PVP 23

Code: 23CE3501

III B.Tech - I Semester - Regular Examinations - NOVEMBER 2025

DESIGN AND DRAWING OF REINFORCED CONCRETE STRUCTURES
(CIVIL ENGINEERING)

Duration: 3 hours Max. Marks: 70

Note: 1. This question paper contains two Parts A and B.
2. Part-A contains 2 very long answer questions. Answer any 1 Question. Each Question carries 28 Marks.
3. Part-B contains 5 essay answer questions. Answer any 3 questions. Each Question carries 14 marks.
4. All parts of Question paper must be answered in one place.

BL – Blooms Level CO – Course Outcome

PART – A

		BL	CO	Max. Marks
1	Design a reinforced concrete footing for a rectangular column of section 300 mm x 500 mm supporting an axial factored load of 1600 kN. The safe bearing capacity of the soil at site is 200 kN/m^2. Adopt M20 grade concrete and Fe 415 HYSD bars. Sketch the details of reinforcement.	L4	CO3 CO4 CO5	28 M
	OR			
2	Design a two-way slab panel of size 4 m × 3 m simply supported on all sides, subjected to a live load of 4 kN/m^2. Use M20 concrete and Fe 415 steel. Determine slab thickness, reinforcement, and spacing using IS coefficients.	L4	CO2 CO3 CO5	28 M

PART – B

			BL	CO	Max. Marks
3	a)	Define modular ratio and explain its significance in reinforced concrete design.	L2	CO1 CO4	7 M
	b)	What is the difference between working stress method and limit state method?	L2	CO1 CO4	7 M
4		A simply supported beam with clear span 6000 mm, b=400 mm, d=560 mm carries a limit state load of 175 kN/m (including self-weight, dead load and live load). It is reinforced with 4 bars of 28 mm diameter tension steel (A_{st}=2464 mm^2) which continue right into the support. Take M20 concrete and Fe415 steel. Design the shear reinforcement.	L3	CO2 CO3 CO4	14 M
5		Determine the design procedure for calculation of reinforcement required to withstand Torsion in a beam.	L3	CO2 CO3 CO4	14 M
6		Design a short RCC column to carry an axial load of 1800 kN. It is 3m long, effectively held in position and restrained against rotation at both ends. Use M20 concrete and Fe415.	L4	CO3 CO4 CO5	14 M

7	Design a one-way slab for room of size 4 m × 8 m supported on 300 mm thick masonry wall on both sides of short span. The Live load is 2.5 kN/m^2. Use M20 concrete and Fe415 steel. Sketch the reinforcement details.	L4	CO2 CO3 CO5	14 M

III B.Tech - I semester - Regular exam - Nov - 2025

Design and Drawing of Reinforced concrete Structures - 23CE3501 (PVP23)
(CIVIL ENGINEERING)

Scheme of Evaluation

Note: For Design problems the procedure shall be awarded 60% marks.

Part - A - Each question - 28 M.

Q. 1. Given Data: — 2 mark

Load calculation — 2 marks

Area of footing — 2 marks

Calculation of Depth of footing

a) One-way shear criterion — 2 marks

b) Two-way shear criterion — 2 marks

c) Bending-Moment criterion — 2 marks

Calculation of Area of reft.

i) For longer direction — 3 marks

ii) For shorter direction — 3 marks

Check for ~~dd~~ Development length — 2 marks

Diagram — 8 marks.

Q2) Given Data - 1 mark

Check for one way or two way slab - 1 mark

Assumption of Depth of slab - 1 mark

Finding effective span - 1 mark

Finding Design load. - 1 mark

Design Moment Calculation - 2 marks

Design shear Calculation - 2 marks

Minimum depth reauired. - 2 marks

Design of main rft.

i) along shorter span (middle strip) - 2 marks

ii) along longer span (middle strip) - 2 marks

iii) Reinforcement in edge strip. - 2 marks

Check for shear - 2 marks

Check for Deflection - 2 marks

Torsional reinforcement - 2 mark

Slab diagram - 6 marks

(ii)

Part-B — Each question - 14 marks

3 a) Definition — 1 mark
Formula — 1 mark
significance - 5 points — 5 marks } 7 marks

3 b) 5-7- Differences — 7 marks.

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4. Given data: — 1 marks
Factored shear Force. — 1 marks
Nominal shear stress — 2 marks
Design shear strength of concrete — 3 marks
Design shear reinforcement — 3 marks
spacing of stirrups — 2 marks
Diagram. — 3 marks

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5. Equivalent shear force — 2 marks
Equivalent Normal shear stress — 2 marks

Finding Torsional moment.
i) Longitudinal rft — 2.5 marks
ii) Torsional rft — 2.5 marks
Distribution of longitudinal rft — 2.5 marks
Distribution of transverse rft. — 2.5 marks

6. Given data - 1 mark

Factored load. - 2 marks.

Calculation of Gross Area - 2 marks

slenderness ratio - 1 mark

Minimum eccentricity - 1 mark

Area of steel. - 2 marks

Dia of lateral ties - 1 mark

Pitch of lateral ties. - 1 mark

Diagram - 3 marks

7. Given data. - 1 mark

Finding type of slab - 1 mark

Assumption of Depth of slab - 1 mark

Finding effective span. - 2 marks

Finding Design load & Factored Moment - 2 mark

Finding depth reqd. - 1 mark

Area of tensile steel - 1 mark

Area of Distribution steel - 1 mark

check for shear - 1 mark

check for Deflection - 1 mark

Diagram - 2 marks

1. Design a reinforced concrete footing for a rectangular column of section 300 mm x 500 mm supporting an axial factored load of 1600 KN. The safe bearing capacity of the soil at site is 200 KN/m². Adopt M20 grade concrete and Fe 415 HYSD Bars. Sketch the details of reinforcement.

Solution:

Given data: Column size - 300 mm x 500 mm

Factored load - w_c = 1600 KN

soil bearing capacity = q_u = 200 KN/m²

Grade of concrete = M20 = f_{ck}

Yield strength of steel = f_y = 415 N/mm²

Load calculation:

self weight of footing, w_f = 10% of w_c

$$= \frac{10}{100} \times 1600$$

$$= 160 \text{ KN}$$

$$\therefore w_c + w_f = 1600 + 160$$

$$= 1760 \text{ KN}$$

Area of footing

$$A = \frac{w_c + w_f}{q_u} = \frac{1760}{200} = 8.8 \text{ m}^2$$

considering length to ~~width~~ width ratio of footing same as ratio of column i.e., $\frac{500}{300} = 1.667$

$$\therefore y = 1.667x$$

Area of footing $= x \times y$

$= x \times 1.667x$

$= 1.667x^2$

$\Rightarrow \quad 1.667x^2 = 8.8$

$$x^2 = \frac{8.8}{1.667}$$

$$x = \sqrt{\frac{8.8}{1.667}}$$

$= 2.297$ m

$\simeq 2.3$ m

$\therefore \quad y = 1.667 \times 2.3 = 3.83$ m

$\simeq 3.85$ m

Soil pressure due to column load only $= \dfrac{1600}{2.3 \times 3.85}$

$= 180.688$ kN/m^2

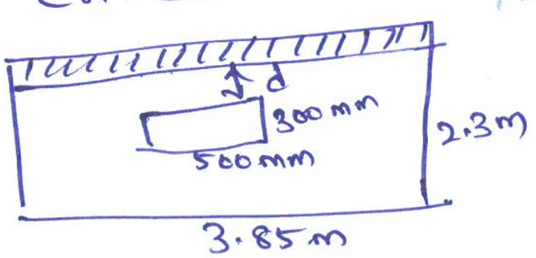
Factored soil pressure $= 1.5 \times 200$

