NAME:

ROLL NO:

TEST :

CIVIL ENG4

DATE:

## Section-A

Q-1-a) Classification of Glass

1) Crown Glass : It is an ordinary type of glass used in interior panelling and framing in buildings.

Composition: Consists of minor percentage of soda and silica with carbon content varying from 22 - 30%.

2) Flint Glass: Relatively superior to the crown glass and can be used in interior panelling as well as on exterior walls due to its better heat resistance and thermal insulation

Composition: Percentage of soda ash in range of 30-35%. and silica in minor proportion.

3) Pyrex Glass: It is the most superior variety of glass available in the market and possesses excellent thermal insulation, heat resistance and also resistant to weathering effects.

Used mainly in exterior wall framings.

Composition: Percentage of ~~silicat~~ silica in the range of (55-65%) with soda ash and small percentage of carbon and nickel.

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(ii) Properties of glass:

a) Possesses excellent heat resistant and thermal insulating properties.

b) Glass fibres are flexible and strong and can be used in reinforcement in composites.

c) Higher strength to weight ratio.

d) Posses very high melting and boiling points due to strong covalent bonds in them.

e) Glass offers relatively lesser toughness and lower ductility and malleability.

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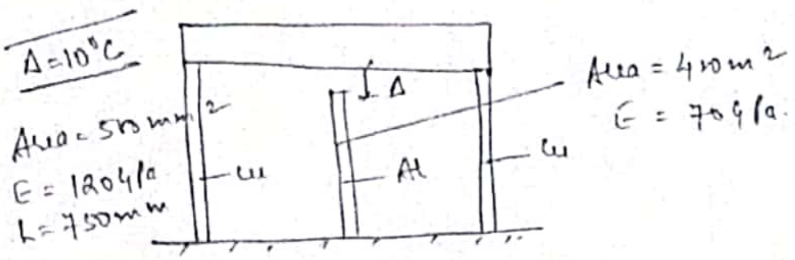
Q-1-b)




Given $\Delta = 0.18mm$.

Sol^n: change in Temp $(\Delta T) = 95-10 = \overline{85°C}$

$\underline{\text{strain prevented in copper rod}} = \left(\frac{L_{cu}\alpha_{cu}\Delta T}{L_{cu}}\right) = (16.8\times10^{-6}\times 85) = \underline{1.428\times10^{-3}}$

$\therefore$ stress induced in copper rod $= (\sigma_{cu}) = (E\epsilon_{cu})$

$\therefore \sigma_{cu} = 120\times10^{3}\times1.428\times10^{-3} = \underline{171.36\ N/mm^2}$

Free expansion of Al rod allowed $= 0.18mm$

Total Temp elongation of Al rod $= (750-0.18)\times 23.1\times10^{-6}\times 85 = \underline{1.47223mm}$

$\therefore$ strain prevented $= \frac{1.47223-0.18}{(750-0.18)}$ $\underline{2.0547\times10^{-3}}$

$\Rightarrow$ from rigid bar configuration

$$\left(L\alpha\Delta t + \frac{\sigma L}{E}\right)_{cu} = \left(L\alpha\Delta t - \frac{\sigma L}{E}\right)_{Al} - 0.18$$

$$0.18 + \left(750\times16.8\times10^{-6}\times85 + \frac{\sigma_{cu}\times750}{120\times10^{3}}\right) = \left[(750-0.18)\times23.1\times10^{-6}\times85 - \frac{\sigma_{Al}\times(750-0.18)}{70\times10^{3}}\right]$$

$$1.251 + 6.25\times10^{-3}\sigma_{cu} = 1.4723 - 0.01071\sigma_{Al}$$

$$\therefore 6.25\times10^{-3}\sigma_{cu} + 0.01071\sigma_{Al} = 0.2213 \quad —(1)$$

from $\Sigma y = 0$

$$2\times\sigma_{cu}\times A_{cu} = \sigma_{Al}\times A_{Al}$$

$$\Rightarrow 2\times\sigma_{cu}\times500 = \sigma_{Al}\times400$$

$$\sigma_{cu} = 0.4\,\sigma_{Al} \quad —(2)$$

from eq (1) and (2)

$\sigma_{cu} = 6.7\ N/mm^2$ (T) in each copper rod.

$\sigma_{Al} = 16.75\ N/mm^2$ (C)

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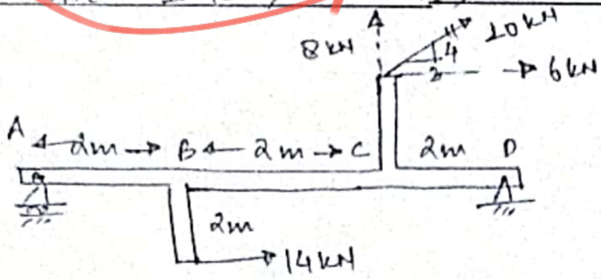
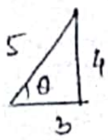
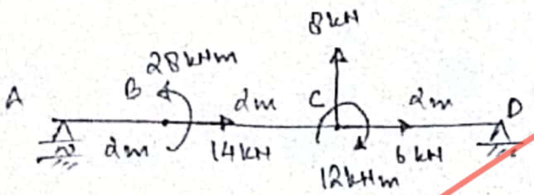
Q-1-c)




$\theta = 53.13^\circ$

Soln: (1) Resolving force on beam



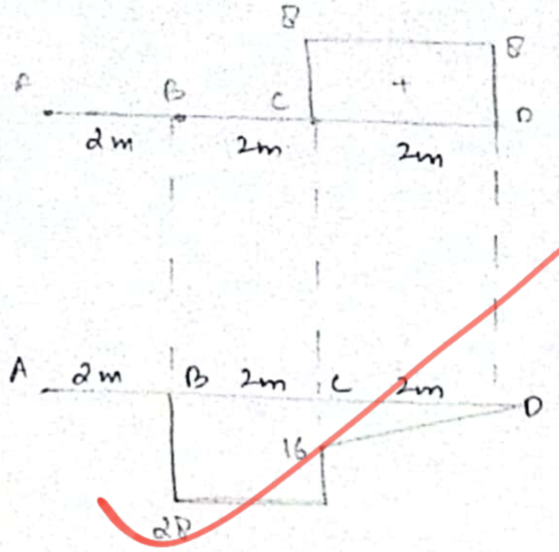

(2) Reactions

$$V_A + V_D + 8 = 0$$

$$\Sigma M_D = 0 \quad V_A\times6 - 28 + 8\times2 + 12 = 0$$

$$\Rightarrow V_A = 0 \text{ and } V_D = 8\,kN\,(\downarrow)$$

③ SFD

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Q-1-d)

$W_{auto} = 210\,N$

$\Delta = 0.25\,m$

$f = 0.5\,N$

$v = 50\,mm/s$

Soln:

force carried by each spring $= \dfrac{210}{4} = 50\,N$

$\therefore (k)_{spring} = \left(\dfrac{50}{0.25}\right) = 200\,N/m$

$\therefore k_{eq} = 4 \times k = 800\,N/m$ (parallel arrangement of springs)

$\therefore C_{cr} = 2m\sqrt{\dfrac{k_{eq}}{m}} = 2\sqrt{k_{eq}\,m} = 2\sqrt{800 \times \left(\dfrac{210}{9.81}\right)} = 255.42\,Ns/m$.

