CBCS SCHEME

18CV53

USN

Fifth Semester B.E. Degree Examination, Feb./Mar. 2022

Design of RC Structural Elements

Time: 3 hrs. Max. Marks: 100

Note: 1. Answer any FIVE full questions, choosing ONE full question from each module.
2. Assume any missing data.
3. Use of IS-456, SP-16 chart permitted.

Module-1

1 a. Explain the following:
i) Partial safety factors for loads and materials.
ii) Explain the principles of limit state design. **(10 Marks)**

b. Explain under reinforced section, over-reinforced section, balance section with neat sketches and also show that $X_{ulim} = 0.53d$, for Fe250 grade of steel. **(10 Marks)**

OR

2 a. Briefly explain the step by step procedure for short term deflection and long term deflection. **(10 Marks)**

b. A flanged beam of T-section is simply supported over an effective span of 8m. The beam has effective flange width of 1400mm, thickness of flange as 150mm, breadth of web as 300mm and effective depth of 450mm. It is reinforced with 4 bars of 25mm diameter in tension and 3 bars of 16mm diameter in compression, check the beam for deflection. Use M_{20} grade concrete and Fe415 steel. **(10 Marks)**

Module-2

3 A R.C.C beam of rectangular section 300 × 600mm is reinforced with 4 bars of 20mm diameter with an effective cover 50mm, effective span of the beam is 6m. Assuming M_{20} concrete and Fe250 steel. Determine the central concentrated load P, that can be carried by the beam in addition to its self weight. **(20 Marks)**

OR

4 a. Find the steel for a rectangular section 300 × 600mm to support a load of 80kN/m with span of 6m (effective) and cover 40mm (effective) adopt M_{20} concrete Fe415 steel. **(10 Marks)**

b. A singly reinforced beam 250 × 500mm is reinforced with 4 – 16mm diameter and cover 40mm (effective) with effective span 6m. Determine the central point load that can be applied at mid span adopt M_{20} concrete Fe500 steel. **(10 Marks)**

Module-3

5 A T-beam slab floor gas 125mm thick slab forming part of T-beam which are of 8m clear span. The end bearing are 450mm thick, spacing of T-beams is 3.5m. The live load on the floor is $3kN/m^2$. Design one of the intermediate beams. Use M_{20} concrete and Fe415 steel. **(20 Marks)**

OR

6 A rectangular beam is to be simply supported on supports of 300mm width. The clear span of the beam is 6m. The beam is to have width of 230mm. The characteristic superimposed load is 12kN/m. Using M_{20} and Fe500 steel, design the beam and sketch details of reinforcement. **(20 Marks)**

Module-4

7 A hall has clear dimensions 3m × 9m with wall thickness 230mm. The live load on the slab is $3kN/m^2$ and finishing load $1kN/m^2$ may be assumed. Use M_{20} grade concrete and Fe415 steel. Design the slab, check for shear and deflection. **(20 Marks)**

OR

8 Design a dog-legged stairs for an building in a room measuring 3.6 × 5.2m clear span. The vertical distance between the floors is 3.2m. Consider LL $3kN/m^2$. Use M20 concrete and Fe415 grade of steel. Assume stairs are supported on 300mm wall at the outer edges of landing slabs consider Rise = 160mm and Tread = 300mm. **(20 Marks)**

Module-5

9 a. Distinguish between short column and long column. **(05 Marks)**

b. Design a circular pin ended column 400mm diameter and helically reinforced with an unsupported length 4.5m to carry a factored load 900kN. Assume M_{30} concrete and Fe415 steel. **(15 Marks)**

OR

10 Design a Isolated rectangular footing of uniform depth for the column size of 230mm × 300mm supporting an axial service load of 850kN. The safe bearing capacity of soil is $150kN/m^2$. Adopt M_{20} grade concrete and Fe415 grade steel sketch the reinforcement details. **(20 Marks)**

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AY: 2020-21

Department: Civil Engg.
IA Test No:
Subject with Sub. Code: Design of RC structural elements (18CV53)
Semester / Division: V.
Name of Faculty: Prof. Parvati B. Oni

Q.No.	Solution and Scheme	Marks
1. a.	i) Partial safety factors for loads & material – To account for the different conditions like for material strength, load etc, different partial factors are used for loads and materials.	
	Design strength = $\frac{\text{characteristic strength}}{\gamma_m}$	
	Design load = γ_f x characteristic load.	
	As per clause 36.4.2 page 68 of IS 456-2000, $\gamma_m = 1.5$ for concrete and $\gamma_m = 1.15$ for steel. Similarly clause 36.4.1 page 68 of code gives γ_f values in table 18 for different values for different load combinations and different limit states.	
	ii) Partial safety factor for materials – IS 456-2000 recommendations – Partial safety factors for materials to be multiplied and characteristic strength is	

Material	Limit state: Collapse	Deflection	cracking
Concrete	1.5	1.0	1.3
Steel	1.15	1.0	1.0

ii) A degree of loading or other actions imposed on a structure can result in limit state where the structure's condition no longer fulfils its design criteria such as fitness for use, structural integrity, durability and so on. All actions likely to occur during a structural design life are considered during LSD method to ensure the structure remains fit for use with appropriate levels of reliability. LSD involves estimating the subject loads on a structure, choosing the size of members to check and selecting the appropriate design criteria. LSD requires two principle criteria to be satisfied :- the ultimate limit state and serviceability limit state.

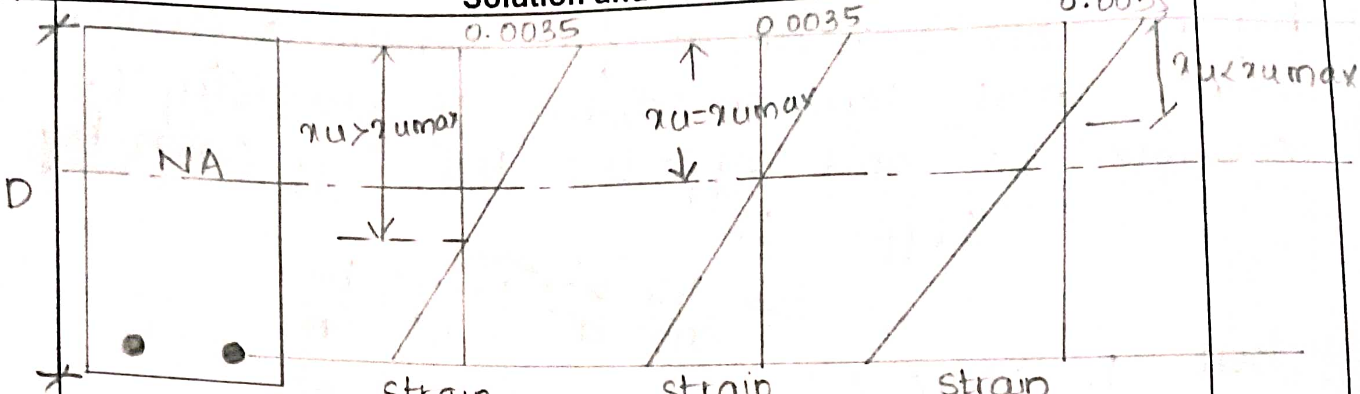
b. Under reinforced section – An under reinforced section is the one in which steel percentage P_t is less than critical or limiting percentage P_{tlim}. Due to this, actual NA is above the balanced NA ie $x_u < x_{umax}$.