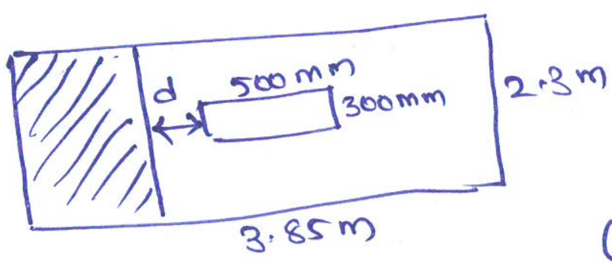
$= 300$ kN/m^2

Calculation of depth of footing:

a) By one-way shear criterion:

Critical section is at "d" from face to column





Shear force in longer direction $= 300 \times 3.85 \times \left[\frac{2.3-0.3}{2} - d\right]$

$= 1155 \left[1 - d\right]$

$= 1155 - 1155d$

Shear force in shorter direction $= 300 \times 2.3 \left[\frac{3.85-0.5}{2} - d\right]$

$= 690 \left[1.675 - d\right]$

$= 1155.75 - 690d$ — (1)

Shear force in shorter direction is more.

∴ Shear force resisted by the concrete $= \tau_c \times d$

for 0.2% reinforcement & M20 concrete

From table 20 of IS-456-2000.

$\tau_c = 0.32$

∴ S.F resisted by concrete $= \frac{0.32}{1000} \times 10^6 \times 2.3 \times d$

$= 736d$ → (2)

Equate (1) & (2)

$1155.75 - 690d = 736d$

$1155.75 = 736d + 690d$

$1155.75 = 1426d$

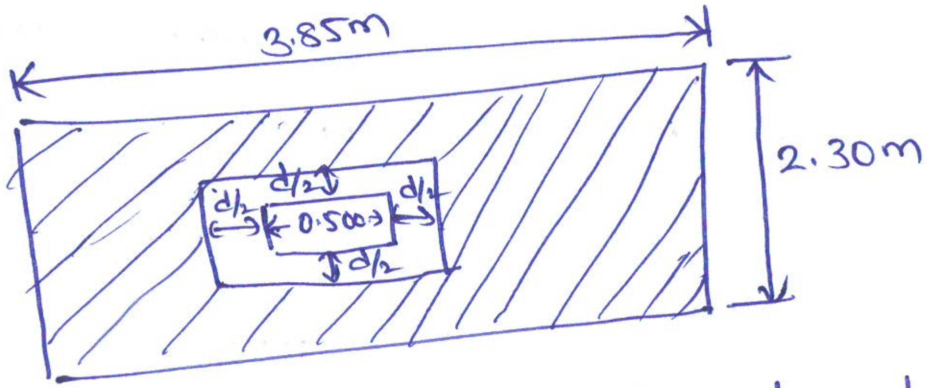
$\Rightarrow d = \frac{1155.75}{1426}$

$= 0.810$ m —— (A)

P.T.O →

b. **Depth of footing by two way shear criterion:**

Critical section is at $d/2$ from face of column.



Shear force at critical section due to shaded area

$$= 300 \times \left[3.85 \times 2.30 - (0.3+d) \times (0.5+d)\right]$$

$$= 300\left[8.855 - (0.15 + 0.8d + d^2)\right]$$

$$= 2,656.5 - 300(0.15 + 0.8d + d^2)$$

$$= 2611.5 - 240d - 300d^2 \rightarrow (3)$$

Punching shear resisted by section $= \tau_c \times A$

where $\tau_c = 0.25\sqrt{f_{ck}}$

$$= 0.25\sqrt{20}$$

$$= 1.118 \text{ N/mm}^2$$

A = Area of critical section

= perimeter $\times d$

$$= [(0.3+d) + (0.5+d)] \times d$$

$$= (0.8 + 2d)d$$

$$= 0.8d + 2d^2$$

$$\text{shear force resisted} = 1.118 \times \frac{10^6}{10^3} \times \left[0.8d + 2d^2\right]$$

$$= 894.4d + 2{,}236d^2 \rightarrow (4)$$

Equating (3) & (4)

$$2611.5 - 240d - 300d^2 = 894.4d + 2236d^2$$

$$2611.5 = 894.4d + 240d + 2236d^2 + 300d^2$$

$$2611.5 = 1134.4d + 2536d^2$$

$$\Rightarrow 2536d^2 + 1134.4d - 2611.5 = 0$$

if $a = 2536$

$b = 1134.4$

$c = -2611.5$

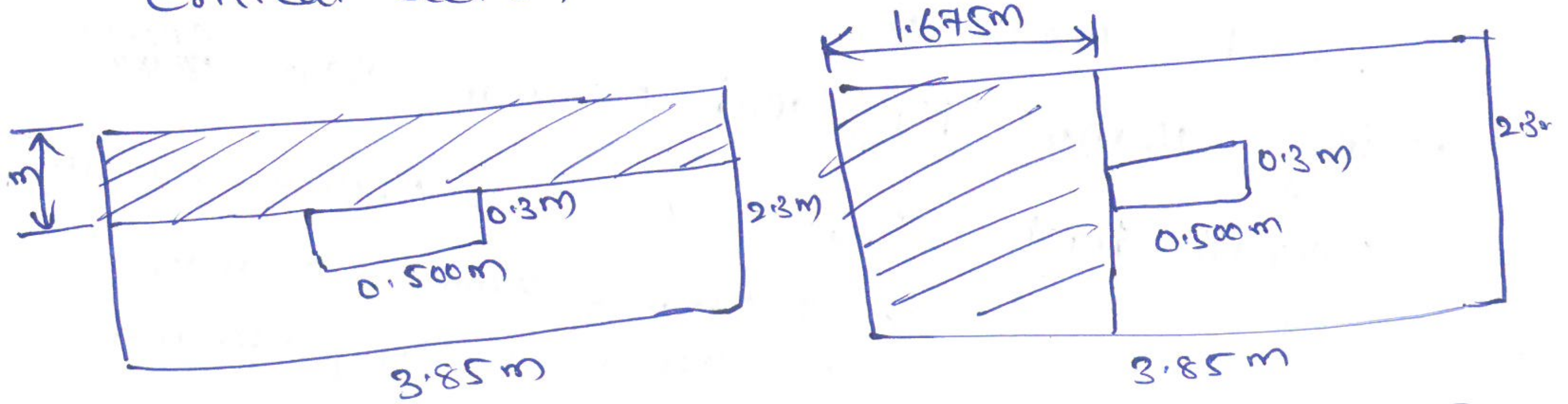
$$d = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

$\Rightarrow d = 0.8156$ (or) -1.2630

$\Rightarrow d = 0.8156$ m ——— (B)

c) Depth of footing by bending moment criterion:

Critical section is at the face of column



(5)

B.M in longer direction $= 300 \times 2.3 \times 1.675 \times \frac{1.675}{2}$

$= 967.94$ kNm $\rightarrow$ (5)

B.M in shorter direction $= 300 \times 3.85 \times \frac{1m \times 1m}{2}$

$= 577.5$ kNm

$\therefore$ B.M in longer direction is more.

