18CE503

Hall Ticket Number:

III/IV B.Tech (Regular\Supplementary) DEGREE EXAMINATION

February, 2021

Civil Engineering

Fifth Semester

Design of Concrete Structures

Time: Three Hours

Maximum: 50 Marks

Answer ANY FIVE questions from Unit-I to Unit-V. (5X10 = 50 Marks)

		UNIT I	
1.	a)	Explain transformed area method.	3M
	b)	What are the advantages of doubly reinforced beams over singly reinforced beams?	3M
	b)	Find the moment of resistance of a singly reinforced beam with width 250mm, overall depth is 400mm and clear cover of 25mm. Use Fe_{250} and M_{20} grades. Assume 5-20mm diameter bars as tension reinforcement. Use WSM.	4M
2.	a)	Design a doubly-reinforced beam of 350mm x 700mm(effective) which is subjected to a bending moment of 400kN-m. Use Fe_{415} steel and M_{20} grade concrete. Assume d'=50mm and f_{sc}=335.3Mpa.	10M
		UNIT II	
3.	a)	A reinforced concrete beam 250mm wide and 400mm effective depth is subjected to ultimate design shear force of 150kN at the critical section near supports. The tensile reinforcement at the section near supports is 0.5%. Design the shear stirrups near the support using LSM. Also design the minimum shear reinforcement at the mid span. Assume M_{20}concrete and Fe_{250} steel.	10M
4.	a)	A reinforced concrete beam is supported on two walls of 750mm thick, spaced at a clear distance of 6m. The beam carries a superimposed load of 9.8kN/m. Design the beam using M_{20} and Fe_{415} grades. Take f_{sc}=240N/mm^2.	10M
		UNIT III	
5.	a)	A simply supported one-way slab of clear span 3m is supported on masonry walls of thickness 350mm. The slab is used for residential purpose. Design the slab using M_{30} and Fe_{415} grades. The live load on slab is 3kN/m^2.	10M
6.	a)	The slab of a residential building of size 4.3mx6m is simply supported on all four sides on 230mm walls. Assuming an imposed load of 2kN/m^2 and load due to finishes of 1kN/m^2, design the floor slab. Use M_{25} concrete and Fe_{415} steel. Assume mild exposure condition.	10M
		UNIT IV	
7.	a)	Design a circular column to carry an axial load of 1000kN. Use M_{20} concrete and Fe_{415} steel.	10M
8.	a)	A corner column of size 400mm x 400mm located in the lowermost storey of a system of braced frames, is subjected to a factored loads. P_u=1300kN, M_{ux}=190kN-m. M_{uy}=110kN-m. The effective length of the column is 2975mm. Design the reinforcement in the column, assume M_{25} and Fe_{415} steel.	10M
		UNIT V	
9.		Design a square footing of uniform thickness for an axially loaded square column of 550mmx550mm. The safe bearing capacity of soil is 250kN/m^2. Load on column is 950kN. Use M_{20} concrete and Fe_{415} steel.	10M
10.		Design a square footing of uniform thickness for an axially loaded square column of 500mmx500mm. The safe bearing capacity of soil is 220kN/m^2. Load on column is 880kN. Use M_{20} concrete and Fe_{415} steel.	10M

SCHEME

DESIGN OF CONCRETE STRUCTURES

by

G. Prasanth Babu.

1(a) The concept of modular ratio makes it possible, for the purpose of analysis, to transform the composite section into an equivalent homogenous section made up entirely of one material. Evidently, this transformation must not alter the magnitude, direction and line of action of the resultant forces in the material 2 due to the flexural stresses. Considering the resultant force dF_2 in an infinitesimal element of material 2 having thickness dy, located at a distance 'y' from the neutral axis.

$$dF_2 = f_{2y}(b_2 dy)$$

$$dF_2 = m f_{1y}(b_2 dy) = f_{1y}(m b_2)\, dy$$

The use of transformed section concept may be limited to determining the neutral axis as the centroidal axis of the transformed area. The stresses induced in the two materials due to given moment can be determined by applying the basic equations of static equilibrium.

(b)

→ When the size of the beam is restricted from architecture point of view.

→ When the structure is subjected to reversal of stresses.

→ Structure subjected to impact load.

→ To resist more moment of resistance.

→ To increase the ductility of the structure.

(c) Given

$b = 250mm$, $D = 400\,mm$, $C_c = 25mm$

$A_{st} = 5\text{-}20\phi$, Fe 250, M20

$$m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3\times7} = 13.33$$

$$\frac{bx^2}{2} = m\,A_{st}(d-x)$$

$$\frac{250x^2}{2} = 13.33\left(5\times\frac{\pi}{4}(20)^2\right)(365-x)$$

$$d = 400-25-\frac{20}{2} = 365\,mm$$

$$x = 177.29$$

$$K = \frac{m\sigma_{cbc}}{m\sigma_{cbc}+\sigma_{st}} = \frac{13.33(7)}{13.33(7)+140}$$

$$= 0.39$$

$$X_c = (0.39)(365) = 145.635\,mm$$

$$X > X_c$$

$\therefore$ The section is over reinforced section

Momen of resistance $= \frac{1}{2}\,\sigma_{cbc}\cdot b\cdot x_c\left(d-\frac{x_c}{3}\right)$

