CBCS SCHEME

USN | | | | | | | | | | 21CV53

Fifth Semester B.E. Degree Examination, June/July 2024
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. Use of IS456-2000, SP-16 is permitted.
3. Assume any missing data suitably.

Module-1

1 a. Explain the difference between working stress and limit state method of design. **(08 Marks)**

b. Write brief short notes on:
i) Partial safety factor for loads
ii) Partial safety factor for material
iii) Characteristic load
iv) Characteristic strength. **(12 Marks)**

OR

2 a. Explain the terms balanced section, under reinforced and over reinforced section. **(06 Marks)**

b. A simply supported beam of rectangular cross section 250mm × 450mm overall depth is used over an effective span of 4.0m. The beam is reinforced with 3 bars of 20mm diameter, Fe415 HYSD bars at an effective depth of 400mm. Two anchor bars of 10mm diameter bars are provided. The self weight of beam together with the dead load on the beam is 4kN/m service load on the beam is 10kN/m. Use M20 grade of concrete. Compute: i) Short term deflection ii) Long term deflection. **(14 Marks)**

Module-2

3 a. Derive from fundamentals the expression for the area of stress block $0.36 f_{ck} x_u$. **(08 Marks)**

b. A rectangular R.C. beam section 230mm × 500mm is used as a simply supported beam for an effective span of 6.0m. The beam consists of tensile reinforcement of 4000mm^2 and centre of the reinforcement is placed at 35mm from bottom edge. What total maximum udl can be allowed on the beam? Use M20 concrete and Fe415 grade steel. **(12 Marks)**

OR

4 a. Derive the moment of resistance equation for a singly reinforced rectangular section. **(10 Marks)**

b. A RCC beam 250mm wide and 450mm deep is reinforced with 3nos of 20mm diameter bars of grade Fe-415 on the tension side with an effective cover of 50mm. If the shear reinforcement of 2 legged 8mm vertical stirrups at a spacing of 160mm centre to centre is provide at a section. Determine the design shear strength of the section. Assume M-20 concrete has been used. **(10 Marks)**

Module-3

5 Design a reinforced concrete beam of rectangular section using the following data.
Effective span = 5m, width of beam = 250mm, overall depth = 500mm service load including dead load and live load = 40kN/m, effective cover = 50mm. Adopt M20 concrete and Fe-415 grade steel. Sketch the reinforcement details. **(20 Marks)**

OR

6 A T-beam slab floor has 125mm thick slab forming a part of tee beam. Which are 8.0m clear span? The end bearings are 450mm thick. Spacing of T beam is 3.5m c/c. The live load on the floor is $3kN/m^2$. Design one of the intermediate T beams. Use M-20 concrete and Fe-415 grade steel. **(20 Marks)**

Module-4

7 Design a reinforced concrete slab for a room of size 3m × 5m (clear). The slab is supported on a wall of 300mm thickness with corners held down. Two adjacent edges of the slab are continuous and other two edges discontinuous. The live load on slab is $3kN/m^2$. Assume floor finish of $1kN/m^2$. Use M20 grade concrete and Fe-415 grade steel sketch the reinforcement details. **(20 Marks)**

OR

8 Design a dog legged stair for an office building in a room measuring 2.8m × 5.8m clear vertical distance between the floor is 3.6m. The width of flight is to be 1.25m. Assume live load of $3kN/m^2$. Use M-20 concrete and Fe-415 grade steel. Assume the stairs are supported on 230mm at the outer edges of landing stairs. Sketch the reinforcement details. **(20 Marks)**

Module-5

9 a. Design a short column of size 450mm × 600mm. The column is subjected to a factored load of 3000kN. Use M20 concrete and Fe415 grade steel. **(10 Marks)**

b. Design the reinforcement for a column of size 300mm × 400mm having an effective length of 2.5m. Moment about the major axis of the column is 100kN-m and axial load of 800kN. Use M25 concrete and Fe500 steel provide the reinforcement on two sides. **(10 Marks)**

OR

10 Design a square footing to carry a column load of 1200kN from a 400mm square column. The bearing capacity of soil is $120kN/m^2$. Use M20 concrete and Fe-415 grade steel. Sketch the reinforcement details. **(20 Marks)**

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Subject Title : Design of R.C. Structural Element   Subject Code : 21CV53

Question Number	Solution	Marks Allocated
1.a.	Explain the difference between working stress and limit state method of design.	