Since $F$ (damping force) $= c \cdot \dot{x}$

$4 \times 0.5 = c \times (50 \times 10^{-3})$

$\Rightarrow \overline{c = 10\,Ns/m \times 4} = \overline{40\,Ns/m}$

$\therefore$ damping ratio $(\xi) = \left(\dfrac{c}{c_{cr}}\right) = \left(\dfrac{40}{255.42}\right) = \overline{0.1566}$

Transmissibility $(T_r) = \dfrac{x(T)}{x(s)} = \dfrac{\sqrt{1+(2\eta\xi)^2}}{\sqrt{(1-\eta^2)^2 + (2\eta\xi)^2}}$

at Resonance $(\eta = 1)$

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$$\therefore \frac{x(T)}{x(L)} = \frac{\sqrt{1+(2\times1\times0.1566)^2}}{\sqrt{(2\times1\times0.1566)^2}}$$

$$\therefore x(T) = x(L) \times 3.3457$$

$$\therefore x(T) = 10 \times 3.3457 = \underline{\overline{33.457mm}} \text{ Ans.}$$

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Q-1-c) <u>Properties of Aluminium :</u>

1) Aluminium possess sufficient ductility and malleability, so that it can be drawn into required forms easily.

2) Aluminium posses high tensile strength and can be used at locations where large amount of stresses are generated.

3) Due to its ~~high~~ high MP, it does not melt when subjected to high Temperatures.

4) High hardness and high wear resistance.

<u>Aluminium as structural Material :</u>

a) Due to its high <u>strength to weight</u> ratio it can be used to erect high rise structures.

b) It can be used in <u>plastic design</u> of frames.

c) It can be easily fabricated and moulded to different shapes as needed in different structural locations.

d) Due to its simplicity in design and ease of erection, it is preffered mostly as substitute for steel.

e) It offers economy in construction.

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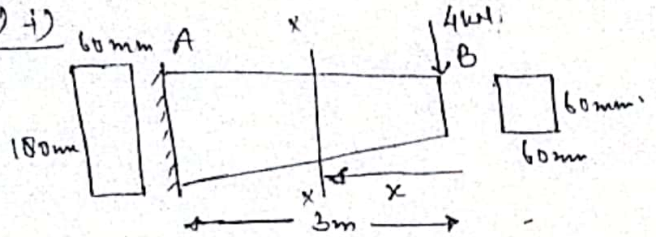
Q-3-a) i)




Soln: Let us consider a section say (x-x) at 'x' m from free end.

$\therefore$ BM @ (x-x) = $(-4x)$

depth @ (x-x) $= 60 + \left(\frac{180-60}{3}\right) \times x = \underline{60 + kx}$ where ($k = 40$ mm/m)

$I_{xx} = \left(\frac{b \times d_x^3}{12}\right)$

$\therefore$ bending stress @ x-x $= \left(\frac{M_x}{I_{xx}} \times y\right) = \frac{(4x) \times 12}{b \times d_x^3} \times \left(\frac{d_x}{2}\right)$

$$\sigma_{xx} = \frac{4x \times 6}{b d_x^2} = \frac{4x \times 6}{b \times (60+kx)^2}$$

For Maximum bending stress $\left(\frac{d\sigma_{xx}}{dx} = 0\right)$

$$\therefore \frac{d\sigma_{xx}}{dx} = \frac{d}{dx}\left[\frac{\cancel{4 \times 6}\ 24x}{b(60+kx)^2}\right] = 0$$

$$= \frac{24}{b}\left[\frac{(60+kx)^2 \cdot 1 - x \times 2(60+kx)k}{(60+kx)^4}\right] = 0$$

$$\Rightarrow (60+kx)^{\cancel{2}} = 2x\,(\cancel{60+kx})\,k$$

$$60 + kx = 2kx$$

$$\Rightarrow kx = 60$$

$$x = \left(\frac{60}{k}\right) = \left(\frac{60}{40}\right) = \underline{1.5\text{m}} \text{ (at centre)} \quad \text{(Ans)}$$

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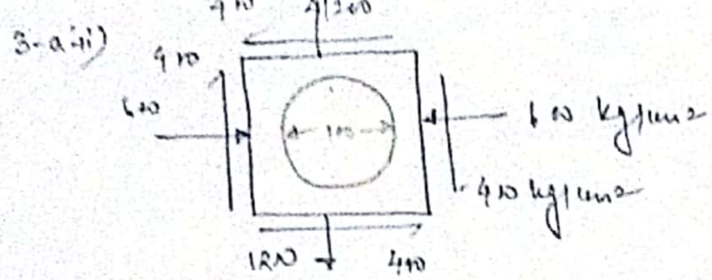
$$\therefore (\sigma_{xx})_{max} = \frac{(4x) \times 10^6 \times 6}{60 \times (120)^2}$$

and $d_x$ @ $x = 1.5$m

$d_x = 60 + 40 \times 1.5 = 120$ mm

$$= \frac{4 \times 1.5 \times 10^6 \times 6}{60 \times 120^2} = \underline{41.67 \text{ N/mm}^2} \quad \text{(Ans)}$$




Soln:

$\sigma_x = -610 \text{ kg/cm}^2$

$\tau_{xy} = -410 \text{ kg/cm}^2 = \tau_{yx}$

$\sigma_y = 1210 \text{ kg/cm}^2$

∴ Principal stresses

$$(\sigma_{p_1}/\sigma_{p_2}) = \frac{\sigma_x+\sigma_y}{2} \pm \frac{1}{2}\sqrt{(\sigma_x-\sigma_y)^2+4\tau_{xy}^2}$$

$$= \frac{-610+1210}{2} \pm \frac{1}{2}\sqrt{(-610-1210)^2+4\times 410^2}$$

$$(\sigma_{p_1}/\sigma_{p_2}) = (300 \pm 984.885)$$

∴ $\sigma_{p_1} = 1284.885 \text{ kg/cm}^2$ $\quad \sigma_{p_2} = -684.885 \text{ kg/cm}^2$

and $\tan 2\theta_p = \frac{2\tau_{xy}}{(\sigma_x-\sigma_y)} = \frac{-2\times 410}{-610-1210} = 0.444$

$\Rightarrow \theta_{p_1} = \overline{\underline{11.98^\circ}} \quad \theta_{p_2} = \overline{\underline{101.98^\circ}}$

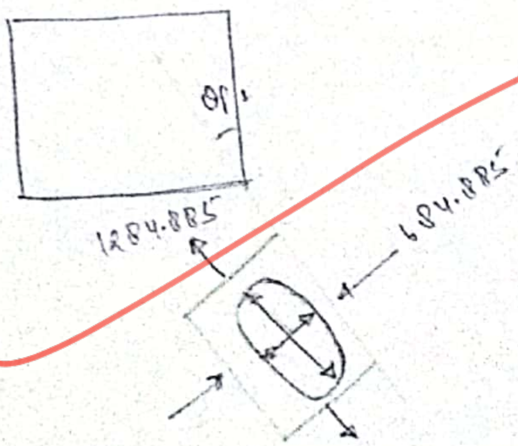
Check @ $\theta_{p_1} = 11.98^\circ$

$$\sigma_x' = \sigma_x \cos^2\theta + \sigma_y \sin^2\theta + 2\tau_{xy}\cos\theta\sin\theta$$

$$= -610\cos(11.98)^2 + 1210\sin(11.98)^2 + 2\times(-410)\cos(11.98)\sin(11.98)$$

$$= -684.885 \text{ kg/cm}^2$$

Sketch:




DIRECTIONS.