Over reinforced section – In this section, the steel percentage is more than limiting percentage due to which the NA falls below the balanced ie $x_u > x_{umax}$.

Balanced section – In balanced section, the strain in steel and strain in concrete reach their maximum values simultaneously. The % of steel in this section is known as critical limiting steel.

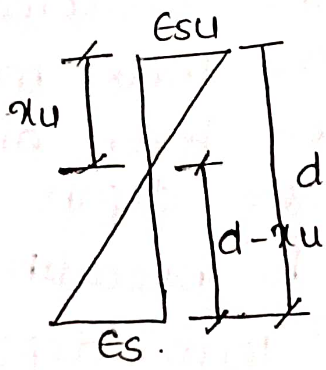
ie $x_u = x_{umax}$

Q.No.	Solution and Scheme	Marks



strain — over reinforced   strain — Balanced   strain — Under reinforced.

From the fig we have,



$$\frac{\epsilon_s}{d-x_u} = \frac{0.0035}{x_u}$$

$$\therefore \frac{x_u}{d-x_u} = \frac{0.0035}{\epsilon_s}$$

$$\frac{x_u}{d} = \frac{0.0035}{\epsilon_s + 0.0035}$$

To avoid compression failure, IS 456 recommends, strain corresponding to 0.0035

ie $\epsilon_{smin} = 0.87\frac{f_y}{E} + 0.002$

$$\therefore \frac{x_{ulim}}{d} = \frac{0.0035}{0.87\frac{f_y}{E_s} + 0.002 + 0.0035} = \frac{0.0035}{0.87\frac{f_y}{E_s} + 0.0055}$$

$\therefore \frac{x_{ulim}}{d}$ for Fe 250 steel

$$= \frac{0.0035}{\frac{0.87 \times 250}{2 \times 10^5} + 0.0055}$$

$= \cancel{0.762}$ 0.531

$$\therefore \boxed{x_{ulim} = 0.53d}$$

2. a The short term deflection can be calculated by the usual methods for elastic deflections using short term modulus of elasticity of concrete E_c and effective MI I_{eff} given by

$$I_{eff} = \frac{I_r}{1.2 - \frac{M_r}{M}\frac{z}{d}\left(1-\frac{x}{d}\right)\frac{b_w}{b}}$$

But $I_r \leq I_{eff} \leq I_{gr}$ where

I_{eff} = MI of cracked section

M_r = cracking moment = $\frac{f_{cr} I_{gr}}{y_t}$

M = max moment under service load.

z = lever arm

x = depth of NA, b_w = breadth of web.

b = breadth of compression face.

Long term deflections occur due to combined effect of shrinkage and creep.

Calculating shrinkage deflection – It may be calculated as

$$a_{cs} = k_3 \psi_{cs} L^2.$$

where k_3 is a constant = 0.5 for cantilevers.

ψ_{cs} = shrinkage curvature = $k_4 \frac{\epsilon_{cs}}{D}$

$$k_4 = 0.65 \times \frac{P_t - P_c}{\sqrt{P_t}} \leq 1.0$$

$P_t = \frac{100 A_{st}}{bd}$ and $P_c = \frac{100 A_{sc}}{bd}$.

Deflection due to creep –

$$a_{cc(perm)} = a_{icc(perm)} - a_{i(perm)}.$$

b. Given –

$b = b_w = 300\,mm$, $D = 600\,mm$, $b_f = 1400\,mm$

$d = 600 - 50 = 550\,mm$ $f_{ck} = 20\,MPa$ $f_y = 415\,MPa$.

$L = 8m = 8000mm$.

Q.No.	Solution and Scheme	Marks

$A_{st} = 4 \times \frac{\pi}{4} \times 25^2 = 1963.49\ mm^2$

$A_{sc} = 3 \times \frac{\pi}{4} \times 16^2 = 603.18\ mm^2$

$P_t = \frac{1963.49 \times 100}{1400 \times 550} = 0.254$

$P_c = \frac{603.18}{1400 \times 550} \times 100 = 0.078$

$f_s = 0.58 \times 415 \times 1 = 240.7\ N/mm^2$

From fig 4 of IS 456-2000,

$F_1 = 1.42$.

From fig 5 of IS 456 2000, $F_2 = 1.0$.

From fig 6 of IS 456-2000, $F_3 = 0.8$ (for $\frac{b_w}{b_f} = 0.214$)

$\therefore$ Basic $\left(\frac{L}{d}\right) = \frac{8000}{550} F_1 F_2 F_3 = 14.54 \times 1.42 \times 1 \times 0.8$

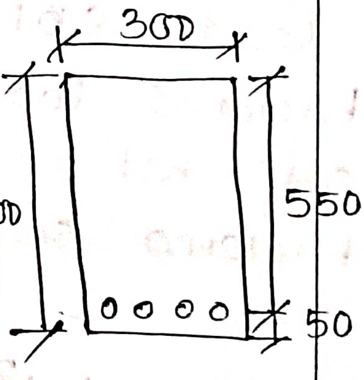
$= 16.52$

$\left(\frac{L}{d}\right)_{per} = 20$ from Cl. 23.2.1(a) of IS 456-2000.

$\therefore$ Basic $\left(\frac{L}{d}\right) < \left(\frac{L}{d}\right)_{per}$ Hence ok.