Sl. NO	Working Stress Method	Limit State Method
1.	This method is based on the elastic theory which assumes that concrete and steel are elastic and the stress strain curve is linear for both.	This method is based on the actual stress - strain curves of steel and concrete. For concrete, the stress strain curve is non - linear.
2.	In this method the factor of safety are applied to the yield stresses to get permissible stresses.	In this method, Partial safety factors are applied to get design values of stresses.
3.	No factor of safety is not known.	Design loads are obtained by multiplying partial safety factors of load to the working loads.
4.	Exact margin of safety is not known.	Exact margin of safety is known.
5	This method of safety gives thicker sections, so less economical.	This method is more economical as it gives thinner sections.
6.	This method assumes that the actual loads, permissible stresses and factors of safety are known. So it is called as deterministic method.	This method is based upon the probabilistic apporach which depends upon the actual data or experience, hence it is called as non-deterministic method.

2×4 = 8 marks

1. b. Write brief short notes on:

i) Partial safety factor for loads.

ii) Partial safety factor for materials.

iii) Characteristic load.

iv) Characteristic strength.

i) Partial Safety factor for loads.

In structural engineering, partial safety factors of loads (γ_f) are used to account for uncertainties in the actual loads acting on a structure. These factors increase the design loads to ensure a sufficient margin of safety.

To consider potential variations in load magnitudes due to Statistical fluctuations [eg. wind gusts, live loads].

Inaccuracies in load estimation.

Unexpected events.

[Partial Safety Factor for loads = $\frac{\text{Ultimate load}}{\text{Design load}}$.

ii) Partial Safety factor for materials.

These factors are used in structural design to account for uncertainties in material properties.

To ensure a sufficient margin of safety by reducing the nominal material strength to a design value.

Applied to characteristic material strengths [eg. yield strength of steel, Compressive strength of concrete] to obtain design strengths. They are typically greater than 1.0. Partial Safety Factor = $\frac{\text{Ultimate strength}}{\text{Actual working strength}}$.

iii) Characteristic Load.

The characteristic load is the value of a load for which there is a 95% probability of not being exceeded during the life of the Structure.

To ensure safety, design loads are typically obtained by multiplying the characteristic loads by suitable parital safety factors. These factors account for uncertainties in load estimation, material properties and construction practices.

iv) Characteristic strength

This refers to the strength level below which a specified proportion (usually 5%) of all valid test results are expected to fail.

Essentially, its a statistically determined value that gives you a high degree of confidence (95%) that the actual strength of the concrete will be at least that high.

Its crucial for designing structures as it allows engineers to account for the inherent variability in concrete production and strength.

2. a. Explain the terms balanced section, under reinforced and over reinforced section.

i) Balanced Section

In balanced section, the strain in steel and strain in concrete reach their maximum values simultaneously. The percentage of steel in this section is known as critical or limiting steel percentage. The depth of neutral Axis (NA) is $x_u = x_{umax}$.

ii) Under Reinforced Section

An under reinforced section is the one in which steel percentage is less than critical or limiting percentage ($p_{t.lim}$). Due to this the actual NA is above the balanced NA and $x_u <$ x_{umax}. 3 X 2 = 6 m

iii) Over Reinforced Section

In the over reinforced section the percentage of is more than limiting percentage due to which NA falls below the balanced NA and $x_u > x_{umax}$. Because of higher percentage of steel yield does not take place in steel and failures occurs when the strain in extreme fibres in concrete reaches its ultimate value.

2b A simply supported beam of rectangular cross section 250mm x 450mm overall depth is used over an effective span of 4.0m. The beam is reinforced with 3 bars of 20mm diameter, Fe415 HYSD bars at an effective depth of 400mm. Two anchor bars of 10mm diameter bars are provided. The self weight of beam together with the dead load on the beam is 4kN/m Service load on the beam is 10 kN/m. Use M20 grade of concrete. Compute : i) Short term deflection ii) Long term deflection.

$E_s = 2.1 \times 10^5 \, N/mm^2$, $E_c = 5000\sqrt{2} = 22,360.68 \, N/mm^2$.

$m = \frac{280}{3} \times 7 = 13.33$. $f_{cr} = 0.7\sqrt{20} = 3.13 \, N/mm^2$

$A_{sc} = 157.14 mm^2$ $A_{st} = 942.85 mm^2$.

i) <u>Short Term Deflection</u>

Depth of NA $\frac{Dx^2}{2} = m \cdot A_{st} (d - x)$.

$125b = 13.33 \times 942.85(420 - x)$.

$x = 156.47 mm$ 01m

$I_{cr} = \frac{250 \times 156.47^3}{3} + 13.33 \times 942.65 (400 - 156.47)^2$.

$= 10.65 \times 10^8 mm^4$ 01M.

$I_{gr} = \frac{250 \times 450^3}{12} = 18.98 \times 10^8 mm^4$ 01M.

$M = \frac{wl^2}{8} = \frac{14 \times 4^2}{8} = 28$ kN-m or 28×10^6 N-mm 01M.