[Major axis along $\sigma_{p_2}$
i.e $\theta_{p_2} = 101.98^\circ$

and minor axis along $\sigma_{p_1}$
i.e $\theta_{p_1} = 11.98^\circ$
with vertical]

$$\therefore \epsilon_{p_1} = -\frac{\sigma_{p_1}}{E} - \mu\frac{\sigma_{p_2}}{E} = -\frac{684.885}{2.1\times10^6} - \frac{0.28\times1284.885}{2.1\times10^6} = -4.974\times10^{-4}$$

$$\epsilon_{p_2} = \frac{\sigma_{p_2}}{E} - \mu\left(-\frac{\sigma_{p_1}}{E}\right) = \frac{1284.885}{2.1\times10^6} + \frac{0.28\times684.885}{2.1\times10^6} = 7.031\times10^{-4}$$

$\therefore$ Major axis of Ellipse $= (7.031\times10^{-4}\times100 + 100) = \underline{100.0703\text{ mm}}$

Minor axis of Ellipse $= (-4.794\times10^{-4}\times100 + 100) = \underline{99.952\text{ mm}}$

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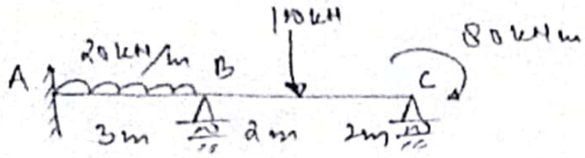
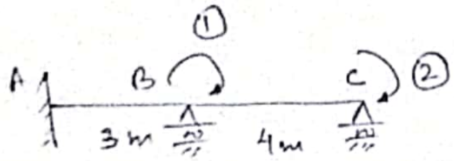
Q-3-b)




Soln: ① $D_k = 2\ (\theta_B, \theta_C)$

② Released structure (coordinate system)




③ FEM's.

$M_{FAB} = -\frac{wl^2}{12} = -\frac{20\times3^2}{12} = -15\text{ kNm}$, $M_{FBA} = 15\text{ kNm}$.

$M_{FBC} = -\frac{Wl}{8} = -\frac{100\times4}{8} = -50\text{ kNm}$, $M_{FCB} = 50\text{ kNm}$

④ Stiffness Matrix:

column 1:

$K_{11} = \frac{4EI}{3} + \frac{4EI}{4} = \frac{7EI}{3}$

$K_{21} = K_{12} = \frac{2EI}{4} = 0.5EI$

column 2:

$K_{12} = K_{21} = \frac{0.5EI}{l}$

$K_{22} = \frac{4EI}{4} = EI$

$$\therefore [K] = EI\begin{bmatrix}\frac{7}{3} & 0.5\\ 0.5 & 1\end{bmatrix}$$

④ Analysis:

$$\begin{bmatrix}\theta_B\\ \theta_C\end{bmatrix} = [K]^{-1}\left\{\begin{bmatrix}M_{final}\\ f_{final}\end{bmatrix} - \begin{bmatrix}M_{fixed}\\ f_{fixed}\end{bmatrix}\right\}$$

$M_{f_B} = -50+15 = -35$ kN/m

$M_{f_C} = 50$ kN/m

$$\therefore \begin{bmatrix}\theta_B\\ \theta_C\end{bmatrix} = \frac{1}{EI}\begin{bmatrix}\frac{7}{3} & 0.5\\ 0.5 & 1\end{bmatrix}^{-1}\left\{\begin{bmatrix}0\\ 80\end{bmatrix} - \begin{bmatrix}-35\\ 50\end{bmatrix}\right\}$$

$$\begin{bmatrix}\theta_B\\ \theta_C\end{bmatrix} = \frac{1}{EI}\begin{bmatrix}1 & -0.5\\ -0.5 & \frac{7}{3}\end{bmatrix}\begin{bmatrix}35\\ 30\end{bmatrix} \times \left(\frac{1}{2.083}\right)$$

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$\therefore \theta_B = \left(\frac{20}{EI}\right)$ , $\theta_C = \left(\frac{52.5}{EI}\right)$

⑤ from slope deflection Eqn:

$M_{AB} = -15 + \frac{2EI}{3}(2\theta_A + \overset{\to 0}{\theta_B} - 0) = (-15 + 13.33) = -1.67$ kN/m.

$M_{BA} = 15 + \frac{2EI}{3}(2\theta_B) = 41.67$ kN/m.

$M_{BC} = -50 + \frac{2EI}{4}(2\theta_B + \theta_C) = -3.75$ kN/m

$M_{CD} = 50 + \frac{2EI}{4}(2\theta_C + \theta_B) = 112.5$ kN/m

$\therefore \theta_B = \left(\frac{9.6}{EI}\right)$ and $\theta_C = \left(\frac{25.2}{EI}\right)$

⑥ from slope deflection Eqns:

Final end Moments

$M_{AB} = -15 + \frac{2EI}{3}(\theta_B) = -8.6$ kN/m.

$M_{BA} = 15 + \frac{2EI}{3}(2\theta_B) = 27.8$ kN/m

$M_{BC} = -50 + \frac{2EI}{4}(2\theta_B + \theta_C) = -27.8$ kN/m.

$M_{CD} = 50 + \frac{2EI}{4}(2\theta_C + \theta_B) = 80$ kN/m.

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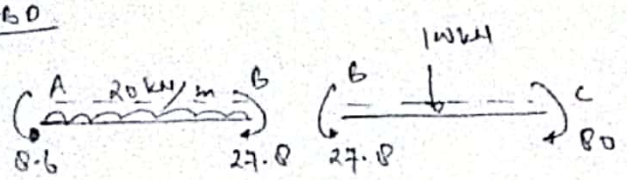
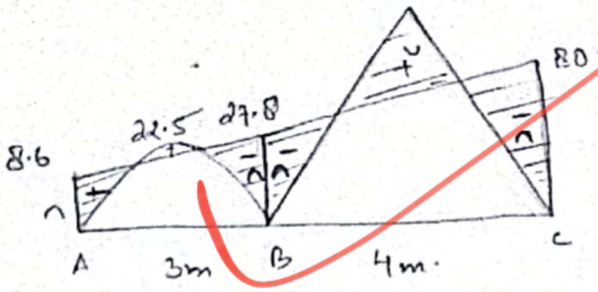
⑥ FBD




⑦ BMD

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Q-3-c) Mortars are classified according to its properties as:

1) On The Basis of Unit Weight

a) Light Weight Mortars: The unit weight of such mortar varies from (12-15 kN/m³) and is generally used in constructions where dead loads desired are very less.

b) Moderately heavy Mortars: The unit weight of these mortars varies in the range of (15-24) kN/m³ and is generally used in ordinary engineering and construction works.

c) Heavy Mortar: Unit ~~wea~~ weight of such mortars are greater than 24 kN/m³ and is used in heavy weight structures such as dams, foundations etc.

2) On The Basis of Binding Material

a) Lime Mortar: Binding material here is lime with suitable proportions of fine aggregates and water.

b) Cement Mortar: Binding material here is cement (portland cement) of varying grades.

c) Gypsum Mortar: If the binding material used is gypsum, mortar is termed as 'gypsum mortar'.

d) Gauged Mortar: Mixture of (lime and cement) in suitable proportions along with adequate proportions of water and aggregates forms gauged mortar.

These mortars possesses excellent workability and also economy in construction is achieved.

3) On The Basis of Adulterants:

a) Sand Mortar: If the adulterant i.e fine aggregates used is sand along with binder.

b) Surkhi Mortar: If the wastes from bricks or powdered bricks are used as adulterants.

4) On The Basis of Properties being Imparted

a) Fire Resistant Mortars: Pozzolana cements along with surkhi in ~~to~~ adequate proportions can develop excellent resistance to heat.

b) Heat Absorbent Mortars: High Alumina cement as binder material is capable of withstanding extreme Temperatures.

c) Sound Absorbent Mortar: Rice husk ash, saw dust used as aggregates have excellent sound damping properties.