$$= \frac{1}{2}\times7\times250\times177\times145.635\left(365-\frac{145.635}{3}\right)$$

$$= 7149.7\,KN\text{-}m$$

2. Given

$b = 350mm$, $d = 700mm$, $M = 400KN\text{-}m$, $d' = 50mm$

Fe415, M20, $f_{sc} = 335.3\,MPa$

$$X_{umax} = 0.48d$$

$$= 0.48\times700 = 336mm$$

$$Mu_2 = Mu - Mu_{limit}$$

$$Mu_{limit} = 0.138 f_{ck} b d^2$$

$= 0.138 \times 20 \times 350 \times 700 \times 700$

$= 473.34 KN\text{-}m$

$Mu_2 = Mu - Mu_{limit}$

$= 600 - 473.34 = 126.7 KN\text{-}m$

$Mu_2 = f_{sc} A_{sc} (d - d')$

$126.7 \times 10^6 = 335.3 (A_{sc})(700 - 50)$

$A_{sc} = 581.34 \, mm^2$

$Mu = 126.7 \times 10^6 = 0.87 f_y A_{st} (d - d')$

$126.7 \times 10^6 = 0.87 \times 415 \times A_{st_2} (700 - 50)$

$A_{st_2} = 539.87 mm^2$

$Mu_{limit} = 473.34 \times 10^6 \, N\text{-}mm$

$Mu_{limit} = 0.87 f_y A_{st_1} (d - 0.42 x_{umax})$

$A_{st_1} = 2345.78 \, mm^2$

3. Given $b = 250mm$, $d = 400mm$

$V_u = 150 KN$

$$P = \frac{100 A_{st}}{bd} = 0.5\%$$

$$\tau_v = \frac{V_u}{bd} = \frac{150 \times 10^3}{250 \times 400} = 1.5 N/mm^2$$

$\tau_c = 0.48 N/mm^2$ from IS 456:2000

$\tau_{cmax} = 2.8 N/mm^2$

$\therefore \tau_v < \tau_{cmax}$

$\tau_v > \tau_c$

Hence shear reinforcement is required.

$V_s = V_u - \tau_c * bd$

$= 150 \times 10^3 - 0.48 \times 250 \times 400$

$= 102\,KN$

Use two legged vertical stirrups of 10mm diameter

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

$$= \frac{0.87 \times 250 \times 2 \times \frac{\pi}{4}(10)^2 * 400}{102 \times 10^3}$$

$= 134\,mm$

Maximum spacing $= 0.75d = 300mm$

Hence provide 10mm dia 2-legged vertical stirrups @ 130mm c/c near the supports.

@ mid span, the shear will be less so provide minimum shear reinforcement for 10mm diameter

$$S_v = \frac{0.87 f_y\, A_{sv}}{0.4\, b} = 340mm$$

Since maximum spacing is restricted to 300mm. Hence provide 2 legged vertical stirrups at 300mm centre to centre in the mid span.

4. Given

Width = 750mm, $l = 6m$, load = 9.8 KN/m

M20, Fe415, $f_s = 240 N/mm^2$

The minimum depth of beam is based on limit state of serveciability requirements. For simply Supported beam $\frac{L}{d} = 20$

Take avg value $\frac{L}{d} = 15$

$$d = \frac{L}{15} = \frac{6000}{15} = 400mm$$

$$D = 400 + 30 = 430\ mm \text{ (say)}$$

Assume width, $b = 300mm$

Self weight $= 0.43 \times 0.3 \times 25$

$= 3.225\ KN/m$

External load $= 9.8 KN/m$

Total load, $W = 9.8 + 3.225$

$= 13.025\ KN/m$

Factored load, $W = 13.025 \times 1.5$

$= 19.537 KN/m$

Effective span:

1. clear span + Effective depth $= 6 + 0.4 = 6.4m$
2. clear span + $\frac{\text{Support width}}{2} + \frac{\text{support width}}{2}$

$$= 6 + \frac{0.75}{2} + \frac{0.75}{2} = 6.75m$$

$\therefore$ Effective span, $l = 6.4m$

$$M = \frac{wl^2}{8} = \frac{19.537(6.4)^2}{8} = 100.02 KN\text{-}m$$

$$Mu = 0.36 f_{ck} \frac{x_{umax}}{d}\left(1 - 0.42\frac{x_{umax}}{d}\right) bd^2$$

$$\frac{x_{umax}}{d} = 0.48$$

$$100.02 \times 10^6 = 0.36 \times 20 \times 0.48 \left(1 - 0.42(0.48)\right) 300 d^2$$

$$d = 347.6 mm$$

Assume 25mm nominal cover and use 20mm bars with 8mm stirrups.

$$D = 347.6 + 25 + 10 + 8 = 390.6 mm$$

$$D = 430 mm$$

$$\therefore d = 430 - 25 - 10 - 8 = 387 mm$$

Since available depth is more than required we will have a under reinforced section.