$M_r = \frac{f_{cr} I_{gr}}{y_r} = \frac{3.13 \times 18.98 \times 10^6}{225} = 26 \times 10^6$ N/mm 01m.

$Z = d - \frac{x}{3} = 400 - \frac{156.47}{3} = 348.34 mm$ 01M.

$I_{eff} = \frac{I_r}{1.2 - \left[\left(\frac{M_r}{M}\right)\left(\frac{z}{d}\right)\left(1 - \frac{x}{d}\right)\left(\frac{b_w}{b}\right)\right]} = \frac{10.45 \times 10^8}{1.2 - \left[\left(\frac{26 \times 10^8}{28 \times 10^6}\right)\left(\frac{348.34}{400}\right)\left(1 - \frac{156.47}{400}\right) \cdot 1\right]}$

$I_{eff} = 14.83 \times 10^8 mm^4$. $I_r < I_{eff} < I_{gr}$ 01M.

Max. short term Deflection $= \frac{5}{384} \cdot \frac{wl^4}{E_c I_{eff}}$.

$= \frac{5}{384} \times \frac{14 \times 4000^4}{(22360.68 \times 14.93 \times 10^8)} = 1.39 mm$ 01M.

Long Term Deflection

a) Deflection due to Shrinkage

$a_{cs} = k_3 \psi_{cs} L^2$ → $k_3 = 0.125$ for ss beam, $\psi_{cs} = k_4 \left(\frac{\varepsilon_{cs}}{D}\right)$

$\varepsilon_{cs} = 0.0003$; $P_t = \frac{100 \times 300\pi}{250 \times 400} = 0.942\%$, $P_c = \frac{100 \times 50\pi}{250 \times 400} = 0.15\%$

$P_t - P_c = 0.785 > 0.25 < 1$ Ok and $k_4 = 0.72 \times \frac{0.785}{\sqrt{0.942}} = 0.58$

$\psi_{cs} = \frac{0.58 \times 0.0003}{450} = 3.866 \times 10^{-7}$

$a_{cs} = k_3 \psi_{cs} L^2 = 0.125 \times 3.866 \times 10^{-7} \times 4000^2 = 0.773mm$ 03M

b) Creep deflection [$a_{cc\,per} = a_{icc\,per} - a_{i\,per}$.

$a_{icc\,per} = k_w \left(\frac{wl^3}{E_{ce} \times I_{eff}}\right)$ $E_{ce} = \frac{E_c}{1+\theta}$. θ - creep coeff 1.6 for 28 day

$E_{ce} = \frac{E_c}{1+6}$.

$a_{icc\,per} = 2.6 \times$ short term deflection $= 2.6 \times a_{i\,per}$

$= 2.6 \times 1.39 = 3.614$ mm.

Creep deflection $a_{cc\,per} = 3.614 - 1.39 = 2.224$ mm.

Total Long Deflection = Shrinkage deflection + Creep deflection

$= 0.773 + 2.224 = 3.013$ mm

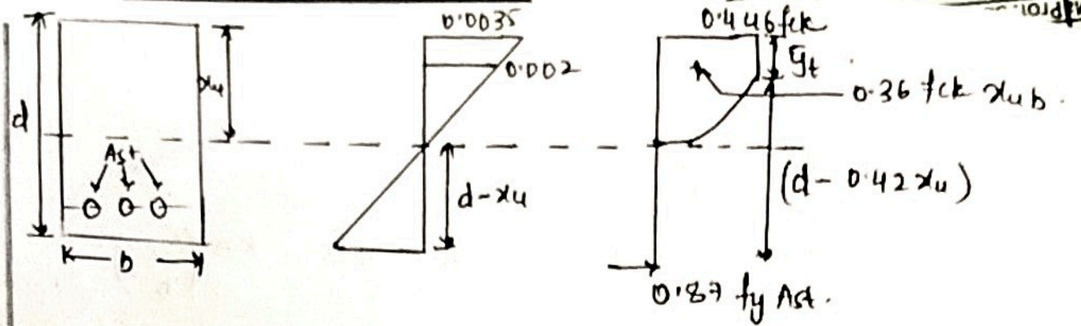
Total Deflection = short term + Long term

$= 1.39 + 3.013 = 4.402$ mm 03M

14M.

