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Section: GH
Subject: Design of Reinforced Concrete Structures

(Assignment)

[2079 Bhadra] (Regular)

Q.1 Describe the difference between working stress and limit stress design. Explain characteristic strength and load.

→

WSM	LSM
1. Traditional method for RC design.	1. Most used method
2. Assume both concrete & steel as elastic (elastic stress-strain diagram)	2. Aims for a comprehensive and rational solution by considering safety at ultimate loads and serviceability at working loads.
3. Designed for working load.	3. Uses multiple factor of safety to provide adequate safety and adequate serviceability
4. Uneconomical compared to LSM	4. Economical design.

→ Characteristic strength (f_{ck})

↳ The value of strength below which not more than 5% sample are expected to fail

↳ $f_{ck} = f_m - k s$

↳ $M\underbrace{20}_{f_{ck}}$

[k → depend on confidence interval for 5% → k = 1.64]

→ characteristic load (L_c)

↳ That value of load which has 95% of probability of not being exceeded during its life time of the structure.

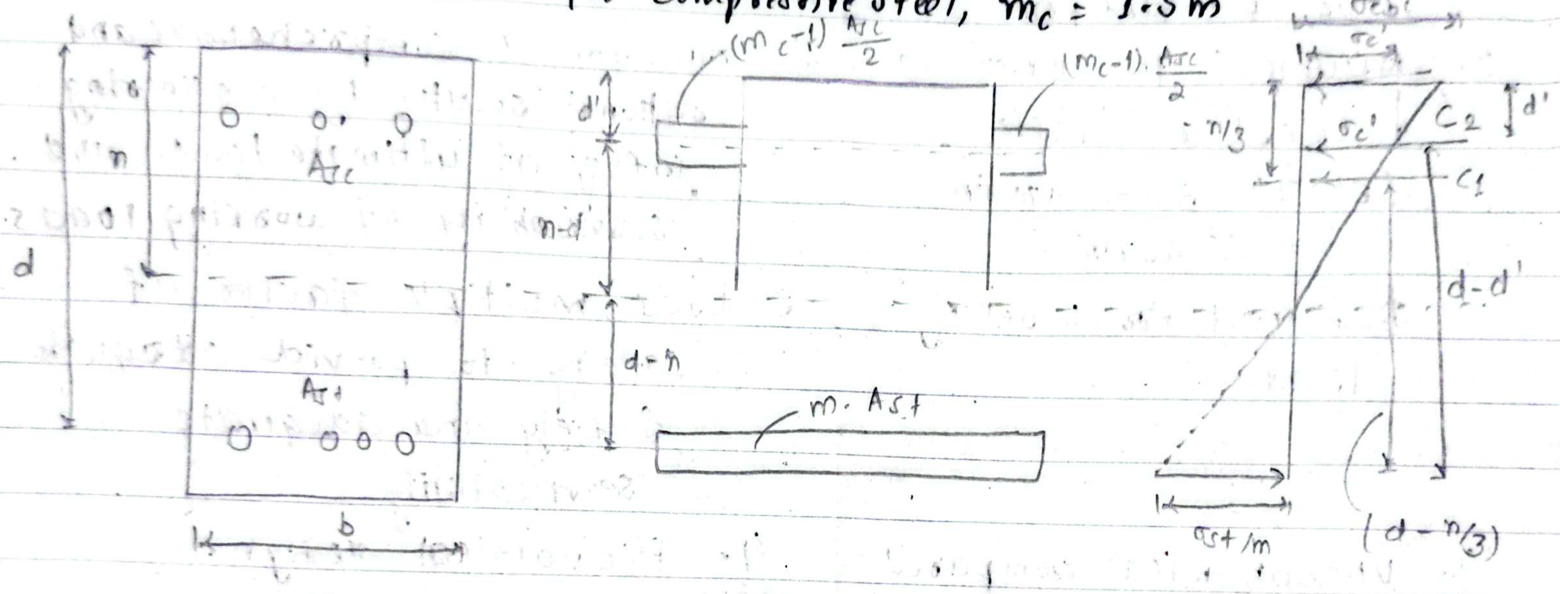
↳ $L_c = L_m + k s$

1· (b): Derive equation for BM carrying capacity of doubly reinforced rectangular section using assumptions for WSM given by IS 456: 2000.

→ Soln.

Modular ratio for tensile steel, $m = \frac{280}{3\sigma_{cbc}}$

" " for compressive steel, $m_c = 1.5m$



(a) (b) Equivalent section in terms of concrete (c) Stress diagram

→ Eqv. concrete area of tensile steel ~~area~~ $= m \cdot A_{st}$

Stress $= \sigma_{st}/m$.

→ compression area.

concrete $= b \cdot n - A_{sc}$

steel eqv. concrete $= m_c A_{sc}$

Net $= bn - A_{sc} + m_c A_{sc}$

$= bn + (m_c - 1) A_{sc}$

stress in eqv. concrete area $(m_c A_{sc}) = \sigma_c'$

" " compression steel, $\sigma_{sc} = m_c \sigma_c' = 1.5 m \sigma_c'$

- Critical Neutral axis: (n_c):

From stress diagram

$$\frac{\sigma_{cbc}}{\sigma_{st}/m} = \frac{n_c}{d-n_c}$$

- Actual NA (n)

Moment of compression area about NA = moment of tensile area about NA

or $b \cdot n \cdot \frac{n}{2} + (m_c - 1) A_{sc} * (n-d') = m\, A_{st} * (d-n)$

or $\frac{b\, n^2}{2} + (1.5m - 1) A_{sc} (n-d') = m\, A_{st} (d-n)$

Stresses: σ_{cbc} → Max. comp. stress in concrete.

σ_c' → stress in eqv. concrete at level of compression steel

σ_{st}/m → stress in eqv. concrete at level of tensile steel.

$m_c \sigma_c' = 1.5 m \sigma_c' = \sigma_{sc}$ → stress in compression steel.

σ_{st} → stress in tensile steel

→ Moment of resistance → calculated by taking moment of comp. forces about centroid of tensile reinforcement

$M_r = M_1 + M_2$

M_1 → MOR of similar balanced section w/o compression steel.

M_2 → Addition MOR provided by comp. steel.

→ $M_1 = C_1 * a$

C_1 → compressive force caried by concret.

$a = (d - n/3)$

$M_1 = \frac{1}{2} \sigma_{cbc}\, bn\,(d - n/3)$. or $R bd^2$ — (I)

→ $M_2 = C_2 * (d - d')$

C_2 → comp. force carried by compressive steel.

C_2 = eqv. area in term of concrete * compr. stress

$= (m_c - 1) A_{sc} * \sigma_c'$

$M_2 = (1.5m - 1) A_{sc} * \sigma_c' (d - d')$ — (II)

→ $[M_r = M_1 + M_2]$ — (III)

σ_c' → ?

$$\frac{\sigma_{cbc}}{n} = \frac{\sigma_c'}{n - d'}$$ (similar triangle)

→ $\underline{n < n_c}$ → under-reinforced

σ_{st} ✓

$\sigma_{cbc}' < \sigma_{cbc}$

Thus,

- $\frac{\sigma_{cbc}'}{\sigma_{st}/m} = \frac{n}{d - n}$] → σ_{cbc}' ✓
- $\frac{\sigma_{cbc}'}{n} = \frac{\sigma_c'}{(n - d')}$] → σ_c' ✓

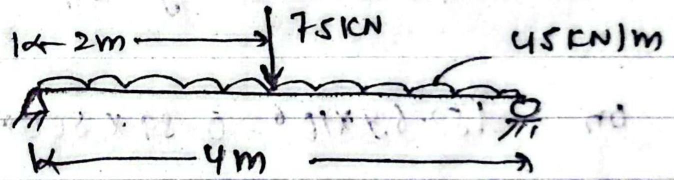
→ $n > n_c$ → over reinforced.

σ_{cbc}

$\sigma_c' = \frac{\sigma_{cbc}}{n}(n-d')$

Q.1.(c) A simply supported RCC beam of effective span 4m and overall dimensions 250 mm × 475 mm is subjected to superimposed load of 45 KN/m excluding its self wt. with point load 75 KN at midspan. Design the beam for limit state of collapse in flexure. Also, check whether the beam is safe in deflection or not. Consider effective cover to be 45 mm. Take M25 concrete and TMT bars. All the loads are in service level.

→ Soln:-

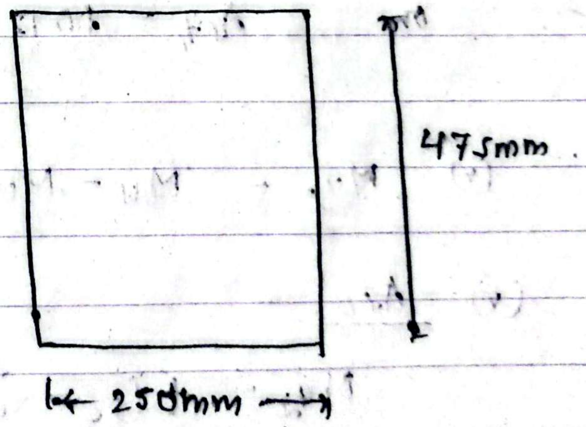


(i) Load

$w_u = 45 \times 1.5 = 67.5$ KN/m

$W_u = 75 \text{ KN} \times 1.5 = 112.5$ KN.

Max^m Bending Moment



$M_u = \frac{112.5 \times 4}{4} + \frac{67.5 \times 4^2}{8}$

$= 247.5$ KNm

$\left(\frac{WL}{4} + \frac{wl^2}{8}\right)$

d'

(ii) Effective depth $(d) = 475 - 45 = 430\,mm$.

$M25 \rightarrow f_{ck} = 25\,N/mm^2$

$TMT \rightarrow Fe500 \rightarrow f_y = 500\,N/mm^2$.

$b = 250\,mm$

(iii) $M_{u,lim} = 0.133 f_{ck} bd^2$ [For Fe500]

$= 0.133 * 25 * 250 * 430^2$

$= 153.69\,KNm < M_u$

so required doubly reinforced section

(iv) A_{st_1}:

$$M_{u,lim} = 0.87 f_y A_{st_1} d \left(1 - \frac{A_{st} f_y}{bd f_{ck}}\right)$$

or $$153.69 * 10^6 = 0.87 * 500 * A_{st_1} * 430 \left(1 - \frac{A_{st_1} * 500}{250 * 430 * 25}\right)$$

or $A_{st_1} = 1012.30\,mm^2$.

(v) $M_{u_2} = M_u - M_{u,lim} = 247.5 - 153.69 = 93.81\,KNm$.

(v) A_{sc}

$$M_{u_2} = (f_{sc} - f_{cc}) A_{sc} (d - d')$$

or $$93.81 * 10^6 = (f_{sc} - 0.446 * 25) * A_{sc} * (430 - 45)$$

For f_{sc}

$$\frac{d'}{d} = \frac{45}{430} = 0.105$$

~~Estimating~~ $f_{sc} = 412 - \frac{(412-395)}{(0.15-0.1)}(0.105-0.1)$

$= 410.3\ N/mm^2$

Thus

$A_{sc} = 610.45\ mm^2$.

(vi) A_{st_2}

$C = T$

$(f_{sc} - f_{cc}) A_{sc} = 0.87\ f_y\ A_{st_2}$

or $(410.3 - 0.446 * 25) * 610.45 = 0.87 * 500 * A_{st_2}$

$\Rightarrow A_{st_2} = 560.14\ mm^2$

(vii) $A_{st} = A_{st_1} + A_{st_2} = 1572.44\ mm^2$

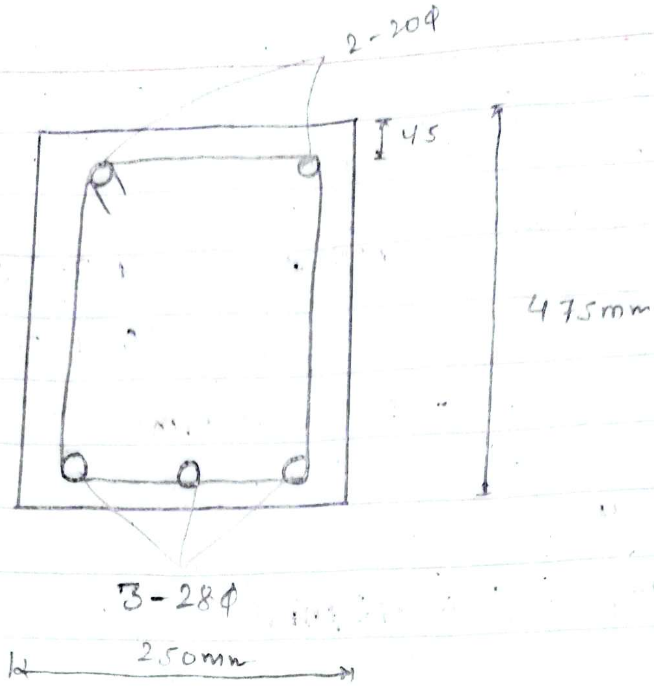
$A_{sc} = 610.45\ mm^2$

say providing 28mm bar in tension region

No. of bars $= \frac{1572.44}{\frac{\pi * 28^2}{4}} = 2.55 \approx 3$

say provide 20 mm bar in compression

No. of bars $= \frac{610.45}{\pi * 10^2} = 1.94 \approx 2$.



(viii) Min^n reinforcement (tension)

$$A_s = \frac{0.85 * bd}{f_y}$$

$$= \frac{0.85 * 250 * 430}{500}$$

$$= 182.75 \text{ mm}^2 < A_{st}$$

ok

(ix) Max^m, $A_{st,max} = 0.04 * b D =$

$$= 0.04 * 250 * 475$$

$$= 4750 \text{ mm}^2 > A_{st}$$

ok

Also,

$> A_{sc}$

ok

2.(a) A SS RC beam 300mm wide and 400mm deep (effective) is reinforced with a 4-20mm diameter bars. Design the shear reinforcement if M25 grade of concrete and TOR steel bar is used and beam is subjected to a SF of 130KN and torsional moment 45KN-m at service state.

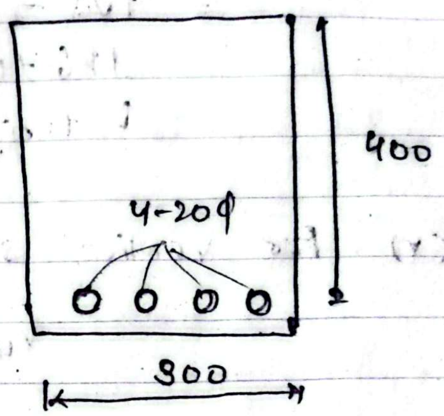
→ Soln:-

$f_y = 415$ N/mm² (TOR)

$f_{ck} = 25$ N/mm² (M25)

$V_u = 130 * 1.5 = 195$ KN

$T_u = 45 * 1.5 = 67.5$ KNm.



i) Nominal shear stress
(Cl. 40.1)

$$\tau_v = \frac{V_u}{bd} = \frac{195}{400 * 300} = 1.625 \text{ N/mm}^2.$$

ii) Design shear strength of concrete (τ_c) [Cl. 40.2].

Table 19,

$$P_t = \frac{100 * A_{st}}{bd} = \frac{100 * (\overset{1256.64}{\pi * 10^2 * 4})}{300 * 400} = 1.05\%.$$

For M25,

$$\left.\begin{array}{l} 1.0 \rightarrow 0.64 \\ 1.25 \rightarrow 0.70 \end{array}\right] \rightarrow 1.05 \Rightarrow \cancel{0.688}\ 0.652 \text{ N/mm}^2$$

$$\tau_c = 0.652 \text{ N/mm}^2 > \tau_v$$

(Required shear reinforcement)

(iii) Maximum shear stress,

$\tau_{c,max} = 3.1 \, N/mm^2 > \tau_v$ — OK

(table 20)

(No redesign required)

(iv) $V_{us} = V_u - \tau_c * bd$

$= 195 - 0.652 * 300*400$

$= 116.76 \, KN$

∴ design shear reinforcement for this.

(v) For vertical stirrups,

$$V_{us} = \frac{0.87 f_y A_{sv} d}{S_v}$$

say provide 2-legged 8mm-ϕ stirrups.

$$A_{sv} = 2 * \frac{\pi}{4} * 8^2 = 100.53 \, mm^2$$

or $$116.76*10^3 = \frac{0.87 * 415 * 100.53 * 400}{S_v}$$

$\Rightarrow S_v = 124.34 \, mm$

(vi) spacing,

Max^m spacing,

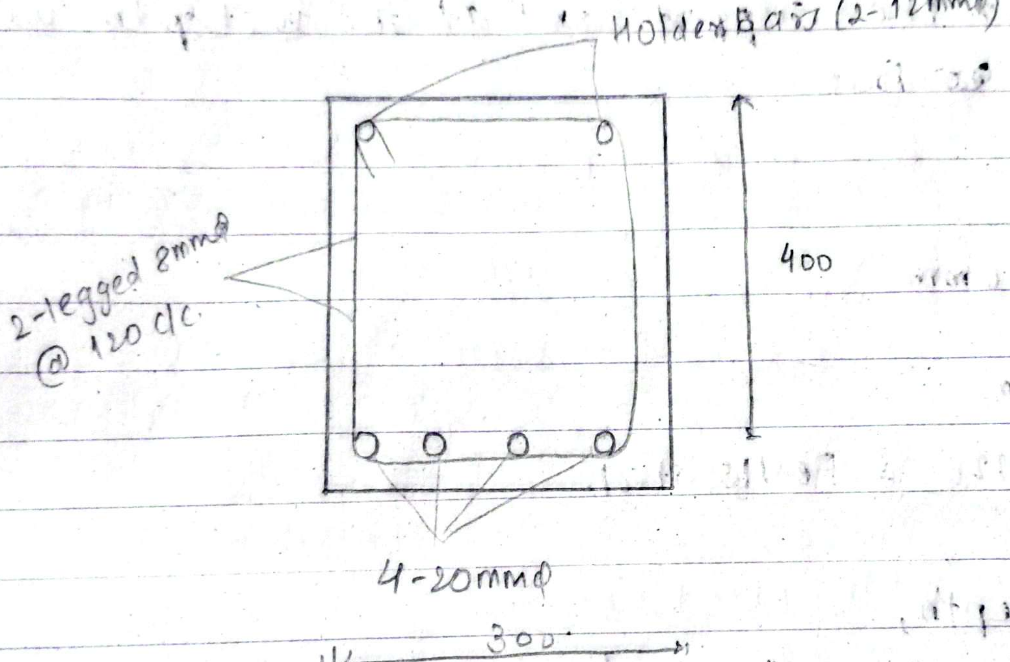
$$\frac{A_{sv}}{bS_v} \geq \frac{0.4}{0.87 f_y}$$ (Cl. 26.5.1.6)

or $$S_v \leq \frac{100.53 * 0.87 * 415}{0.4 * 300}$$

$\leq 300.47 \, mm$

Also, $S_{max} = 0.75 * d$
$= 0.75 * 400$
$= 300$ mm
'or'
$S_{max} = 300$ mm
(26.5.1.5)

Thus, provide 8-2-legged 8-φ stirrups (vertical) at 120 mm c/c



Q 2(b) A rectangular slab panel 5m × 4m (clear span) is continuous over three edges and discontinuous over one short edge. The slab is to rest on 250mm wide beam. The slab is subjected to live load of 4 kN/m² and floor finish of 1.5 kN/m². Design the slab and check whether the provided section satisfies the deflection criteria. Also, sketch the arrangement of reinforcement bars at the support and at the mid span with torsional bars.