Moment of resistance in longer direction

$M_r = 0.138 \, f_{ck} b d^2$

$= 0.138 \times 20 \times 2300 \times d^2$

$= 6348 \, d^2$ Nmm $\rightarrow$ (6)

Equating (5) & (6), we get

$967.94 \times 10^3 \times 10^3 = 6348 d^2$

$\Rightarrow d = \sqrt{\frac{967.94 \times 10^6}{6348}}$

$= 390.486$ mm

$= 0.391$ m $\rightarrow$ (C)

out of equations A, B, & C, (B) is highest

$\therefore d = 0.815$ m

Using 16 mm dia, and 50 mm clear ~~space~~ cover

Overall depth $D = 0.815 + 0.008 + 0.050$ m

$\Rightarrow$ 815 mm + 8 mm + 50 mm

= 873 mm say 875 mm

$\therefore d = 875 - 8 - 50 = \underline{817 \text{ mm}}$

(6)

Area of reinforcement

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

For longer direction

$$967.94 \times 10^6 = 0.87 \times 415 \times A_{st} \times 817 \left[1 - \frac{A_{st} \times 415}{2300 \times 817 \times 20}\right]$$

$$= 294623.85\, A_{st} - \frac{122209897\, A_{st}^2}{37582000}$$

$$3.2517\, A_{st}^2 - 294623.85\, A_{st} + 967.94 \times 10^6 = 0$$

Let $a = 3.2517$, $b = -294623.85$, $c = 967.94 \times 10^6$

$$\therefore A_{st} = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

$\therefore A_{st} = 87200\ mm^2$ (or) $3411.5\ mm^2$

$\therefore A_{st} = 3411.5\ mm^2$

Using 16 ϕ bars, spacing $= \frac{201}{3411.5} \times 2300$

$= 135.512\ mm$

$\therefore$ Provide 16ϕ @ $135\ mm$.

For shorter direction

$$577.5 \times 10^6 = 0.87 \times 415 \times A_{st} \times 817 \left[1 - \frac{A_{st} \times 415}{2300 \times 817 \times 20}\right]$$

$\therefore A_{st} = 1999\ mm^2$

Minimum reinforcement @ 0.12% $= \frac{0.12}{100} \times 3850 \times 1000$

$= 4620\ mm^2 > 1999\ mm^2$

As per IS code provisions

$$\frac{\text{Reinforcement in central band}}{\text{Total reinforcement in shorter direction}} = \frac{2}{\beta + 1}$$

where $\beta = \frac{y}{x} = \frac{3.85}{2.3} = 1.67$

Hence Reinforcement in Central band $= \frac{2}{2.67} \times 4620$

$= 3460.67\ mm^2$

This reinforcement shall be distributed in central 1.9 m band width

spacing of 16 ϕ bars $= \frac{201}{3460.67} \times 1900$

$= 110.35$

$\cong 110\ mm$

$\therefore$ Provide 16 ϕ @ 110mm c/c in central band.

Balance area of steel to be distributed in outer bands $= 4620 - 3460.67$

$= 1159.33\ mm^2$

Spacing of 16 ϕ bars $= \frac{201}{1159.33} \times 1950 = 338.08$

Maximum spacing allowed $= 300\ mm$ or d (817 mm)

$\therefore$ Provide 16 ϕ bars @ 300 mm c/c in outer bands.

Check for development length

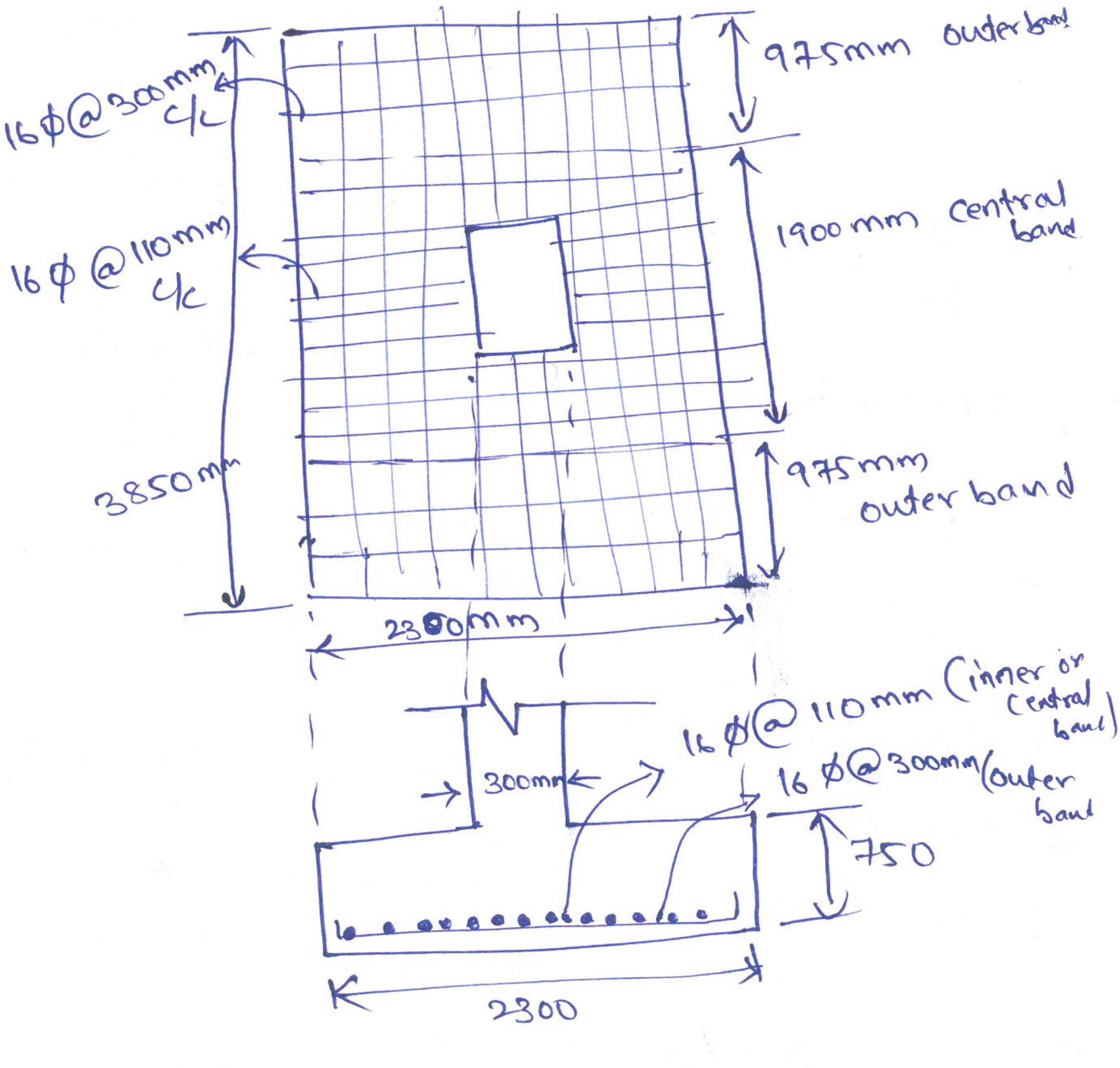
$$L_d = \frac{0.87 f_y \phi}{4 \tau_{bd}} = \frac{0.87 \times 415 \times 16}{4 \times 1.92} = 752.2 \text{ mm} \cong 750 \text{ mm}$$

↳ from Table in IS456

Provide 90° bend at free end of reinforcement as per IS provisions, anchorage value of 90° bend is 8ϕ = 128 mm

∴ Anchorage length = 750 + 128 = 878 > 750

Hence ok



2. Design a two-way slab panel of size 4m x 3m simply supported on all sides, subjected to a live load of 4 kN/m². Use M20 concrete and Fe 415 steel. Determine slab thickness, reinforcement, and spacing using IS coefficients.