3. Area of steel $A_{st} = \frac{\pi}{4} \times 20^2 \times 4$

$= 1256.63\ mm^2$



$\frac{x_u}{d} = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b d}$

$\frac{x_u}{d} = \frac{0.87 \times 250 \times 1256.63}{0.36 \times 20 \times 300 \times 550} = \cancel{0.381}\ 0.23$

But $\frac{x_{ulim}}{d} = 0.53$

$\therefore x_{ulim} > x_u$. Hence under reinforced s/n.

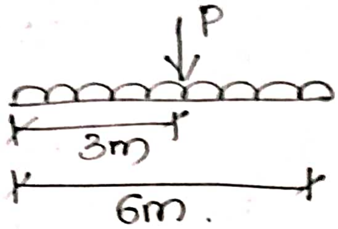
$\therefore x_u = 0.23 \times 550 = 126.5\ mm$.

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

$$= 0.87 \times 250 \times 1256.63 \times 550 \left(1 - \frac{1256.63 \times 250}{300 \times 550 \times 20}\right)$$

$$= 136.01 \times 10^6 \text{ Nmm}$$

Let P be the concentrated load in kN at centre and w be the self wt/length.



$$w = 0.3 \times 0.6 \times 25$$

$$= 4.5 \text{ kN/m}.$$

∴ Factored self wt $= 1.5 \times 4.5$

$$W_u = 6.75 \text{ kN/m}.$$

Design load $= w_u = 1.5 \times P$.

∴ Design moment $= M_u = \frac{W_u L^2}{8} + \frac{P_u L}{4}$

$$\Rightarrow 136.01 \times \cancel{10^6} = \frac{6.75 \times 6^2}{8} + \frac{P \times 1.5 \times 6}{4}$$

$$\Rightarrow 136.01 \times \cancel{10^6} = 30.37 + 2.25P$$

$$\therefore \boxed{P = 46.95 \text{ kN}}$$

4.
a. Given $b = 300$ mm $D = 600$ mm, $L_{eff} = 6$ m

$d = 600 - 40 = 560$ mm $f_{ck} = 20 \frac{N}{mm^2}$ $f_y = 415 \frac{N}{mm^2}$

Load $= 80$ kN/m

Factored load $= 1.5 \times 80 = 120$ kN/m

Self wt $= 0.3 \times 0.6 \times 25 = 4.5$ kN/m

Factored self wt $= 1.5 \times 4.5 = 6.75$ kN/m.

∴ Total $W_u = 126.75$ kN/m.

$$\therefore M_u = \frac{W_u L^2}{8} = \frac{126.75 \times 6^2}{8} = 570.37 \text{ kNm}.$$

$$M_{ulim} = 0.138 f_{ck} b d^2 = 0.138 \times 20 \times 300 \times 560^2$$

$$= 259 \text{ kNm}$$

∴ $M_u > M_{ulim}$ Hence the section is doubly reinforced.

Hence $A_{st} = A_{st1} + A_{st2}$

Q.No.	Solution and Scheme	Marks

$$A_{st1} = \frac{0.36 f_{ck} x_u b}{0.87 f_y} = \frac{0.36 \times 20 \times 0.48 \times 560 \times 300}{0.87 \times 415}$$

$$= 1608.10\ mm^2.$$

$$M_{u2} = M_u - M_{ulim} = 570.37 - 259 = 311.37\ kN/m$$

$$\therefore A_{st2} = \frac{M_{u2}}{0.87 f_y (d-d')} = \frac{311.37 \times 10^6}{0.87 \times 415 (560-40)}$$

$$= 1658.46\ mm^2.$$

$$\therefore \text{Total } A_{st} = A_{st1} + A_{st2} = 1608.10 + 1658.46$$

$$= \underline{3266.56\ mm^2}.$$

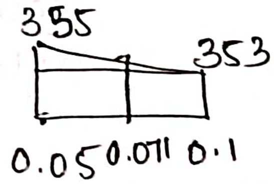
$$A_{sc} = \frac{0.87 f_y A_{st2}}{f_{sc}} = \frac{0.87 \times 415 \times 1658.46}{f_{sc}}$$

Strain $\cancel{f_{sc}}$ $= 0.0035 \left(\frac{x_u - d'}{x_u}\right) = 0.0035 \left(\frac{0.48 \times 560 - 40}{0.48 \times 560}\right)$

$$= 2.979 \times 10^{-3}.$$

From table F for $\frac{d'}{d} = \frac{40}{560} = 0.071$ & $f_y = 415\ \frac{N}{mm^2}$

$$f_{sc} = 353.84\ N/mm^2.$$



$$\therefore A_{sc} = \frac{0.87 \times 415 \times 1658.46}{353.84}$$

$$A_{sc} = \underline{1692.25\ mm^2}$$

b. $b = 250\ mm$ $D = 500\ mm$, $L = 6\ m$, $f_{ck} = 20\ \frac{N}{mm^2}$

$f_y = 500\ N/mm^2$. $d = 500 - 40 = 460\ mm$.

$$A_{st} = 4 \times \frac{\pi}{4} \times 16^2 = 804.24\ mm^2.$$

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

$$= 0.87 \times 500 \times 804.24 \times 460 \left(1 - \frac{804.24 \times 500}{250 \times 460 \times 20}\right)$$

$$M_u = 132.79 \times 10^6\ Nmm$$

But. $M_u = \frac{W_u L^2}{8} + \frac{P_u L}{4}.$

Self wt (factored) w_u = $1.5 \times 0.25 \times 0.5 \times 25 = 4.68$ kN/m

$\therefore \; 132.79 = \frac{4.68 \times 6^2}{8} + \frac{1.5 P_u \times 6}{4}$

$\Rightarrow \; 132.79 = 21.06 + 2.25 P_u$

$\boxed{\therefore P_u = 49.66 \text{ kN}}$

5. $D_f = 125$ mm spacing = 3.5 m, LL = 3 kN/m², $f_{ck} = 20$ N/mm², $f_y = 415$ N/mm², clear span = 8 m,

Dimensions of beam –

Depth = $\frac{1}{12}$th to $\frac{1}{15}$th span.

$= \frac{1}{12} \times 8000$ to $\frac{1}{15} \times 8000$

$= 667$ mm to 533.3 mm

Let d = 600 mm, D = 650 mm, $b_w = \frac{1}{2}$ d to $\frac{1}{3}$d.

$\therefore$ b = 250 mm.

Effective span –

clear span + d = 8 + 0.6 = 8.6 m.

c/c bearings = 8 + 0.45 = 8.45 m

eff span = 8.45 m

$b_f = \frac{L_o}{6} + b_w + 6D_f$ $\qquad L_o = 8.45$ m.