Steel reinforcement

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

$$100.02 \times 10^6 = 0.87 \times 415 \times A_{st} \times 387 \left[1 - \frac{A_{st} \times 415}{387 \times 300 \times 20}\right]$$

$$A_{st} = 843 mm^2$$

$\therefore$ No. of 20mm dia bars

$$843 = n \times \frac{\pi}{4}(20)^2$$

$$n = 2.68$$

Provide 3 - 20ϕ bars at mid span

$$\text{Actual } A_{st} = 3 \times \frac{\pi}{4}(20)^2 = 942.47 mm^2$$

Actual moment of resistance

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

$$= 110 \times 10^6 \text{ N-mm}$$

ul

m

ϕ

$$= \frac{19.537 \times 6.4}{2} - 19.537\left(\frac{0.75}{2} + 0.387\right)$$

$$= 77.63\,kN$$

$$\tau_v = \frac{V_u}{bd} = \frac{47.63 \times 10^3}{300 \times 387} = 0.41\,N/mm^2$$

$$\frac{100 A_{st}}{bd} = \frac{100\left(2 \times \pi/4\,(20)^2\right)}{300 \times 387}$$

$$= 0.55\%$$

$\therefore$ $\tau_c = 0.5\%$ @ interpolation

Since $\tau_v < \tau_c$, no shear reinforcement

We have to provide minimum reinforcement

as per code

$$\frac{A_{sv}}{bS_v} = \frac{0.4}{0.87 f_y}$$

$$S_v = 302\,mm$$

$$0.75d = 0.75 \times 387 = 290\,mm$$

$\therefore$ provide 8mm dia 2 legged stirrups at 290mm c/c throughout the beam and provide 2-10ϕ holding bars at top

Check for development length:

$$l_d \le \frac{1.3 M_1}{V} + l_0$$

$$M_{u limit} = 0.36 \frac{x_{umax}}{d}\left[1 - 0.42\frac{x_{umax}}{d}\right] bd^2 f_{ck}$$

$$= 124 \times 10^6 \text{ N-mm}$$

$M_r < M_{u limit}$

$\therefore$ The section is under reinforced section.

Hence OK

The code recommends that atleast $1/3^{rd}$ the positive moment reinforcement in simple members shall extend along the same phase of the member into the support. However, bend one bar upwards, at a distance x_1 from the support.

Bending moment at 'x_1':-

$$x_1 = \frac{wl}{2} \times x_1 - \frac{wx_1^2}{2} = \frac{2}{3}\frac{wl^2}{8}$$

This should be $2/3^{rd}$ of the maximum moment.

$$= \frac{2}{3}\frac{19.547 \times (6.4)^2}{8}$$

$$x_1 = 1.35\text{m}$$

However the bars are to be taken further by a distance d (or) 12ϕ

$\therefore$ Distance of point from centre of the support

$$1.35 - 0.387 = 0.963\text{m}$$

Shear reinforcement

The critical section for shear is at a distance 'd' from the phase of the support.

$$V_u = \frac{W_u l}{2} - W_u\left[\frac{0.75}{2} + 0.387\right]$$

Only 2-20ϕ bars are available @ the supports

$$0.36 f_{ck} b X_u = 0.87 f_y A_{st}$$

$$0.36 \times 20 \times 300 \times X_u = 0.87 \times 415 \times 2 \frac{\pi}{4} (20)^2$$

$$X_u = 105 mm$$

$$M_u = 0.87 f_y A_{st} (d - 0.42 X_u)$$

$$= 0.87 \times 415 \times \frac{\pi}{4} (20)^2 \times 2 \left(387 - 0.42 \times 105\right)$$

$$= 77.78 \times 10^6 N\text{-}mm$$

$$V = \frac{WL}{2} = \frac{19.537 \times 6.4}{2}$$

$$= 62.52 KN$$

$$L_d = \frac{0.87 f_y \phi}{1.25 \times 1.6 \times 4 \tau_{bd}}$$

$$= 940 mm$$

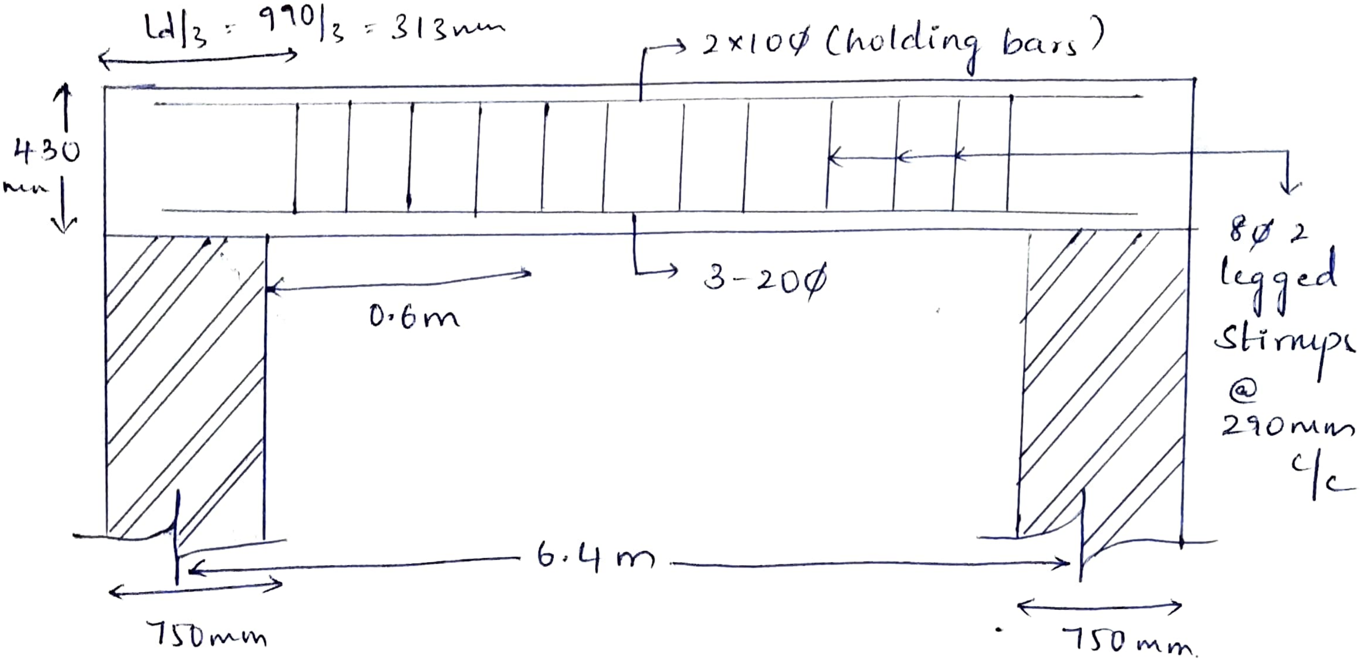
$$l_o = \frac{W_s}{2} - c = \frac{750}{2} - 40$$