Solution:

Given data:

Slab size - 4m x 3m

Slab is supported on 4 sides

Live load - 4 kN/m²

Grade of concrete - M20

Grade of steel - Fe 415

* $\frac{l_y}{l_x} = \frac{4}{3} = 1.33 < 2$. Hence it is two way slab

* Assuming $D = 150$ mm [Assuming $\frac{l}{d} = 25$
$\Rightarrow d = \frac{3000}{25} = 120$ mm]

$d = 150 - 15 - 4$
$= 131$ mm (Assuming clear cover = 15 mm
Diameter of main bar = 8mm)

* Effective span:

Effective span in x-direction: (Assuming support width as 300mm)

i) Centre to centre $3 + 0.3 = 3.3$ m

ii) Clear span + Effective depth $= 3 + 0.131 = 3.131$ m

$\therefore l_x = 3.131$ m

Similarly in y-direction $l_y = 4.131$ m

* **Design load (w_u)**

unit wt. of RCC

self wt. of slab = $0.15 \times 1 \times 25$ = 3.75 kN/m

Finishing load = 1×1 = 1 kN/m

Live load = 4×1 = 4 kN/m

∴ Total load = $3.75 + 1 + 4$ = 8.75 kN/m

Factored or design load = 1.5×8.75

= 3.125 kN/m

* Since the slab is supported on all the four sides and its corners are held down.

* **Design Moment & Shear**

As $\frac{l_y}{l_x} = 1.33$

$$\alpha_x = 0.079 + \frac{0.085 - 0.079}{1.4 - 1.3} \times (1.33 - 1.3)$$

$$= 0.079 + \frac{0.006}{0.1} \times 0.03$$

$$= 0.0808$$

$$\alpha_y = 0.056$$

$$M_{ux} = \alpha_x w_u . l_x^2$$

$$= 0.0808 \times 13.125 \times 3.131^2 = 10.396 \text{ kN m}$$

$$= 10.396 \times 10^6 \text{ Nmm}$$

$$M_{uy} = \alpha_y . w_u . l_x^2$$

$$= 0.056 \times 13.125 \times 3.131^2 = 7.20 \text{ kNm}$$

$$= 7.20 \times 10^6 \text{ Nmm}$$

Maximum shear force = V_u

$$V_u = w_u \cdot \frac{l_x}{2}$$

$$= 13.125 \times \frac{3.131}{2}$$

$$= 20.547 \text{ kN}$$

* Minimum depth required (d_{reqd})

$$d_{reqd} = \sqrt{\frac{M_u}{R_u \cdot b}}$$ [$R_u = 2.76$ for M20 concrete & Fe 415 steel]

$$= \sqrt{\frac{10.396 \times 10^6}{2.76 \times 1000}}$$

$= 61.373$ mm < $d_{assumed}$ Hence OK

* Design of main reinforcement:

i) along shorter span in X-direction (middle strip):

Width of middle strip $= \frac{3}{4} \times l_y$

$$= \frac{3}{4} \times 4.131$$

$$= 3.098$$

$d = 131$ mm.

$$M_u = 0.87 f_y A_{st} d \left[1 - \frac{A_{st} f_y}{b d f_{ck}}\right]$$

$$10.396 \times 10^6 = 0.87 \times 415 \times A_{st} \times 131 \left[1 - \frac{415 A_{st}}{1000 \times 131 \times 20}\right]$$

$A_{st} = 62900$ mm & 220.5 mm

$\therefore A_{st} = 220.5$ mm

Using 8mm ϕ bars $A_\phi = \frac{\pi}{4} \times 8^2 = 50.3 \text{ mm}^2$

$$\text{spacing} = \frac{1000 \times A_\phi}{A_{st}} = \frac{1000 \times 50.3}{220.5 \text{mm}} = 228.11 \cong 200 \text{mm}$$

$\therefore$ spacing = 200 mm (spacing is less than 3 d and 300 mm)

$$Ast\,min = \frac{0.12}{100} bD = \frac{0.12 \times 1000 \times 150}{100} = 180\ mm^2$$

$$Ast\ provided = \frac{1000 \times 50.3}{200} = 251.5\ mm^2 > 180\ mm^2$$

Hence OK

$\therefore$ Provide 8mm ϕ @ 200 mm c/c in middle strip of width 3.098 m.

ii) Along longer span in y- direction (middle strip):

$$\text{Width of middle strip} = \frac{3}{4} l_x$$

$$= \frac{3}{4} \times 3.131$$

$$= 2.348\ m$$

Effective depth along y-direction

$$d = 131 - 4 - 4 = 123\ mm$$

$$M_u = 0.87 f_y\, Ast\, d \left[1 - \frac{f_y\, Ast}{bd\, f_{ck}}\right]$$

$$7.20 \times 10^6 = 0.87 \times f415 \times Ast \times 123 \left[1 - \frac{415 \times Ast}{1000 \times 123 \times 20}\right]$$

$$\therefore Ast = 162.8\ mm^2 < Ast\ min$$

$$\therefore \text{provide } Ast = 180\ mm^2$$

$$\text{Spacing of 8mm dia. bars} = \frac{1000 \times 50.3}{180}$$

$$= 279.44 \simeq 250\ mm$$

$\therefore$ spacing is 250.00 which is less than 3d (or) 300 mm

iii) Reinforcement in edge strip:

A_{st} min $= 180 \text{ mm}^2$

$\therefore$ In edge strip also provide 8 mm ϕ bars @ 250 mm c/c spacing which is less than 5d (or) 450 mm

Width of edge strip in x-direction $= \frac{1}{2}(4.131 - 3.098)$

$= 0.5165$ m

Width of edge strip in y-direction $= \frac{1}{2}(3.131 - 2.348)$

$= 0.3915$ m

* Check for shear

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

$= \frac{20.547 \times 1000}{1000 \times 131}$

$= 0.157 \text{ N/mm}^2$

$$P_t = \frac{100 A_{st}}{bd} = \frac{100 \times 220}{1000 \times 131} = 0.168\%$$

For $P_t = 0.168\%$ & M20 concrete From table 20 IS 456

$$\tau_c = 0.28 + \frac{0.36 - 0.28}{0.25 - 0.15} \times (0.168 - 0.15)$$

$= 0.294 \text{ N/mm}^2$

For 150 mm thick slab $K = 1.30$

$\therefore \tau_c = 0.294 \times 1.30 = 0.38 > \tau_v$

$\therefore$ No shear rft. required.