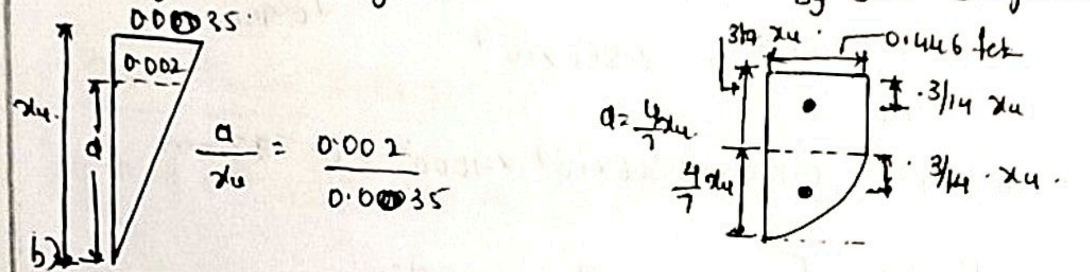
Module - 2.

3. a. Derive from fundamentals the expression for the area of stress block $0.36 f_{ck} x_u$.



C/S Strain Diagram Stress Diagram.

Now by Strain Diagram By Stress Diagram



$$\frac{a}{x_u} = \frac{0.002}{0.0035}$$

03M.

Area of Stress block $= \square + D = \frac{3}{7} x_u \times 0.446 \times f_{ck} + \frac{2}{3} \times \frac{4}{7} x_u \times 0.446 f_{ck}$ 02M.

$= 0.36\, f_{ck}\, x_u$

08M.

3.b. A rectangular R.C. beam section 230mm × 500mm is used as a simply supported beam for an effective span of 6.0m. The beam consists of tensile reinforcement of 4000mm² and centre of the reinforcement is placed at 35mm from bottom edge. What total maximum udl can be allowed on the beam? Use M20 concrete and Fe415 grade steel.

Step 1) $d = D - d' = (500 - 35) = 465\,mm$

$$\frac{x_u}{d} = \frac{0.87 f_y A_{st}}{0.36 f_{ck} bd} \Rightarrow \frac{0.87 \times 415 \times 4000}{0.36 \times 20 \times 230 \times 465} = 1.87.$$ 4×3 =12M

Step 2) Limiting value of $\frac{x_{umax}}{d} = 0.48$ for $f_y = 415\,N/mm^2$

$\frac{x_u}{d} > \frac{x_{umax}}{d}$ ∴ Hence Section is overreinforced.

Step 3) Ultimate moment of Resistance. $M_u = 0.138 f_{ck} bd^2$ or

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

$= 0.36 \times 20 \times 230 \times 465^2 \times 0.48(1 - 0.42 \times 0.48)$.

$= 137.22 \times 10^6$ N-mm or 137.22 KN-m.

Unfactored

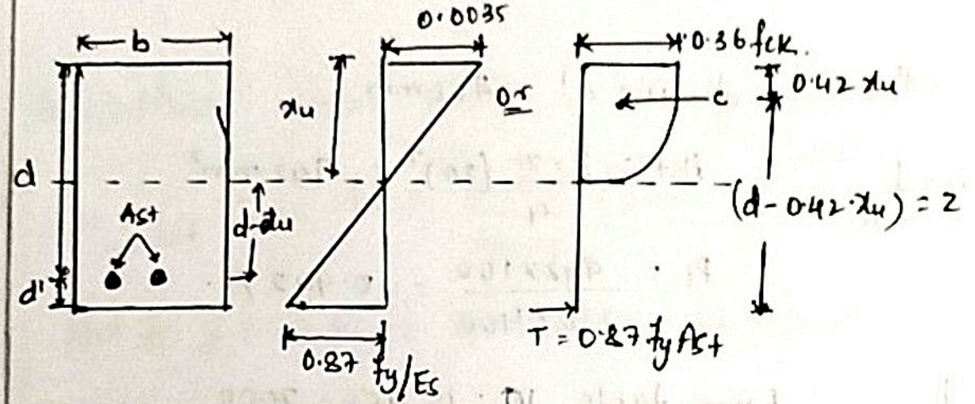
step 4) Max. Udl

4.a

Unfactord load moment : $\frac{M_u}{1.5} = 91.48$ kN-m.

step 4) Max. Udl on beam = $\frac{wl^2}{8} = 91.48$ or $w = \frac{8 \times 91.48}{6^2} = 20.33$ kN/m.

4. a. Derive the moment of resistance equation for a singly reinforced rectangular section.



0.2 M

$C = T = 0.36 f_{ck} x_u b = 0.87 f_y A_{st}$

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

0.2 M

i) For Under reinforced failure $x_u < x_{umax}$.

then $M_u = T \times z$ (lever Arm) $= 0.87 f_y A_{st} \left[d - 0.42 \left\{ \frac{0.87 f_y A_{st}}{0.36 f_{ck} b} \right\}\right]$

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

0.3 M

ii) For Moment of Resistance for balanced section $x_u = x_{ulim}$.

$M_{ulim} = C \times z = 0.36 f_{ck} x_u b (d - 0.42 x_u)$.

Replace x_u by x_{ulim} or x_{umax}.