(3) On the Basis of Strength Impacted

| Type | strength ($N/mm^2$) | |
|---|---|---|
| $L_1$ | 0.3 - 0.5 | Light |
| $L_2$ | 0.5 - 0.7 | |
| $M_1$ | ≥ 3 | Medium |
| $M_2$ | 2 - 3 | |
| $M_3$ | 0.7 - 1.5 | |
| $H_1$ | ≥ 10.5 | High |
| $H_2$ | 6.5 - 7.0 | |

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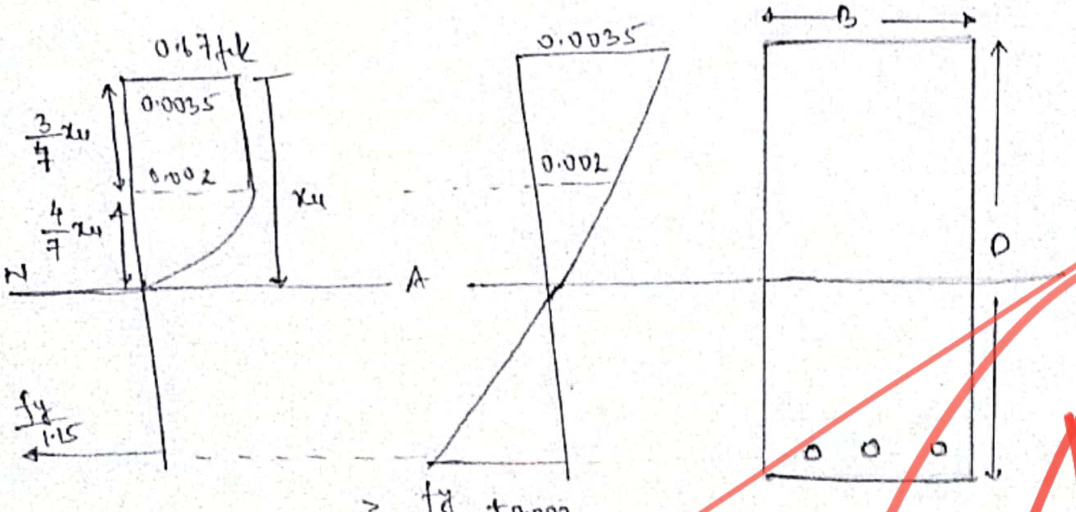
## Section - B

Q-5-a) Stress strain curve

a) concrete and steel




Stress diagram — Strain diagram — Beam ys.

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→ Maxm compressive strain in concrete is assumed as 0.0035 in flexure and maxm stress in concrete as 0.67 fck, in addition an additional partial fos of 1.5 is considered

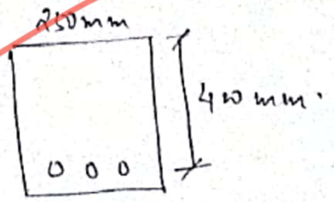
→ Maxm tensile strain in rf shall not be less than $\frac{f_y}{1.15E_s} + 0.002$ where a partial fos = 1.15 is considered for rf.

→ stress diagram is part rectangular upto $(\frac{3}{7}x_u)$ and part parabolic $(\frac{4}{7}x_u)$ and may vary in shape according to the test results as per IS 456:2000.

Q.5 b)

$V_u = 10$ kN  M20

$T_u = 4$ kNm  $p_t(\%) = 0.25\%$.



Soln:

Equivalent shear force $(V_{ue}) = \left(V_u + \frac{1.6 T_u}{B}\right)$

$\therefore V_{ue} = \left(10 + \frac{1.6 \times 4}{0.25}\right) = 35.6$ kN

$\therefore$ Nominal shear stress $(\tau_{ve}) = \left(\frac{V_{ue}}{Bd}\right) = \frac{35.6 \times 10^3}{250 \times 400} = 0.356\ N/mm^2$

and $\tau_c = 0.36\ N/mm^2$ @ $p_t(\%) = 0.25\%$.

and since $(\tau_{ve} - \tau_c) \simeq 0$ (very less)

Therefore design the beam for Minm shear rf

Assuming 2 legged 8mm shear stirrups

$\therefore \frac{A_{sv}}{B S_v} \geq \frac{0.4}{0.87 f_y}$

$\frac{2 \times \frac{\pi}{4} \times 8^2}{250 \times S_v} \geq \frac{0.4}{0.87 \times 415}$

$\therefore S_v \leq 362.96$ mm $\ngtr$ 300 mm (adopt)

$\therefore$ Provide 2 legged 8mm shear stirrups @ 300 mm c/c.

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Since the beam is subjected to Torsion and $D < 450$ mm.

$\therefore$ torsional and side face r/f are required in the beam. (Ans)

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5-(i) $\phi = 20$ mm bolts $\quad f_u = 1040\ N/mm^2 \quad$ 10.9 s grade bolts

threads in shear plane

a) slip not permitted

slip resistance $= \left[\frac{\mu_f\, n_e \times k_h \times f_o}{\gamma_{mf}}\right]$ where $f_o \to$ proof load.

$\therefore f_o = 0.7 \times f_u \times A_{net} = \left(0.7 \times 1040 \times 0.78 \times \frac{\pi}{4} \times 20^2\right) \times 10^{-3} = 178.39$ kN

$\therefore$ slip resistance $= \left(\frac{0.5 \times 1 \times 1 \times 178.39}{1.25}\right) = 71.356$ kN

for four such bolts (strength) $= 4 \times 71.356 = 285.427$ kN (Ans)

b) slip permitted.

Bolts will behave as shear and bearing type

$\therefore$ a) $V_{dsb} = \left(\cancel{\quad} \times \cancel{\frac{\pi}{4} \times 0.78} \times 20^2 \times \frac{1040}{\sqrt{3} \times 1.25}\right) \times 10^{-3} \times 4 =$ ~~423.67 kN~~ 470.83 kN (245)

b) $V_{dpb} = \left(2.5 k_b \times d \times t_{min} \times \frac{f_{up}}{1.25}\right) = \left(2.5 \times 1.0 \times 20 \times 12 \times \frac{410}{1.25} \times 10^{-3} \times 4\right) = 787.2$ kN

$\therefore$ ~~Bolt~~ strength = Minm ( $V_{dsb}$, $V_{dpb}$ ) = ~~423.67 kN~~ ~~(Ans)~~

470.83 kN (Ans)

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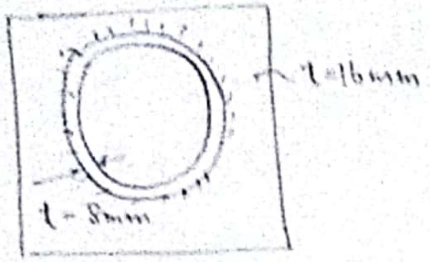
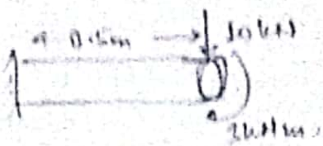
5-d)




Shop weld

T = 3kNm

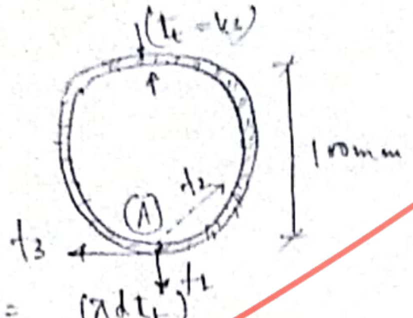



Soln:

Moment @ weld = 10×0.5 = 5kNm

Torsion @ weld = 3kNm

Weld size




Area of weld = $(\pi d t_t)$

$\therefore f_1$ (direct shear stress) $= \left(\frac{10\times10^3}{\pi\times d\times t_t}\right) = \left(\frac{10\times10^3}{\pi\times100\times t}\right) = \left(\frac{31.83}{t}\right)$ N/mm$^2$

$(I_p)_{weld} = \int r^2 dA = \int \left(\frac{d}{2}\right)^2 \times \pi d t = \left(\frac{\pi d^3 t}{4}\right)$

and $I_{xx} = I_{yy} = \left(\frac{\pi d^3 t}{8}\right)$

$\therefore f_2$ (bending stress as shear) $= \frac{5\times10^6\times8}{\pi\times100^3\times t}\times50 = \frac{636.619}{t}$ N/mm$^2$

$f_3$ (torsional shear stress) $= \frac{T}{I_p}\times r = \frac{3\times10^6\times4}{\pi\times100^3\times t}\times50 = \frac{190.986}{t}$ N/mm$^2$

→ In plane shear stresses ($f_1$ and $f_3$) @ A

$\therefore f_R = \sqrt{f_1^2+f_2^2+2f_1f_2\cos 90}$ $\quad$ ($\because \theta = 90°$)

$= \sqrt{\left(\frac{31.83}{t}\right)^2+\left(\frac{190.986}{t}\right)^2} = \frac{193.62}{t}$ N/mm$^2$.