* **Check for deflection:**

$$P_t = 0.168\ \%$$

$$f_s = 0.58\, f_y \left[\frac{A_{st}\ \text{reqd}}{A_{st}\ \text{provided}}\right]$$

$$= 0.58 \times 415 \left[\frac{220}{220}\right]$$

$$= 240.7\ N/mm^2$$

for $P_t = 0.168\%$ & $f_s = 240.7\ N/mm^2$

$$K_t = 1.6$$

$$\left(\frac{l}{d}\right)_{max} = 20 \times 1.6 = 32$$

$$\left(\frac{l}{d}\right)_{provided} = \frac{3131}{131} = 23.90$$

$\therefore \left(\frac{l}{d}\right)_{max} > \left(\frac{l}{d}\right)_{provided}$ Hence OK

* **Torsional reinforcement at corners**

Mesh size $= \frac{l_x}{5} = \frac{3.131}{5} = 0.6262$ ~~mm~~ 626 mm

Area of torsional reinforcement $= \frac{3}{4} \times 220$

$= 165\ mm^2$

Using 8mm ϕ bars $A_\phi = \frac{\pi}{4} \times 8^2 = 50.3$

Spacing $= \frac{1000 \times 50.3}{220} = 228.63$ mm

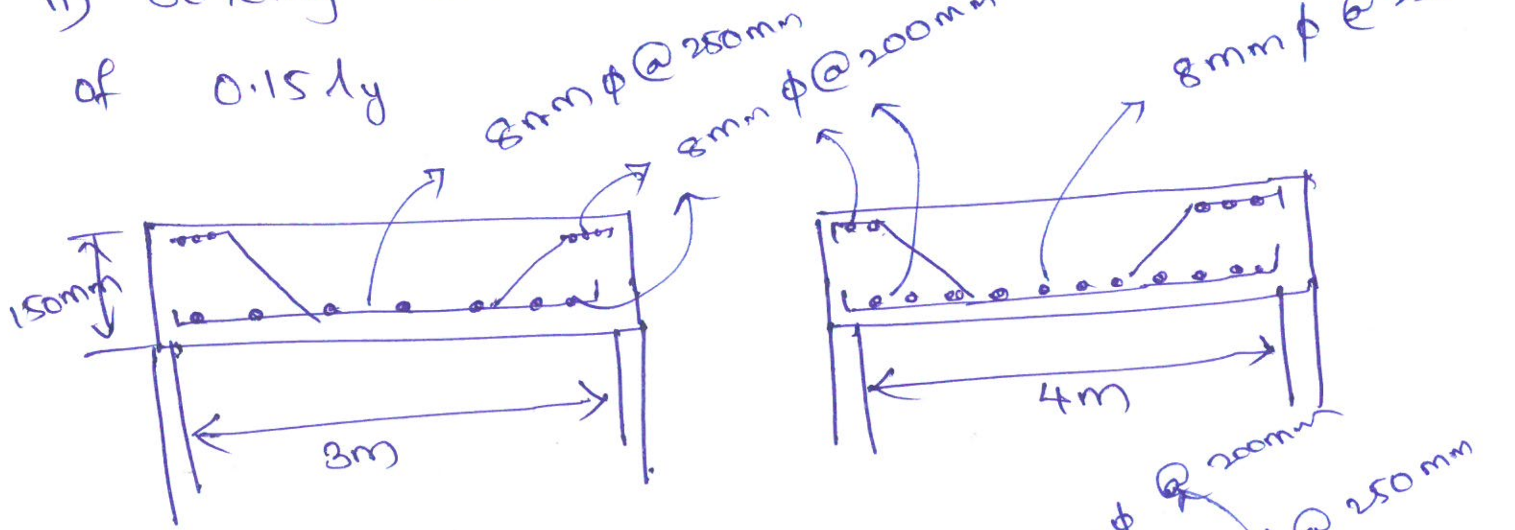
$\cong 200$ mm

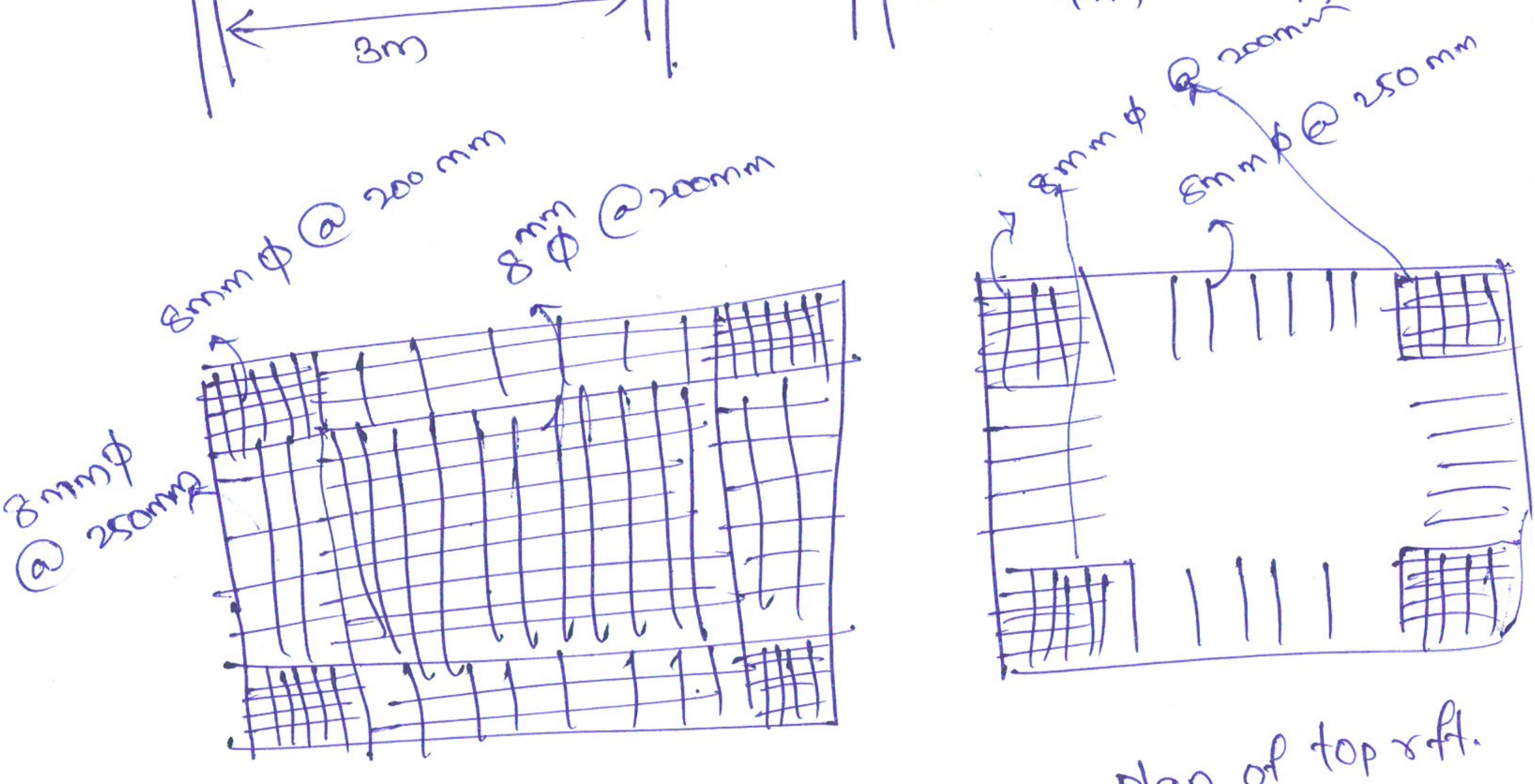
$\therefore$ Provide 8 mm mesh bars @ 200 mm c/c in mesh

* Arrangement of reinforcement.

i) Bending half of main bars at a distance of 0.15 l_x

ii) Bending half of main bars at a distance of 0.15 l_y





Plan of bottom rft.

Plan of top rft.

3) a) Define modular ratio and explain its significance in reinforced concrete design.

Answer : The modular ratio is the ratio of the modulus of elasticity of steel to the modulus of elasticity of concrete. It is denoted by 'm'

$$\boxed{m = \frac{E_s}{E_c}}$$

where:

m = modular ratio

E_s = modulus of elasticity of steel

E_c = modulus of elasticity of concrete

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

where:

σ_{cbc} = Permissible compressive stress in concrete.