→ Soln.

$w = 250mm$

$L_x = 4m$

$L_y = 5m.$

Assume, M20 & Fe415 steel.

(i) Overall depth,

Assume, $D = \frac{L_x}{30} = \frac{4000}{30}$

$= 133.33$

Take, $D = 160mm$

$d = 160 - 30 = 130mm$ (effective cover = 30mm)

(ii) Effective length,

$l_e = l_{c/c} = 4000 + 250 = 4.25m$

or

$l_{cs} + d = 4000 + 130 = 4.13m$

Thus, $l_x = 4.13$ m
$l_y = 5.13$ m

(iii) Type of slab.

$$\frac{l_y}{l_x} = \frac{5.13}{4.13} = 1.242 < 2$$

Two-way slab.

(iv) Load on slab.

Self wt $= 25 * 0.16 = 4$ kN/m²
FF $= 1.5$ kN/m²
LL $= 4$ kN/m²

For 1m width,
Total load, $w_u = 1.5(4+1.5+4) = 14.25$ kN/m.
(factored)

(v) Moment :
As continuous over three edges and 1 short edge discontinous → case ② Table 26

$M_x = \alpha_x w l_x^2$
$M_y = \alpha_y w l_x^2$

$\alpha_x^- = 0.0493$, $\alpha_y^- = 0.037$
$\alpha_x^+ = 0.0373$, $\alpha_y^+ = 0.028$.

$M_{x}^{-} = 0.0493 * 14.25 * 4.13^2 = 11.983\,kNm$

$M_{x}^{+} = 0.0373 * 14.25 * 4.13^2 = 9.066\,kNm$

$M_{y}^{-} = 0.037 * 14.25 * 4.13^2 = 8.993\,kNm$

$M_{y}^{+} = 0.028 * 14.25 * 4.13^2 = 6.806\,kNm$

(vi) Check depth
(from max^n BM)

(Annex G)
$$d = \sqrt{\frac{11.983 * 10^6}{0.138 * 20 * 1000 \cancel{*130}}} = 65.89\,mm$$
< d adopted (= 130 mm) ok

(vii) Area of reinforcement

(Annex G-1.1)
$$M_{u,lim} = 0.87\, f_y\, A_{st}\, d \left(1 - \frac{A_{st}\, f_y}{b d f_{ck}}\right) \quad (1)$$

For, $M_{x}^{-} = 11.983\,kN\,m$, $d = 130\,mm$
$A_x^{-} = 266.65\,mm^2$

For $M_x^{+} = 9.066\,kN\,m$, $d = 130\,mm$
$A_x^{+} = 199.50\,mm^2$

For $M_y^{-} = 8.993\,kNm$, $d = 150 - 20 \cancel{/2} + 20 - 20/2 = 100\,mm$

$A_y^{-} = 263.48\,mm^2$

For $M_y^{+} = 6.806\,kNm$, $d = 100\,mm$
$A_y^{+} = 196.52\,mm^2$

(viii) Spacing provided & reqd.:

$$S = \frac{1000 * A_s}{A_{st}}$$

Say, providing 8mmϕ bars, $A_s = \frac{\pi}{4} * 8^2 = \cancel{100.00}$ 50.3 mm²
(4.26.3.3)

For A_x^-, $S_x^- = \frac{1000 * 50.3}{266.65} = 188.64$ mm
For A_x^+, $S_x^+ = \frac{1000 * 50.3}{199.50} = 252.13$ mm
< 3*d = 390 mm or 300 mm

For A_y^-, $S_y^- = 190.90$ mm
For A_y^+, $S_y^- = 255.95$ mm.
< 3*d = 300 mm or 300 mm

check

Providing, 8mmϕ bars at 180 mm spacing for all.

x) $A_{st,min} = 0.12\% * bD = \frac{0.12 * 1000 * 160}{100} = 192$ mm².
OK

) Reinforcement provided,

$$A_{st,x^+} = A_{st,x^-} = A_{st,y^+} = A_{st,y^-} = \frac{1000 * 50.3}{180}$$

$$= 279.44 \text{ mm}^2$$

(xi) Check for deflection:

(Cl. 23.2.1.)

$$\frac{L}{d} < \alpha \beta \gamma \lambda \delta$$

(a) For continuous,

$\alpha = 26$

(b) $\beta = 1$ (span < 10 m)

(c) $f_s = 0.58 * 415 * \frac{266.65}{279.44} = 229.68\ N/mm^2$

$$P_t = \frac{100 * 279.44}{1000 * 130} = 0.214\%$$

From fig. 4,

$\gamma = 1.73$

(d) $\lambda = 1$ (NO compression reinforcement)

(e) $\delta = 1$ (Not flanged)

$$d > \frac{L}{\alpha * \gamma} > \frac{4000}{26 * 1.73}$$

$d > 88.93$

OK

Safe

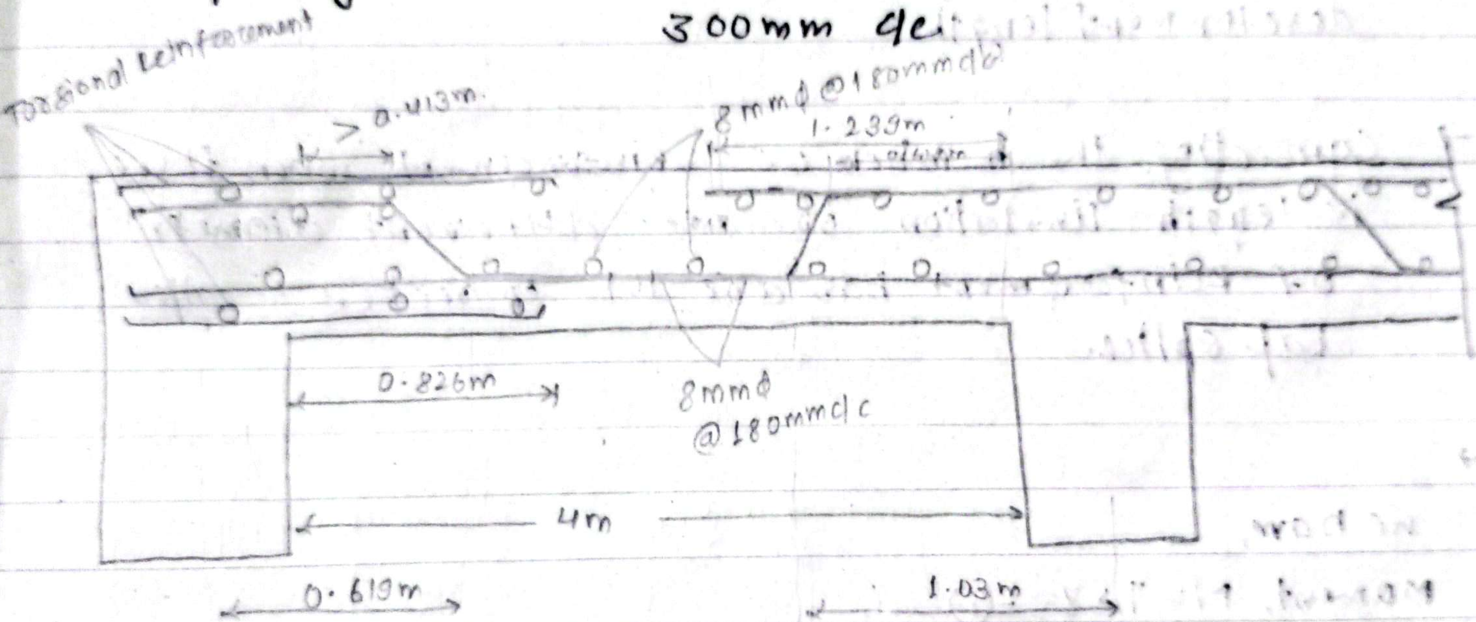
(xii) Torsional Reinforcement.

Area $= \frac{3}{8} \times A_{st,req} = \frac{3}{8} \times 266.25 = 99.84\,mm^2$

length $= \frac{l_{e}}{5} = \frac{4.13}{5} = 0.826\,m = 826\,mm.$

Spacing $= \frac{1000 \times 50.3}{99.84} = 563.81$

proving 8 mm-ϕ torsional reinforcement in 4 layers at 300 mm c/c



Longitudinal Section

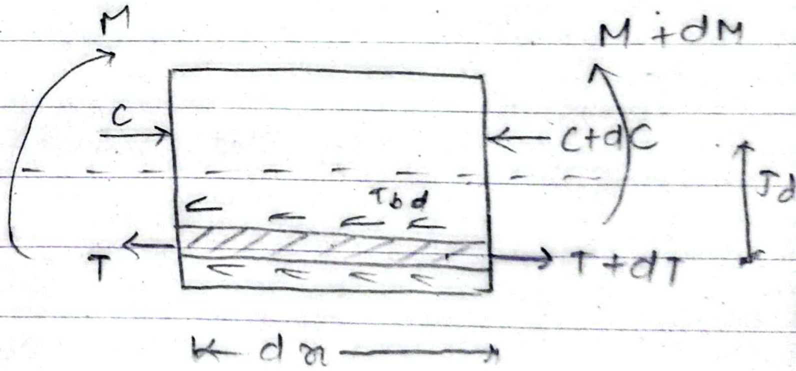
Q 3(a) Define development length and lap splice. Derive the expression $L_d \leq \frac{1.3 M_1}{V_u} + L_0$ at simply supported end, where symbols have their usual meaning.

→ Sol^n:

→ Minimum length of bar which must be embedded in the concrete beyond any section to prevent the bar from slipping out of concrete is called development length.

→ Connecting the member of reinforcement when there is length limitation or different diameter of reinforcement bar are to be connected is called Lap splice.

→



we have,

Moment, $M = T * Z$ —(i)

Also,

$M + dM = (T + dT) * J_d$

or $M + dM = \underbrace{T * J_d}_{M} + dT\, J_d$

or $dM = dT * J_d$

$\Rightarrow \left[dT = \frac{dM}{J_d} \right]$ —(ii)

CamScanner

$$\tau_{bd} \cdot \pi \cdot \phi \cdot dx = \frac{dM}{dx \; J_d} \qquad \left\{ \because \tau_{bd} = \frac{\phi f_s}{4 \tau_{bd}} \right\}.$$

$$\therefore \tau_{bd} = \frac{dM}{dx} \cdot \frac{1}{J_d \cdot \pi \phi} \Rightarrow \frac{V}{J_d \cdot \pi \phi} \quad \text{— (iii)}$$

Also,

$$\tau_{bd} = \frac{\phi f_s}{4 L_d} \quad \text{— (iv)}$$

Equating (iii) & (iv)

$$\frac{V}{J_d \cdot \pi \phi} = \frac{\phi f_s}{4 L_d}$$

$$\text{Or} \quad L_d = \frac{\pi \phi^2}{4} \cdot f_s \, J_d * \frac{1}{V}$$

$$= T * J_d * \frac{1}{V}$$

$$= \frac{M}{V}$$

$$\therefore \left[L_d = \frac{M}{V} \right]$$

M → Moment capacity of the section (M_1)
V → factored shear at section.

Thus,

$$L_d = \frac{M_1}{V}$$

If anchorage is required,

$$L_d \leq \frac{M_1}{V} + L_0$$

For simply supported at end

$$\left[L_d \leq 1.3 \frac{M_1}{V} + L_0 \right]$$

Q. 3(b) Design an unbraced rectangular RC column having clear ~~ht~~ height 6m with x-section dimension 400mm * 350mm subjected to design axial load of 600KN, design bending moments 100KNm about major axis and 50KNm about minor axis. consider M20 concrete and Fe415 steel.

→ Soln.

$b = 350mm$

$D = 400mm$

$l = 6000mm$] ← Taking $l = l_{eff}$ [$l_{ex} = l_{ey} = 6000mm$]

$P_u = 600KN$

$M_{ux} = 100KNm$

$M_{uy} = 50KNm$

$f_{ck} = 20MPa$, $f_y = 415MPa$,

(I) Slenderness ratio,

$$\frac{l_{eff}}{b} = \frac{6000}{350} = 17.14 > 12$$

$$\frac{l_{eff}}{D} = \frac{6000}{400} = 15 > 12$$

(long column)

(II) Additional moments

$$M_{ax} = \frac{P_u D}{2000}\left(\frac{l_{ex}}{D}\right)^2$$

$$= \frac{600 * 400}{2000}\left(\frac{6000}{400}\right)^2$$

$$= 27KNm$$

$$M_{ay} = \frac{P_u b}{2000}\left(\frac{l_{ey}}{b}\right)^2 = \frac{600 \times 10^3}{2000} \times 350 \left(\frac{6000}{350}\right)^2$$

$$= 30.857 \text{ KNm}$$

(iii) Reduction factors (K_x & K_y)

$$P_{uz} = 0.45 f_{ck} A_c + 0.75 f_y A_s$$

Say providing 1.2% of steel,

$$P_{uz} = 0.45 * 20 * (0.988 * 400 * 350) + 0.75 * 415 * 0.012 * 400 * 350$$

$$= 1767.8 \text{ KN}$$

For X-axis,

P_{bx}

d'/D $\left(d' = 40 + \frac{25}{2} = 52.5\right)$

(Table 60) (SP-16)

$$\frac{d'}{D} = \frac{52.5}{400} = 0.131$$

Taking next higher value, 0.15

$$K_1 = 0.196, \quad K_2 = 0.203$$

$$\therefore P_{bx} = \left(0.196 + 0.203 * \frac{1.2}{20}\right) * 20 * 350 * 400$$

$$= 582.904 \text{ KN}$$

For Y-axis,

$$\frac{d'}{b} = \frac{52.5}{350} = 0.15$$

$$(K_1 = 0.196, \; K_2 = 0.203)$$

$$P_{by} = \left(0.186 + 0.203 * \frac{1.2}{20}\right) * 20 * 350 * 400$$

$$= 582.904 \text{ KN}$$

$$k_x = \frac{P_{uz} - P_u}{P_{uz} - P_{bx}} \leq 1$$

$$= \frac{1767.8 - 600}{1767.8 - 582.904}$$

$$= \underline{0.985}.$$

$k_y = 0.985.$

(iv) Final additional moments.

$M_{ax} = 27 \times 0.985 = 26.595 \, kN\text{-}m$

$M_{ay} = 30.857 \times 0.985 = 30.394 \, kN\text{-}m.$

(v) Minimum eccentricity

X-

$e_{min} = \frac{l}{500} + \frac{D}{30} = 25.33 \, mm$ or $20 \, mm$] $25.33 \, mm$

Y-

$e_{min} = \frac{l}{500} + \frac{b}{30} = 23.67 \, mm$ or $20 \, mm$] $23.67 \, mm.$

$M_{ax} = 600 \times 0.02533 = 15.20 \, kN\text{-}m < 26.595 \, kN\text{-}m$ ~~27 kNm~~

$M_{ay} = 600 \times 0.02367 = 14.20 \, kN\text{-}m < 30.394 \, kNm.$

(vi) Thus,

$M_{ux} = M_x + M_{ax} = 100 + 26.595 = 126.595 \, kNm$

$M_{uy} = 50 + 30.394 = 80.394 \, kNm.$

(vii) $\frac{P}{f_{ck}} = \frac{1.2}{20} = 0.06$

$\frac{P_u}{f_{ck} bD} = \frac{600 \times 10^3}{20 \times 350 \times 400} = 0.214$

Thus, $\frac{d'}{D} = 0.151$ → Taking 0.15.

Chart 45

$\frac{M_{x_1}}{f_{ck} b D^2} = 0.098$

$M_{ux_1} = 109.766$ KN-m

$\frac{d'}{b} = 0.15$

Chart 45

$\frac{M_{y_1}}{f_{ck} b^2 D} = 0.098$

$M_{uy_1} = 96.049$ KNm.