Available distance beyond the face of the support $= l_o + \frac{750}{2} = 710mm$

Code recommends that each bar should extended by a distance $\frac{ld}{3} = \frac{940}{3} = 313mm$

$$l_d \leq \frac{1.3 M_1}{V} + l_o$$

Hence OK



5. Given

Clear span = 3m

Thickness = 350 mm, M20, Fe415

L.L = 3 KN/m²

<u>Thickness of slab</u>

$$d = \frac{span}{25} = \frac{3000}{25} = 120mm$$

Adopting a clear cover of 20mm & using 10mm dia bars, the total depth is computed as D = 150mm

<u>Effective span</u>:

1. clear span + effective depth = 3 + 0.12 = 3.12m
2. c/c of supports + $\frac{sw}{2} + \frac{sw}{2}$ = 3.35m

Therefore, L = 3.12m

<u>Loads</u>

Self wt of slab = 0.15 * 25 = 3.75 KN/m²

Finishes = 1.5 KN/m²

Live load = 3 KN/m² (given)

So, total service load = 8.25 KN/m²

$W_u = 1.5 * 8.25 = 12.375\ KN/m^2$

Ultimate moments and shear forces

$M_{u\,limit} = 0.138\ f_{ck}\ bd^2$

$= 0.138 * 30 * 1000 * 120^2$

$= 59.62\ KN\text{-}m$

$M_u < M_{u\,limit}$, section is under-reinforced.

Tension reinforcement

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

$$15.06 \times 10^6 = 0.87 \times 415 \times A_{st} * 120\left(1 - \frac{A_{st} \times 415}{1000 \times 120 \times 30}\right)$$

$A_{st} = 362.78\ mm^2 < A_{st\,min}\ (144\ mm^2)$

Use 10mm dia bars

$$S = \frac{1000\left(\frac{\pi}{4}(10)^2\right)}{362.78} = 205\ mm$$

Therefore adopt a spacing of 200mm c/c

Alternate bars are bent up at supports

Distribution bars

$A_{st} = 0.12\ \%$

Provide 8mm dia bars

$A_{st} = 144\ mm^2$

$$S = \frac{1000 * \frac{\pi}{4}(8)^2}{144} = 349\ mm$$

Therefore, provide 8mm dia bars @ 300mm c/c

Check for shear stress

$$\tau_v = \frac{V_u}{bd} = \frac{19.305 \times 10^3}{1000 \times 120}$$

$= 0.161 \, N/mm^2$

$$P_t = \frac{100 A_{st}}{bd} = \frac{100 \times 362 \times 0.5}{1000 \times 120}$$

$= 0.151\%$

$\tau_c = 0.68 \, N/mm^2$

Hence OK

Check for deflection control

$$\left(\frac{L}{d}\right)_{max} = \left(\frac{L}{d}\right)_{basic} * K$$

$(L/d)_{max} = 20 * 1.55 = 31$

$(L/d)_{provided} = \frac{3120}{120} = 26 < 31$

Hence, the deflection criteria is satisfied.

6. Given

$l_x = 4.3m, \; l_y = 6m$

$l_y / l_x = 1.4 < 2$

$\therefore$ It is a two way slab

$\frac{L}{D} = 0.8 \times 30 = 24$ as per IS456:2000 @ Fe415 steel

Hence, $D = \frac{4300}{24} = 179mm$

$\therefore D = 175mm$

Assume 10mm ϕ bars & mild exposure

$d_x = 175 - 15 - 5 = 155\,mm$

$d_y = 155 - 10 = 145\,mm$

Effective span

$L_x = 4300 + 155 = 4455\,mm$

$L_y = 6000 + 145 = 6145\,mm$

$L_y/L_x = \frac{6145}{4455} = 1.38$

Loads

Self wt of slab $= 0.175 \times 25 = 4.375\,KN/m^2$

wt of finishes $= 1.0\,KN/m^2$

Imposed load $= 2\,KN/m^2$

Total load, $w = 7.375\,KN/m^2$

$W_u = 1.5 \times 7.375 = 11.06\,KN/m^2$

Moments

$L_y/L_x = 1.38$

$a_x = 0.098$

$a_y = 0.0515$

$M_x = a_x W_u L_x^2 = 0.098 \times 11.06 \times 4.455^2 = 21.51\,KN\text{-}m/m$

Check the depth for maximum bending moment

For a balanced section effective depth is $d = 79\,mm < 155\,mm$

The slab is under-reinforced section.