→ Outplane stresses ($f_q$ and $f_2$)

$$\therefore f_e = \sqrt{3f_q^2 + f_2^2} = \sqrt{3\times\left(\frac{193.62}{t}\right)^2 + \left(\frac{636.619}{t}\right)^2} = \frac{719.548}{t}\ N/mm^2$$

∴ <u>for weld to be safe in shear</u>

$$f_e \le \frac{f_u}{\sqrt{3}\times 1.25} \qquad (\gamma_{mw} = 1.25 \text{ for shop weld})$$

$$\therefore \frac{719.548}{t} \le \frac{410}{\sqrt{3}\times 1.25}$$

$$\Rightarrow t \ge 3.7996 \quad \Rightarrow (ks) \ge \cancel{3.7}\ 3.7996$$

{ As $f_u$ is 8N : 2000, $k = 0.7$ }

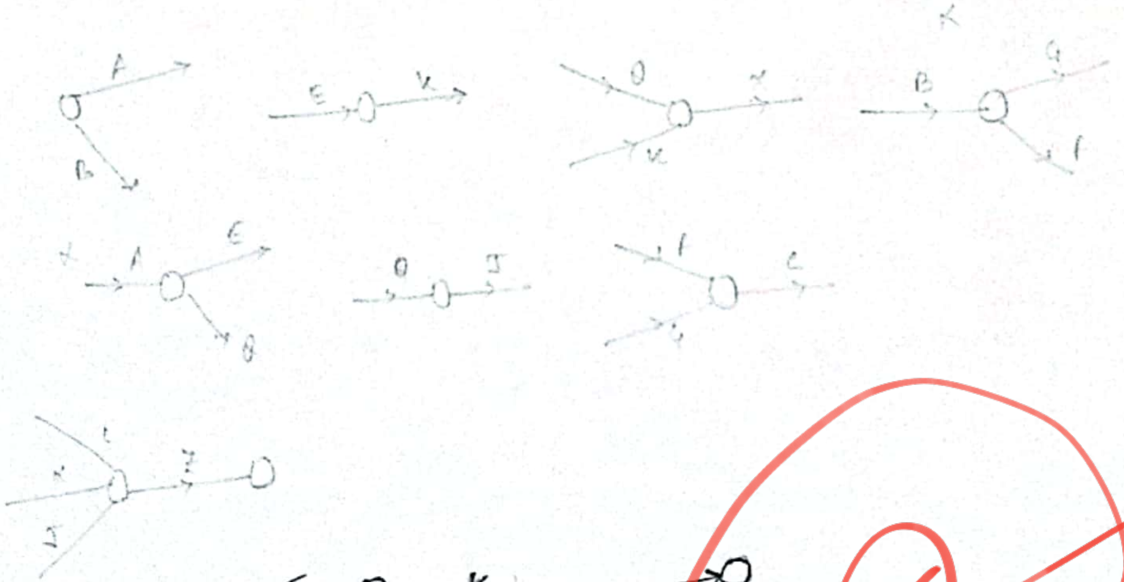
$$\therefore s \ge \left(\frac{3.7996}{0.7}\right) = 5.428 \approx \underline{6mm}$$

∴ Adopt weld size <u>$s = 6mm$</u> Ans.

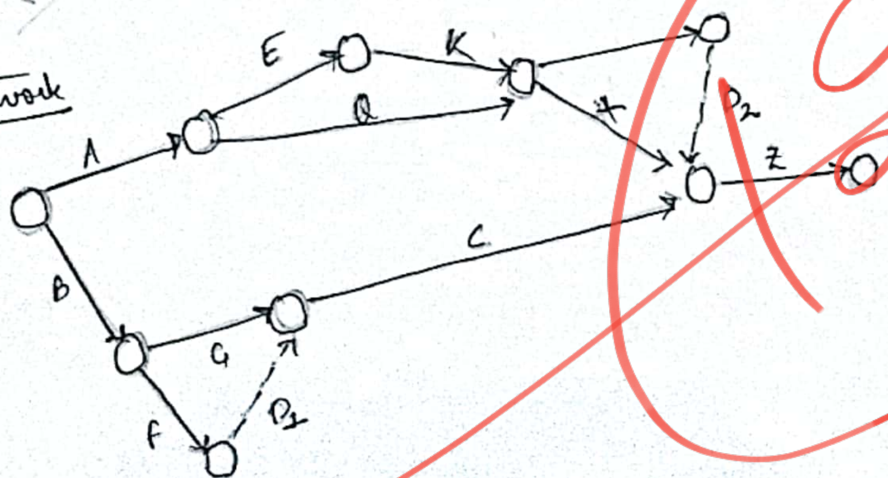
---

5.e)

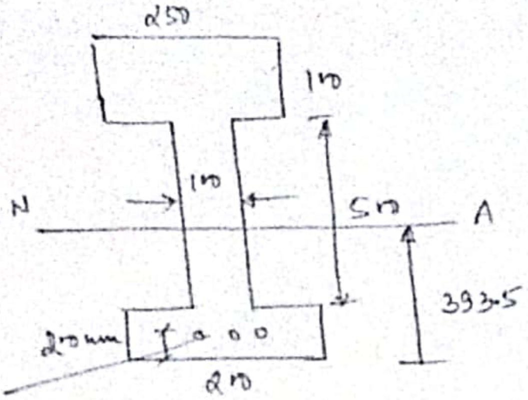
<u>Rough:</u>




<u>Actual Network</u>



Q6.a)




All dim in 'mm'

LL = 20 kN/m.

A = 605 mm$^2$

$f_0$ = 1100 N/mm$^2$

Soln: ① Calcn of loads:

$DL = [0.25\times0.1 + 0.5\times0.1 + 0.2\times0.2]\times25 = \underline{2.875\ kN/m}$

$LL = 20\ kN/m$

$M_d = \frac{W_d l^2}{8} = \frac{2.875\times12^2}{8} = 51.75\ kNm/m$

$M_L = \frac{W_L \times l^2}{8} = \frac{20\times12^2}{8} = 360\ kNm$

At centre.

$P_T = P_0 \times A = 1100\times605\times10^{-3} = 665.5\ kN$

loss = 15%. $\therefore K = 0.85$

$A_{beam} = (250\times100 + 500\times100 + 200^2)$
$= \underline{115000\ mm^2}$

A)

At Transfer stage (@ centre)

At Top

$f_{Top} = \left(\frac{P_T}{A} + \frac{M_d}{I_{xx}}\times y_T - \frac{P_T\times e}{I_{xx}}\times y_T\right)$ where $e = 393.5 - 100 = 293.5$ mm

$f_{Top} = \left[\frac{665.5\times10^3}{115000} + \frac{51.75\times10^6}{13900\times10^6}(800-393.5) - \frac{665.5\times10^3\times293.5}{13900\times10^6}\times(800-393.5)\right]$

$\Rightarrow (5.786 + 1.5134 - 5.4122) = \underline{1.887\ N/mm^2}$

At bottom

$f_{bottom} = \left[\frac{P_T}{A} - \frac{M_d}{I_{xx}}\times y_B + \frac{P_T\times e}{I_{xx}}\times y_B\right]$