(viii) α_n

$\frac{P_u}{P_{uz}} = \frac{600}{1767.8} = 0.339$ $\left.\begin{matrix} 0.2 \to 1 \\ 0.8 \to 2.0 \end{matrix}\right] \Rightarrow 0.339 \to 1.232$

$\boxed{\alpha_n = 1.232}$

(ix) For Biaxial Bending,

$$\left(\frac{M_{ux}}{M_{ux_1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy_1}}\right)^{\alpha_n}$$

$$= \left(\frac{126.595}{109.76}\right)^{1.232} + \left(\frac{80.394}{96.04}\right)^{1.234}$$

$= 1.995$

(unsafe)

Increase $P_t = 2.4\%$

$P_{uz} = 1232.38\ KN$

$P_{bx} = P_{by} = 582.901\ KN$

$$K_x = K_y = \frac{1232.38 - 600}{1232.36 - 582.90}$$

$$\approx 0.974$$

$M_{ux} = 100 + 0.974 \times 27 = 126.298\ KN\text{-}m$

$M_{uy} = 50 + 0.974 \times 30.857 = 80.055\ KN\text{-}m$

$$\frac{P}{f_{ck}} = \frac{2.4}{20} = 0.12$$

$$\frac{P_u}{f_{ck}\,bD} = 0.214$$

chart 45

$$\frac{M_{x_1}}{f_{ck}\,bD^2} = 0.153 \qquad \frac{M_{y_1}}{f_{ck}\,b^2D} = 0.153$$

$M_{x_1} = 171.36\ KN\text{-}m \qquad M_{y_1} = 149.94\ KNm$

$\alpha_n = 1.232$

$$\left(\frac{126.298}{171.36}\right)^{1.232} + \left(\frac{80.055}{149.94}\right)^{1.232}$$

$$= 1.148$$

unsafe

Again, $P_t = 3\% < 4\%$

$P_{uz} = 1225.35\,KN$

$P_{bx} = P_{by} = 582.90\,KN$

$K_x = K_y = \dfrac{1225.35 - 600}{1225.35 - 582.90} = 0.973$

$M_{ux} = 126.298\,KN\text{-}m$

$M_{uy} = 80.055\,KNm$

$\dfrac{P}{f_{ck}} = 0.15$

Chart 45

$\dfrac{M_{ux_1}}{f_{ck}\, b D^2} = 0.177$

$M_{ux_1} = 198.24\,KNm$

$\dfrac{M_{uy_1}}{f_{ck}\, b^2 D} = 0.177$

$M_{uy_1} = 173.46\,KNm$

$$\left(\frac{126.298}{198.24}\right)^{1.232} + \left(\frac{80.055}{173.46}\right)^{1.232}$$

$= 0.959 \leq 1$

Safe

(x) Reinforcement

$A_s = 3\%$ of $350 \times 400 = 4200\,mm^2$

Providing 28-mm bar;

No. of bars $= \dfrac{4200}{\pi * 14^2} = 6.78 \approx 8$

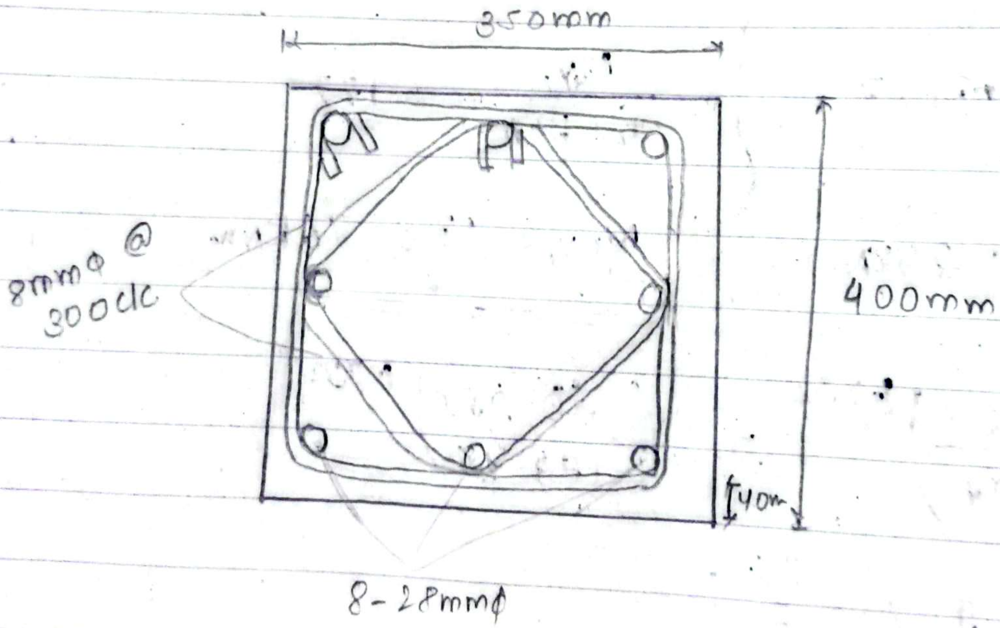
Thus 8-28mmϕ bars are provided

(XI) Lateral ties

Diameter | $\frac{1}{4} \times 28 = 7$mm or 6mm } **8mm**

Pitch | 350mm, $16 \times 28 = 448$mm, 300mm } **300mm**

Thus provide 8mm-ϕ ties at 300mm c/c



4.(a) Discuss about requirements for good detailing. Also describe bar bending schedule.

→ Requirements of good detailing.

① Simple

↳ Intersection between building components (beam, column, slab) should not be congested and complex.

② Minimum spacing of reinforcement must be maintained such that aggregate can pass through them

③ Spacing should be within maximum spacing of reinforcement.

④ Sufficient cover should be provided.

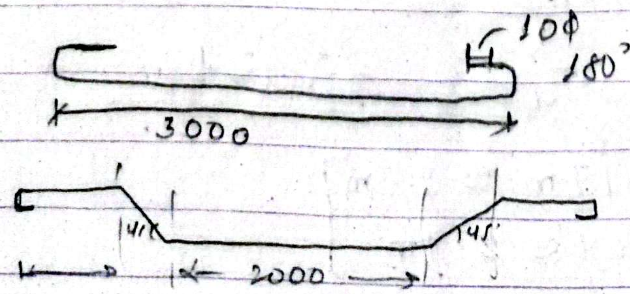
⑤ Lap length should be sufficient at proper location.

→ BBS

↳ providing detailing of bar bending including diameter, ~~of~~ length, weight and number of bars required such that sufficient quantity of reinforcement is available and wastage is minimum.

↳ Helps in estimation of structure.

eg.



Q.4(b) Design a footing for a rectangular column of size 30cm × 35cm reinforced with 8-20 mm dia bar. The column is subjected to a factored axial load and moment of 1000KN and 80KN-m resp. The allowable bearing capacity of soil is 140KN/m². At a depth of 1.6m. Use M25 concrete and TMT bar for column and footing both. Sketch all the reinforcement required.

→ Sol^n:-

$a = 30cm$

$b = 35cm$

$P_u = 1000KN$

$M_{ux} = 80KN\text{-}m$

$q_a = 140KN/m^2$

$f_{ck} = 25N/mm^2 \} \ f_y = 500N/mm^2$

$D_f = 1.6m$

① ~~Assume self wt. & filling wt. to be 10%.~~

② Assume $e < L/6$

$$\frac{P}{A} + \frac{M}{Z} \leq q_a$$

or
$$\frac{\frac{1000}{1.5}}{B*L} + \frac{80/1.5}{BL^2/6} \leq 140$$

say, $\frac{L}{B} = \frac{b}{a} = \frac{35}{30} \simeq 1.2$

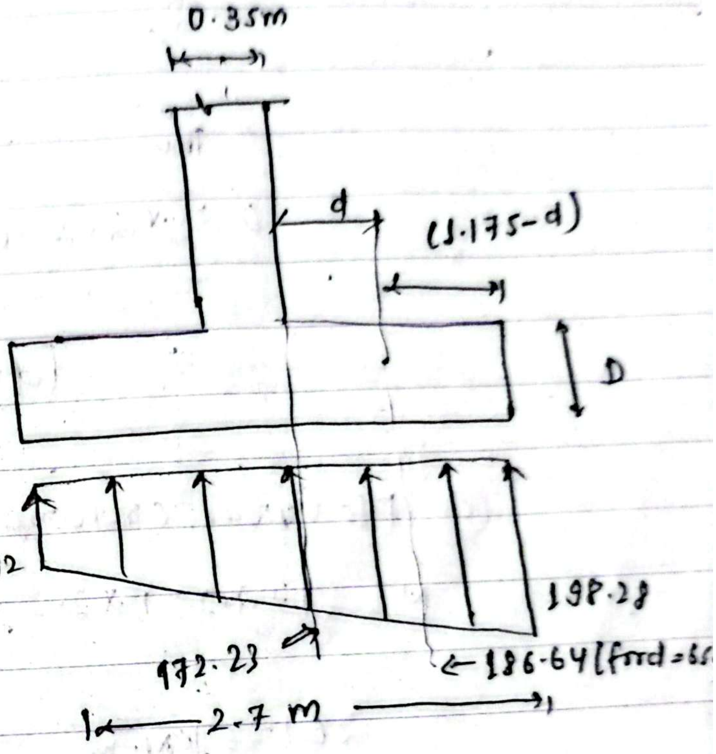
$$\Rightarrow \begin{bmatrix} B = 2.17m \approx 2.2m \\ L = 2.64 \approx 2.7m \end{bmatrix}$$

[078BCE178]

③ Factored soil pressure,

$$q_u = \frac{P_u}{A} \pm \frac{M}{Z}$$

$$= \frac{1000}{2.2 * 2.7} \pm \frac{80}{\frac{2.2 * 2.7^2}{6}}$$



$q_{u,max} = 198.28 \, KN/m^2$

$q_{u,min} = 138.42 \, KN/m^2$

④ Depth calulation.

ⓐ Two wayshear,

$$q_{av} = q = \frac{198.28 + 138.42}{2}$$

$$= 168.35 \, KN/m^2$$

$$V = 168.35 * [2.7 * 2.2 - (0.35+d) * (0.3+d)]$$

$$\tau_c = 0.25\sqrt{f_{ck}} = 0.25\sqrt{25} = 1.118 \text{ MPa} = 1.118 \, N/mm^2$$

$$1.118 * 10^{-3} * [(0.3+d+0.35+d)] * 2 * d = 168.35 [2.7 * 2.2 - (0.35+d)(0.3+d)]$$

Solving,

[d = 321mm]

(b) one-way shear

taking d = 650mm.

$$SF = \left(\frac{198.28 + 186.64}{2}\right) * \left(\frac{1175 - d}{1000}\right) * 2.2$$

assume, % of steel = 0.25%.

For M25, $\tau_c = 0.36\ N/mm^2$
(M27. (Table 19)

Thus,

$$0.36 * 2.2 * d = 2.2 * \underbrace{\left(\frac{198.178 + 186.69}{2}\right)}_{192.384} * \left(\frac{1175 - d}{1000}\right)$$

⇒ [d = 409mm]

(c) Flexural criteria.

$$M = (172.23) * 2.2 * \frac{1.175^2}{2}$$

$$= 261.56\ kN\text{-}m$$

For Fe500,

$$d = \sqrt{\frac{261.56 * 10^6}{0.133 * 25 * 2200}}$$

= 189.09mm

Thus, adopt d = 409mm

$$D = 409 + 50 + \frac{16}{2} + 16 = 483 \simeq 500mm$$

$$d = 500 - 50 - \frac{16}{2} - 16 = 426mm.$$

⑤ Reinforcement calculation

For longer direction,

$M = 261.56 \times 10^6 = 0.87 \times 500 \times 426 \times A_{st}\left(1 - \frac{A_{st} \times 500}{2200 \times 426 \times 25}\right)$

$\Rightarrow A_{st,x} = 1456.76 mm^2$

For shorter dir^n

$M_{shorter\ dir^n} = \frac{(173.36 \times 2.7) \times 0.95^2}{2} = 211.22$ kN-m

$q = \frac{P}{A} \pm \frac{M}{\frac{B^2L}{6}}$

$d = 410mm$

$b = 2700mm$

$q_{u,max} = 205.08$ kN/m²

$q_{u,min} = 131.62$ kN/m²

Then

$A_{st,y} = 1210.79 mm^2$

column face =

$131.62 + \frac{(205.08 - 131.62)}{2.2} \times \left(\frac{2.2 - 0.3}{2} + 0.3\right)$

0.95

$= 173.36$ kN/m²

⑥ Providing 16mm bar → $A = 201.06 mm^2$

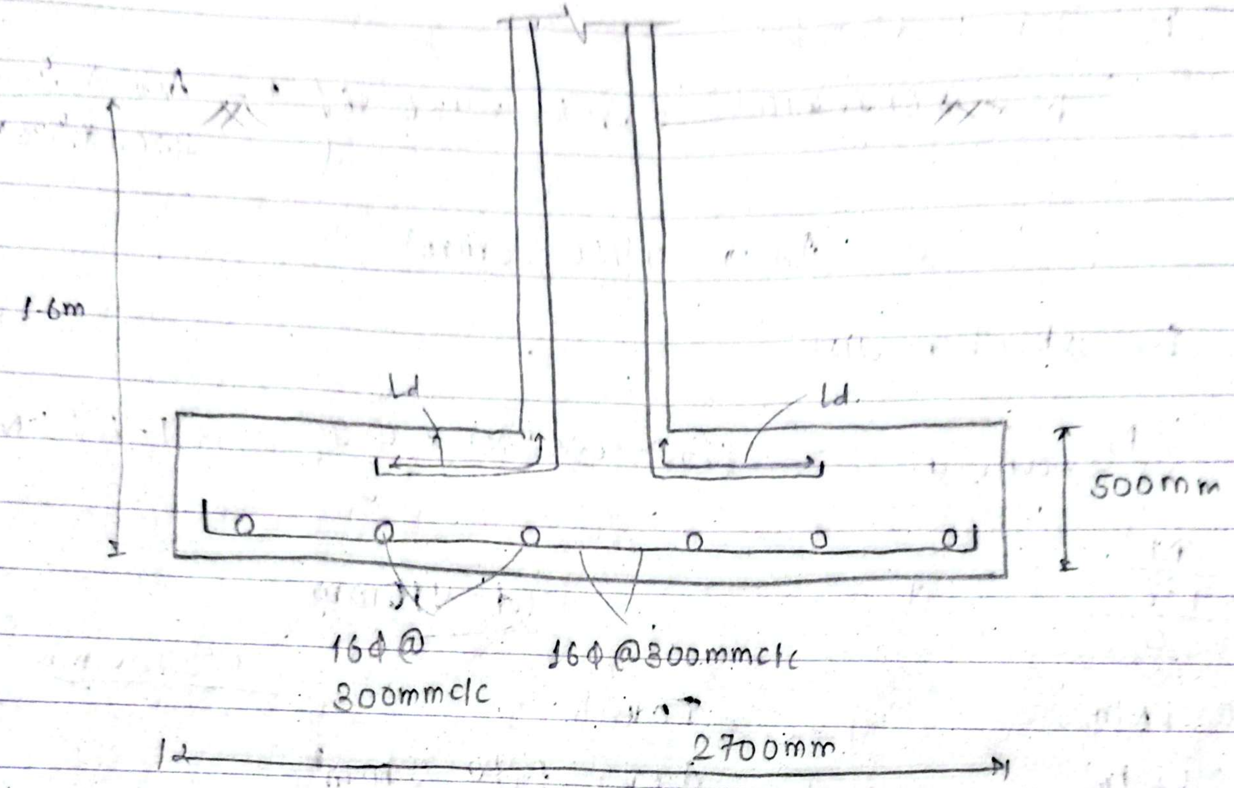
Spacing in longer dir^n = $\frac{2200 \times 201.06}{1456.76}$

$= 303mm$

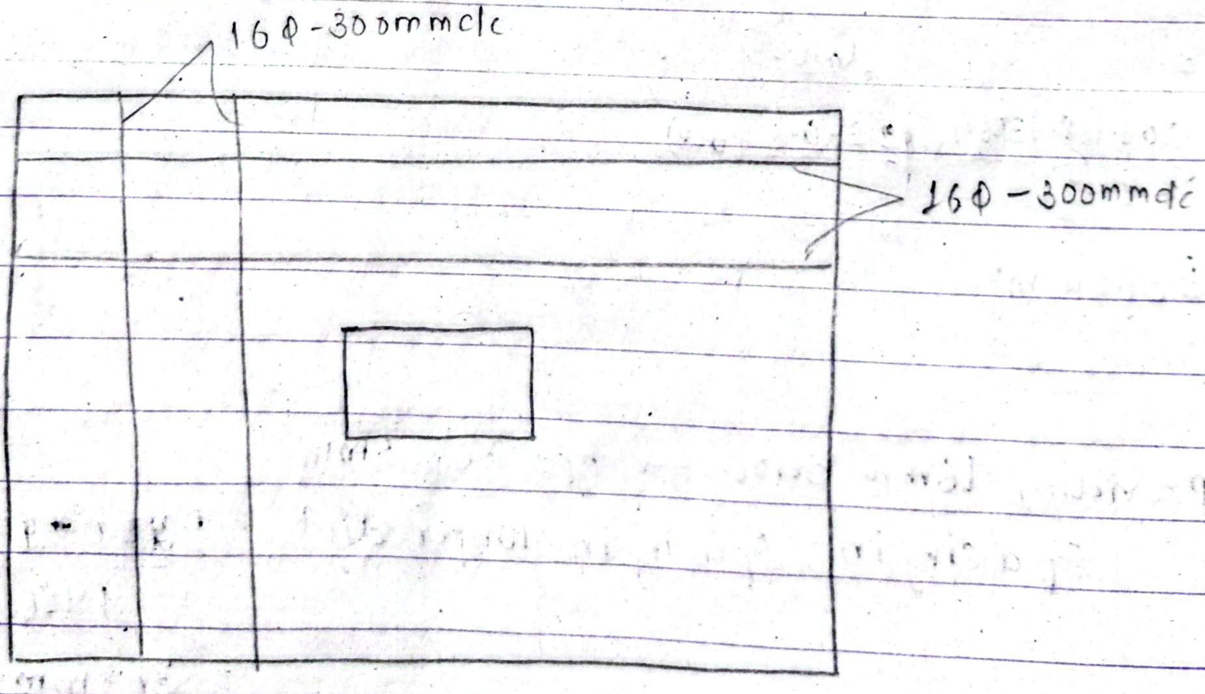
Spacing in shorter dir^n = $\frac{2700 \times 201.06}{1210.79 mm^2}$

$= 448.35mm$

Providing 16mm bars at 300mm c/c



section



Plan

[2080 Bhadra] (Regular)

[078BCE178]

Q.1.(a) Discuss different types of flexural sections according to amount of reinforcements in tension zone. Describe different types of failure modes in above mentioned sections.