$\therefore M_{ulim} = 0.36 f_{ck} b d^2 \left[\frac{x_{ulimax}}{d} \left(1 - 0.42 \frac{x_{umax}}{d}\right)\right]$ — ②.

As can be calculated by $\left[\frac{0.36 f_{ck} b x_{umax}}{0.87 f_y}\right]$

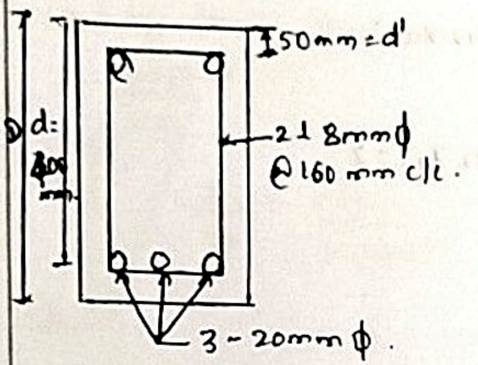
iii) $x_u > x_{umax}$ Over Reinforced section

$M_{ulim} = 0.36 \left(\frac{x_{umax}}{d}\right) \times \left[1 - 0.42 \times \frac{x_{umax}}{d}\right] f_{ck} b d^2$ — ③.

0.3 M

08 M

4.b. A RCC beam 250mm wide and 450mm deep is reinforced with of 20mm diameter bars of grade Fe415 on the tension side an effective cover of 50mm. If the shear reinforcement of 2legged 8mm vertical stirrups at a spacing of 160mm centre to centre. is provide at a section. Determine the design shear strength of the section. Assume M20 concrete has been used.



$d = D - d' = 400mm$. — 01M.

$A_{st} = 3 \times \frac{\pi}{4}(20)^2 = 942\ mm^2$ — 01M

$P_t = \frac{942 \times 100}{250 \times 400} = 0.942\%$. — 01M

From table 19 · IS 456 - 2000

$\tau_c = 0.61\ N/mm^2$ — 01M

$V_{uc} = 0.61 \times 250 \times 400 = 61000 N$ or $61 KN$ — 01M

Strength of shear reinforcement $V_{us} = \frac{0.87 \times 415 (2 \times \pi/4 \times 8^2) \times 400}{160}$

$= 90,742 N$ or $90.742\ KN$ — 01M

$V_u = V_{us} + V_{uc} = 90.742 + 61 = 151.742\ KN$ — 01M.

From table 20, IS 456 - 2000, $\tau_{cmax} = 2.8\ N/mm^2$ — 01M.

Upper limit of strength $V_{max} = \tau_{cmax} \times bd = 2.8 \times 250 \times 400$

$= 280\ KN > V_u$ — 01M.

The design strength of section for shear $= 151.742\ KN$. — 01M

10 M.

Module - 3

5. Design a reinforced concrete beam of rectangular section using the following data.
Effective span = 5m, width of beam = 250mm, overall depth = 500mm
service load, including dead load and live load = 40 kN/m effective cover = 50mm. Adop M20 concrete and Fe415 grade steel.
Sketch the reinforcement details.

i) $M_u = \frac{W_u l^2}{8} = \frac{1.5 \times 40 \times 5^2}{8} = 187.5$ kN-m , $V_u = \frac{1.5 \times 40 \times 5}{2} = 150$ kN 02M.

ii) $M_{ulim} = 0.36 f_{ck} bd^2 \cdot \frac{x_{umax}}{d}\left(1 - 0.42\frac{x_{umax}}{d}\right)$.

$= 0.36 \times 20 \times 250 \times 450^2 \times 0.48(1 - 0.42 \times 0.48)$

$= 140$ kN-m

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

$d = 500 - 50 = 450$ mm. 02M.

$M_u > M_{ulim}$ Hence Design a doubly reinforced section.

$f_{sc} = E_{sc} \cdot E_s$, $f_{sc} = \frac{0.0035}{x_{umax}}(x_{umax} - d') \times 2 \times 10^5 = 538$ N/mm² 01M.

But $f_{sc} \not> 0.87 f_y \Rightarrow 0.87 \times 415 = 361$ N/mm² 01M.

$A_{sc} = \frac{M_u - M_{ulim}}{f_{sc}(d - d')}$ or $\frac{187.5 - 140}{361(450 - 50)} = 329$ mm² 01M

Provide 2, 16mm ϕ ; A_{sc} provided = 402 mm²

$A_{st2} = \frac{f_{sc} A_{sc}}{0.87 f_y} \Rightarrow \frac{361 \times 329}{0.87 \times 415} = 329$ mm² 01M

$A_{st1} = \frac{0.36 f_{ck}\, b\, x_{umax}}{0.87 f_y} \Rightarrow \frac{0.36 \times 20 \times 250\,(0.48 \times 450)}{0.87 \times 415} = 1077$ mm² 01M.