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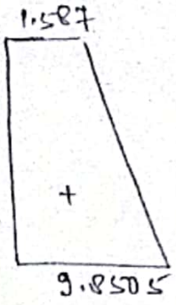
$$\therefore f_{bottom} = \left[\frac{665.5\times10^3}{115000} - \frac{51.75\times10^6}{1390\times10^6}\times 393.5 + \frac{665.5\times10^3\times 293.5}{1390\times10^6}\times 393.5\right]$$

$$= (5.786 - 1.465 + 5.5295) = \underline{9.8505\ N/mm^2}$$

At centre
(Transfer stage)




B) At final stage (considering LL and losses)

at Top

$$f_{top} = \left[\frac{k\times P_T}{A} + \frac{M_d}{I_{xx}}\times y_T + \frac{M_l}{I_{xx}}\times y_T - \frac{kP_T\times e\, y_T}{I_{xx}}\right]$$

$$= \left[\frac{0.85\times665.5\times10^3}{115000} + \frac{(51.75+360)\times10^6}{1390\times10^6}\times(800-393.5)\right) - \frac{0.85\times665.5\times10^3\times293.5\times(800-393.5)}{1390\times10^6}$$

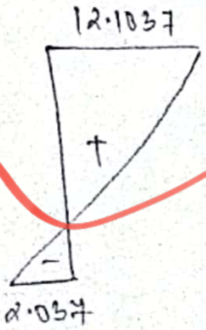
$$= (4.9181 + 12.041 - 4.855) = \underline{12.1037\ N/mm^2}$$

at bottom

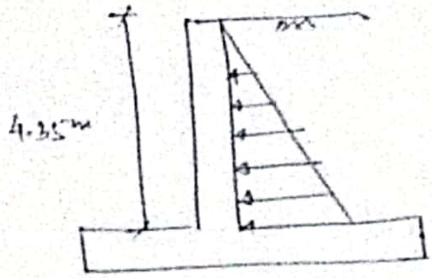
$$f_{bottom} = \left[\frac{kP_T}{A} - \frac{(M_d+M_l)}{I_{xx}}y_B + \frac{kP_T\times e\times y_B}{I_{xx}}\right]$$

$$= (4.9181 - 11.656 + 4.7) = \underline{-2.037\ N/mm^2}$$

At final stage
(@ centre)



Q.6.b)




$\gamma = 18\ kN/m^3$

$k_a = 1/3$

Fe 415

M20

Soln: ① Design for flexure.

a) base pressure @ stem $= (k_a \gamma H) = \left(\frac{1}{3}\times 18\times 4.35\right) = 26.1\ kN/m^2$

b) Maxm factored BM $= \left(\frac{1}{2}k_a \gamma H^2\right)\times\frac{H}{3} = \left(\frac{1}{2}\times\frac{1}{3}\times 18\times 4.35^2\times\frac{4.35}{3}\right) = 82.31\ kNm/m$

$\therefore (BMu) = (82.31\times 1.5) = 123.46\ kNm/m$

c) $d_{req} = \sqrt{\frac{BMu}{QB}} = \sqrt{\frac{123.46\times 10^6}{0.138\times 20\times 1000}} = 211.507\ mm \approx 220\ mm$ (say)

d) $(A_{st})_{req} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6\,BMu}{f_{ck}\,Bd^2}}\right]Bd$

$= \frac{0.5\times 20}{415}\left[1 - \sqrt{1 - \frac{4.6\times 123.46\times 10^6}{20\times 1000\times 220^2}}\right]\times 1000\times 220$

$(A_{st})_{req} = 1893.1035\ mm^2 \approx 1900\ mm^2$

$\therefore$ spacing of 16 mm $\phi$ bars $= \left(\frac{1000}{1900}\times\frac{\pi}{4}\times 16^2\right) = 105.82\ mm \approx 100\ mm$ (say)

② Design for shear

Maxm factored shear $(V_u) = \frac{1}{2}\times 1.5\times k_a \gamma H^2 = 0.5\times 1.5\times\frac{1}{3}\times 18\times 4.35^2 =$ ~~85.15 kN/m~~ $85.15\ kN/m$

Nominal shear stress $(\tau_v) = \left(\frac{V_u}{Bd}\right) = \left(\frac{85.15\times 10^3}{1000\times 220}\right) = 0.387\ N/mm^2$

$P_t(\%) = \left(\frac{A_{st}\ provided}{Bd}\times 100\right) = \left(\frac{1900}{1000\times 220}\times 100\right) = 0.864\ \%$

$\therefore \tau_c$ (from Table) $= 0.56 + \frac{0.62-0.56}{0.25}(0.864-0.75) = 0.587\ N/mm^2$

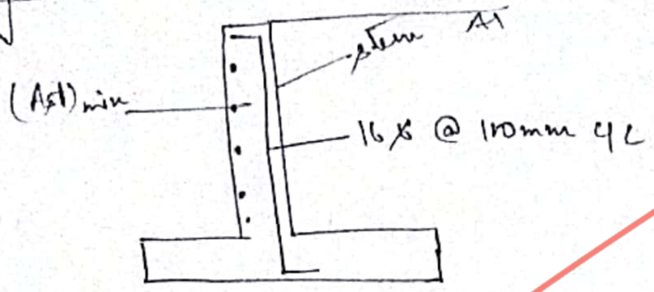
$\because \tau_c \not> \tau_v < (\tau_c)_{max}$ where $(\tau_c)_{max} = 0.625\sqrt{f_{ck}}$

$= 2.795\ N/mm^2$

∴ The cantilever stem slab is safe in shear.

Positions of curtailment: 50% curtailment of 2/3 bars upto depth $\left(\frac{H}{2}\right)$ from Top can be done as moment on the upper half portion is less.

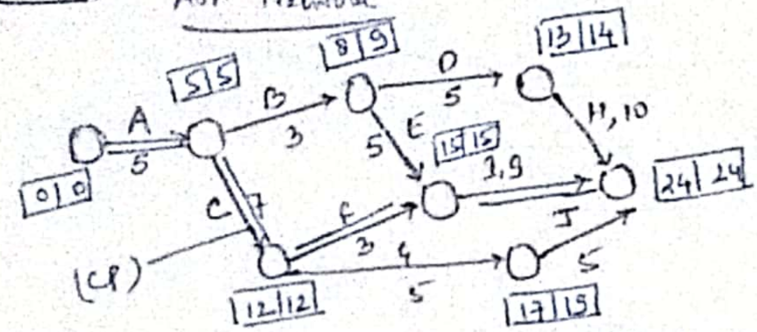
(3) Detailing



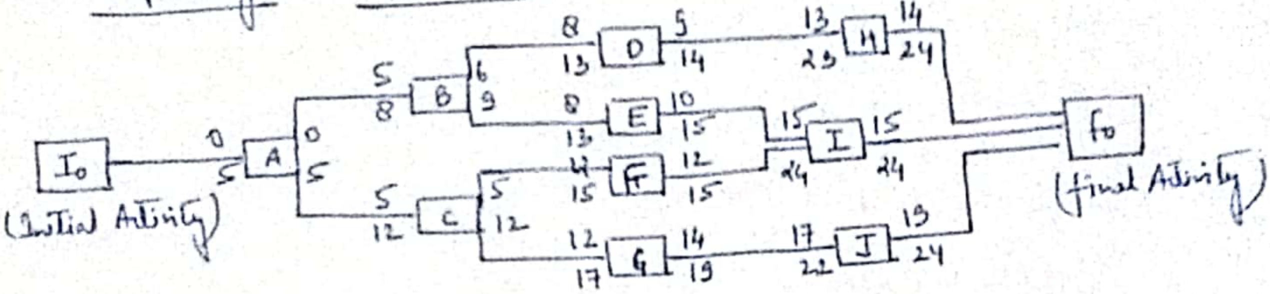

$$Ast_{min} = (0.12\% \text{ of } BD) = \left(\frac{0.12}{100} \times 1000 \times 245\right) \quad \{\text{Assuming eff cover} = 25mm\}$$

$$= \underline{294 mm^2}$$

Q-6-c-i) AoA Network




Soln: Corresponding AoN Network




| Activity | EST | EFT | LST | LFT | TF ( $T_{Lj} - T_{Ei} - t_{ij}$ ) |
|---|---|---|---|---|---|
| A | 0 | 5 | 0 | 5 | 0 |
| B | 5 | 8 | 6 | 9 | 1 |
| C | 5 | 12 | 5 | 12 | 0 |
| D | 8 | 13 | 9 | 14 | 1 |
| E | 8 | 13 | 10 | 15 | 2 |
| F | 12 | 15 | 12 | 15 | 0 |
| G | 12 | 17 | 14 | 19 | 2 |
| H | 13 | 23 | 14 | 24 | 1 |
| I | 15 | 24 | 15 | 24 | 0 |
| J | 17 | 22 | 19 | 24 | 0 2 |

iii) Critical Path : A → C → F — I

Total Project duration = 24 days. Ans

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6-c-ii) Economic life of an equipment: It is defined as the duration or life of an equipment till its financial evaluation takes place or the equipment is still serving its justifying its purpose in terms of economy.