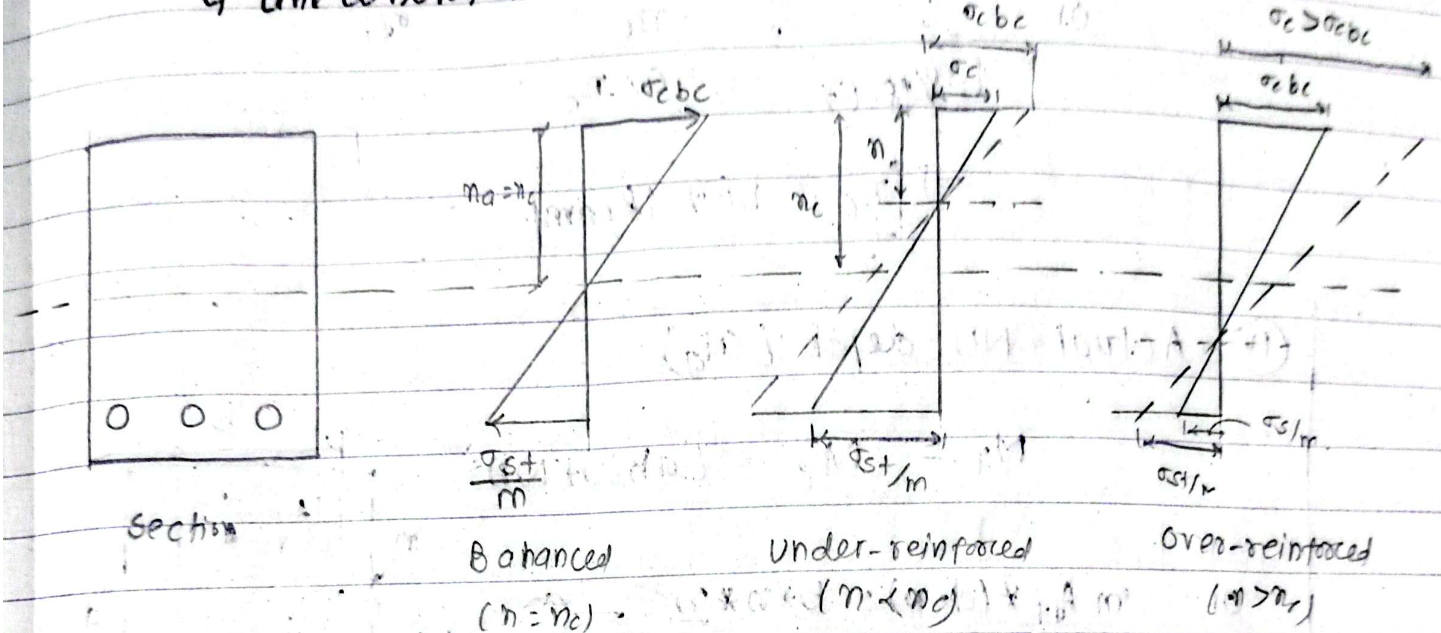
→ Types of flexural sections are:-

(1) Balanced section
- ↳ NA depth = critical NA depth.
- ↳ Both σ_s & σ_c are permissible
- ↳ Hypothetical section
- ↳ limiting states in steel and concrete are reached simultaneously.
- ↳ Failure is expected to occur by simultaneous initiation of crushing of steel & yielding of steel.

(2) Under-reinforced section
- ↳ reinforcement provided is less than that in balanced section
- ↳ Ast ↓↓ ⇒ NA shifts up] NA depth < critical NA depth
- ↳ Steel reaches its full capacity
- ↳ Before concrete reaches its permissible capacity steel (being ductile) gives sufficient warning before failure.
- ↳ Failure is ductile

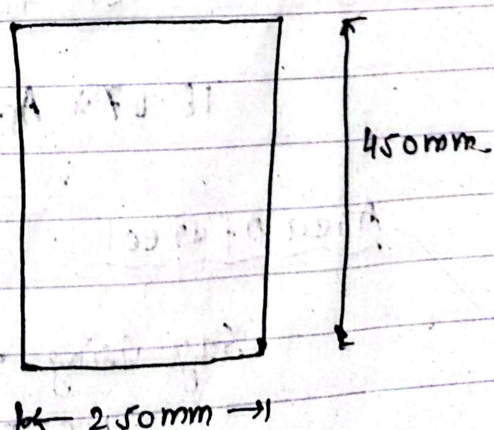
(3) Over-reinforced section
- ↳ reinforcement provided is more than that in balanced section.
- ↳ Ast ↑↑ ⇒ NA drops down] NA depth > critical NA depth.
- ↳ concrete reaches its full capacity

↳ Before steel reaches its failure point (permissible stress) concrete will break suddenly
↳ Brittle failure (without early warning)
↳ steel cannot utilize its full capacity
↳ uneconomical section.



Q.1(b) Determine the area of steel required and the moment of resistance for a RC beam 250 mm × 450 mm. Consider M15 grade of concrete and mild steel. Use WSM.

→ Soln. say, effective ~~depth~~ cover = 50mm,
($d = 450 - 50 = 400$mm)



(i) Permissible stress

concrete; $\sigma_{cbc} = 5 N/mm^2$ (Table 21)

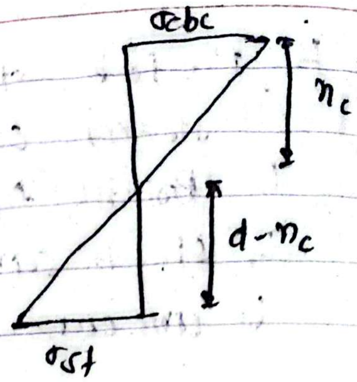
Steel, $\sigma_{st} = 130 N/mm^2$ (Table 22)

Mild steel → Fe250.

(ii) Modular ratio (m)

$$m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3 \times 5}$$
$$= 18.67$$

(iii) Critical NA depth (n_c)



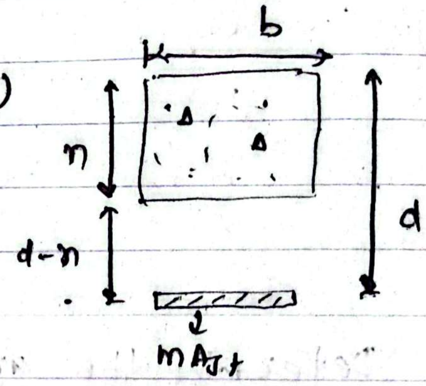
Similar Δ:

$$\frac{\sigma_{cbc}}{\sigma_{st}/m} = \frac{n_c}{d-n_c} \quad -(i)$$

or $$\frac{5}{130/18.67} = \frac{n_c}{400-n_c}$$

=> $[n_c = 167.18\,mm]$

(iv) Actual NA depth (n_a)



$M_T = M_C$ (about NA)

or $$m A_{st} * (d-n) = b*n*\frac{n}{2} \quad -(ii)$$

or ~~18.67*1~~

Considering balanced section,

Ast from eqn (ii)

$$18.67 * A_{st} * (400-167.18) = 250 * \frac{167.18^2}{2}$$

Area of steel => $[A_{st} = 803.74\,mm^2]$

Say using 20mm bar,

No. of bar $= \frac{803.74}{\frac{\pi * 20^2}{2}} = 2.56 \approx 3$

providing 3 bars of 20mm on tension side

$(A_{st})_{prov} = 942.48\ mm$

say clear cover = 30mm

effective cover = $30 + \frac{20}{2} = 40mm$

$d = 450 - 40 = 410mm.$

Moment of resistance:

Max^m moment, $M = \frac{1}{2} * \sigma_{cbc} * b * x_c * \left(d - \frac{x_c}{3}\right)$

$= \frac{1}{2} * 5 * 250 * 167.18 * \left(400 - \frac{167.18}{3}\right)$

$= 35.97\ KNm.$

For A_{st},

$M = \sigma_{st} * A_{st} * \left(d - \frac{x_c}{3}\right)$

on 3

Q. 2. (a) Describe anchorage and flexural bond stress. Derive equation for development length and bondstress.

→ Soln:

Anchorage Bond stress

↳ Anchorage bond arises over the length of anchorage provided for a bar or near the end (or cut-off point) of a reinforcing bar. This bond resists the pulling out of the bar if it is in tension or conversely pushing in of bar if it is in compression.

- Flexural bond arises in flexural members on account of shear or a variation in bending moment, which in turn causes a variation in axial tension along the length of a reinforcing bar.

Flexural bond stress, (u_f)

$$u_f = \frac{V}{(\Sigma O) * (z)}$$

ΣO → total perimeter of bar at the beam section under consideration.

z → lever arm.

V → transverse shear force.

- Anchorage bond stress (u_{av})

$$u_{av} = \frac{\phi f_s}{4 L_d}$$

L_d → development length

f_s → tensile stress (Gen. $0.87 * f_y$)

ϕ → dia. of rod.

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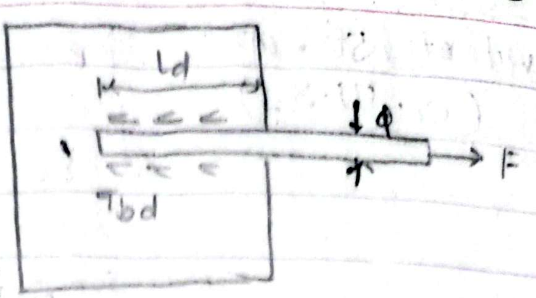
[078BCE178]

Bond stress (τ_{bd})



$(F = f_s * A_{st}) \leq$ Resistance

Force

Resistance

or, $0.87 * f_y * A_{st} = \tau_{bd} * \pi \phi * L_d$

or, $0.87 f_y * \frac{\pi \phi^2}{4} = \tau_{bd} * \pi * \phi * L_d$

or, $\tau_{bd} = \frac{0.87 f_y * \phi}{4 L_d}$

or, $\left[\tau_{bd} = \frac{f_s * \phi}{4 L_d}\right]$ m.

Q.2(b) Design a rectangular RC beam having dimension 200mm * 400mm subjected to design design BM 100KN-m, design shear force 80KN (at critical section) and design torsional moment 15KN-m. Consider M25 grade concrete and Fe 415 grade steel.

→ Soln:-

①

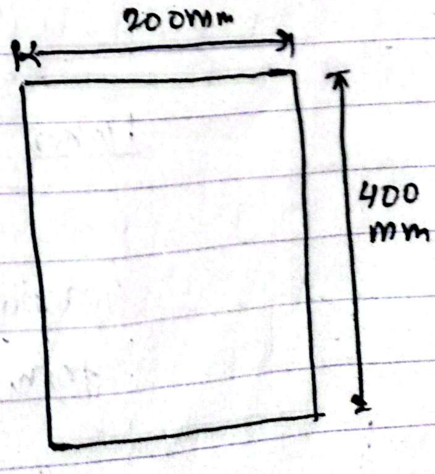
$M_u = 100 KN\text{-}m$

$V_u = 80 KN$

$T_u = 15 KNm$

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

$f_y = 415 N/mm^2$



② Equivalent shear (Cl. 41.3.1) $V_e = V_u + 1.6 \frac{T_u}{b}$

$= 80 + \frac{1.6 \times 15}{0.2}$

$= 200 KN.$

Assume, effective cover 50mm

Then,

$d = 400 - 50 = 350mm.$

③ Equivalent nominal shear stress (τ_{ve})

• $\tau_{ve} = \tau_e = \frac{V_e}{bd} = \frac{200 \times 10^3}{200 \times 350} = 2.86 N/mm^2.$

(Cl. 41.3.1)

• $\tau_{c,max} = 3.1 N/mm^2$ (For M25)

[Table. 20]

Taking 0.25% of steel.

From Table 19, For M25,

$\tau_c = 0.36 N/mm^2$

Here,

$\tau_c < \tau_{ve} < \tau_{c,max}.$

Torsional reinforcement is required in the form of longitudinal and transverse reinforcement.

CamScanner

(4) For longitudinal reinforcement,

(Cl. 41.4.2) $M_{e_1} = M_u + M_t$

$$M_t = \frac{T_u\left(1 + D/b\right)}{1.7}$$

$$= \frac{15 * \left(1 + \frac{400}{200}\right)}{1.7}$$

$= 26.47$ KN-m.

$$M_{e_1} = M_u + M_t$$
$$= 100 + 26.47$$
$$= 126.47 \text{ KN-m.}$$

$\because M_t < M_u \rightarrow$ NO need for compression reinforcement side

Also,

$$M_{e_1} = 0.87 f_y A_{st} d \left(1 - \frac{f_y A_{st}}{f_{ck} bd}\right)$$

or $126.47 * 10^6 = 0.87 * 415 * A_{st} * 350 \left(1 - \frac{415 * A_{st}}{25 * 200 * 350}\right)$

or $[A_{st} = 1633.88 \text{ mm}^2]$

Min^n reinforcement

(Cl. 26.5.1) $A_s = \frac{0.85 bd}{f_y} = \frac{0.85 * 200 * 350}{415}$

$= 143.37 \text{ mm}^2 < A_{st}$

OK

Provide 28mm bar,

$$\text{No of bar} = \frac{1633.88}{\pi * 14^2} = 2.65 \approx 3$$

Provide 3- 28mm bar

$$A_{st} = 3 * \frac{\pi}{4} * 28^2 = 1847.26\,mm^2.$$

Check Max^m reinforcement (Cl. 26.5.1) $= 0.04 * b * D$

$= 0.04 * 200 * 400$

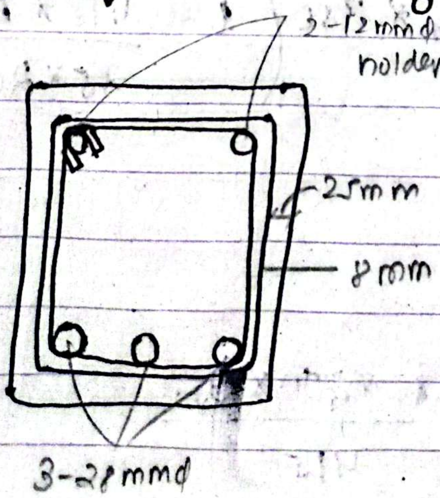
$= 3200\,mm^2 > A_{st}$

ok

⑤ Transverse reinforcement

$$\frac{A_{sv}}{S_v} = \frac{T_u}{b_1 d_1 (0.87 f_y)} + \frac{V_u}{2.5 d_1 (0.87 f_y)} \quad \text{— (I)}$$

$$\frac{A_{sv}}{S_v} = \frac{(\tau_{ve} - \tau_c)\, b \cdot \cancel{s_v}}{0.87 f_y} \quad \text{— (II)}$$



Assume 2-12mmϕ holder bar

Now,

$$b_1 = 200 - 2*25 - 2*8 - \left(\frac{28}{2} + \frac{28}{2}\right)$$

$$= 106\,mm$$

$$d_1 = 400 - 2*25 - 2*8 - \left(\frac{28}{2} + \frac{12}{2}\right)$$

$$= 314\,mm$$

$$d = 400 - 25 - 8 - \frac{28}{2} = 353\,mm$$

[078BCE178]

$P_t = \frac{A_{st} * 100}{bd} = \frac{1847.26 * 100}{200 * 353} = 2.61\%$

[τ_c] Table 19

$2.50 \rightarrow 0.88$
$2.75 \rightarrow 0.90$] $2.61 \rightarrow 0.89$

$\tau_c = 0.89\ N/mm^2 < \tau_{ve} = 2.86 N/mm^2$

Adopt 2-legged 8 mm stirrups.

$A_{sv} = 2 * \pi/4 * 8^2 = 100.53 mm^2$

From (I),

$\frac{100.53}{S_v} = \frac{75 * 10^6}{106 * 314 * 0.87 * 415} + \frac{80 * 10^3}{2.5 * 314 * 0.87 * 415}$

solving $S_v = 65.68 mm$.

From (II),

$\frac{100.53}{S_v} = \frac{(2.86 - 0.89) * 200}{0.87 * 415}$

$\Rightarrow S_v = 92.12 mm$

(I) < (II)

Thus,

$S_v = 92.12 mm$

Taking ~~s~~ taking $S_v = 90 mm$

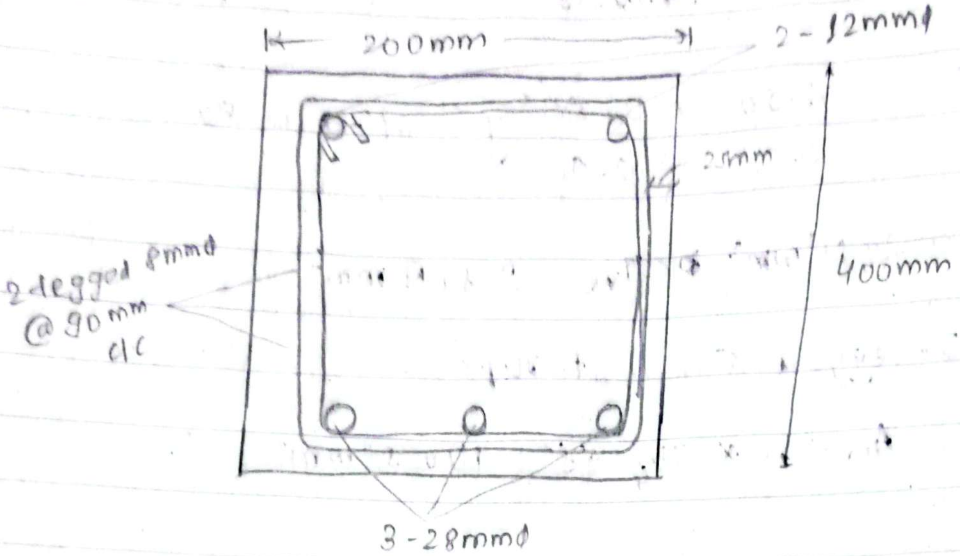
From 26.5.1.7(a),

$x_1 = 200 - 25 * 2 - 2 * 8/2 = 142 mm$

$y_1 = 400 - 25 * 2 - 2 * 8/2 = 342 mm$

$$S_{max} \begin{cases} x_1 = 142 mm \\ \frac{x_1 + y_1}{4} = 121 mm \\ 300 mm \end{cases}$$

Thus adopt 2-legged 8mm stirrups @ 90mm c/c.