Reinforcement

Using tensile reinforcement formula, $A_{st} = 402\,mm^2$

% of steel is 0.2595

Provide 10mm dia bars @ 190mm c/c

$$Mu/bd^2 = \frac{11.3 \times 10^6}{1000 \times 145^2} = 0.538$$

$$pt = 0.1526, \quad Ast = \frac{0.1526 \times 1000}{145 * 100} = 221 mm^2$$

Deflection

$$pt = \frac{100 Ast}{bd} = \frac{100 \times 413}{1000 \times 155}$$

$$= 0.266\%$$

$$Fs = \frac{0.58 \times 415 \times 402}{413} = 234 MPa$$

$K = 1.61$

$L/d = 20 \times 1.61 = 32.2$

$$\text{depth ratio} = \frac{4455}{155} = 28.74 < 32.2$$

Hence OK

Shear

$$d = \frac{155 + 145}{2} = 150 mm$$

$$Vu = Wu (0.5 Lx - d)$$

$$= 11.06 (0.5 \times 4.3 - 0.15)$$

$$= 22.12 KN/m$$

$$Shear = \frac{22.12 * 1000}{1000 \times 150}$$

$$= 0.148 MPa$$

$d_x = 175 - 15 - 5 = 155\,mm$

$d_y = 155 - 10 = 145\,mm$

Effective span

$L_x = 4300 + 155 = 4455\,mm$

$L_y = 6000 + 145 = 6145\,mm$

$L_y / L_x = \frac{6145}{4455} = 1.38$

Loads

Self wt of slab $= 0.175 \times 25 = 4.375\,KN/m^2$

wt of finishes $= 1.0\,KN/m^2$

Imposed load $= 2\,KN/m^2$

Total load, $w = 7.375\,KN/m^2$

$W_u = 1.5 \times 7.375 = 11.06\,KN/m^2$

Moments

$L_y / L_x = 1.38$

$a_x = 0.098$

$a_y = 0.0515$

$M_x = a_x W_u L_x^2 = 0.098 \times 11.06 \times 4.455^2 = 21.51\,KN.m/m$

Check the depth for maximum bending moment

For a balanced section effective depth is $d = 79\,mm < 155\,mm$

The slab is under-reinforced section.

Reinforcement.

Using tensile reinforcement formula, $A_{st} = 402\,mm^2$

$p_t = 0.266$, $\tau_c = 0.368$ MPa

Hence safe in shear

Cracking

Spacing of steel $< 3d = 3 \times 145$

$= 435$ mm or

300mm in both directions

Diameter of steel reinforcement $< 175/8 = 21$mm.

Hence, no calculation is required for cracking.

Development Length

$$L_d \leq \frac{1.3 M_1}{V_u} + L_0$$

Length of embedment $= 230 -$ clear side cover

$= 230 - 25$

$= 205$mm $> L_d/3$

$$L_d = \frac{(0.87 \times 415 \times 10)}{4 \times 1.4 \times 1.6} = 403 \text{ mm}$$

$L_d/3 = 135$mm < 205mm

Hence safe.

7. Assume 2% of $A_g = A_{sc}$

$$P = 0.4 f_{ck} (A_g - A_{sc}) + 0.6 f_y A_{sc}$$

$$P = 0.4 f_{ck} \left(A_g - \frac{2}{100} A_g\right) + 0.6 f_y \frac{2}{100} A_g$$

$$P = 0.4 f_{ck} \left(\frac{\pi}{4} d^2 - 0.02 \frac{\pi}{4} d^2\right) + 0.012 f_y \frac{\pi}{4} d^2$$

$1.5 \times 1000 \times 1000 = 0.4 \times 20 \times 0.98 \left(\frac{\pi}{4} d^2\right) + 4.98 \left(\frac{\pi}{4} d^2\right)$

$d^2 = 149105.36$

$d = 386.14 mm \simeq 387 mm$

$Ag = \frac{\pi}{4} d^2 = \frac{\pi}{4} (387)^2$

$= 117106.086$

$Asc = 2\%$ of Ag

$= \frac{2}{100} (117106.086)$

$= 2342.12 mm^2$

Assume 16 mm diameter bars

$n \times \frac{\pi}{4} d^2 = 2342.12$

$n \times \frac{\pi}{4} (16)^2 = 2342.12$

$n = 11.6 \simeq 12$ bars

Actual reinforcement provided $= 12 \times \frac{\pi}{4} (16)^2$

$= 2412.7 mm^2$

Spiral reinforcement

Let the diameter of helix be 6 mm

Outside dia of helix $= 387 - 40 \times 2$

$= 307 mm$

Area of core $= \frac{\pi}{4} (307)^2 = 74022.99 mm^2$

for M20 concrete, $f_{ck} = 20 N/mm^2$

for Fe415 steel, $f_y = 415 N/mm^2$

$$= 0.36\left(\frac{Ag}{Ac} - 1\right)\frac{fck}{fy}$$

$$Ag = Ac + Asc$$

$$Ac = Ag - Asc$$

$$Ac = 114693.38 \, mm^2$$

$$= 0.36\left(\frac{117106.086}{114693.38} - 1\right)\frac{20}{415}$$

$$= 0.000964$$

Diameter of helix upto centre

$$= 387 - 2 \times 40 - 8$$

$$= 294.5 \, mm$$

$$\text{Vol. of spiral } (V_h) = \frac{\text{Circumference} * \text{c/s area}}{S}$$