Factors on which Economic life of an Equipment depends are:

a) Physical use of equipment
b) change in Technology
c) change in pattern / fashion
d) Accidents / Errors

Concrete Pumps: These are the mechanical devices that are used for transport of concrete to the location it is needed in the construction.

Various types of concrete pumps used in engg works are.

a) Rotary Pumps
b) Squeeze Pressure Pumps
c) Pneumatic type Pumps

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Q-8-a) Rcc concrete footing design

Thickness of wall = 230 mm

Load (Including self wt) = 200 kN/m.

$q_0 = 150\ kN/m^2$

M 20, Fe 415, $\tau_{bd} = 1.2\ N/mm^2$

Soln: $P_T = 200\ kN/m$ (Assuming 20% wt of soil)

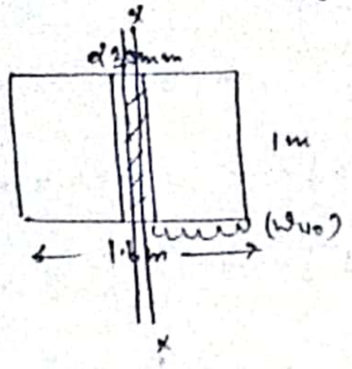
Area of footing required $= \left(\frac{P_T}{q_0}\right) = \left(\frac{200 \times 1.2}{150}\right) = \cancel{1.33\,m^2}\ 1.6m^2$

$\therefore$ Provide footing of size ~~1.45m x 1.25m~~ (say). 1.6m x 1m

∴ factored design soil pressure $(w_{uo}) = \left(1.5 \times \frac{P_T}{A}\right) = \left(\frac{1.5 \times 200}{1.6}\right) = \underline{187.5 \, kN/m^2}$

② Design for flexure.

( Critical section @ midway b/w edge of wall and centre of wall ) — At (x-x)

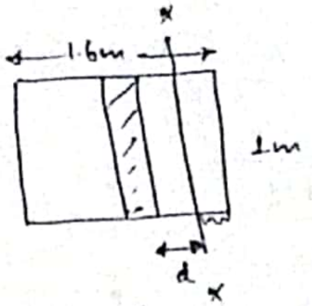



$$\therefore (M_{u_{xx}})_{max\,m} = \frac{W_{uo} \times}{2}\left[\frac{1.6-0.23}{2} + \frac{0.23}{4}\right]^2 \times 1 = \cancel{103.218}\ 51.684 \, kNm/m$$

$$\therefore d_{req} = \sqrt{\frac{BM_u}{QB}} = \sqrt{\frac{51.684 \times 10^6}{0.138 \times 20 \times 1000}} = 136.844 \text{ mm} \approx \text{say } \underline{(140\text{ mm})}$$

③ Design for shear ( 1 way shear )

Critical section @ 'd' from face of wall




$$(V_u)_{xx} = W_{uo}\left[\frac{1.6-0.23}{2} - 0.14\right] \times 1 = \underline{102.1875 \, kN/m}$$

$$\therefore \text{Nominal shear stress } (\tau_v) = \left(\frac{V_u}{Bd}\right) = \left(\frac{102.1875 \times 10^3}{1000 \times 140}\right) = \underline{0.73 \, N/mm^2}$$

$\because$ $\tau_v > \tau_c$

Increase the depth of footing (say) d = 400 mm.

$$\therefore (\tau_v) = \left(\frac{V_u}{Bd}\right) = \left(\frac{102.1875 \times 10^3}{1000 \times 400}\right) = 0.2554 \, N/mm^2 < (\tau_c)$$

(safe in shear)

∴ Adopt $\underline{d = 400\text{ mm}}$

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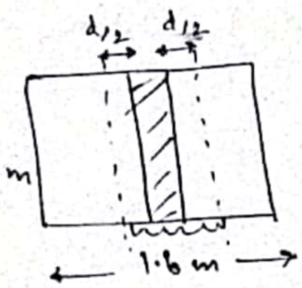
④ check for punching shear

critical section @ '$d/2$' all around wall face



$\therefore (\tau_{vp})_{dev} = \left(\dfrac{\text{Net punching force}}{\text{Resisting } c/s \text{ area}}\right)$

$$= \frac{P_u - W_{u0}[0.23+0.4]\times 1}{d[(0.23+0.4)+1]\times 0.4} = 139.474 \text{ kN/m}^2$$

$= 0.139 \text{ N/mm}^2$

and $(\tau_{vp})_{per} = k_B \times 0.25\sqrt{f_{ck}}$ where $k_b = \left(0.5 + \frac{b}{a} \ngtr 1.0\right)$

$\therefore k_b = 0.5 + \left(\dfrac{0.23}{1}\right) = 0.73 < 1$

$\therefore (\tau_{vp})_{per} = 0.73 \times 0.25\sqrt{20} = 0.816 \text{ N/mm}^2$

$\because (\tau_{vp})_{devp} < (\tau_{vp})_{per}$

$\Rightarrow$ safe in punching shear

⑤ Design of R/f

$$A_{st\,req} = 0.5\frac{f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6\,BM_u}{f_{ck}Bd^2}}\right]Bd$$

$$= \frac{0.5\times 20}{415}\left[1 - \sqrt{1 - \frac{4.6\times 51.684\times 10^6}{20\times 1000\times 400^2}}\right]\times 1000\times 400 = 364.16 \text{ mm}^2$$

Also $p_t(\%) = 0.25\%$ (say)

$\therefore A_{st} = \left(\dfrac{0.25}{100}\times 1000\times 400\right) = 1000 \text{ mm}^2$

$\therefore$ Provide $A_{st}$ = 1000 mm² (say)

$\therefore$ spacing of 24 ϕ bars = $\left(\dfrac{1000}{1000}\times\dfrac{\pi}{4}\times 24^2\right) = 452.16$ mm $\ngtr$ ~~3000 mm~~ 300 mm

$\therefore$ Provide 24 ϕ bars as tensile R/f @ 300 mm c/c

(6) Check for Bond:

$$(L_d)_{req} = \frac{\phi \times 0.87 f_y}{4 \tau_{bd}} = \frac{24 \times 0.87 \times 415}{4 \times 1.2 \times 1.6} = \underline{1128.28\,mm}.$$

and from $\boxed{C = T}$

$$0.36 f_{ck} B x_u = 0.87 f_y A_{st}$$

$$\therefore x_u = \left(\frac{0.87 \times 415 \times 1000}{0.36 \times 20 \times 1000}\right) = \underline{50.145\,mm}$$

$$\therefore M_{u1} = 0.87 f_y A_{st} (d - 0.42 x_u) = 0.87 \times 415 \times 1000 (400 - 0.42 \times 50.145) \times 10^{-6}$$
$$= \underline{136.81\,kNm}.$$

$$(V_u)_{max} = 102.1875\,kNm.$$

→ $\therefore (L_d)_{provided} \leq \frac{M_{u1}}{V_u} + L_0$

check $\left(\frac{M_{u1}}{V_u}\right) = \frac{136.81 \times 10^3}{102.1875} = \underline{1338.87\,mm} > (L_d)_{req}$