$A_{st} = A_{st1} + A_{st2} = 329 + 1077 = 1406$ mm².

Provide 3, 25mm ϕ . A_{st} provided = 1473 mm² 01M.

Shear reinforcement : $\tau_v = \frac{V_u}{bd} = \frac{150 \times 1000}{250 \times 450} = 1.33$ N/mm² 01M

$P_t = \frac{100 \times 1473}{250 \times 450} = 1.3\%$ 01M

table 19 IS 456 - 2000, $\tau_c = 0.68$ N/mm², 01M.

table 20 IS 456 - 2000, $\tau_{cmax} = 2.8$ N/mm² for M20.

$\therefore \tau_c < \tau_v < \tau_{cmax}$, hence shear reinforcements are required,

$V_{us} = V_u - \tau_c bd = 150 \times 10^3 - 0.68 \times 250 \times 450 = 73.5 \times 10^3$ N or 73.5 kN 01M.

Using 8mm ϕ, 2L stirrups @ spacing of

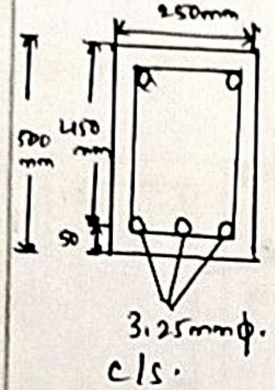
$S_v = \frac{0.87 f_y A_{sv} d}{V_{us}} = \frac{0.87 \times 415 (2 \times \pi/4 \times 8^2) \times 450}{73.5 \times 10^3} = 221$ mm

Adopt a spacing of 200mm c/c

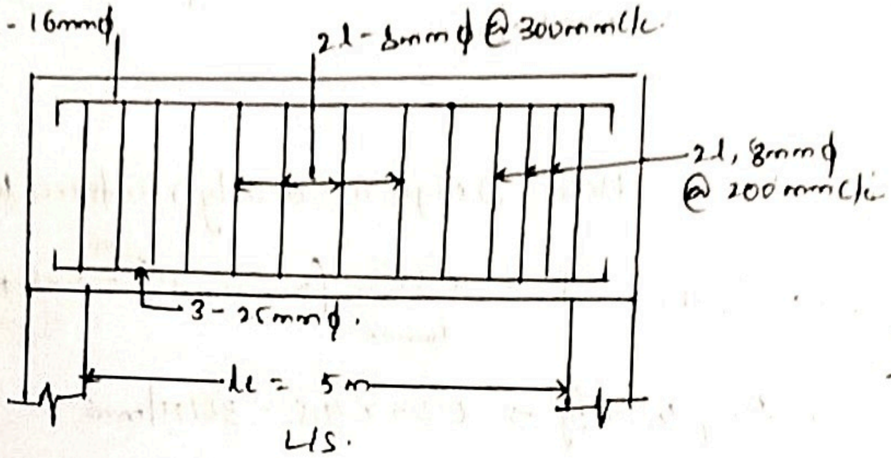
near support and 300mm towards centre of span.

Max. spacing = $0.75d = 0.75 \times 450 = 337.5$

Provide is Ok and satisfactory



C/S.



L/S.

02+02

20M

6. A T-beam slab floor has 125mm thick slab forming a part of the beam. Which are 8.0m. clear span? The end bearings are 450mm thick. Spacing of T-beam is 3.5m c/c. The live load on the floor is $3 kN/m^2$. Design one of the intermediate T-beams. Use M_{20} concrete and Fe_{415} grade steel.

Dimension of beam.

Overall depth $= \frac{1}{12}$ to $\frac{1}{15}$ of span $= \frac{8000}{12}$ to $\frac{8000}{15} = 666.6$mm to 533.33mm.

Provide $D = 650$mm, and $d = 600$mm

width of web b_w: $\frac{1}{2}$ to $\frac{1}{3}$ of d. ⇒ $b_w = 250$mm 02M

Effective span: least of clear span + eff. depth $= 8 + 0.6 = 8.6$m

clear span + end bearing $= 8 + 0.45 = 8.45$m 01M.

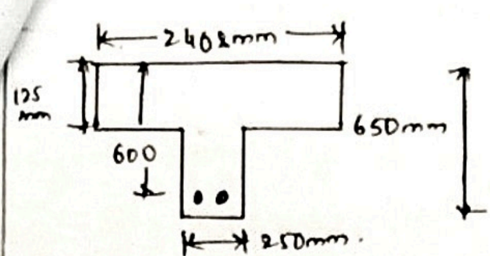
01 Provide $l_e = 8.45$m

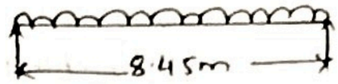
Flange width: (a) $b_f = \frac{l_0}{6} + b_w + 6D_f$

$l_0 = 8$m

$= \frac{8000}{6} + 230 + 6 \times 125 = 2408$mm

or $b_f = \frac{L_1 + L_2}{2} + b_w = \left[\left(\frac{3.5 - 0.23}{2}\right) + \left(\frac{3.5 - 0.23}{2}\right)\right] + 0.23 = 3.5$m





02M.