Q. 3(a) Define interaction diagram with its features and neat sketches. Discuss on the modes of failure for compression members in eccentric compression within the interaction diagram.

→ ~~Sol~~

Interaction diagram is a complete graphical representation of the design strength of a uniaxially eccentrically loaded column of given proportions. It serves as a failure envelope.

Features:

(i) X- axis : $\dfrac{P_u}{f_{ck}\, bD}$

(ii) Y-axis : $\dfrac{M_u}{f_{ck}\, bD^2}$

(iii) Curves : $\dfrac{p}{f_{ck}}$

(iv) curves for reinforcement distributed equally on two sides (chart 27 to 38) and distributed equally on four sides (chart 39 to chart 50)

(v) chart for Rectangular section (chart 27 to 50) and circular section (chart 51 to chart 62)

(vi) chart for different values of

$\frac{d'}{D} = 0.05, 0.10, 0.15$ and 0.20

(vii) chart for different grade of steel

Fe250, Fe415 and Fe500

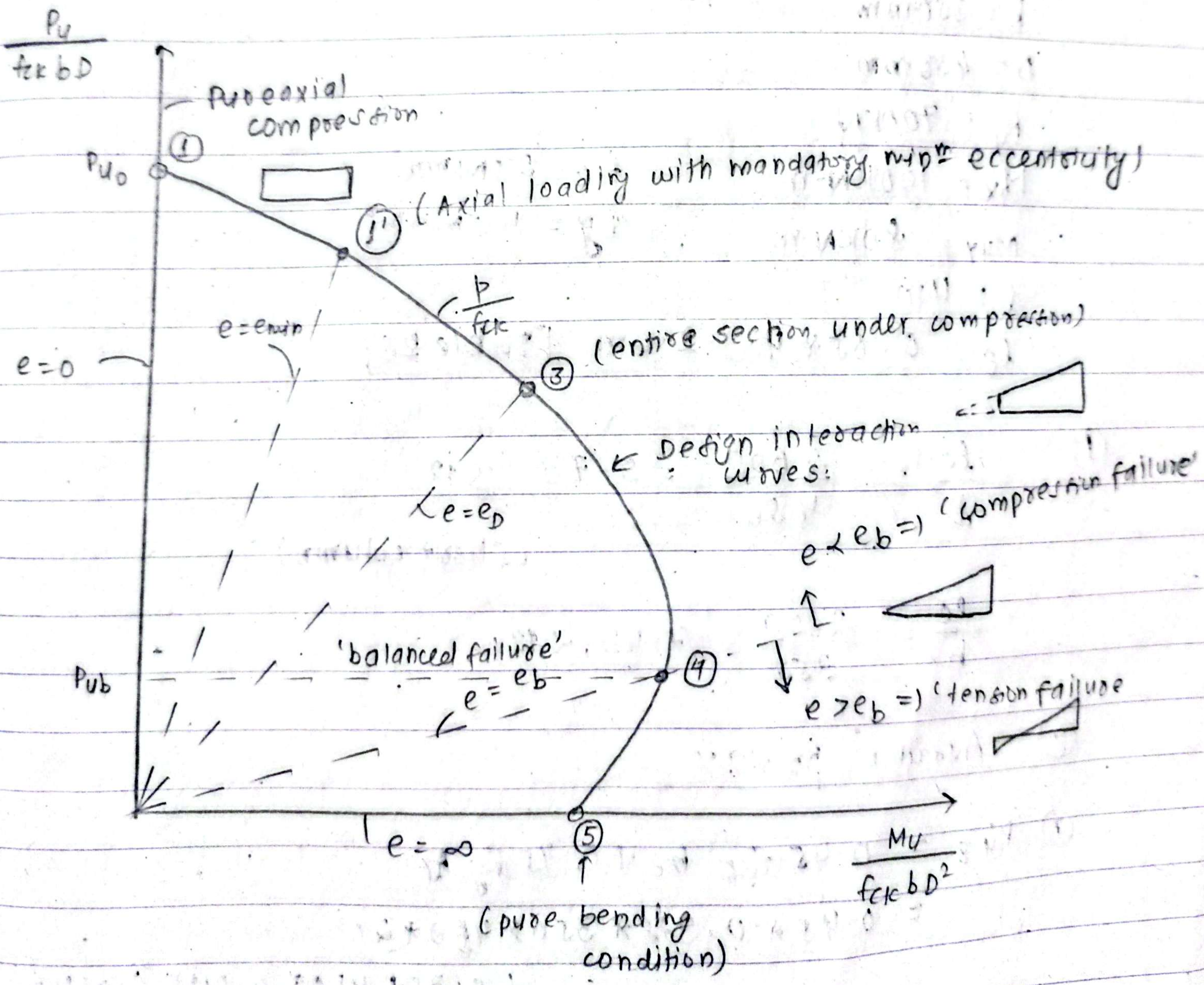


Fig: Interaction Diagram.

Q. 3-(b) Design a braced rectangular RC column having clear ~~span~~ height of 4m with cross sectional dimension of 450mm * 350mm subjected to design axial load of 700KN, design bending moments of 100KN-m about major axis and 80KN-m about minor axis. The column is held effectively at both ends and restrained against rotation at one end. Consider M25 concrete and Fe415 steel.

→ Soln:-

$b = 350mm$

$D = 450mm$

$P_u = 700KN$

$M_{ux} = 100KN\text{-}m$ $f_{ck} = 25N/mm^2$

$M_{uy} = 80KN\text{-}m$ $f_y = 415N/mm^2$

$l = 4m$

$l_e = 0.65 * 4 = 2.6m$ [Table 28].

① $\frac{l_e}{D} = \frac{2600}{450} = 5.77 < 12$

(short column)

$\frac{l_e}{b} = \frac{2600}{350} = 7.43 < 12.$

② Assume $p = 1.2\%$.

③ $P_{uz} = 0.45 f_{ck} A_c + 0.75 f_y A_s$

$= 0.45 * 0.988 * 350 * 450 * 25$

$+ 0.75 * 415 * 0.012 * 350 * 450$

$= 2144.4 KN.$

[078BCE178]

(4) $\frac{P_u}{P_{uz}} = \frac{700}{2144.4} = 0.33$

(5) $\frac{p}{f_{ck}} = \frac{1.2}{25} = 0.048$; $\frac{P_u}{f_{ck}\, b\, D} = \frac{700 \times 10^3}{25 \times 350 \times 450} = 0.18$

For x,

$\frac{d'}{D} = \frac{50}{450} = 0.11$

↳ Take $d'/D = 0.15$

Chart 45

$\frac{M_{ux_1}}{f_{ck}\, b D^2} = 0.091$

$\Rightarrow M_{ux_1} = 0.091 \times 25 \times 350 \times 450^2 = 161.24\, kNm$

For y

$\frac{d'}{b} = \frac{50}{350} = 0.143$

↳ Take $\frac{d'}{b} \simeq 0.15$.

Chart 45,

$\frac{M_{uy_1}}{f_{ck}\, b^2 D} = 0.091$

$M_{uy_1} = 0.091 \times 25 \times 350^2 \times 450 = 125.41\, kN\text{-}m$.

(6) α_n

$\left.\begin{array}{l} 0.2 \rightarrow 1 \\ 0.8 \rightarrow 2 \end{array}\right] \rightarrow 0.33 \Rightarrow 1.217$

$\alpha_n = 1.217$

For Biaxial Bending,

$\left(\frac{M_{ux}}{M_{ux_1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy_1}}\right)^{\alpha_n}$

$$\left(\frac{100}{161.24}\right)^{1.217} + \left(\frac{80}{125.41}\right)^{1.217}$$

$$= 1.138$$
$$> 1$$

Not safe

Next trial,

Take $p = 2\%$

$\rightarrow P_{uz} = 2325\ KN$

$\rightarrow \frac{P_u}{P_{uz}} = \frac{700}{2325} = 0.30$

$\rightarrow \alpha_n = 1.167$

$\rightarrow \frac{P}{f_{ck}} = 0.08$, $\frac{P_u}{f_{ck} bD} = 0.18$

chart 45

$\frac{M_{ux_1}}{f_{ck} bD^2} = 0.1$

$M_{ux_1} = 177.19\ KN\text{-}m.$

$\frac{M_{uy_1}}{f_{ck} b^2 D} = 0.1$

$M_{uy_1} = 137.8\ KN\text{-}m$

$\rightarrow$ Thus,

$$\left(\frac{100}{177.19}\right)^{1.167} + \left(\frac{80}{137.8}\right)^{1.167}$$

$$= 1.043 > 1$$

Next trial,

Take $p = 2.5\%$ $(< 4\%)$ ok

→ $P_{uz} = 2953.1\,kN$

→ $\frac{P_u}{P_{uz}} = 0.237$

→ $\alpha_n = 1.0617$

→ $\frac{p}{f_{ck}} = 0.1$; $\frac{P_u}{f_{ck} b D} = 0.18$

charts,

$$\frac{M_{ux_1}}{f_{ck} b D^2} = 0.128 \qquad \frac{M_{uy_1}}{f_{ck} b^2 D} = 0.128$$

$M_{ux_1} = \cancel{0.128}$ $226.8\,kNm$ $\qquad$ $M_{uy_1} = \cancel{0.72}$ $176.41\,kNm$.

→ Thus,

$$\left(\frac{100}{226.8}\right)^{1.0617} + \left(\frac{80}{176.4}\right)^{1.0617}$$

$$= 0.85 \leq 1$$

safe.

(4) Thus, $A_s = 2.5\%$ of $350 * 450 = 3937.5\,mm^2$

provide 28 mm ϕ bars.

$$\text{No. of bars} = \frac{3937.5}{\pi * 14^2} = 6.39 \approx 8$$

$(A_s)_{provided} = 8 * \pi * 14^2 = 4926.02\,mm^2$

$$\frac{4926.02}{350 * 450} * 100 = 3.13\% \; (< 4\%)$$

ok

Lateral ties [26.5.3.2.(C)]

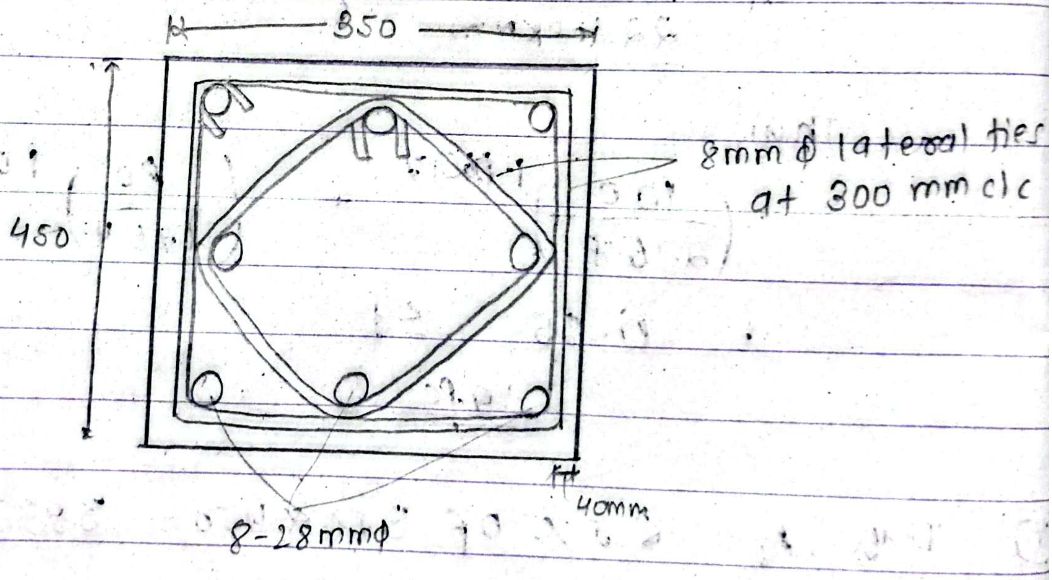
Diameter $\begin{cases} 1/4 \times 28 = 7mm \\ \text{or} \\ 6mm \end{cases}$ > max

provide 8mm dia ties.

Pitch $\begin{cases} b = 350mm \\ 16 \times 28 = 448mm \\ 300mm \end{cases}$ < (min)

Provide ~~spa~~ pitch of 300 mm

Thus provide 8 mm dia lateral ties at pitch of 300mm.



[078BCE178]

Q 4.(a) Explain the ductility requirement for a RC beam with a neat sketch.

→

- Longitudinal reinforcement → Min. 2 bars at top and bottom throughout
- Compression reinforcement = ≥ 0.24% of X-section area.
- Shear reinforcement = stirrups at both end up to 2d; closely spaced.
- Stirrup spacing → Max $\begin{cases} d/4 \\ 8 \times \text{dia of smaller longitudinal bar} \\ 100\,mm \end{cases}$
- Anchorage → 135° hook with 10 × bar diameter extension; properly confined.

Q. 4.(b): Design a two-way slab simply supported on all four edges for a room 6m × 4m clear span in size supported on 230mm thick walls. The superimposed load is 4 kN/m² and corners are not held down. Use M25 mix and Fe 415 grade steel.

→ Solution:-

$L_x = 4m$

$L_y = 6m$

$w = 230mm$

$w_I = 4 kN/m^2$

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

$f_y = 415 N/mm^2$.

Simply supported on four edges.

(i) Depth of slab:

$\left(\frac{L}{d} < \alpha \beta \gamma \lambda \delta\right)$

(shorter span)

$$d > \frac{\cancel{6000}\ 4000}{20 \times 1.3}$$

$d > \cancel{230.76mm}$ 153.85mm.

Take $\left[\begin{array}{l} D = 250mm \\ d = 230mm \end{array}\right]$

(ii) Effective span,

$$l_{eff} = \left.\begin{array}{l} l_{cs} + d \\ = l_{c/c} \end{array}\right\} \begin{array}{l} \rightarrow 4.23m \\ \rightarrow 4.23m \end{array}$$

Take, $l_x = 4.23m$

$l_y = 6.23m$

$\frac{l_y}{l_x} = \frac{5.23}{4.23} = 1.472 < 2$
(Two way slab)

(iii) Load calculation,

$w_s = 4 \, kN/m^2$

$FF = 1 \, kN/m^2$

dead load $= 25 * 0.25 = 6.25 \, kN/m^2$

Total factored load, $w_u = 1.5(4+1+6.25)$
$= 16.875 \, kN/m.$

(iv) Bending moment.

[Annex D-2-1] →

$M_x = \alpha_x w l_x^2$

$M_y = \alpha_y w l_x^2$

From table 27,

$\frac{l_y}{l_x} = 1.472 \Big] \quad \alpha_x = 0.103 \quad \alpha_y = 0.0474$

Thus,

$M_x = 0.103 * 16.875 * 4.23^2$
$= 31.1 \, kNm$

$M_y = 0.0474 * 16.874 * 4.23^2$
$= 14.31 \, kNm.$

(5) Area of reinforcement,

$$M_x = 0.87\, f_y\, A_{st}\, d\left(1 - \frac{f_y A_{st}}{f_{ck} b d}\right)$$

For, $M_x = 31.1\,kNm$, $d = 250 - 15 - \frac{8}{2} = 231m$

$A_{st\,x} = 383.46\,mm^2$.

For, $M_y = 14.31\,kNm$, $d = 250 - 15 - 8 - 8/2 = 223m$.

$A_{st\,y} = 180.15\,mm^2$.

(vi) Minimum reinforcement,

0.12% of $bD = 300\,mm^2$

Thus,

$A_{st\,x} = 383.46\,mm^2$

$A_{st\,y} = 300\,mm^2$

(vii) Spacing,

Assume 8mm bar provided.

$A_s = \pi/4 \times 8^2 = 50.3\,mm^2$.

For, $A_{st\,x} = 383.46\,mm^2$

$$S_x = \frac{1000 \times 50.3}{383.46} = 131.17\,mm.$$

For, $A_{st\,y} = 300\,mm^2$

$$S_y = \frac{1000 \times 50.3}{300} = 167.7\,mm.$$

[578BCE178]

spacing check,

3d or 300

3 × 230
||
690mm or 300mm

S < 300mm ok.

Provide 8mm bars at 120mm spacing in shorter span and 160mm spacing in longer span.