$$= \frac{\pi d \left(\pi/4 \, \phi_s^2\right)}{S}$$

$$= \frac{\pi (294.5)\left(\pi/4 \, (6)^2\right)}{S}$$

$$= \frac{(939.19)(78.54)}{S}$$

Vol. of core per height / length

$$V_c = Ac \times 1$$

$$V_c = 114652.08 \, mm^3$$

$$\frac{V_h}{V_c} = \frac{26549.5}{S} * \frac{1}{114652.08}$$

$$= \frac{0.23}{S}$$

$$\frac{0.23}{S} = 0.000964$$

$$S = 238.5\,nm$$

Pitch $\ngtr$ 75mm

$\ngtr \frac{1}{6}$ (core dia) $= 64.5mm$

$\nless$ 25mm

$\nless 3\phi_s = 3(6) = 18mm$

Hence keep the pitch $= 238.5 \simeq 240mm$.

8. $\lambda = \frac{l_{eff}}{b} = \frac{2975}{400} = 7.43 < 12$

$\therefore$ The given column is a short column, Hence the column may be designed as a short column. Check for minimum eccentricity

$$e_x = \frac{190 \times 10^6}{1300 \times 10^3} = 145mm$$

$$e_y = \frac{110 \times 10^6}{1300 \times 10^3} = 84.6mm$$

Min. eccentricity as per code

$$e_x = \frac{190 \times 10^6}{1300 \times 10^3}$$ is minimum eccentricity.

$$e_x = e_y = \frac{2975}{500} + \frac{400}{30} = 19.28 < 20$$

$$e_{min} = 20mm$$

$\therefore$ Min eccentricities are less than applied eccentricities

Trail section [longitudinal reinforcement]

Designing the column for uniaxial eccentricity

$P_u = 1300\,KN$ & $M_u = 1.15\sqrt{M_{ux}^2 + M_{uy}^2}$

$= 1.15\sqrt{(190)^2 + (110)^2}$

$= 252.4\,KN\text{-}m$

Assuming $d' = 60$

$$\frac{d'}{D} = \frac{60}{400} = 0.15$$

$$\frac{P_u}{f_{ck}\,bD} = \frac{1300 \times 10^3}{25 \times 400 \times 400} = 0.325$$

$$\frac{M_u}{f_{ck}\,bD^2} = \frac{252.4 \times 10^6}{25 \times (400)^3} = 0.157$$

From chart 45

$$\frac{p}{f_{ck}} = 0.14$$

$p = 3.5\,\%$

$$A_{s\,req} = \frac{3.5 \times 400 \times 400}{100} = 5600\,mm^2$$

$$n \times \frac{\pi}{4}(25)^2 = 5600$$

$n = 12$

∴ provide 12-25ϕ bars as longitudinal bars distributed equally on all sides.

Check for safety under bi-axial loading

$$\left[\frac{M_{ux}}{M_{ux_1}}\right]^{\alpha_n} + \left[\frac{M_{uy}}{M_{uy_1}}\right]^{\alpha_m} \leq 1$$

Due to symmetry $M_{ux_1} = M_{uy_1}$

$$\frac{P_u}{f_{ck} b D} = 0.325$$

$$P_{provided} = \left(12 \times \frac{\pi}{4}(25)^2\right) \times \frac{100}{(400)^2}$$

$$= 3.68\%$$

$$\frac{p}{f_{ck}} = \frac{3.68}{25} = 0.147$$

$$\frac{d'}{D} = 0.18$$

$$\frac{M_{ux_1}}{f_{ck} b D^2} = 0.165$$

$M_{uy_1} = M_{ux_1} = 264\ kN\text{-}m$

$= 264 \times 10^6\ N\text{-}mm > M_{ux}$ & M_{uy}

Values of P_{uz} & α_n

$$P_{uz} = 0.45 f_{ck} A_g + (0.75 f_y - 0.45 f_{ck}) A_{sc}$$

$$= 0.45 \times 25 \times 400^2 + (0.75 \times 415 - 0.45 \times 25) \times 12 \times \frac{\pi}{4}(25)^2$$

$$= 3568\ kN$$

$$\frac{P_u}{P_{uz}} = \frac{1300}{3568} = 0.364$$

It is between 0.2 and 0.8

$$\alpha_n = 0.667 + 1.667\left(\frac{P_u}{P_{uz}}\right)$$

$$= 1.273$$

$$\left(\frac{190}{264}\right)^{1.273} + \left(\frac{110}{264}\right)^{1.273} < 1$$

$$0.985 < 1$$

Hence the trail section is safe under applied loading.