Significance of Modular Ratio in Reinforced Concrete Design:

a. Converts Steel Area into Equivalent Concrete Area.

b. Used in tranformed section Method.

c. Ensures Compatibility of Strain

d. Essential in Working stress Design (WSD)

e. Helps compute cracking and Deflection

3 b) What is the difference between Working stress method and limit state method?

Answer:

Working stress method (WSM)	Limit state Method (LSM)
1. Based on elastic theory	1. Based on ultimate load theory & probabilistic concept
2. Uses F.O.S for material strength	2. Uses F.O.S for both loads and materials
3. Ensures stresses stay within permissible (Working) stresses	3. Ensures safety by checking Ultimate limit state and serviceability limit state
4. Assumes linear stress-strain relationship	4. Considers non-linear and realistic stress-strain behaviour
5. More conservative, resulting in heavier sections.	5. More economical, resulting in optimized sections.
6. older, traditional method, rarely used in modern design	6. Modern and widely adopted method by most design codes.
7. Suitable for simple or low-risk structures	7. Suitable for all types of structures, including complex and high-risk designs.

4. A simply supported beam with clear span 6000 mm b = 400 mm, d = 560 mm carries a limit state load of 175 kN/m (including self-weight, dead load and live load). It is reinforced with 4 bars of 28 mm diameter tension steel (A_{st} = 2464 mm^2) which continue right into the support. Take M20 concrete and Fe 415 steel. Design the shear reinforcement.

Solution:

Given data: clear span of beam = 6000 mm = L = 6 m

b = 400 mm

d = 560 mm

(self wt + dL + LL) limit state load w_u = 175 kN/m

A_{st} = 4 bars of 28 mm dia

= 2464 mm^2

Grade of steel = 415 MPa = f_y

Grade of concrete = 20 MPa = f_{ck}

Factored shear force (V_u)

$$V_u = \frac{w_u \cdot L}{2} = \frac{175 \times 10^3 \times 6}{2} = 525000 \text{ N}$$

- Nominal shear stress (τ_v)

$$\tau_v = \frac{V_u}{bd} = \frac{525000}{400 \times 560} = 2.343 \ N/mm^2$$

For M_{20} concrete, $\tau_{cmax} = 2.8 \ N/mm^2$

$\therefore \tau_v < \tau_{cmax}$ (Hence OK)

Design shear strength of concrete

Area of steel available near supports is 4 bars of 28mm diameter

$$A_{st} = 2464 \ mm^2$$

$$P_t = \frac{100 \times A_{st}}{bd} = 1.1 \ \%$$

For M_{20} concrete and $P_t = 1.1\ \%$

$$\tau_c = 0.62 + \frac{0.67 - 0.62}{1.25 - 1.00} \times (1.1 - 1.0)$$

$$= 0.62 + \frac{0.05}{0.25} \times 0.1$$

$$= 0.62 + 0.02$$

$$= 0.64 \ N/mm^2$$

$\therefore \tau_v > \tau_c$, Hence shear reinforcement is to be designed.

Design of shear reinforcement:

shear taken by stirrups = V_{us}

$V_{us} = V_u - \tau_c \cdot b \cdot d$

$= 525000 - (0.64 \times 400 \times 560)$

$= 525000 - 143360$

$V_{us} = 381640$ N

Use 8 mm ϕ 2 legged stirrups

$A_{sv} = 2 \times \frac{\pi}{4} \times 8^2 = 100.5$ mm^2

Spacing of stirrups, $S_v = \frac{0.87 f_y A_{sv} \cdot d}{V_{us}}$

$= \frac{0.87 \times 415 \times 100.5 \times 560}{381640}$

$= 53.24$ mm

$S_v = 50$ mm

Spacing of nominal shear reinforcement

$S_v = \frac{0.87 f_y A_{sv}}{0.4 b} = \frac{0.8 \times 415 \times 100.5}{0.4 \times 400} = 208.53$ mm

Check for spacing

The spacing of stirrups should be minimum of follow

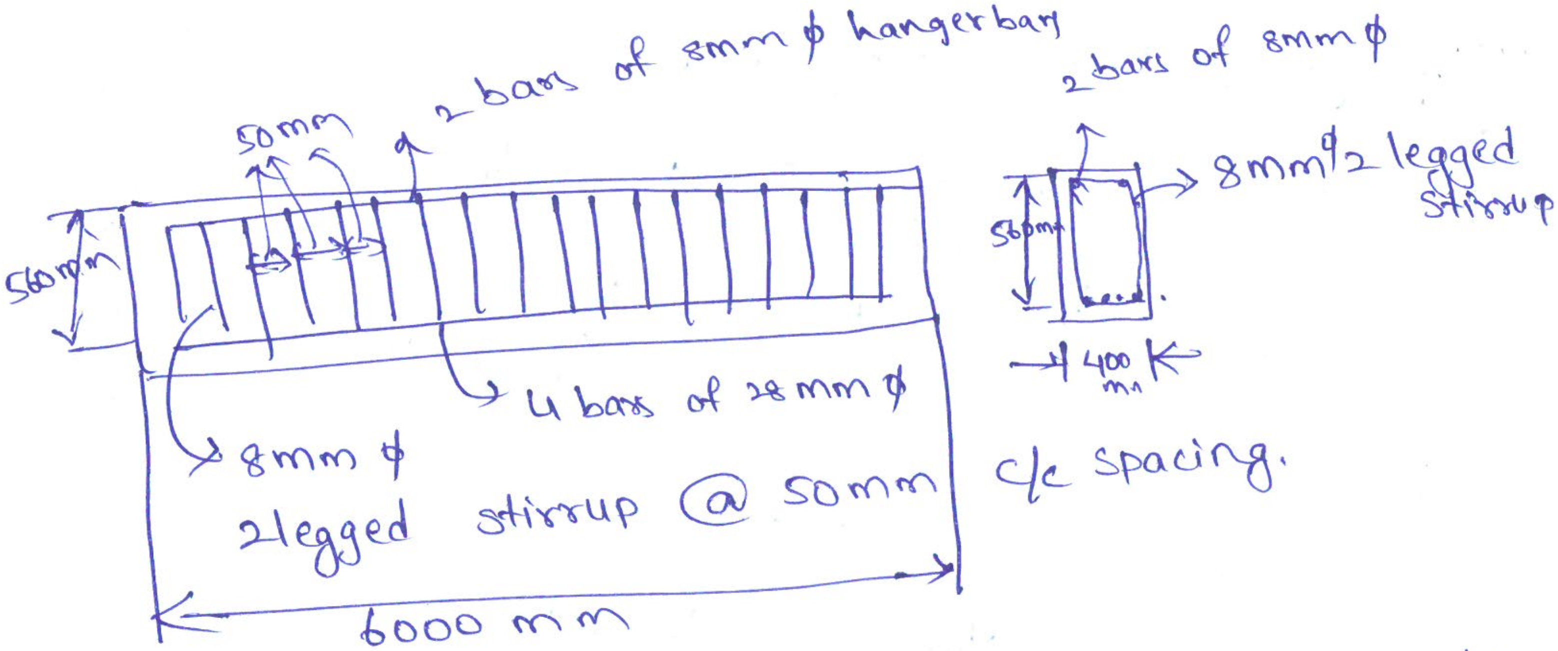
i) $0.75d = 0.75 \times 560 = 420$ mm

ii) 300 mm

iii) 50 mm

iv) 208.53 mm

∴ Provide 2 legged 8 mm ϕ stirrups @ 50 mm c/c throughout len



5. Determine the design procedure for calculation of reinforcement ~~to~~ required to withstand torsion in a beam.

Solution: 1. Find Equivalent shear Force (V_e)

$$V_e = V_u + \frac{1.6 T_u}{b} \quad \text{(clause 41.3 of IS456)}$$

2. find Equivalent Normal shear stress (τ_{ve})

$$\tau_{ve} = \frac{V_e}{bd}$$

if $\tau_{ve} < \tau_{cmax}$, OK

or if $\tau_{ve} > \tau_{cmax}$, then redesign the section.