$b_f = \frac{8450}{6} + 250 + 6 \times 125 = 2408$ mm

$b_f = 0.5(L_1 + L_2) + b_w = 3.5$ m $= 3500$ mm

$\therefore$ $b_f = 2408$ mm.

Design moment (M_u) and shear force (V_u) –

Load from slab –

SW of slab = $0.125 \times 1 \times 1 \times 25 = 3.125$ kN/m²

FF = 0.6 kN/m²

LL = 3 kN/m²

Total W = 6.725 kN/m²

Load of slab on beam = $6.725 \times 1 \times 3.5 = 23.577$ kN/m

Self wt of rib =

Width of rib = 250 mm

Depth of rib = 650 − 125 = 525 mm

Q.No.	Solution and Scheme	Marks

$\therefore$ Self wt of rib $= 0.25 \times 0.525 \times 1 \times 25 = 3.28\ kN/m$

Weight of plaster to rib $= 0.5\ kN/m$

$\therefore$ Total load on beam $= 23.577 + 3.28 + 0.5$
$= 27.3\ kN/m$

Factored load $W_u = 1.5 \times 27.3 = 40.95\ kN/m$

$$M_u = \frac{W_u L^2}{8} = \frac{40.95 \times 8.45^2}{8} = 365.5\ kNm$$

$$V_u = \frac{1}{2} \times W_u L = \frac{1}{2} \times 40.95 \times 8.45 = 173\ kN$$

Design of longitudinal bars –

$x_{ulim} = 0.48d = 0.48 \times 600 = 288\ mm$

$M_{ulim} = 0.446 f_{ck}\, b_f\, D_f\, (d - 0.5 D_f)$
$= 0.446 \times 20 \times 2408 \times 125 (600 - 0.5 \times 125)$
$= 1443.145 \times 10^6\ Nmm$

$\therefore$ $M_u < M_{ulim}$. It can be designed as singly reinforced.

$x_u = D_f = 125\ mm$

$M_u' = 0.36 f_{ck}\, b_f\, D_f\, (d - 0.42 D_f)$
$= 0.36 \times 20 \times 2408 \times 125 (600 - 0.42 \times 125)$
$= 1186.54 \times 10^6\ Nmm$

Thus $M_u < M_u'$ ie NA is within the flange.

Then $365.5 \times 10^6 = 0.87 \times 415 \times A_{st} \times 600 \left(1 - \frac{A_{st}}{2408 \times 600} \times \frac{415}{20}\right)$

$$1687.21 = A_{st}\left(1 - \frac{A_{st}}{69628.9}\right)$$

$\therefore$ $A_{st} = 1730\ mm^2$

Provide 6 bars of 20mm diameter

A_{st} prov $= 6 \times \frac{\pi}{4} \times 20^2 = 1885\ mm^2$.

Design of shear reinforcement –

$$P_t = \frac{1885}{250 \times 600} \times 100 = 1.257$$

From table no 19 of IS 456.

$$\tau_{c\,max} = 2.8\ N/mm^2$$

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

$$V_{us} = V_u - \tau_c bd$$

$$= 173000 - 0.67 \times 250 \times 600$$

$$V_{us} = 72500\ N$$

Using 2 legged 8mm ϕ stirrups,

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

$$72500 = \frac{0.87 \times 415 \times 2 \times \frac{\pi}{4} \times 8^2 \times 600}{S_v}$$

$$\therefore S_v = 300.39\ mm$$

Max spacing allowed is $0.75 \times 600 = 450\ mm$ or 300mm whichever is less.

Check for deflection control-

As it is simply supported,

$$\left(\frac{L}{d}\right)_{basic} = 20 \qquad P_t = 1.257.$$

Ast reqd = $1730\ mm^2$ Ast prov = $1885\ mm^2$.

$$f_s = 0.58 \times \frac{1730}{1885} \times 415 = 220.9\ N/mm^2$$

From fig 4 of IS 456 - 2000, $F_1 = 1$.

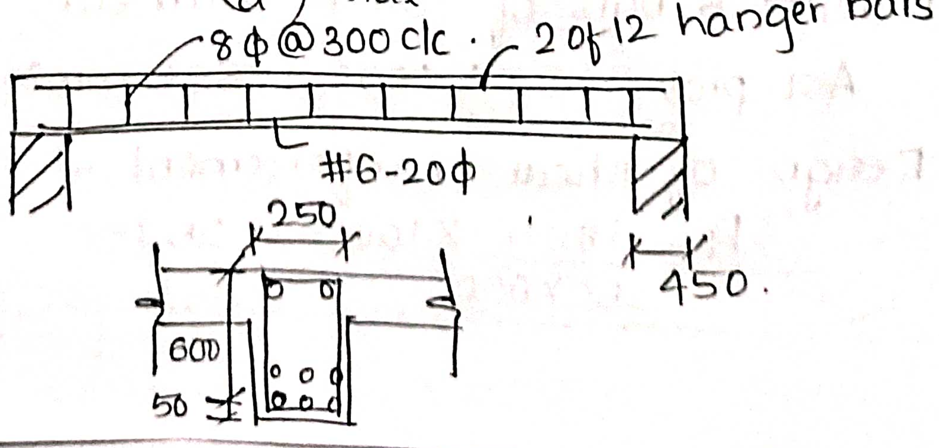
No compression steel $\therefore F_2 = 1$.

$$\frac{b_w}{b_f} = \frac{250}{2408} = 0.104. \quad \therefore \quad F_3 = 0.8$$

$$\left(\frac{L}{d}\right)_{max} = F_1 F_2 F_3 \times \left(\frac{L}{d}\right)_{basic} = 1 \times \cancel{0.104} \times 0.8 \times 20$$

$$= 16.$$

$$\therefore \left(\frac{L}{d}\right)_{prov} < \left(\frac{L}{d}\right)_{max}.$$



Q.No.	Solution and Scheme	Marks
6.	Width of beam = 230 mm	

Depth of beam = $\frac{1}{12}$ to $\frac{1}{15}$ span

$= \frac{1}{12} \times 6000$ to $\frac{1}{15} \times 6000$

$= 500$ to 400 mm

$\therefore$ d = 400 mm.

D = 400 + 50 = 450 mm.

Eff span -

c/c between supports = 6 + 0.23 = 6.23 m

clear span + d = 6 + 0.4 = 6.4 m.