(viii) Provided reinforcement

shorter span,

$$(A_{st,x})_{prov} = \frac{1000 * 50.3}{120} = 419.17\,mm^2$$

Longer span

$$(A_{st,y})_{prov} = \frac{1000 * 50.3}{160} = 314.37\,mm^2$$

(ix) check for deflection (Cl. 23.2)

$$\frac{L}{d} < \alpha \beta \gamma \lambda \delta$$

$$d > \frac{l_x}{\alpha \beta \gamma \lambda \delta}$$

$\alpha = 20$ (continuous)

$$P_t = \frac{100 * 419.17}{1000 * 230} = 0.18\%$$

$$f_s = 0.58 * 415 * \frac{383.46}{419.17} = 220\,MPa$$

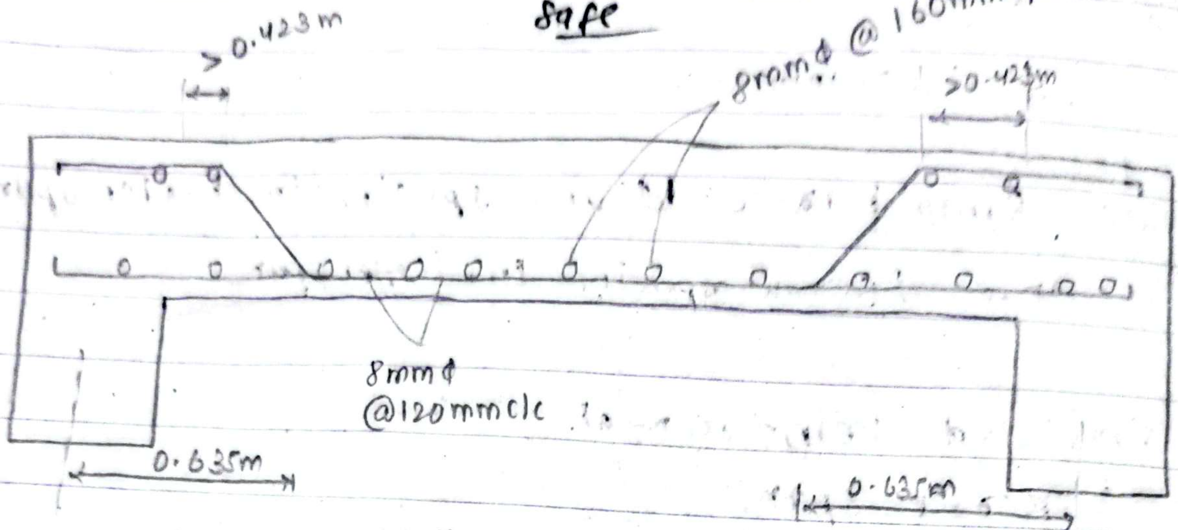
from fig 4 $\gamma = 1.9$

$$d > \frac{4.23}{20 \times 1.9}$$

$$d > 111.31\ mm$$

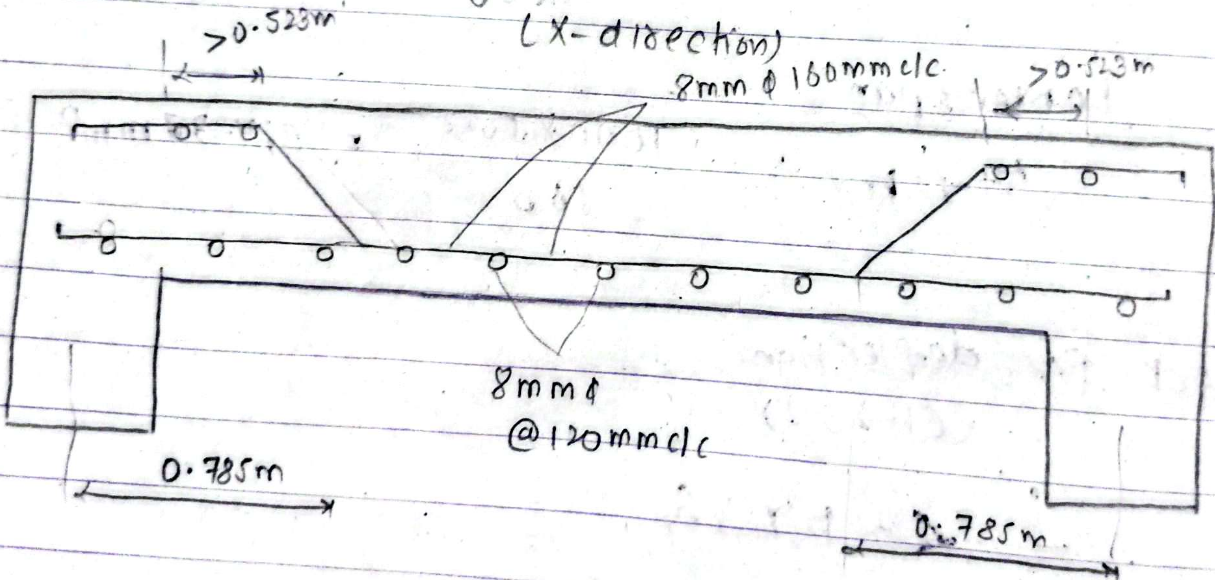
ok

safe



Longitudinal section

(X-direction)



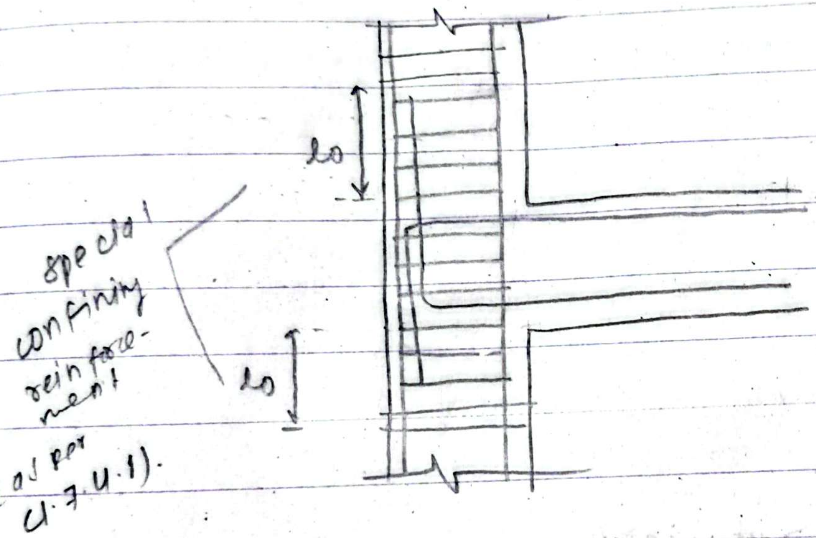
Longitudinal section

(Y-direction)

[078BCE178]

Q5(a) Discuss special confining reinforcement for ductile detailing of column with neat sketches.

→ In confinement zone
4.A both-ends.
(l_0) length ≥ greater of
{ 450mm
1/6 * clear height
column dimension.



(as per cl. 7.4.1).

Q.5(b) Design an isolated footing supporting a square column 400mm * 400mm with 8-20ϕ longitudinal reinforcement. The column is subjected to axial service load of 1000KN. Consider base of footing is 1m below ground level. Take allowable bearing capacity of soil 100KN/m³, M15 concrete and Fe415 steel. Also check for load transfer from column to footing.

→ Solution:-

$a = 400mm$
$b = 400mm$ } column.
8-20ϕ rebar

$P = 1000KN$

$P_u = 1.5 * 1000 = 1500 KN$

$q_a = 100 KN/m^3$

$f_{ck} = 15 N/mm^2$

$f_y = 415 N/mm^2$

(1) Area of footing required,

$$A_{req} = \frac{1.15 * P}{q_a} = \frac{1.15 * 1000}{100} = 11.5 m^2.$$

(2) Size of footing

say, square footing, $L = B$

$L * L = 11.5$

$\Rightarrow L = 3.39$

Adopt, $\underline{L = B = 3.5m}$

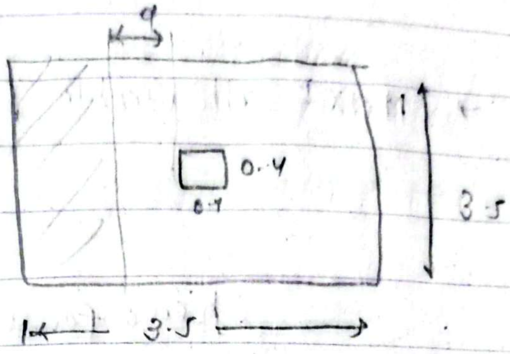
(3) Factored bearing pressure on footing,

$$q_u = \cancel{Pa}\ \frac{P_u}{A_{adopted}} = \frac{1500}{3.5 * 3.5}$$

$$q_u = 122.4\ kN/m^2.$$

(4) Depth of foundation:

(a) one way shear



Critical shear (V_u)

$$= q_u * B * \left(\frac{L-b}{2} - d\right)$$

$$= 122.4 * 3.5 * \left(\frac{3.5-0.4}{2} - d\right)$$

$\rightarrow V_u = 428.4(1.55 - d)$

$\rightarrow$ $\tau_c \rightarrow$ % steel assume 0.25%

$\tau_c = 0.35\ N/mm^2$] Table 19 (M15)

Shear force resisted, $V_c = \tau_c * B * d$

$$= 0.35 * 3.5 * d * 10^3$$

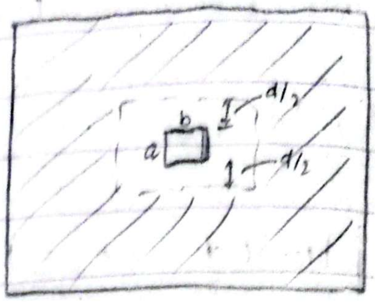
$$= 1225d$$

Equating-

$428.4(1.55 - d) = 1225d$

$\Rightarrow$ [d: 402mm]

(b) Two way shear (punching shear)



→ Shear force at critical section,

$= q_u * (L * B - (b+d)*(a+d))$

$= 122.4 * [3.5 * 3.5 - (0.4+d)^2]$

→ Max^m allowable punching stress

$\tau_c = 0.25\sqrt{f_{ck}} = 0.25\sqrt{15} = 0.968 \, N/mm^2$

Shear force resisted $= (b+d+a+d) * 2 * d * \tau_c$

$= (0.8+2d) * 1.936d.$

equating

$122.4(3.5 * 3.5 - (0.4+d)^2) = (0.8+2d) * 1.936d * 10^3$

or. [d = 436 mm]

© Moment criteria.

$M = (w a^2/2)$

$= (q_u * B) * \left(\frac{L-b}{2}\right)^2 * \frac{1}{2}$

$= 122.4 * 3.5 * \left(\frac{3.5-0.4}{2}\right)^2 * \frac{1}{2}$

$= 514.61$ kN m.

For Fe415,

$d = \sqrt{\frac{514.61 * 10^6}{0.138 * 15 * 3500}}$

$=$ ~~788.3mm~~ 266.51 mm

[078BCE178]

⑤ Adopt effective depth of $d = 440mm$

$D = 440 + 50 + \frac{16}{2} + 16 \approx 520mm$

$d = 520 - 50 - \frac{16}{2} - 16 = 446mm.$

⑥ Flexural reinforcement,

$$514.6 * 10^6 = 0.87 * 415 * A_{st} * 446 * \left(1 - \frac{415 * A_{st}}{15 * 3500 * 446}\right)$$

$\Rightarrow A_{st} = 3400.75\ mm^2$

Providing 16 mm bars

Total No. of bars $= \cancel{\frac{1000 * \pi * 8^2}{3400.75}}$ $\frac{3400.75}{\pi * 8^2}$

$= 16.91.$

spacing $= \frac{3500 * \pi * 8^2}{3400.75} = 206.93mm.$

$A_{st,min} = 0.12\%$ of $b * D = \frac{0.12}{100} * 3500 * 520$

$= 2184mm^2 < A_{st}$

ok.

~~For 0.25%, $A_{st} = 0.25\% * 3500 * 520$~~

~~$= 4550\ mm^2.$~~

Provide reinforcement at 200mm c/c.

$A_{st,prov} = 3518.58mm^2.$

7. Bearing (load transfer) from column to footing
(Cl. 34.4)

(Bearing stress at column-footing interface) $\sigma_u = \frac{1.5 * P}{A_c} = \frac{1.5 * 1000 * 10^3}{400 * 400}$

$= 9.375 N/mm^2$

Permissible bearing stress $= 0.45 f_{ck} \sqrt{\frac{A_g}{A_c}}$

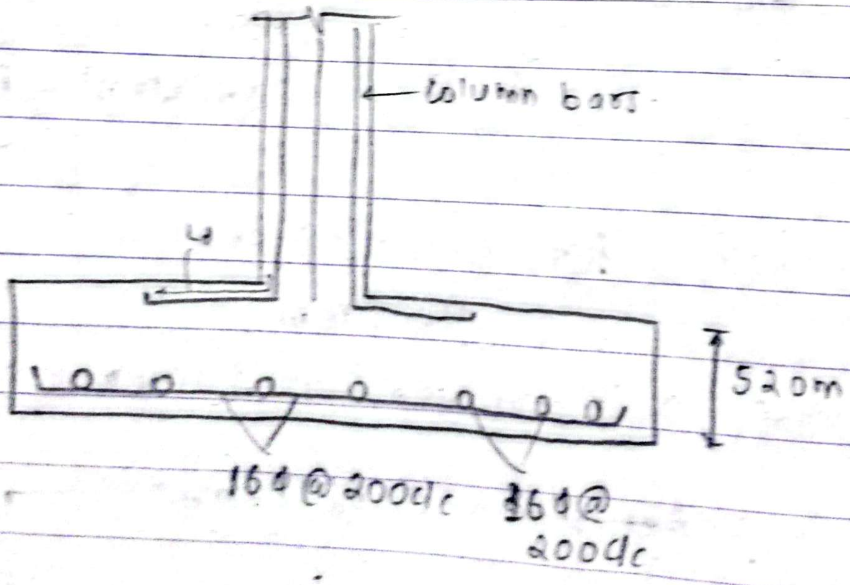
$= 0.45 * \cancel{20}\ 15 \left(\cancel{\sqrt{\frac{(3500)^2}{(400)^2}}}\right)^2$

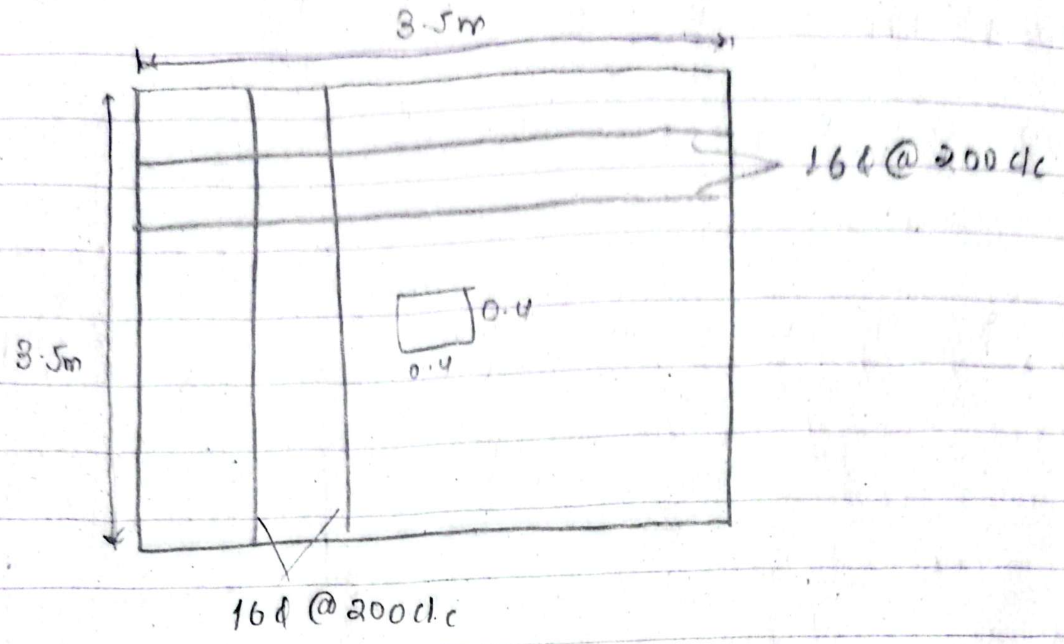
$= \cancel{59.06 N/mm^2}$ $13.5 N/mm^2 > \sigma_u$

Ok

But $\sqrt{\frac{A_g}{A_f}} = \sqrt{\frac{3500^2}{400^2}} \not> 2$

Bearing alone is adequate → no dowel bars required for load transfer.





Plan

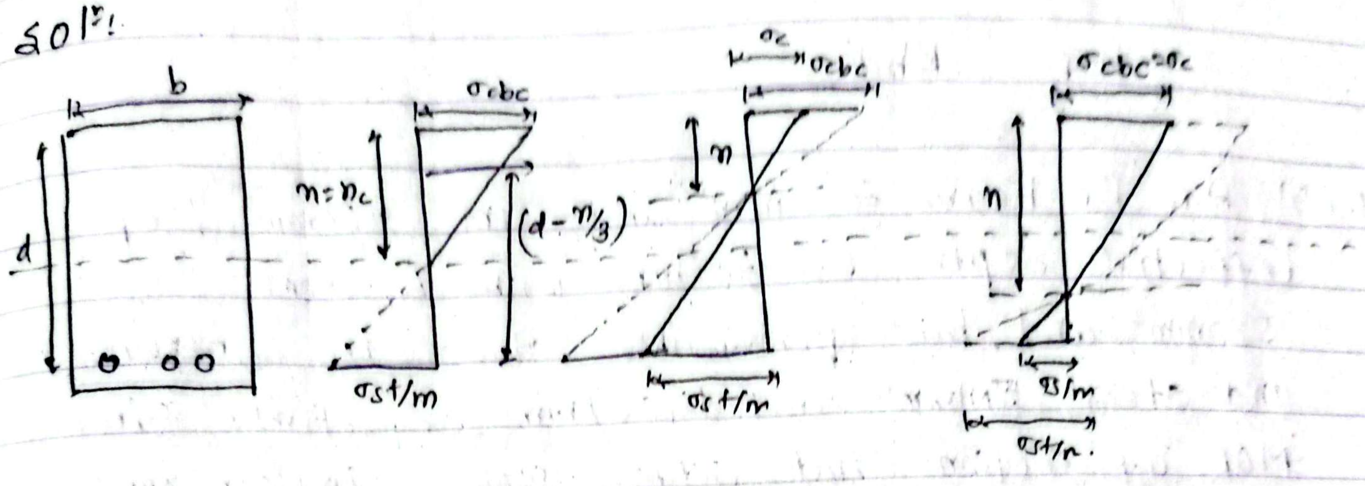
[2080 Baisakh]

२२ ३.

[078BCE148]

Q.1(a) What are balanced, under-reinforced and over-reinforced sections? Explain with neat sketches of stress distribution and MOR expression.

→ Soln:



section Balanced ($n=n_c$) Under-reinforced ($n<n_c$) Over-reinforced ($n>n_c$)

For Balanced section,

$$MOR = C*l \text{ 'or' } T*l$$

$$= \left(\frac{1}{2}*\sigma_{cbc}*b*n\right)*\left(d-\frac{n}{3}\right)$$

'or'

$$\sigma_{st}*A_{st}*(d-n/3)$$

For under reinforced section

$$MOR = T*l$$

$$= \sigma_{st}*A_{st}*\left(d-\frac{n}{3}\right)$$

For overreinforced section

$$MOR = C*l$$

$$= \left(\frac{1}{2}*\sigma_{cbc}*b*n\right)*\left(d-\frac{n}{3}\right)$$

$$= \left(\frac{1}{2}*\sigma_{cbc}*b*(kd)\right)*\left(d-\frac{kd}{3}\right)$$

$$MOR = \frac{1}{2} \sigma_{cbc} * k \left(1 - \frac{k}{3}\right) bd^2$$

$$= \frac{1}{2} \sigma_{cbc} * k * j * b d^2$$

$$MOR = Rbd^2$$

Q1: (b) An RC beam 230mm wide and 400mm deep (effective depth of 380 mm) with A steel of 580mm² and has permissible stress in concrete and steel 5N/mm² and 140N/mm² respectively. Find MOR of section and actual stresses in concrete and steel.