Find τ_c from table 19 of IS 456

if $\tau_{ve} > \tau_c$, Design for shear.

if $\tau_{ve} < \tau_c$, Provide Nominal shear rft.

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

3. Provide Torsional rft

It is provided in form of Longitudinal rft & transverse reinforcement.

i) Longitudinal rft:

Find equivalent Bending moment.

$$M_e = M_u + M_T$$

where $M_T = \frac{T_u(1 + D/b)}{1.7}$.

a) if $M_T < M_u$ then longitudinal steel required to resist M_e is calculated and distributed as per codal provisions

b) if $M_T > M_u$, then As per IS 456, then provide rft at compression face for beam to resist Moment M_e'

where $M_e' = M_T - M_u$

c) Distribution of Longitudinal rft as per clause 26.5.1.7 of IS 456: 2000

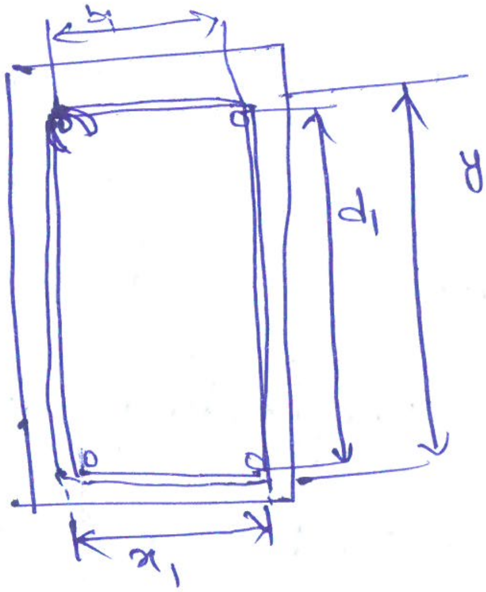
ii) Transverse reinforcement.

Transverse rft is provided in form of closed links or hoops

$$0.87 f_y \cdot \frac{A_{sv}}{x} = \frac{T_u}{b d_1} + \frac{V_u}{2.5 d_1}$$

but $0.87 f_y \cdot \frac{A_{sv}}{x} \not< (\tau_{ve} - \tau_c) \cdot b$

where x is spacing of stirrups



b_1 & d_1 is c/c distance b/w corner bars in width & depth respectively

a) Distribution of transverse reinforcement.

as per clause 26.5.1.7 of IS 456

6. Design a short RCC column to carry an axial load of 1800 kN. It is 3m long, effectively held in position and restrained against rotation at both ends. Use M20 concrete and Fe 415.

Solution:

Given data:

$L = 3\,m$

$P = 1800\,kN$

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

$f_y = 415\,N/mm^2$

Effectively held in position and restrained against rotation

$l_{eff} = 0.65\,l = 0.65 \times 3000\,mm$

$= 1950\,mm$

Factored load $P_u = 1.5 \times P$

$= 1.5 \times 1800$ KN

$= 2700$ KN

Assuming 1% steel $A_{sc} = 0.01 A_g$

$A_c = A_g - A_{sc}$

$= A_g - 0.01 A_g$

$= 0.99 A_g$

$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$

$2700 \times 10^3 = 0.4 \times 20 \times 0.99 A_g + 0.67 \times 415 \times 0.01 A_g$

$= 7.92 A_g + 2.78 A_g$

$= 10.7 A_g$

$A_g = 252336.44 \text{ mm}^2$

∴ Provide column of size 505 mm × 505 mm

∴ A_g provided $= 255025 \text{ mm}^2$

Slenderness ratio:

$$\frac{l_{eff}}{b} = \frac{1950}{505} = 3.86 < 12$$

Hence it is a short column

Minimum eccentricity

$$e_{min} = \frac{l}{500} + \frac{D}{30}$$

$$= \frac{3000}{500} + \frac{505}{30}$$

$$= 22.83 \text{ m}$$

$$\frac{e_{min}}{D} = \frac{22.83}{505} = 0.045 < 0.05$$

Hence the column can be designed as axially loaded short column.

Area of steel (Asc)

$$A_{sc} = 0.01 A_g = 0.01 \times 255025$$

$$= 2550.25 \text{ mm}^2$$

Use 20 mm ϕ bars $A_\phi = \frac{\pi}{4} \times 20^2 = 314 \text{ mm}^2$

No. of bars required $= \frac{2550.25}{314} = 8.12 \cong 10$ bars

$\therefore$ Provide 10 – 20 mm dia bars.

Lateral ties:

Diameter of lateral ties should be more than

i) $\frac{1}{4} \times 20 = 5$ mm

ii) 6 mm

$\therefore$ Use 6 mm ϕ links.