$\therefore$ Eff span = 6.23 m.

Design moment and shear -

Imposed load = 12 kN/m

self wt. = 0.23 × 0.45 × 1 × 25

= 2.58 kN/m

Design load -

Wu = 1.5 × 12 + 1.5 × 2.58

= 21.87 kN/m

$$Mu = \frac{21.87 \times 6.23^2}{8} = 106.10 \text{ kNm}.$$

$Mulim = 0.133 f_{ck} b d^2 = 0.133 \times 20 \times 230 \times 400^2$

= 97.88 kNm.

$\therefore$ Mu > Mulim

Hence design it as doubly reinforced.

$$Ast_1 = \frac{0.36 \times f_{ck} \times b \times x_{ulim}}{0.87 \times f_y}$$

$$= \frac{0.36 \times 20 \times 230 \times 0.46 \times 400}{0.87 \times 500}$$

= 700.468 mm²

Mu - Mulim = 106.10 - 97.88 = 8.22 kNm

Stress in tensile steel = $0.87 f_y Ast_2$

Lever arm = d - d' = 400 - 50

= 350 mm

$\therefore 0.87 \times 500 \times A_{st2} \times 350 = 8.22 \times 10^6$.

$\therefore A_{st2} = 53.99 \text{ mm}^2$.

Min A_{st} =

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

$$\therefore A_s = \frac{0.85 \times bd}{f_y} = \frac{0.85 \times 230 \times 400}{500} = 156.4 \text{ mm}^2$$

$\therefore A_{st} = A_{st1} + A_{st2} = 700.468 + 156.4 = 856.86 \text{ mm}^2$

To find Asc –

$$\epsilon_{sc} = 0.0035 \left(\frac{x_{ulim} - d'}{x_{ulim}} \right)$$

$$= 0.0035 \left(\frac{0.46 \times 400 - 50}{0.46 \times 400} \right) = 0.728 \times 0.0035 = 2.54 \times 10^{-3}$$

From stress strain curve corresponding to $\frac{f_y}{1.15}$, $f_{sc} = 425 \text{ N/mm}^2$.

$\therefore f_{sc} A_{sc} = 0.87 f_y A_{st2}$

$\Rightarrow 425 \times A_{sc} = 0.87 \times 500 \times 156.4$

$\therefore A_{sc} = 160.08 \text{ mm}^2$.

Provide 6 bars of 16mm dia at tension zone. and 2 bars of 12mm dia in compression zone.

Design for shear –

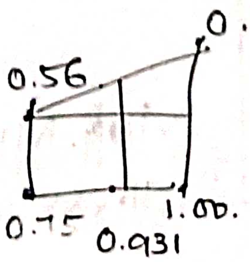
$$V_u = \frac{W_u L}{2} = \frac{21.87 \times 6.23}{2} = 68.125 \text{ kN}$$

$$\tau_v = \frac{V_u}{bd} = \frac{68.125 \times 10^3}{230 \times 400} = 0.74 \text{ N/mm}^2.$$

$$P_t = \frac{A_{st} \times 100}{bd} = \frac{856.86}{230 \times 400} \times 100 = 0.931$$

From table 19 of IS 456,

$\tau_c = 0.603 \text{ N/mm}^2$



From table 20,

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

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

Q.No.	Solution and Scheme	Marks
	∵ Shear reinforcement is to be designed.	

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

$= 68,125 - 0.603 \times 230 \times 400$

$= 12649$ N.

Using 2 legged 6mm stirrups,

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

$$\therefore S_v = \frac{0.87 \times 500 \times 2 \times \frac{\pi}{4} \times 6^2 \times 400}{12649}$$

$= 777.88$ mm

max spacing permitted $= 0.75d = 0.75 \times 400$
$= 300$ mm

∴ Provide 6mm stirrups @ 300 mm c/c.

<u>Check</u> for deflection –

$\left(\frac{L}{d}\right) = 20$, $P_t = 0.931$

$$f_s = 0.58 \times f_y \times \frac{856.86}{6 \times \frac{\pi}{4} \times 16^2} = 205.90 \text{ N/mm}^2$$

From fig 4, $F_1 = 1.10$.

$$P_c = \frac{A_{sc}}{bd} \times 100 = \frac{160.08}{230 \times 400} \times 100 = 0.174$$

From fig 5, $F_2 = 1.05$.

From fig 6, $F_3 = 0$.

$$\max\left(\frac{L}{d}\right) = F_1 F_2 F_3 \times \left(\frac{L}{d}\right)_{basic} = 1.10 \times \cancel{0.174}\ 1.05 \times 1 \times 20$$

$= 23.1$ mm

$$\left(\frac{L}{d}\right) = \frac{6000}{400} = 15 < 23.1$$

Hence satisfactory.

7. $\frac{L_y}{L_x} = \frac{9}{3} = 3 > 2$. Hence design it as one way.

$$\frac{1}{25} \times \text{span} = \frac{1}{25} \times 3000 = 120 \text{ mm}$$

$\therefore d = 125$ mm

$D = 150$ mm

Loads -

DL = 0.15 X 1 X 1 X 25 = 3.75 kN/m.
FL = 1 kN/m.
LL = 3 kN/m.
Total = 7.75 kN/m   Wu = 1.5w = 1.5 X 7.75

Eff span -
i) 3000 + d = 3000 + 125 = 3125 mm
ii) 3000 + w = 3000 + 230 = 3230 mm
L = 3.125 m.

$Mu = \frac{WuL^2}{8} = \frac{1.5 \times 7.75 \times 3.125^2}{8} = 14.19$ kNm

$Vu = \frac{WuL}{2} = \frac{1.5 \times 7.75 \times 3.125}{2} = 18.164$ kN

$x_{ulim} = 0.48d = 0.48 \times 125 = 60$ mm

$M_{ulim} = 0.36 f_{ck} b x_{ulim} (d - 0.42 x_{ulim})$
$= 0.36 \times 20 \times 1000 \times 60 (125 - 0.42 \times 60)$
$= 43.114$ kNm

$Mu < M_{ulim}$. Hence single reinforcement.

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

$14.19 \times 10^6 = 0.87 \times 415 \times A_{st} \times 125 \left(1 - \frac{A_{st}}{1000 \times 125} \times \frac{415}{20}\right)$

$A_{st} = 333$ mm^2.

