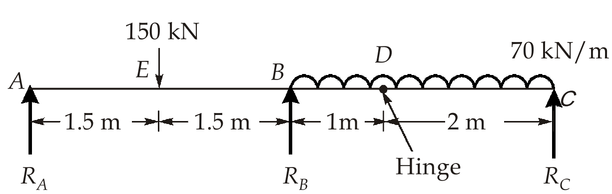
Now, maximum tensile stress will be at point C and its value is given as,

$$\sigma = \frac{M_z}{I_{zz}}\times\bar{y} + \frac{M_y}{I_{yy}}\times z$$

$$= \frac{2.82\times 10^6}{2.26\times 10^7}\times 125.71 + \frac{1.026\times 10^6}{1.79\times 10^6}\times 10$$

$$= 21.42 \text{ N/mm}^2$$

(ii)



As D is hinged

$\therefore$ $$M_D = 0$$

For beam DC,

$$R_C(2) - 70 \times 2 \times \frac{2}{2} = 0$$

$\Rightarrow$ $R_C = 70 \text{ kN } (\uparrow)$

For beam *AD*,

$$R_A(4) - 150(2.5) + R_B(1) - 70 \times 1 \times 1 \times \frac{1}{2} = 0$$

$\Rightarrow$ $4R_A + R_B = 410$ ...(1)

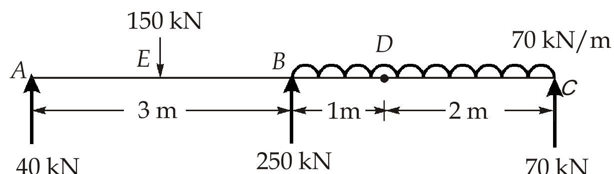
Also, $\Sigma F_y = 0$

$\Rightarrow$ $R_A + R_B + R_C = 150 + 70(3)$

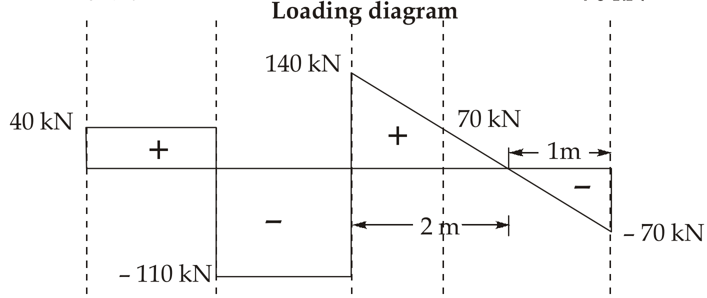
$\Rightarrow$ $R_A + R_B = 290$ ...(2) $[\because R_C = 70 \text{ kN}]$

From (1) and (2),

$$R_A = 40 \text{ kN } (\uparrow) \text{ and } R_B = 250 \text{ kN } (\uparrow)$$



Loading diagram



Shear force diagram

At location of hinge,

$$SF_D = -70 + 70(2) = 70 \text{ kN}$$

$\therefore$ $SF_A = R_A = 40 \text{ kN}$

$$SF_{-B} = R_A - 150 = -110 \text{ kN}$$

$$SF_{+B} = R_A - 150 + R_B = +140 \text{ kN}$$

$$SF_{-E} = R_A = 40 \text{ kN}$$

$$SF_{+E} = R_A - 150 = -110 \text{ kN}$$

$$SF_D = 70 \text{ kN}$$

$$SF_{-C} = R_C = 70 \text{ kN}$$

Q.8 (c) Solution:

(i)

Radius of relative stiffness (l) is given by

$$l = \left[\frac{Eh^3}{12k(1-\mu^2)}\right]^{1/4}$$

$$= \left[\frac{3\times10^5\times(20)^3}{12\times6\times(1-0.15^2)}\right]^{1/4} = 76.42 \text{ cm}$$

Now, $$\frac{a}{h} = \frac{15}{20} = 0.75 < 1.724$$

∴ $$b = \sqrt{1.6a^2+h^2} - 0.675h$$

Equivalent radius of resisting section,

$$= \sqrt{1.6\times(15)^2+(20)^2} - 0.675\times20 = 14.1 \text{ cm}$$

(a) Edge load stress using equation by Teller and Sutherland.

$$S_e = 0.529\frac{P}{h^2}(1+0.54\mu)\times\left[4\log_{10}\left(\frac{l}{b}\right)+\log_{10}(b)-0.408\right]$$

$$= 0.529\times\frac{5000}{(20)^2}(1+0.54\times0.15)\times\left[4\log_{10}\left(\frac{76.42}{14.1}\right)+\log_{10}(14.1)-0.408\right]$$

$$= 26.28 \text{ kg/cm}^2$$

(b) Corner load stress using equation by Kelley

$$S_c = \frac{3P}{h^2}\left[1-\left(\frac{a\sqrt{2}}{l}\right)^{1.2}\right]$$

$$= \frac{3\times5000}{(20)^2}\left[1-\left(\frac{15\times\sqrt{2}}{76.42}\right)^{1.2}\right] = 29.44 \text{ kg/cm}^2$$

(ii)

The construction of WBM roads may be divided into following steps:

1. **Preparation of foundation for receiving the WBM course**: The foundation layer i.e. subgrade, sub base or base course is prepared to required grade and camber on existing road surfaces, the depressions and pot holes are filled and

corrugation are removed by scarifying and reshaping the surface to the required grade and camber.

2. **Provision of lateral confinement**: It may be done by constructing the shoulders in advance to a thickness equal to that of compacted WBM layer.
3. **Spreading of coarse aggregate**: The coarse aggregates are spread uniformly to proper profile to even thickness upon the prepared foundation.
4. **Rolling**: Rolling is started from the edges and then gradually shifted towards the centreline of the road.
5. **Application of screenings**: After the coarse aggregates are rolled adequately, the dry screenings are applied gradually over the surface to fill the interstices in three or more applications.
6. **Sprinkling and Grouting**: After screening, the surface is sprinkled with water, swept and rolled.
7. **Application of binding material**: Binding material is applied due avoid loosening of particles of surface.
8. **Setting and drying**: The WBM road is then allowed to dry for proper setting of wearing course.

■■■■