Maximum BM and SF

Load: self weight of slab = $0.125 \times 1 \times 1 \times 25 = 3.125$

weight of floor finish $= 0.60$

Live load $= 3.00$

Total load $= 6.725\ kN/m^2$

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

self weight of rib $= (0.65 - 0.125)\ 0.25 \times 25 = 3.28$

wt. of plaster $= 0.50$

Total load on beam $= 27.30\ kN/m$

Factored load $= W_u = 1.5 \times 27.30 = 40.95\ kN/m$ 02M.

$M_u = \frac{W_u l_e^2}{8} = \frac{40.95 \times 8.45^2}{8} = 365.5\ kN/m$ 01M

$V_u = \frac{W_u l_e}{2} = \frac{40.95 \times 8.45}{8} = 173\ KN$ 01M.

$M_{ulim} = 0.446 f_{ck} b_f D_f (d - \frac{d_f}{2}) = 0.446 \times 20 \times 2408 \times 125 (600 - 125/2)$

$M_{ulim} = 1443.145 \times 10^3\ N\text{-}mm$ or $1443.145\ KN.m$ 01M.

$M_u < M_{ulim}$. Hence can be designed as singly reinforced section.

Assuming NA coincides with flange ie $= x_u = D_f = 125\ mm$

$M_u' = 0.36 f_{ck} b_f \cdot D_f (d - 0.42 D_f) = 0.36 \times 20 \times 2408 \times 125 (600 - 0.42 \times 125)$ 01M.

$M_u' = 1186.542 \times 10^6\ N\text{-}mm$

$M_u < M_u'$ ie NA is within flange ~~than~~ MR.

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

$365.5 \times 10^6 = 0.87 \times 415 \times A_{st} \times 600 \left[1 - \frac{A_{st} \times 415}{2408 \times 600 \times 20}\right]$.

$A_{st}^2 - 69628.9 A_{st} + 1687.21 \times 69{,}628.9 = 0$.

By solving $A_{st} = 1730\ mm^2$, Provide 6 Nos. 20mm ϕ 02M

Ast Provided = 1885 mm² $P_t = \frac{1885 \times 100}{250 \times 600} = 1.257\%$

Table 19, IS 456 · $\tau_c = 0.67$ N/mm²

$\tau_v = \frac{V_u}{bd} = \frac{173 \times 1000}{250 \times 600} = 1.153$ N/mm²

Table 20, IS 456 · $\tau_{cmax} = 2.6$ N/mm², $\tau_v < \tau_c < \tau_{cmax}$

Shear reinforcement are designed,

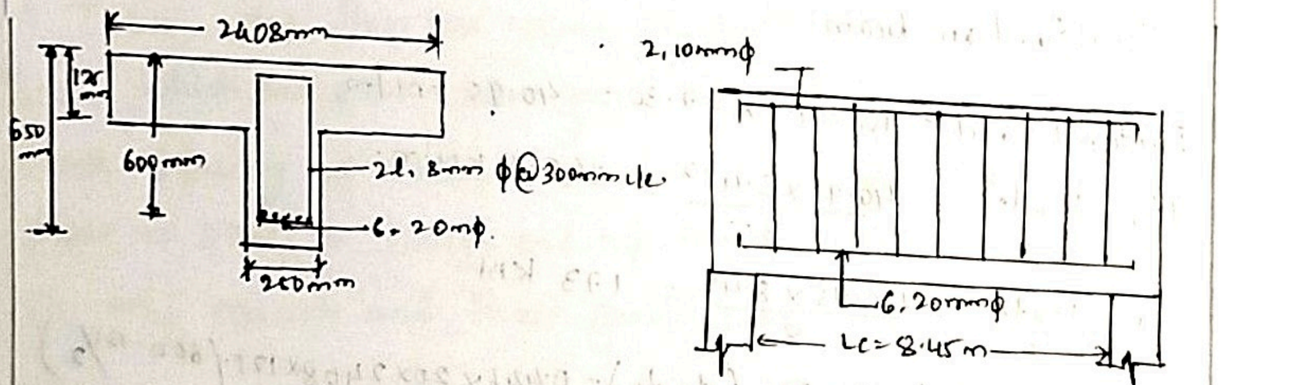
$V_{us} = V_u - \tau_c bd = 173000 - 0.67 \times 250 \times 600 = 72.508$ KN 01M.