Using 10 mm ϕ bars,

$S = \frac{\pi/4 \times 10^2}{333} \times 1000 = 235.8$ mm

Hence provide 10 mm ϕ bars @ 225 mm c/c.

Check for shear -

$\tau_v = \frac{Vu}{bd} = \frac{18.164 \times 1000}{1000 \times 125} = 0.145$ N/mm^2

$P_t = \frac{\pi/4 \times 10^2}{225 \times 125} \times 100 = 0.279.$

τ_c for beams = 0.375 N/mm2
for slabs $\tau_c = 1.3 \times 0.375 = 0.487$ N/mm^2

$\tau_v < \tau_c < 0.5\, \tau_{cmax}$.

Hence slab is safe from shear.

Check for deflection -

$P_t = 0.279$

$f_s = 0.58 \times 415 = 240.7$ N/mm^2

From fig 4, $F_1 = 1.50$

Q.No.	Solution and Scheme	Marks

$\left(\frac{l}{d}\right)_{max} = 1.5 \times 20 = 30$

$\left(\frac{l}{d}\right)_{prov} = \frac{3125}{125} = 25 < \left(\frac{l}{d}\right)_{max}$

∴ Deflection control is satisfactory.

Distribution steel –

$A_s = \frac{0.12bd}{100} = \frac{0.12 \times 1000 \times 125}{100} = 150\,mm^2$

Selecting 8mmϕ bars,

$S = \frac{\pi/4 \times 8^2 \times 1000}{150} = 335\,mm.$

∴ Provide 8mm bars @ 300mm c/c.

8 Floor to floor height = 3.2m.

Height of one flight = $\frac{3.2}{2} = 1.6m = 1600mm$

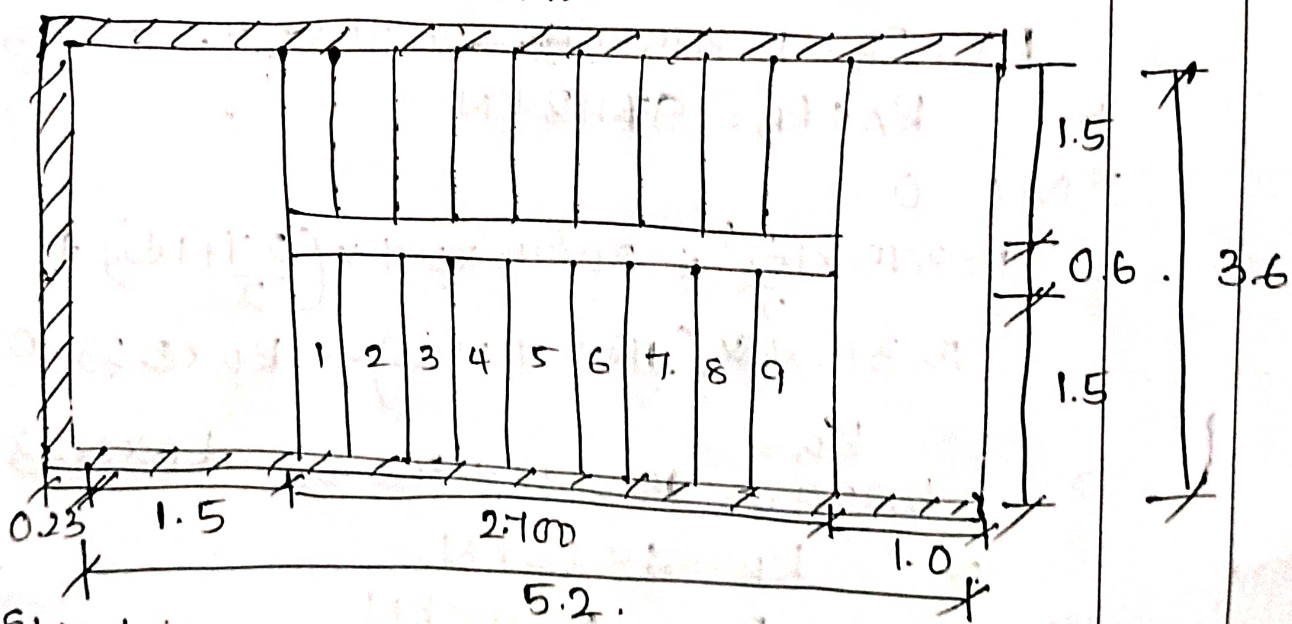
No. of risers = $\frac{1600}{160} = 10$.

∴ No. of treads = 10 − 1 = 9

Assume width of flight = 1.5m.

For 9 treads we need length of 9 × T.

ie 9 × 300 = 2700 mm.



Effective span –

= c/c distance b/w the walls

= 1.5 + 2.7 + 1 + 0.23

= 5.43m

Thickness of waist slab = 1/20th to 1/25th span

ie 260 to 208.

Let us take $t = 250mm$ and $D = 280mm$.

Weight of waist slab $= 0.28\sqrt{1+\left(\frac{160}{300}\right)^2} \times 25$

$= 7.93\ kN/m$.

Weight of steps $= \frac{1}{2} \times \frac{0.16 \times 0.25}{0.25} \times 25 = 2\ kN/m$.

Dead load $= 7.93 + 2 = 9.93\ kN/m$.

In going portion with FL, let us take $DL = 10.5\ \frac{kN}{m}$

In landing portion, $DL = 0.25 \times 1 \times 25 = 6.25\ kN/m$

With finishing material it may be taken as $7.25\ \frac{kN}{m}$

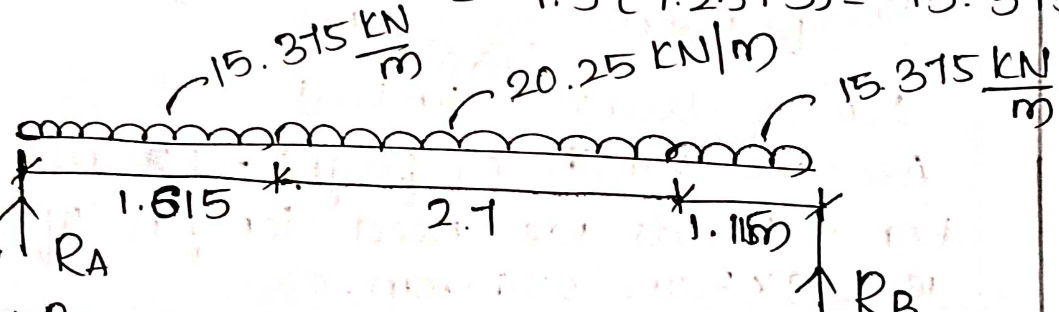
$LL = 3\ kN/m^2$.