9. Given

$W_c = 950KN$

$SBC = 250KN/m^2$, M20, Fe415

Self wt of footing $W_f = 10\%$ of W_c

$= 95KN$

$W_c + W_f = 950 + 95$

$= 1045KN$

Area of footing $= \frac{W_c + W_f}{q_0} = \frac{1045}{250} = 4.18 m^2$

Side of the footing $= \sqrt{4.18} = 2.044 \simeq 2.1m$

$\therefore$ Area of footing $= 2.1 \times 2.1m$

Factored soil pressure due to column load

Only, $P_u = \frac{1.5W_c}{x^2}$

$= \frac{1.5 \times 950}{2.1 \times 2.1} = 323.13 KN/m^2$

Depth of footing

(i) By one way shear criteria $\therefore$ Critical section shall be at a distance 'd' from the face of the column

Shear force due to factored soil pressure at critical section

$= 323.13 * 2.1 (1.1 - d)$

$= 678.573(1.1-d)$ — (1)

Assuming 0.2% of steel,

$\tau_c = 0.32 N/mm^2$

Shear force resisted by the section

$= \tau_c * 2.1 * d$ — (2)

$746.43 - 678.573d = 0.32 \times 2.1 \times d$

$746.43 - 678.573d = 672d$

$d = 0.552 mm$

$= 552 mm$

(ii) By two way shear criteria : considering the critical section at a distance '$d/2$' from the periphery of the shear force

perimeter, $P = 4(0.55+d)$

$P = 2.2 + 4d$

Shear force resisted by the critical section

$= \tau_c * Area$

$= \tau_c * P * depth$

$= 0.25\sqrt{f_{ck}} * (2.2+4d)(d)$

$= 1.118(2.2+4d)(d)$

$= 77.78d + 141.42d^2$ — (3)

Shear force applied due to reaction

$= 323.13((2.1)(2.1) - (0.55+d * 0.55+d)$

$= 678.573(1.1-d) - ①$

Assuming 0.2% of steel,

$\tau_c = 0.32\,N/mm^2$

Shear force resisted by the section

$= \tau_c * 2.1 * d - ②$

$746.43 - 678.573d = 0.32 \times 2.1 \times d$

$746.43 - 678.573d = 672d$

$d = 0.552mm$

$= 552mm$

(ii) By two way shear criteria : considering the critical section at a distance '$d/2$' from the periphery of the shear force

perimeter, $P = 4(0.55+d)$

$P = 2.2 + 4d$

Shear force resisted by the critical section

$= \tau_c * \text{Area}$

$= \tau_c * P * \text{depth}$

$= 0.25\sqrt{f_{ck}} * (2.2+4d)(d)$

$= 1.118(2.2+4d)(d)$

$= 77.78d + 141.42d^2 - ③$

Shear force applied due to reaction

$= 323.13((2.1)(2.1) - (0.55+d * 0.55+d)$

$$= 323.13\left(4.41 - (0.3025 + d^2 + 1.1d)\right)$$

$$= 1425 - \left((0.3025 + d^2 + 1.1d)\right)$$

$$= 1.287m$$

(iii) Depth of footing bending moment Criteria ..

$$Mu = 323.13 * 2.1 * \left(\frac{2.1-0.55}{2}\right) * \left(\frac{2.1-0.55}{4}\right)$$

$$= 203.78 \text{ KN-m}$$

$$Mulimit = 0.36 f_{ck} \frac{x_{umax}}{d}\left[1 - 0.42 \frac{x_{ulimit}}{d}\right) b d^2$$

$$= 0.36 \times 20 \times 0.48 \left(1 - (0.42)(0.48)\right] 2210\, d^2$$

$$203.78 = 6097.98 d^2$$

$$d = 182.8 mm.$$

From the above three cases, $d = 1.287m$

$$D = 1.287 + 50 + 16/2 = 1.345m.$$

Area of steel

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

$$203.78 \times 10^6 = 0.87 \times 415 \times A_{st} * 1345 \left[1 - \frac{415 * A_{st}}{20 \times 2210 \times 1345}\right]$$