→ Soln.

width (b) = 230mm

$d = 380mm$

$D = 400mm$

$A_{st} = 580 \, mm^2$

$\sigma_{cbc} = 5 N/mm^2$

$\sigma_{st} = 140 N/mm^2$

$$m = \frac{280}{3\sigma_{cbc}} = 18.67$$

Now

(i) actual NA depth

$M_T = M_C$ (about NA)

or $m A_{st} * (d - n) = b \cdot n \cdot \frac{n}{2}$

or $18.67 * 580 * (380 - n) = 230 * \frac{n^2}{2}$

or $\boxed{n = 147.85 mm}$

(ii) critical Neutral axis,
stress diagram
(similar Δ)

$$\frac{\sigma_{cbc}}{\sigma_{st}/m} = \frac{x_c}{d - x_c}$$

or $\frac{5}{140/18.67} = \frac{x_c}{380 - x_c}$

or $\boxed{x_c = 152.02\,mm}$

(iii) $x < x_c \rightarrow$ under-reinforced section

(iv) For steel, $\sigma_s = \sigma_{st} = 140\,N/mm^2$

For concrete, $\frac{\sigma_c}{140/18.67} = \frac{147.85}{380 - 147.85}$

$\Rightarrow \sigma_c = 4.77\,N/mm^2$

actual stresses,

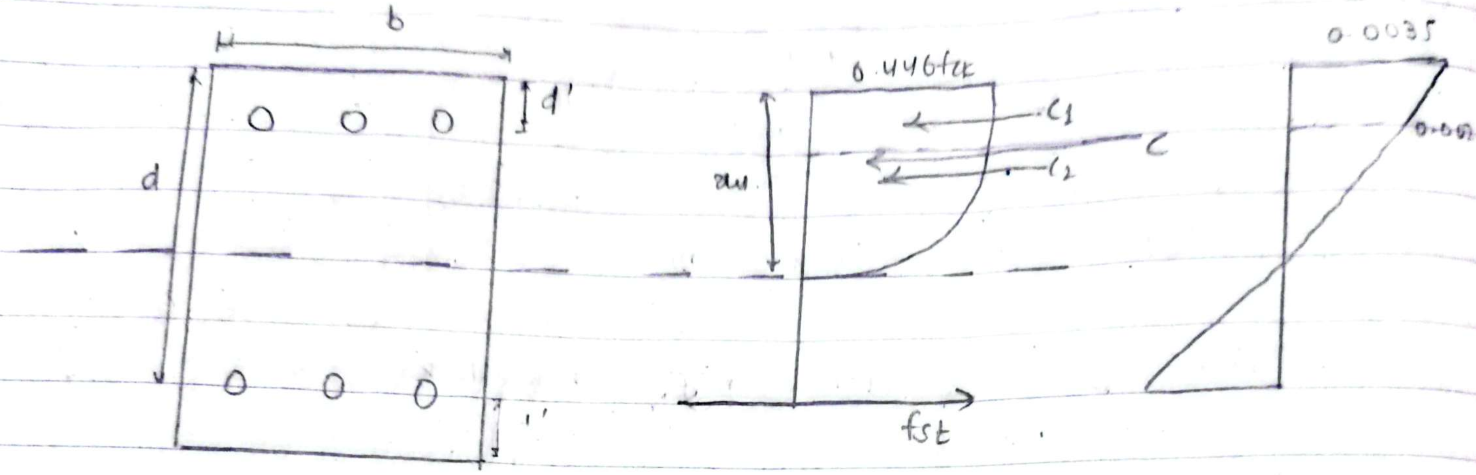
(v) $MOR = \sigma_{st}\, A_{st} * \left(d - \frac{x_a}{3}\right)$

$= 140 * 580 * \left(380 - \frac{147.85}{3}\right)$

$= 26.854\,KNm.$

Q.1. (c) Design steps of doubly reinforced section with neat sketches.

→ Solⁿ:-



Doubly reinforced beam

(1) Given, b, D, d, fck, fy

(2) Calulate factored bending moment (M_u).

(3) Calculate $M_{u,l} = R\,bd^2 f_{ck}$

(4) Check, If $M_u > M_{u,l}$ (Doubly reinforced beam)

$M_u \leq M_{u,l}$ (Singly reinforced beam)

(5) $\Delta M_u = M_u - M_{u,l}$.

(6) Calculate A_{st_1},

$$M_{u,l} = 0.87\, f_y\, A_{st_1}\, d \left(1 - \frac{f_y\, A_{st_1}}{f_{ck}\, bd}\right)$$

[078BCE178]

⑦ Calculate A_{sc}

$$\Delta M_u = (f_{sc} - f_{cc}) A_{sc} (d - d')$$

or $$\Delta M_u = (f_{sc} - 0.446 f_{ck}) A_{sc} (d - d')$$

⑧ Calculate A_{st_2}

$$C = T$$

or $$(f_{sc} - 0.447 f_{ck}) A_{sc} = 0.87 f_y A_{st_2}$$

⑨ $A_{st} = A_{st_1} + A_{st_2}$

$A_{sc} = A_{sc}$

⑩ Check Minimum reinforcement [Cl 26.5.1.]

$$\frac{A_s}{bd} = \frac{0.85}{f_y}$$

$A_{st} > A_s$ ✓

⑪ check Maximum reinforcement

$$A_s = 0.04 bD$$

$$A_{st} < A_s.$$

⑫ Provide necessary number of rebars in compression and tension side

Q. 2.(a) An RC beam with effective depth of 500mm and breadth of 400mm contains 5# 25mm diameter bars, out of which two bars are to be bent up at 45° near the end of support. The beam carrying a UDL of 1000 kN/m over a clear span. Calculate shear ~~reinforcement of~~ resistance of bend up bars and design the additional stirrups if required.

→ Solⁿ:

$d = 500mm$

$b = 400mm$

$w_u = 1.5 * 100 = 150 kN/m$

$l = 6m$

$A_{st} = 5 * \frac{\pi}{4} * 25^2 = 2454.37 mm^2$.

(i) Factored shear force $(V_u) = \frac{w_u l}{2}$

$= \frac{150 * 6}{2}$

$= 450 kN$

(ii) Nominal shear stress,

$\tau_v = \frac{V_u}{bd}$

$= \frac{450 * 10^3}{500 * 400}$

$= 2.25 N/mm^2$.

At support two bars are bent up.

here,

$$P_t = \frac{3 \times \frac{\pi}{4} \times 25^2}{400 \times 500} \times 100 = 0.74\%$$

Assume, M20 grade concrete and Fe415 steel.

From table 19,

$$\left.\begin{array}{l} 0.50 \rightarrow 0.48 \\ 0.75 \longrightarrow 0.56 \end{array}\right] \Rightarrow 0.74\% \Rightarrow 0.557$$

$\tau_c = 0.557 N/mm^2$

$\tau_{c,max} = 2.8 N/mm^2$
(Table 20)

(iii) For bent up bars at 45°,

$\alpha = 45°$

$V_{us}' = 0.87 f_y A_{sv} \sin\alpha$

$= 0.87 \times 415 \times 2 \times \frac{\pi}{4} \times 25^2 \times \sin 45°$

$= 250.64 kN$

(iv) $V_{us} = V_u - \tau_c bd = 450 - 0.557 \times 400 \times 500$

$= 338.6 kN.$

But, $V_{us}' \not> \frac{V_{us}}{2} (= 169.3 kN)$

Thus, for remaining 169.3kN vertical stirrups are provided.

(v) For vertical stirrups

$$169.3 \times 10^3 = V_{us} = \frac{0.87 f_y A_{sv} \times d}{S_v}$$

try providing 2-legged 8 mmφ stirrup

$$169.3 \times 10^3 = \frac{0.87 \times 415 \times 100.53 \times 500}{S_v}$$

or $[S_v = 107.19\ mm]$

<u>check for spacing</u>

(Cl. 26.5.1.6)

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

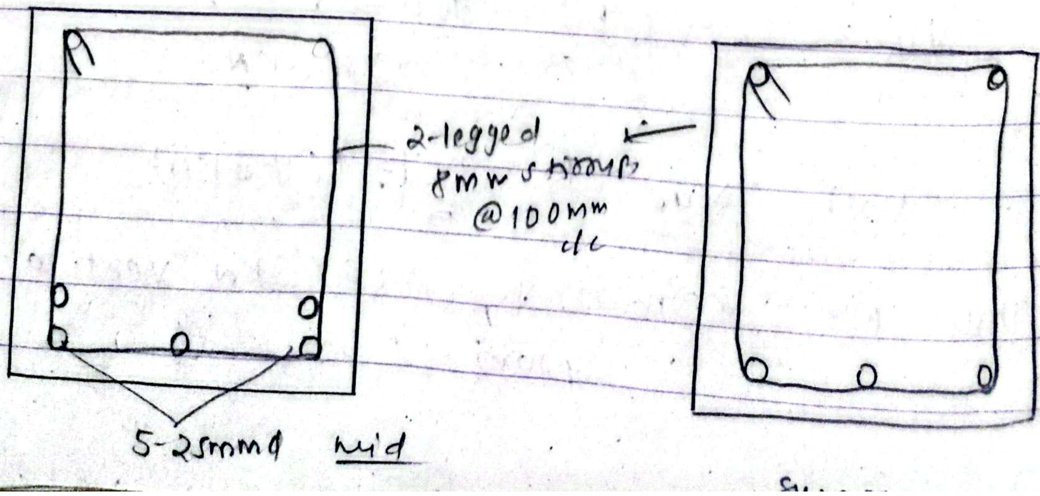
or $$\frac{100.53}{400 \times S_v} \geq \frac{0.4}{0.87 \times 415}$$

$$S_v \leq 226.85 mm$$

(Cl. 26.5.1.5)

$$S_{max} = 0.75d = 0.75 \times 500 = 375 mm$$
or
300mm

Thus, provide 2-legged 8mm dia vertical stirrups at 100 mm c/c



[078BCE178]

Q.2(b) Design a column 400mm × 500mm having unsupported length of 4m with both ends effectively and restrained against rotation at one end with following data.

Factored axial load = 1600 kN

Factored Moment = 150 kNm, 50 kNm

→ Soln:-

$b = 400mm$

$D = 500mm$

$l = 4m$

[Table 28] $l_e = \cancel{0.65} \; 0.80 \times 4000 = 3200mm.$

say effective cover = 50mm

then, $d = 500 - 50$
$= 450mm.$

$f_{ck} = 25 N/mm^2$, $f_y = 500 N/mm^2$.

(I) Slenderness check,

SR $\frac{l_e}{D} = \frac{3200}{500} = 6.4 < 12$

$\frac{l_e}{b} = \frac{3200}{400} = 8 < 12$

It is short column on both axes.

(II) Lets provide 2% steel of gross section

i.e. $p = 2\%$

$M_{ux} = 150 kNm$

$M_{uy} = 50 kNm$

(iii) For biaxial bending,

$P_{uz} = 0.45 f_{ck} A_c + 0.75 \times f_y A_s$

$= 0.45 \times 25 \times 0.98 \times 400 \times 500 + 0.75 \times 500 \times 0.02 \times 400 \times 500$

$= 3705 \text{ kN}$

$$\frac{P_u}{P_{uz}} = \frac{1600}{3705} = 0.432$$

$0.2 \rightarrow 1$
$0.8 \rightarrow 2$] $0.432 \Rightarrow \alpha_n = 1.387$

(iv) Find M_{ux_1} & M_{uy_1}

For X

$$\frac{d'}{D} = \frac{50}{500} = 0.1 \; ; \; \frac{p}{f_{ck}} = \frac{2}{25} = 0.08 \; ; \; \frac{P_u}{f_{ck} bD} = 0.32$$

Chart 48,

$$\frac{M_{ux_1}}{f_{ck} bD^2} = 0.128$$

$M_{ux_1} = 0.128 \times 25 \times 400 \times 500^2 = 320 \text{ kNm}.$

For Y

$$\frac{d'}{b} = \frac{50}{400} = 0.125 \; ; \; \frac{p}{f_{ck}} = 0.08 \; ; \; \frac{P_u}{f_{ck} bD} = 0.32$$

Chart 48, $\left(\frac{d'}{b} = 0.1\right)$

$$\frac{M_{uy_1}}{f_{ck} b^2 D} = 0.128$$

Chart 49 $\left(\frac{d'}{b} = 0.15\right)$

$$\frac{M_{uy_1}}{f_{ck} b^2 D} = 0.116$$

Thus,

$$\frac{M_{uy_1}}{f_{ck} b^2 D} = 0.122$$

$M_{uy_1} = 0.122 \times 25 \times 400^2 \times 500 = 244 kNm$

(V) Check

$$\left(\frac{M_{ux}}{M_{ux_1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy_1}}\right)^{\alpha_n}$$

$$= \left(\frac{150}{320}\right)^{1.387} + \left(\frac{50}{244}\right)^{1.387}$$

$= 0.48 \leq 1$

OK safe

(VI) Longitudinal reinforcement

$A_{sc} = 2\%$ of bD

$= \frac{2}{100} \times 400 \times 500$

$= 4000 mm^2$

(VII) Providing 28 mm ϕ bars

No. of bars required $= \frac{4000}{\pi \times 14^2} = 6.496$

≈ 8

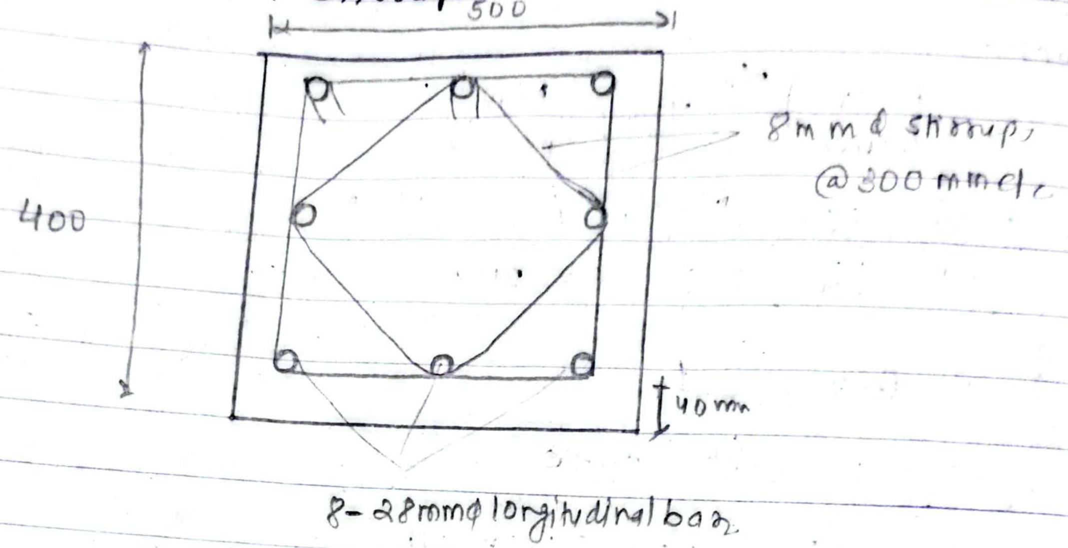
Providing 8 - 28mmϕ bars

(IX) Transverse reinforcement

(26.5.3.2) diameter $\left\{ \begin{array}{l} 1/4 \times 28 = 7mm \\ \text{or} \\ 6mm \end{array} \right\}$ $\geq 7mm$ say 8mm

Pitch $\left\{\begin{array}{l} 400 \\ 16 \times 28 = 448 \\ 300\text{mm} \end{array}\right\} \leq 300\text{mm}$ say 300mm

Provide 8mmϕ stirrups at 300mm c/c.



Q.3.(a) Design a reinforced concrete rectangular slab of size 4m*5m to support an imposed load of 4KN/m² and floor finish of 1KN/m². The slab has two adjacent edges discontinuous with slab resting on 275mm wide beam. Check safety of slab against shear and deflection. Use M20 grade concrete and Fe 415 grade steel. (Design of torsional reinforcement is not required)

→ Sol^n

4m*5m

$w_I = 4\text{KN/m}^2$

$FF = 1\text{KN/m}^2$

$w = 275\text{mm}$

$f_{ck} = 20\text{N/mm}^2$ $f_y = 415\text{N/mm}^2$

[078BCE178]

(#) Two adjacent edges discontinuous → case No. (4) Table 26

(i) Depth,

$$d > \frac{L}{33}$$

$$> \frac{4000}{33}$$

$d > 121.21mm$

Adopt, $D = 150mm$

$d = 150 - 20 = 130mm$ (providing 20 mm effective cover)

(ii) effective length

2.2) $\frac{4000}{12} = 333.33 > 275mm$

so,

$$\left.\begin{array}{l} l_e = l_{cs} + d = 4.13m \\ l_{c/c} = 4.275mm \end{array}\right\}$$

Thus,

$l_x = 4.13m$

$l_y = 5.13m$

(iii) $\frac{l_y}{l_x} = \frac{5.13}{4.13} = 1.242 < 2$

(Two-way slab)

(iv) Load calculation

$w_s = 4 kN/m^2$

$FF = 1 kN/m^2$

Deadload $= 25 * 0.15 = 3.75 kN/m^2$

For 1 m width of slab,

Total factored load, $w_u = 1.5(4+1+3.75)\times 1$

$= \cancel{8.83\ kN/m}$

$= 13.125\ kN/m$

(v) Bending moment and shear force.