∴ Factored load on going per meter width

$= 1.5\,(10.5 + 3) = 20.25\ kN/m$.

and on landing slab per metre width, total load

$= 1.5\,(7.25 + 3) = 15.375\ \frac{kN}{m}$



$R_A + R_B - 15.375 \times 1.615 - 20.25 \times 2.7 - 15.375 \times 1.115 = 0$

$\Rightarrow R_A + R_B = 97.18\ kN$.

$\Sigma M_A = 0$

$\Rightarrow 15.375 \times \frac{1.615^2}{2} + 20.25 \times 2.7 \times \left(\frac{2.7}{2} + 1.615\right) +$
$15.375 \times 1.115 \left(\frac{1.115}{2} + 2.7 + 1.615\right) - R_B \times 5.43 = 0$.

$\Rightarrow 20.05 + 162.11 + 83.52 - R_B \times 5.43 = 0$

$\Rightarrow R_B = 48.92\ kN$.

$\therefore R_A = 48.25\ kN$.

Design Moment -

$M_u = \frac{48.25 \times 5.43}{2} - 15.375 \times 1.615 \left(\frac{5.43 - 1.615}{2}\right)$
$- \frac{2.7}{2} \times 20.25 \times \frac{2.7}{4} - \frac{48.25 \times 5.43}{2}$
$- 15.375 \times 1.115 \left(\frac{5.43 - 1.115}{2}\right)$

$= 130.99 - 47.36 - 18.45 - 130.99 = 36.98$

Q.No.	Solution and Scheme	Marks
	$= 102.79\ kN$	
	$Mu_{lim} = 0.36 f_{ck} b\ 0.48d\ (d - 0.42 \times 0.48d)$	
	$= 0.138 f_{ck} b d^2$	
	$= 0.138 \times 20 \times 1000 \times 250^2$	
	$= 172.5 \times 10^6\ Nmm > Mu$.	
	Hence section is designed as singly reinforced.	
	$Mu = 0.87 f_y A_{st} d \left(1 - \frac{A_{st}}{bd} \frac{f_y}{f_{ck}}\right)$	
	$102.79 \times 10^6 = 0.87 \times 415\ A_{st} \times 250 \left(1 - \frac{A_{st} \times 415}{1000 \times 250 \times 20}\right)$	
	$102.79 \times 10^6 = 90262.5\ A_{st} (1 - 8.3 \times 10^{-5} A_{st})$	
	$102.79 \times 10^6 = 90262.5\ A_{st} - 7.49\ A_{st}^2$	
	$\Rightarrow 7.49\ A_{st}^2 - 90262.5\ A_{st} + 102.79 \times 10^6$	
	$\therefore A_{st} = 1273.3\ mm^2$.	
	Using 16mm ϕ bars, spacing required.	
	$S = \frac{\pi/4 \times 16^2}{1273.3} \times 1000 = 157.9\ mm$	
	Provide 16mm bars at 150 mm c/c.	
	Distribution steel –	
	$A_{st} = 0.12\%$ of gross sectional area.	
	$\frac{0.12}{100} \times 1000 \times 280 = 336\ mm^2$.	
	Using 10mm bars,	
	$S = \frac{\pi/4 \times 10^2}{336} \times 1000 = 283\ mm$.	
	Provide 10 bars at 230mm c/c.	
9.		
a.		

Short column	Long column.
1. If the ratio of eff. length to its least lateral dimension is ≤ 12.	1. If the ratio of effective length of column to its least lateral dimension is ≥ 12.
2. Buckling tendency is very low.	2. Long and cylinder columns buckle easily.
3. The crushing tendency is very high.	3. It has very low crushing tendency.
4. It has more radius of gyration.	4. It has less radius of gyration.
5. Its slenderness ratio is less than 45.	5. Its slenderness ratio is more than 45.

b. $l = 4.5m$. Both ends pinned.

$L = l = 4.5m$.

$$\frac{L}{d} = \frac{4500}{400} = 11.25 < 12.$$

$\therefore$ It can be designed as short column.

Min eccentricity

$$e_{min} = \frac{L}{500} + \frac{D}{30} = \frac{4500}{500} + \frac{400}{30} = 22.33 > 20mm$$

$\therefore e_{min} = 20mm$

$$\frac{e_{min}}{D} = \frac{20}{400} = 0.05.$$

Hence it may be treated as axially loaded column.

Load carrying capacity

$$P_u = 1.05[0.4 f_{ck} A_c + 0.67 f_y A_{sc}]$$

$P_u = 900 kN$. $\cancel{P_u = 1.5 \times 900 = 1350} kN$

Let p be the % of reinforcement used.

$$A_c = A_g\left(1 - \frac{P}{100}\right)$$

$$A_{sc} = \frac{P}{100} A_g$$

$$\Rightarrow \frac{\overset{900}{\cancel{1350}} \times 1000}{1.05} = 0.4 \times 30 \times A_g\left(1 - \frac{P}{100}\right) + 0.67 \times 415 \times \frac{P}{100} A_g$$

$$= A_g\left[12 - 0.12P + \frac{278.05}{100} \times P\right]$$

Q.No.	Solution and Scheme	Marks

$$\cancel{1.28\times10^6} \; 857.14\times10^3 = \frac{\pi}{4}\times400^2\left[12-0.12P+\cancel{2}\,2.78\,P\right]$$

$$\cancel{10.185} \; 6.820 = 12+\cancel{0.158}\;2.66\,P$$

$$\therefore P = 1.947$$

Thus P is between 0.8 to 4%. Hence satisfactory

$$A_{sc} = \frac{P}{100}\times A_g = \frac{1.947}{100}\times\frac{\pi}{4}\times400^2 = 2446.67\ mm^2$$

Provide 8 bars of 20mm ϕ.

$$A_{sc}\ \text{provided} = 8\times\frac{\pi}{4}\times20^2 = 2513.2\ mm^2$$

Helical reinforcement -

Let us try 8mm spirals at 'S' with clear cover of 50mm.

core diameter = $400-2\times50 = 300$ mm.

Area of core $= \frac{\pi}{4}\times300^2 - \cancel{1885}\ 2513.2$

$= 68172.63\ mm^2$

Vol. of core per pitch height s

$= 68172.63\times S$.