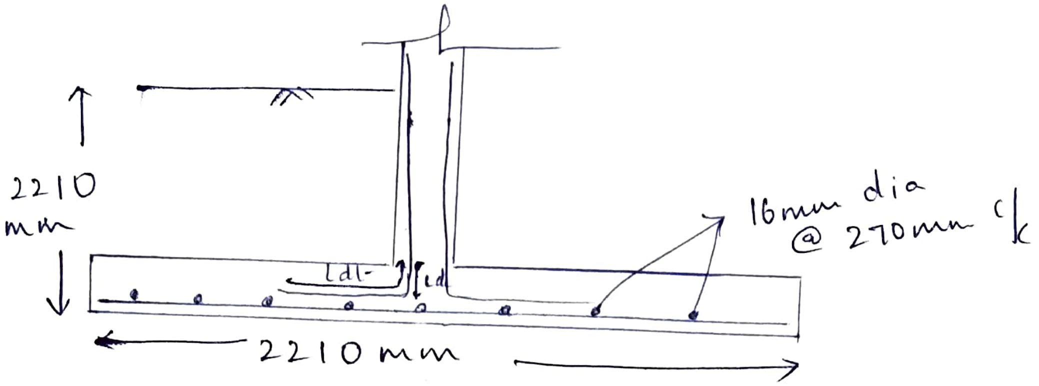
$$A_{st} = 420.87 mm^2$$

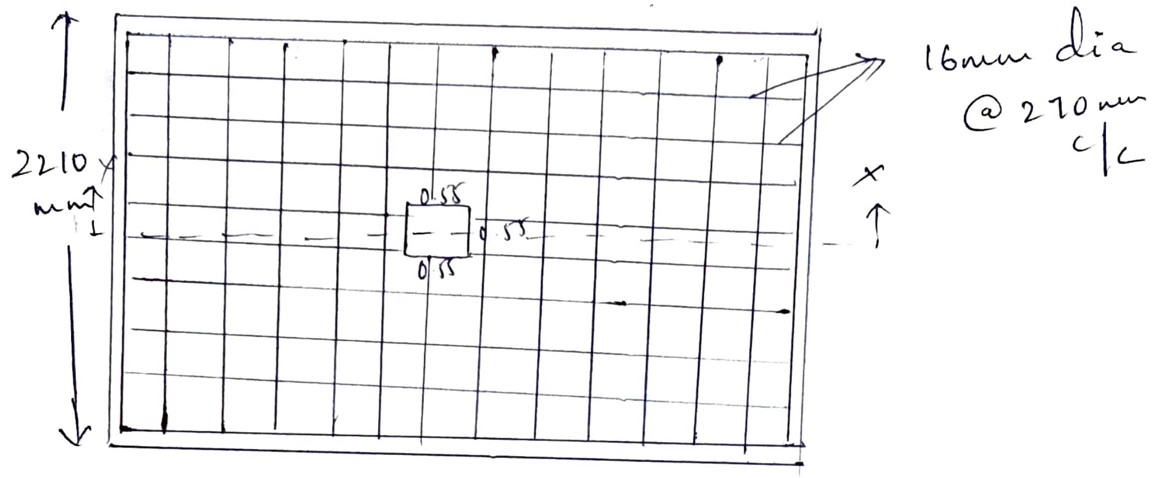
$$A_{st} min = \frac{0.12}{100}\left[2210 \times 1345\right]$$

$$= 356.3 mm^2$$

$$S = \frac{\pi/4 (16)^2 * 2210}{421} = 300 mm$$

Provide 16mm dia bars at 300mm c/c in each direction.





10 Given

$W_c = 880 KN$

Column = 500 x 500 mm

SBC = 220 KN/m²

M20 & Fe415

Self wt of footing W_f = 10% of W_c

= 88 KN

$W_c + W_f = 880 + 88 = 968 KN$

$$\text{Area of footing} = \frac{W_c + W_f}{q_0} = \frac{968}{220}$$

$= 4.4 m^2$

Side of square footing $= \sqrt{4.4} = 2.097m$

$\simeq 2.25m$

$\therefore$ Area of footing $= 2.25 \times 2.25$

Factored soil pressure due to column load only, $P_u = \dfrac{1.5W_c}{x^2}$

$= \dfrac{1.5(880)}{2.25 \times 2.25} = 260.74 KN/m^2$

Depth of footing

(i) By one way shear : @ 'd' from face of the column

$= 260.74 * 2.25(1-d)$

$= 586.66(1-d)$ - (1)

Assume 0.2% of steel, $\tau_c = 0.32 N/mm^2$

Shear force resisted $= \tau_c \times 2.25 \times d$

$586.66 - 586.66d = 0.72d \times 10^3$

$d = 0.448$

$d = 0.45m$

$= 450mm$

(ii) By two way shear :: @ distance 'd/2' from the periphery of the column

$P = 4(0.5+d)$

$P = 2 + 4d.$

Shear force resisted by the critical section

$= \tau_c * \text{Area}$

$= 0.25\sqrt{f_{ck}}\,(2+4d)(d)$

$= 70.7d + 141.42d^2$

Shear force due to reaction

$= 260.74\left((2.25)(2.25) - (0.45+d)(0.45+d)\right)$

$d = 350\,mm$

(iii) Depth of footing by bending moment

@ face of the column.

$M_u = 260.74 \times 2.25 * \left(\frac{2.25 - 0.5}{2}\right)\left(\frac{2.25 \cdot 0.5}{4}\right)$

$= 224.58\,KNm$

$M_{u\,limit} = 0.36 f_{ck} \frac{X_{umax}}{d}\left[1 - 0.42\frac{X_{umax}}{d}\right] b d^2$

$d = 210\,mm$

From above three, $D = 450 + 50 + 16/2$

$D = 508\,mm \simeq 510\,mm$

Area of steel

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

$224.58 \times 10^6 = 0.87 \times 415 \times A_{st} * 450 \left[1 - \frac{415 \times A_{st}}{20 \times 2250 \times 450}\right]$

$A_{st} = 1423.8\,mm^2$

$A_{st\,min} = \frac{0.12}{100}\left[2250 * 510\right] = 1377\,mm^2$

Use 16mm dia bars

$$S = \frac{\pi/4 \, (16)^2 \times 2250}{1423.8} = 317.7 \simeq 300\,mm$$

$\therefore$ provide 16mm dia bars @ 300mm c/c