$M_x = \alpha_x w l_x^2$

$M_y = \alpha_y w l_x^2$]

From table 26,

$\alpha_x^- = 0.061$ } $\alpha_y^- = 0.047$

$\alpha_x^+ = 0.046$ } $\alpha_y^+ = 0.035$

Thus,

$M_x^- = 0.061 * 13.125 * 4.13^2 = 13.656\ kNm$

$M_x^+ = 0.046 * 13.125 * 4.13^2 = 10.298\ kNm$

$M_y^- = 0.047 * 13.125 * 4.13^2 = 10.522\ kNm$

$M_y^+ = 0.035 * 13.125 * 4.13^2 = 7.835\ kNm$

(VI) Reinforcement required,

$$M_u = 0.87\, f_y A_{st} d \left(1 - \frac{A_{st} f_y}{b d f_{ck}}\right)$$

For, $M_x^- = 13.656\ kNm \Rightarrow A_{st,x}^- = 305.88\ mm^2$

$M_x^+ = 10.298\ kNm \Rightarrow A_{st,x}^+ = 227.677\ mm^2$

$M_y^- = 10.522\ kNm \Rightarrow A_{st,y}^- = 232.83\ mm^2$

$M_y^+ = 7.835\ kNm \Rightarrow A_{st,y}^+ = 171.63\ mm^2$

$A_{st,min} = 0.12\%\ bD = 0.12\% * 1000 * 150 = 180\ mm^2$

Thus, $A_{st,y}^+ = 180\ mm^2$.

(vii) Depth check,

For Fe415.

$$d > \sqrt{\frac{13.656 * 10^6}{0.138 * 20 * 1000}}$$

> 70.34 mm

d adopted = 130 mm

ok

(viii) Spacing req^d,

$$S = \frac{1000 * A_s}{A_{st}}$$

Say providing 8mmϕ rebars

$A_s = \frac{\pi}{4} * 8^2 = 50.3 \text{ mm}^2$

For, $A_{st,x}^- = 305.88 \text{ mm}^2$, $S_x^- = 164.44$ mm

$A_{st,x}^+ = 227.677 \text{ mm}^2$, $S_x^+ = 220.93$ mm

$A_{st,y}^- = 232.83 \text{ mm}^2$, $S_y^- = 216.04$ mm

$A_{st,y}^+ = 180 \text{ mm}^2$, $S_y^+ = 279.44$ mm.

$< 3 * 130 = 390$ mm

or

300 mm

ok

Provide, 8mm ~~stirrup~~ rebars along short direction at 160mm c/c & along long direction at 210mm c/c spacing.

(ix) Reinforcement provided,

$$A_{st,x} = \frac{1000 * 50.3}{160} = 314.37 \text{ mm}^2$$

$$A_{st,y} = \frac{1000 * 50.3}{210} = 239.53 \text{ mm}^2.$$

Design shear

$$V_x = \frac{w l_x}{3} = \frac{13.125 \times 4.13}{3} = 18.071\,KN$$

$$V_y = w l_x \left(2 - \frac{l_x}{l_y}\right)$$

$$= 13.125 \times 4.13 \left(2 - \frac{4.13}{5.13}\right)$$

$$= 64.77\,KN$$

'OR' Taken as,

$$V_u = \frac{w_u l_x}{2} = \frac{13.125 \times 4.13}{2} = 27.10\,KN.$$

(x) Check for shear

Nominal shear $\tau_v = \frac{V_u}{bd}$

$$= \frac{27.10 \times 10^3}{1000 \times 130}$$

$$= 0.208\ N/mm^2.$$

$$P_t = \frac{100 \times 314.37}{1000 \times 130} = 0.242\ \%.$$

[Table 19]

M20

for 0.15 → 0.28
0.25 → 0.36] ⇒ 0.242 ⇒ 0.354

$$\tau_c = 0.354\ N/mm^2$$

$$\tau_v < \tau_c$$

safe in shear

[078BCE148]

(x) check for deflection.
(Cl. 23.2.1)

$$\frac{L}{d} < \alpha \beta \gamma \lambda \delta$$

→ For continuous, $\alpha = 26$
→ span < 10m, $\beta = 1$
→ For γ

$$f_s = 0.58 * 415 * \frac{305.88}{314.37} = 234.2 \text{ N/mm}^2$$

$$P_t = \frac{100 * 314.37}{1000 * 130} = 0.242\%$$

From fig. 4,

$\gamma = 1.6$

→ $\lambda = 1$ (no compression reinforcement)
→ $\delta = 1$ (rectangular beam)

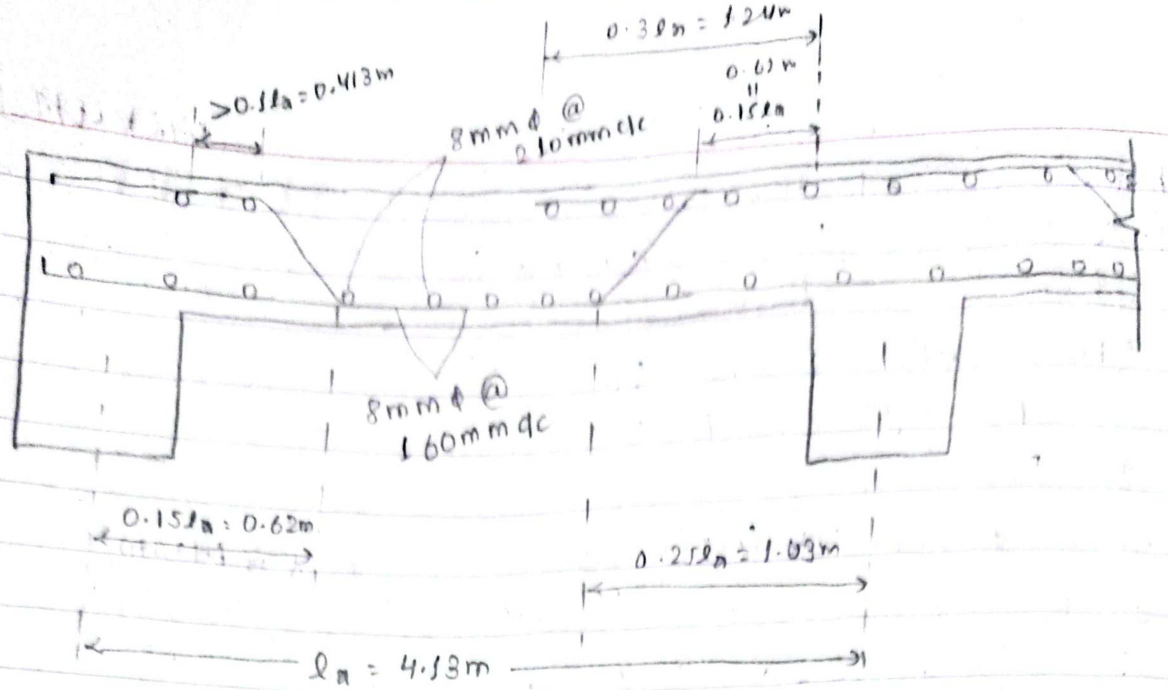
Thus,

$$d > \frac{4000}{26 * 1.6}$$

> 96.15

d adopted = 130 mm

safe in deflection.



Q.3(b) What is philosophy of design of structures in earthquake prone region Explain? Explain about design for strength and ~~safety~~ stability

→ The philosophy of design of structures in earthquake prone region as per IS codes 13920 are:

1. NO Damage during minor earthquakes
   ↳ should remain elastic and suffer no damage.
   ↳ ordinary designs with normal load combination and minimum code requirements.

2. Controlled damage during moderate Earthquake
   ↳ Some non-structural damage is accepted.
   ↳ Structure remain serviceable.
   ↳ Minor repair may be needed but collapse is not expected.

3. Prevent collapse during major earthquakes.
↳ Structure may undergo significant damage but should not collapse.
↳ life safety.
↳ Achieved through ductile design and detailing

Design for strength:

1. seismic force calculation:

Based on shear,

$$V_b = A_h \times W$$

V_b = design base shear

A_h = Hz. seismic coefficient

W = seismic wt. of building

2. Load combination (as per IS 456 and IS 1893)
   - 1.5 (DL + LL)
   - 1.2 (DL + LL ± EL)
   - 1.5 (DL ± EL)
   - 0.9 DL ± 1.5 EL

3. Design for load Resistance.
↳ structures are designed to resist horizontal seismic forces.

Design for Ductility

1. Material requirements
   - Use high-strength, ductile material (Min^m grade of concrete = M40)

2. Structural detailing
   - Special confining reinforcement in columns.
   - Strong column-weak beam design philosophy

$$\Sigma M_c > 1.45 M_b$$

3. Beam detailing
   - Minimum and maximum reinforcement limits.
   - Use closely spaced stirrups near support to resist shear and enhance ductility

4. Column and Beam-column Joint Detailing:
   - Lateral ties closely spaced in critical region.
   - Enhanced shear strength through transverse reinforcement

5. Lap splices and Anchorage:
   - Splices avoided in critical regions
   - proper anchorage length and hooks required for ductile behavior.

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[078BCE178]

Q.4) Design an isolated footing for a 450mm × 500mm sized column, with 6# 20mm diameter bar. Carrying factored axial load of 1100KN and factored uniaxial moment of 120KN-m at column base. Take depth of footing 1.5m and safe-bearing capacity of soil 100KN/m². Use M20 grade concrete and Fe500 grade steel.

→ Soln:-

$P_u = 1100 KN$

$M_{ux} = 120 KNm$

$D_f = 1.5 m$

$q_a = 100 KN/m^2$

$f_{ck} = 20 N/mm^2$

$f_y = 500 N/mm^2$

Now,

① $\frac{P}{A} + \frac{M}{Z} \leq q_a$

or $\frac{\frac{1100}{1.5}}{B \times L} + \frac{\frac{120}{1.5}}{BL^2/6} \leq 100$

Taking $L = B$

$B = L = 2.99 m$

Taking $[L = B = 3m]$

② Factored soil pressure,

$q_u = \frac{P_u}{A} \pm \frac{M_u}{Z} = \frac{1100}{3 \times 3} \pm \frac{120}{\frac{3 \times 3^2}{6}}$

$q_{u,max} = 148.89 KN/m^2$

$q_{u,min} = 95.56 KN/m^2$

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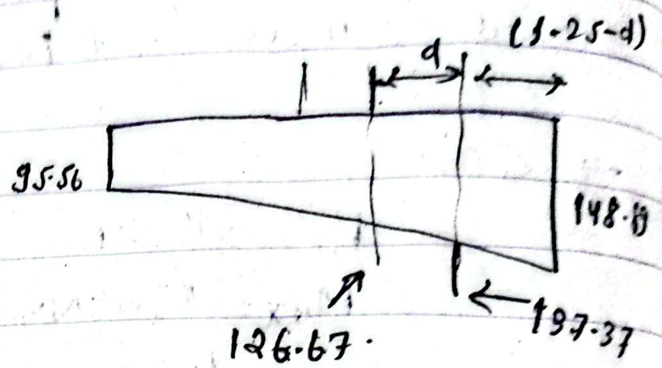
③ Depth calulation

Gen. one-way shear critical so taking that.

assume $d = 600\text{mm}$

$q_{d \text{ from face}} = 137.37\,\text{kN/m}^2$



$SF = \left(\dfrac{148.19 + \cancel{95.56}\,137.37}{2}\right) * (1.25-d) * 3$

$= 428.34\,(1.25-d)$

Say % steel = 0.25%

$\tau_c = 0.36\,\text{N/mm}^2$ (M20)] table 19

$0.36 * 3 * d * 10^3 = 438.34 * (1.25-d)$

$\Rightarrow d = \cancel{0}.361\,\text{mm}$.

Taking:

$d = 400\text{mm}$

$D = 400 + 50 + 16 + \frac{16}{2} = 474\text{mm}$

$\approx 480\text{mm}$

$d = 406\text{mm}$

④ Moment at face

$$M = (126.67 \times 3) \times \frac{1.25^2}{2}$$

$$= 296.88 \text{ kNm}$$

⑤ Reinforcement required.

$$296.88 \times 10^6 = M = 0.87 \underset{}{\overset{500}{f_y}} A_{st} \overset{406}{d} \left(1 - \frac{A_{st} * \overset{500}{f_y}}{\underset{3000}{B} * \underset{406}{d} * \underset{20}{f_{ck}}}\right)$$

solving

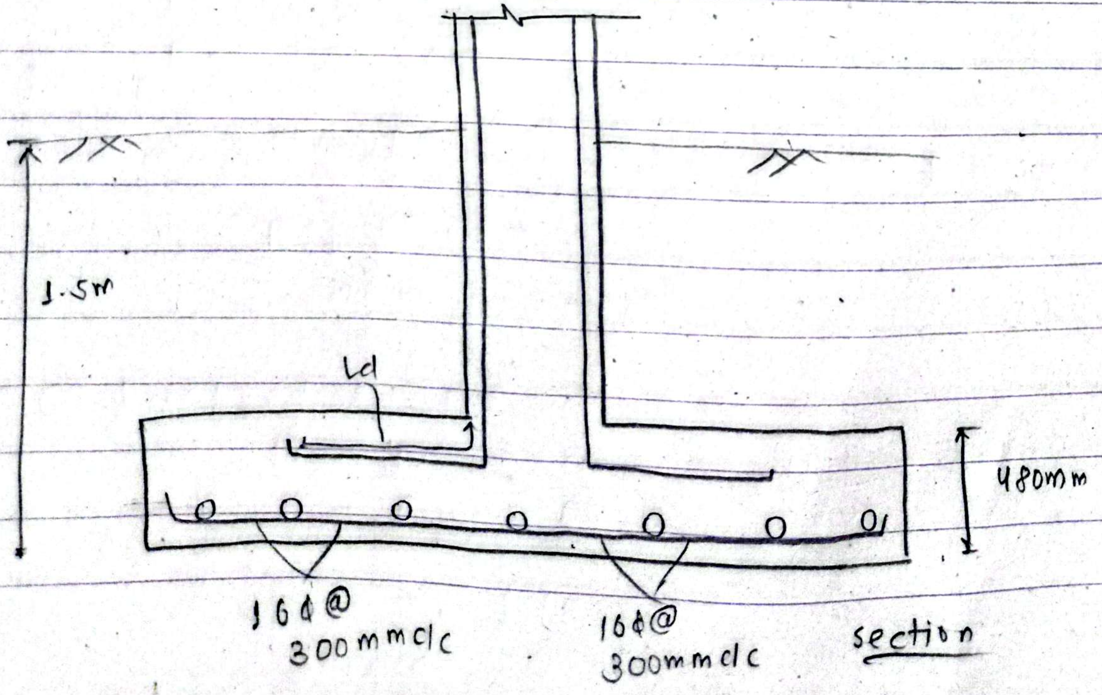
$A_{st} = 1743.37 \text{ mm}^2$

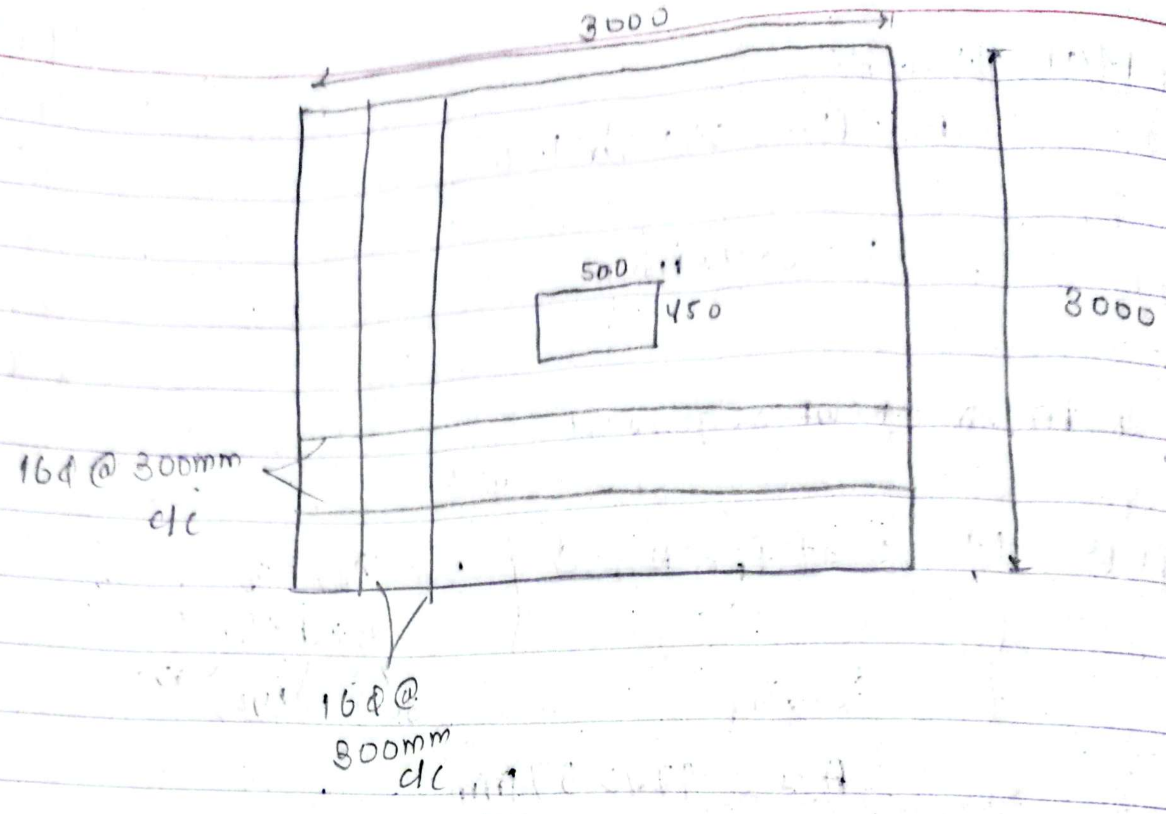
⑥ Providing 16 mm bars $A_T = \pi * 8^2 = 201.06 \text{ mm}^2$

Spacing ~~No of bars~~ $= \frac{3000 * 201.06}{1743.37}$

$= 345.98 \text{ mm}$

Thus, providing 16 mm bars @ 300 mm c/c





Plan