Length of one spiral of 8mm ϕ.

$= \pi(300-8) = 292\pi$.

Volume of one spiral

$$V_{us} = \frac{\pi}{4}\times8^2\times292\pi = 46110\ mm^3$$

According to IS 456,

$$\frac{V_{us}}{V_0} \nless 0.36\left[\frac{A_g}{A_c}-1\right]\frac{f_{ck}}{f_y}$$

Now $A_g = \frac{\pi}{4}\times400^2 = 125663.7\ mm^2$

$$\therefore \frac{46110}{68172.63\times S} \nless 0.36\left[\frac{125663.7}{68172.63}-1\right]\times\frac{30}{415}$$

$\therefore S \geq 30.81$ mm

Max pitch specified = 75mm or $\frac{\text{core diameter}}{6}$

$= 75$ mm or $\frac{300}{6} = 50$mm

Min pitch = 25mm or 3 x dia of helical reinforcement
= 24mm
ie 24 mm
∴ Provide 8mm spirals at 40mm pitch.

10 $P = 850$ kN. col = 230mm x 300 mm
Self wt = ~~200~~ 85 kN
Total load = 935 kN.
SBC = 150 kN/m^2

$A = \frac{935}{150} = 6.233$ m^2

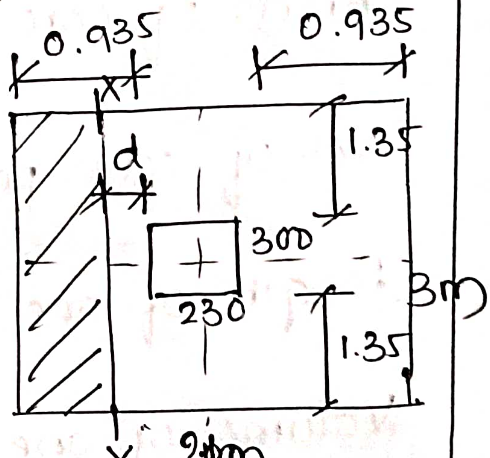
Provide 2.1m x 3.0m footing
∴ Area provided = 2.1 x 3 = 6.3 m^2.

Soil pressure = $q_u = 1.5 \times \frac{935}{6.3} = 222.62$ kN/m^2
= 0.222 N/mm^2

Cantilever projection

$l = \frac{2.1 - 0.23}{2} = 0.935$ m

$l = \frac{3.0 - 0.3}{2} = 1.35$ m



Critical section for one way shear is as shown in the fig

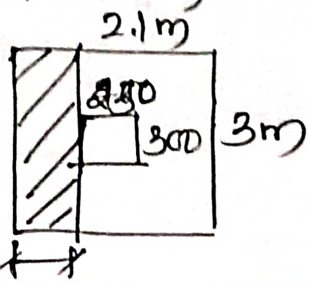
Total shear across x-x.
$V_u = q_u \times$ shaded area
$= 0.222 \times 3000 \times (935 - d)$

Assuming $P_t = 0.20$, τ_c for M20 concrete = 0.32 N/mm^2

Let depth of footing bed. Then.
$0.222 \times 3000 (935 - d) = 0.32 \times 3000 \times d$.
$622710 - 666\ d = 960\ d$.
$d = 382.97$ mm

provide $d = 400$ mm and $D = 475$ mm

Check for bending -



Mu for 3000mm width is given by

$M_u = 0.222 \times 3000 \times 935 \times \frac{935}{2}$
$= 291.11 \times 10^6$ Nmm

Q.No.	Solution and Scheme	Marks

$M_{ulim} = 0.36 \times 20 \times 3000 \times 0.48 \times 400 (400 - 0.42 \times 0.48 \times 400)$

$= 1324.44 \times 10^6 > M_u$.

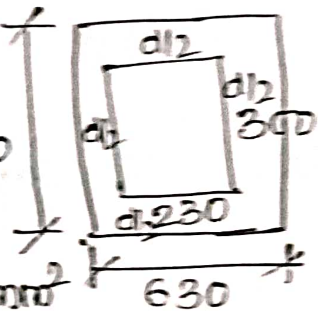
$\therefore$ Depth selected is sufficient.

Check for depth from the consideration of two way shear –

The critical section is at a distance of $\frac{d}{2}$ $= \frac{400}{2} = 200$ from the face of the column.

Perimeter of the section

$= 2[630 + 700] = 2660\,mm$



Area of concrete resisting two way shear

$= 2660 \times 400 = 1064000\,mm^2$

Punching shear.

$V_u = 0.222[3000 \times 2100 - 630 \times 700]$

$V_u = 1300698\,N$.

Equating it to resisting shear,

$1300698 = 1064000\,\tau_c'$

$\tau_c' = 1.22\,N/mm^2$

Two way shear permitted $= k_s \tau_c$

where $k_s = 0.5 + \beta_c$ but not greater than 1.

$\beta_c = \frac{2.1}{3} = \cancel{0.525}\ 0.7$

$\therefore k_s = 0.5 + 0.7 = 1.2$

$\therefore \tau_c = 0.25\sqrt{f_{ck}} = 0.25\sqrt{20} = 1.118\,N/mm^2$

But actual $\tau_c = k_s \tau_c = 1.464\,N/mm^2$

$\therefore$ Footing is safe in two way shear.

Reinforcement –

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

$291.11 \times 10^6 = 0.87 \times 415 \times A_{st} \times 400 \left[1 - \frac{A_{st} \times 415}{3000 \times 400 \times 20}\right]$

$= 144420\,A_{st}\,[1 - 1.729 \times 10^{-5} A_{st}]$

$= 144420\,A_{st} - 2.49\,A_{st}^2$

$\Rightarrow 2.49\,A_{st}^2 - 144420\,A_{st} + 291.11 \times 10^6 = 0$

$A_{st} = 2091.11 \ mm^2$

Using 12mm bars.

$$S = \frac{\pi/4 \times 12^2}{2091.11} \times 3000 = 162.25.$$

Provide 16mm bars @ 170mm c/c in both directions

Hence ok.

Check for development length -

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

τ_{bd} for M20 and Fe 415 steel is $1.92 \ N/mm^2$.

$$L_d = \frac{0.87 \times 415 \times 12}{4 \times 1.92} = 564.14 < 1350 \ mm$$
$$< 935 \ mm$$

Hence development length is available.

Boni
(Prof. Parvati Oui)

HOD

Dean 23/3/22
(Academics)