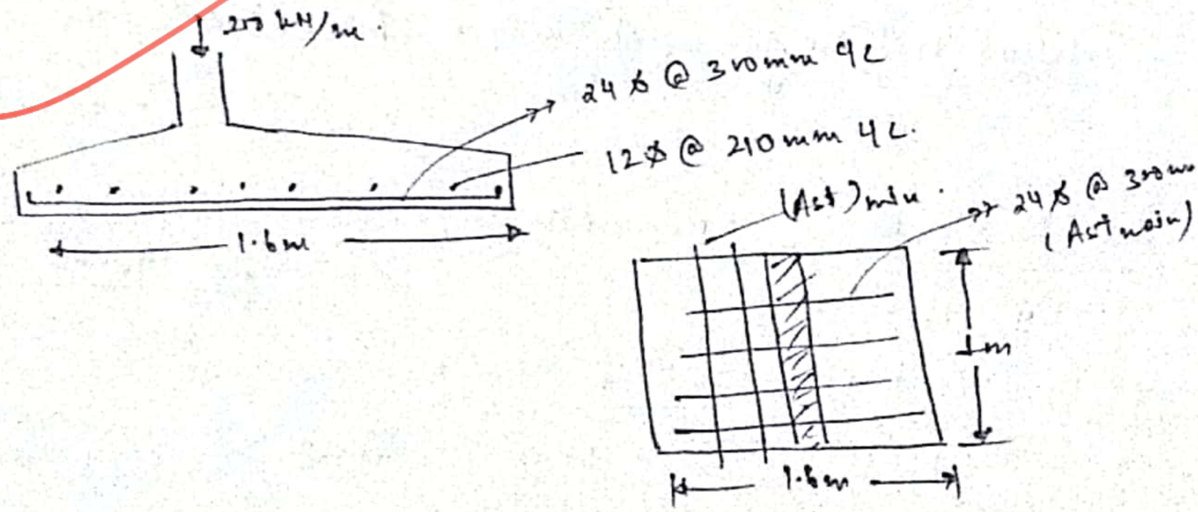
⇒ ok is safe in bond.

→ $(A_{st})_{min} = (0.12\%\ of\ BD) = \frac{0.12 \times 1000 \times (400+40)}{100}$ {Assuming eff cover = 40mm}

$$= \underline{528\,mm^2}$$

$\therefore$ spacing of 12 φ bars $= \left(\frac{1000}{528} \times \frac{\pi}{4} \times 12^2\right) = 214.99\,mm$ say $\underline{210\,mm\ c/c}$

(7) Detailing.

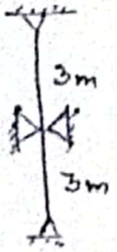


Q.b) column ISHB 350

$L_0 = 6m$ (unsupported length)

Axis:

In y-y dirn — restrained @ mid length




$\therefore l_{eff} = \left(\frac{l}{2}\right) = \underline{3m}$

Since y-y is a minor axis, therefore will undergo buckling class 'b'

& $\alpha = 0.34$

and $f_{cc}$ (euler's crippling stress) = $\frac{\pi^2 E}{\lambda^2}$

where $\lambda = \left(\frac{KL}{r_{min}}\right) = \left(\frac{3000}{28.4}\right) = \underline{105.634}$

$\therefore f_{cc} = \frac{\pi^2 \times 2\times10^5}{(105.634)^2} = \underline{176.8977}\ N/mm^2$

$\therefore (\lambda) \rightarrow$ non dim slenderness ratio $= \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{250}{176.8977}} = 1.188$

and $\phi = 0.5\left[1+\alpha(\lambda-0.2)+\lambda^2\right] = 0.5\left[1+0.34(1.188-0.2)+1.188^2\right] = \underline{1.374}$

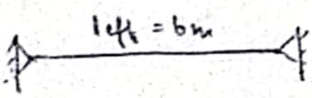
$\therefore f_{cd}$ (design comp stress) $= \frac{f_y/1.1}{\phi+\sqrt{\phi^2-\lambda^2}} = \frac{250/1.1}{1.374+\sqrt{1.374^2-1.188^2}}$

$= \underline{110.135\ N/mm^2}$

$\therefore$ factored load carrying capacity in y-y $(P_c) = f_{cd} \times A_g = 110.135 \times 6671 \times 10^{-3}$

$= \underline{734.71\ kN}$

And in z-z dirn: (Major Axis)




$l_{eff} = L_0 = 6m$

$\therefore \alpha = 0.21$ for buckling class 'a'

$\therefore \lambda = \left(\frac{KL}{r_{min}}\right) = \left(\frac{6000}{142.9}\right) = \underline{41.987}$

and $\lambda$( Non-dimensionless SR) = $\sqrt{\frac{f_y}{f_{cc}}}$

where $f_{cc} = \left(\frac{\pi^2 E}{\lambda^2}\right) = \frac{\pi^2 \times 2\times10^5}{(41.987)^2} = 1119.695\ N/mm^2$

$\therefore \lambda_N = \sqrt{\frac{250}{1119.695}} = 0.4725$

$\therefore \phi = 0.5[1 + 0.21(0.4725-0.2) + 0.4725^2] = \underline{0.6402}$

$\therefore f_{cd} = \frac{f_y/1.1}{\phi + \sqrt{\phi^2 - \lambda_N^2}} = \frac{250/1.1}{0.6402 + \sqrt{0.6402^2 - 0.4725^2}} = \underline{211.962\ N/mm^2}$

$\therefore$ factored load capacity in Z-Z $(P_c) = (f_{cd} \times A_g) = (211.962 \times 6671) \times 10^{-3}$
$= \underline{1414\ kN}$

$\therefore$ factored load capacity = Min of two = $\underline{734.71\ kN}$

$\therefore$ service load capacity $= \frac{734.71}{1.5} = \underline{489.806\ kN}$ (Ans)

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Q-8-c)

1) Overhead cost in construction Project

Overhead cost includes the indirect cost that are incurred during construction and engineering works which includes:

a) cost of Insurance of the equipments used.

b) Insurance, Registration of Land / space in use.

c) Overheads, and expenses incurred in contractor.

d) Site Mobilisations

e) cost of maintenance, and rundown of equipment

f) depreciation costs of equipment

g) cost over time lost in injuries and accidents.

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2) Prime cost: 'Prime cost' refers to the major / direct costs involved in the construction of a project or an activity.
This is the first major cost needed for the start of the project till its execution and completion.

These costs include:

a) Labour, human resources cost.
b) Material cost
c) Equipment / Machinery cost.

3) Prime causes of Accidents in a construction Industry are:

a) Sudden failure of machine or its components.
b) Accidents associated during operation of Machinery.
c) Unskilled supervisors operating the machines.
d) Unhygienic and unsound working conditions.
e) Lack of personal protective equipments / safety kits.
f) Improper safety measures, plans and guidelines.
g) Lack of coordination between the workers, lack of motivation and inadequate training.

4) Earthwork Equipments used for compaction of soil are:

1) Vibratory Rollers: a) These rollers are used for compacting granular soils, gravels or boulders.
b) Compaction of the soil is achieved due to the vibrating or vibroflotation phenomena.
c) used for compaction of soil in foundations, embankments etc.

a) Pneumatic Tyred Rollers:

a) These type of rollers are mainly suited for cohesive soils, can be used for cohesionless soils also.

b) Compaction of soil is achieved mainly by pressure and kneading action

c) Used primary in compaction of core of earthen dams and embankments for oil storage Tanks.

d) Most versatile compaction equipment used in engg constructions.

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