Pitch of ties

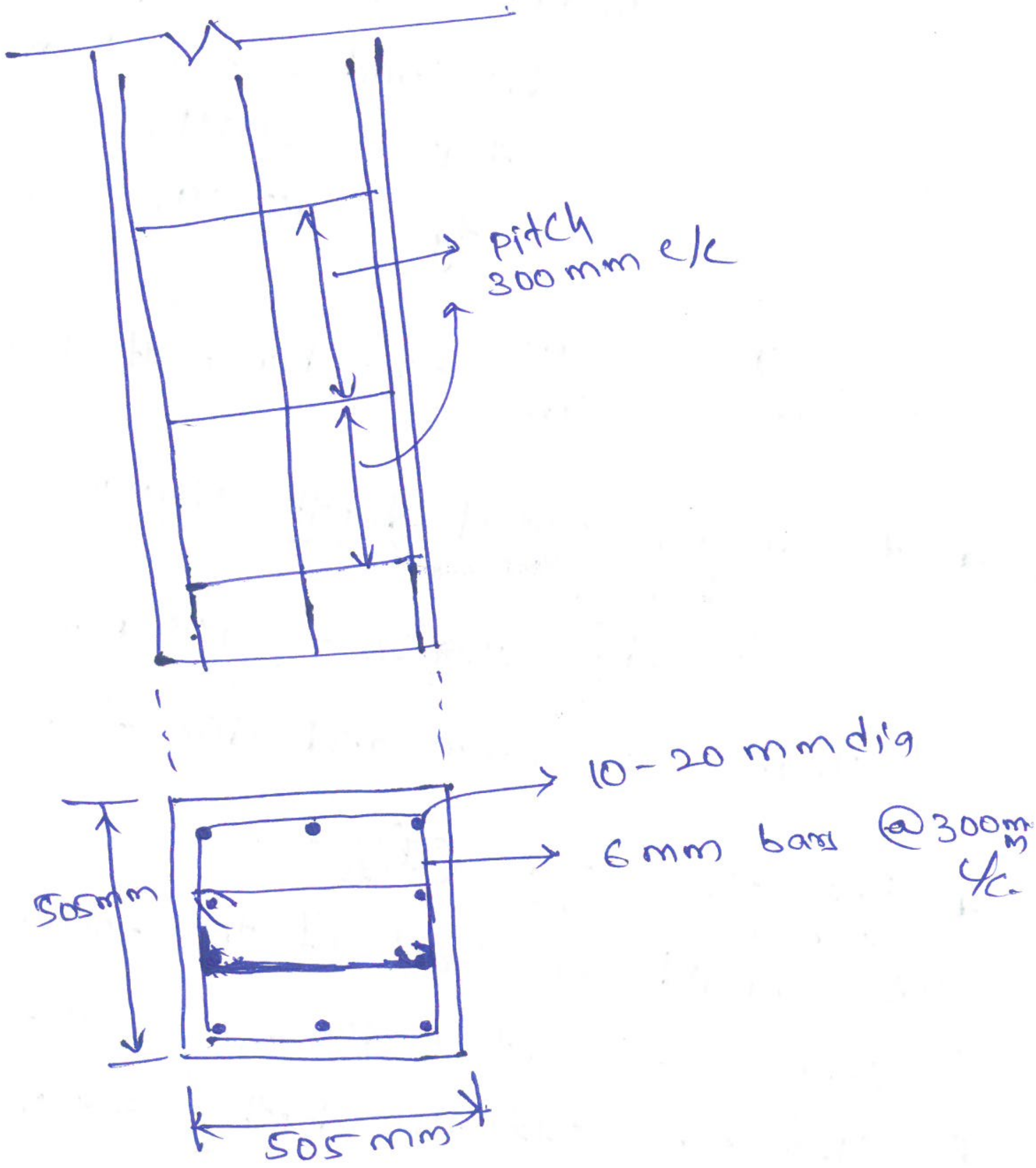
It should not exceed following

i) Least lateral dimension = 505 mm

ii) 16 x 20 = 320 mm

iii) 300 mm

∴ Provide 6mm ϕ @ 300 mm c/c



7. Design a one-way slab for room size 4m x 8m supported on 300 mm thick masonry wall on both sides of short span. The live load is 2.5 kN/m². Use M_{20} concrete and Fe 415 steel. Sketch the reinforcement details.

Solution:

Given data:

$l_y = 8\ m$

$l_x = 4\ m$

thickness of masonry wall = 0.3 m

Live load = 2.5 kN/m²

f_{ck} = 20 MPa

f_y = 415 MPa

$\frac{l_y}{l_x} = \frac{8m}{4m} \geq 2$ hence it is a one way slab.

* Assuming total depth = 180 mm $\left[d = \frac{l}{25} = \frac{4000}{25} = 160mm\right]$

$d = 180 - 20 - 5 = 155$ mm

[clear cover 20mm and dia of main bar = 10 mm]

* Effective span (l_{eff})

It should be least of the following

i) centre to centre distance = 4 + 0.3 = 4.3 m

ii) Clearspan + Effective depth = 4 + 0.155 = 4.155 m

$\therefore l_{eff} = 4.155\ m$

* Design load (w_u) and factored moment (M_u)

Self wt. of slab = 0.180 × 1.0 × 25 (unit wt. of RCC = 25 kN/m³)

= 4.5 kN/m

Imposed load = 2.5 kN/m

Total load "w" = 7 kN/m

Design load (w_u) = 1.5 × 7 = 10.5 kN/m

$$\text{Factored moment} = \frac{w_u \cdot l_{eff}^2}{8}$$

$$= \frac{10.5 \times 4.155^2}{8}$$

= 22.659 kNm

M_u = 22.659 × 10^6 Nmm

* Effective depth required

$$\frac{x_{umax}}{d} = 0.48 \quad [\text{For Fe 415}]$$

$$d_{reqd} = \sqrt{\frac{M_u}{R_u \cdot b}} = \sqrt{\frac{22.659 \times 10^6}{2.76 \times 1000}} = 90.60 < 155,$$

Hence OK

$d > d_{reqd}$. Hence, the section is under reinforced.

* Area of tensile steel (A_{st})

$$M_u = 0.87 f_y A_{st} d \left[1 - \frac{A_{st} f_y}{f_{ck} b d}\right]$$

$$22.659 \times 10^6 = 0.87 \times 415 \times A_{st} \times 155 \left[1 - \frac{A_{st} \times 415}{20 \times 1000 \times 155}\right]$$

A_{st} = 7045 mm² or 429 mm²

∴ A_{st} = 429 mm²

Using 10mm dia. bars, $A\phi = \frac{\pi}{4} \times 10^2 = 78.5\ mm^2$

spacing of 10 mm dia. bars $= \frac{1000 \times A\phi}{A_{st}}$

$= \frac{1000 \times 78.5}{429}$

$= 182.983$

$\cong 180\ mm$

$\therefore$ Provide 10 mm ϕ @ 180 mm c/c

This spacing is less than:

i) $3d = 3 \times 155 = 465$ mm

ii) 300 mm.

Bend alternate bars at $\frac{l}{7} = \frac{4155}{7} = 593$ mm from centre of support.

Distribution steel

Distribution reinforcement is provided in longer direction i.e., 8m, choosing mild steel bars.

$= 0.15\ \%$ of X-sectional Area.

$= \frac{0.15}{100} \times 1000 \times 180 = 270\ mm^2$

using 6mm ϕ bar $= A\phi = \frac{\pi}{4} \times 6^2 = 28.3\ mm^2$

$\therefore$ spacing of 6mm ϕ bar $= \frac{1000 \times A\phi}{A_{st}} = 104.81$ mm

$\therefore$ Provide 6mm ϕ @ 100mm c/c in longer direction.

* **Check for shear**

Factored shear force = $V_u = \frac{w_u \cdot l}{2}$

$$V_u = \frac{10.5 \times 4.155}{2}$$

$$= 21.81375 \text{ kN}$$

$$= 21813.75 \text{ N}$$

* **Nominal shear stress, τ_v**

$$\tau_v = \frac{V_u}{bd} = \frac{21813.75}{1000 \times 155} = 0.14 \text{ N/mm}^2$$

* **Design shear strength of concrete (τ_c)**

$$P_t = \frac{100 \times A_{st}}{bd} \quad \left[A_{st} \text{ at support} = \frac{429}{2} = 214.5\right]$$

$$= \frac{100 \times 214.5}{1000 \times 155} = 0.138 \%$$

From Table 19 of IS 456 for $P_t = 0.138\%$ & M20 Concr.

$$\tau_c = 0.28$$

$\therefore \tau_v < \tau_c$, Hence OK

* **Check for deflection:**

$$P_t = \frac{100 A_{st}}{bd} = \frac{100 \times 429}{1000 \times 155} = 0.27 \%$$

$$f_s = 0.58 f_y \left[\frac{A_{st} \text{ reqd}}{A_{st} \text{ provided}}\right]$$

$$= 0.58 \times 415 \left[\frac{429}{429}\right] = 240 \text{ N/mm}^2$$

for $P_t = 0.138\%$, $f_s = 240 \text{ N/mm}^2$, $k_t = 2$

$\therefore \left(\frac{l}{d}\right)_{max} = 20 \times K_t = 20 \times 2 = 40$

$\left(\frac{l}{d}\right)_{provided} = \frac{4155}{155} = 26.8$

$\therefore \left(\frac{l}{d}\right)_{max} > \left(\frac{l}{d}\right)_{provided}$, Hence OK