Using 2L, 8mm ϕ, Fe415 steel.

$V_{us} = \frac{0.87 f_y A_{sv} d}{S_v}$; $S_v = \frac{0.87 \times 415 \times 2 \times \pi/4 \times 8^2 \times 600}{72500} = 300.39$ mm 02M

Max. spacing = 0.75d = 0.75 × 600 = 450mm.

Hence provide 2L - 8mm vertical stirrups @ 300mm c/c



Module - 4.

7 Design a reinforced concrete slab for a room of size 3m × 5m (clear). The slab is supported on a wall of 300mm thickness with corners held down. Two adjacent edges of the slab are continuous and other two edges discontinuous. The live load on slab is 3kN/m². Assume floor finish of 1kN/m². Use M30 grade concrete and Fe415 grade steel sketch the reinforcement details.

—03M

$\frac{l_y}{l_x} < 2$. It will be two way slab.

Thickness of slab: $l_x < 3.5$m and load is 3kN/m²

From deflection criteria depth of slab required ·

$\frac{lx}{d} = 32$. Hence $d = \frac{3000}{32}$ provided = 100mm   01M.

$D = 125mm$

Effective Span: (eff) As thickness of wall is more than depth of slab

$lx = 3000 + 100 = 3100mm$ or 3.1m.

$ly = 5000 + 100 = 5100mm$ or 5.1m

$\frac{ly}{lx} = \frac{5.1}{3.1} = 1.65$ 01M

BM and SF

Self wt of slab = $0.125 \times 1 \times 1 \times 25$ = 3.125 KN/m².

FF = 1.00 KN/m²

L.L = 3.00 KN/m²

Total load = 7.125 KN/m²

Factored load $W_u = 1.5 \times 7.125 = 10.69$ KN/m²   01M.

Table-4 IS 456:2000 Case 4

In Short span direction : Coefficient for -ve moment

$= 0.075 + 15/25 (0.084 - 0.075) = 0.0804$

+ve moment $= 0.056 + 15/25 (0.063 - 0.056) = 0.0602$.

For long span : Coefficient for -ve moment = 0.047.

Coefficient for +ve moment = 0.035.

Design -ve moment for short span $= 0.0804 \times 10.69 \times 3.1^2 = 8.26$ KNm

-ve moment for long span $= 0.047 \times 10.69 \times 3.1^2 = 4.828$ KNm

+ve moment for short span $= 0.0602 \times 10.69 \times 3.1^2 = 6.18$ KNm

+ve moment for long span $= 0.035 \times 10.69 \times 3.1^2 = 3.500$ KNm   02M.

$x_{ulim} = 0.48 \times 100 = 48mm$

$M_{ulim} = 0.36 f_{ck} b x_{umax} (d - 0.42 x_{umax})$

$= 0.36 \times 20 \times 1000 \times 48 (d - 0.42 \times 48) = 27.592$ KNm $> M_u$ 01M.

∴ Singly reinforced section can be designed.

Design for -ve reinforcement.

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

$8.26 \times 10^6 = 0.87 \times 415 \times A_{st} \times 100 \times \left[1 - \frac{A_{st} \times 415}{1000 \times 100 \times 20}\right]$

$A_{st} = 240.8mm^2$. Using 8mm @ 200mm c/c   02M.

Reinforcement in long direction:

$M_u = 4.828$ kNm $\quad d = 100-8 = 92$ mm.

$$4.828\times10^6 = 0.87\times415\,A_{st}\times92\left[1-\frac{A_{st}}{1000\times92}\times\frac{415}{20}\right]$$

$A_{st} = 150.45\,mm^2 > A_{st\,min} = \frac{0.12}{100}\times1000\times125$.

Provide 8mm ϕ @ 300mm c/c

Provide the same in edge strip also. 02M

Extend 50% of tensile steel upto support and 50% steel up to $0.1l_x$ or $0.1l_y$ of the support as appropriate.

Torsional reinforcement at corner of two discontinuous edges

$A_{st} = 3/4 \times 240.8 = 180.6\,mm^2$

Use 8mm ϕ bars @ 275mm c/c 02M.

Design for +ve moment: In x direction $M_u = 6.18$ kNm

$$6.18\times10^6 = 0.87\times415\,A_{st}\times1000\left(1-\frac{A_{st}}{1000\times100}\times\frac{415}{20}\right)$$

$A_{st} = 177.7\,mm^2$.

Provide 8mm ϕ @ S = 275mm c/c 01M.

In y direction: $M_u = 3.56$ too small.

Provide 8mm ϕ @ 300mm c/c 01M.

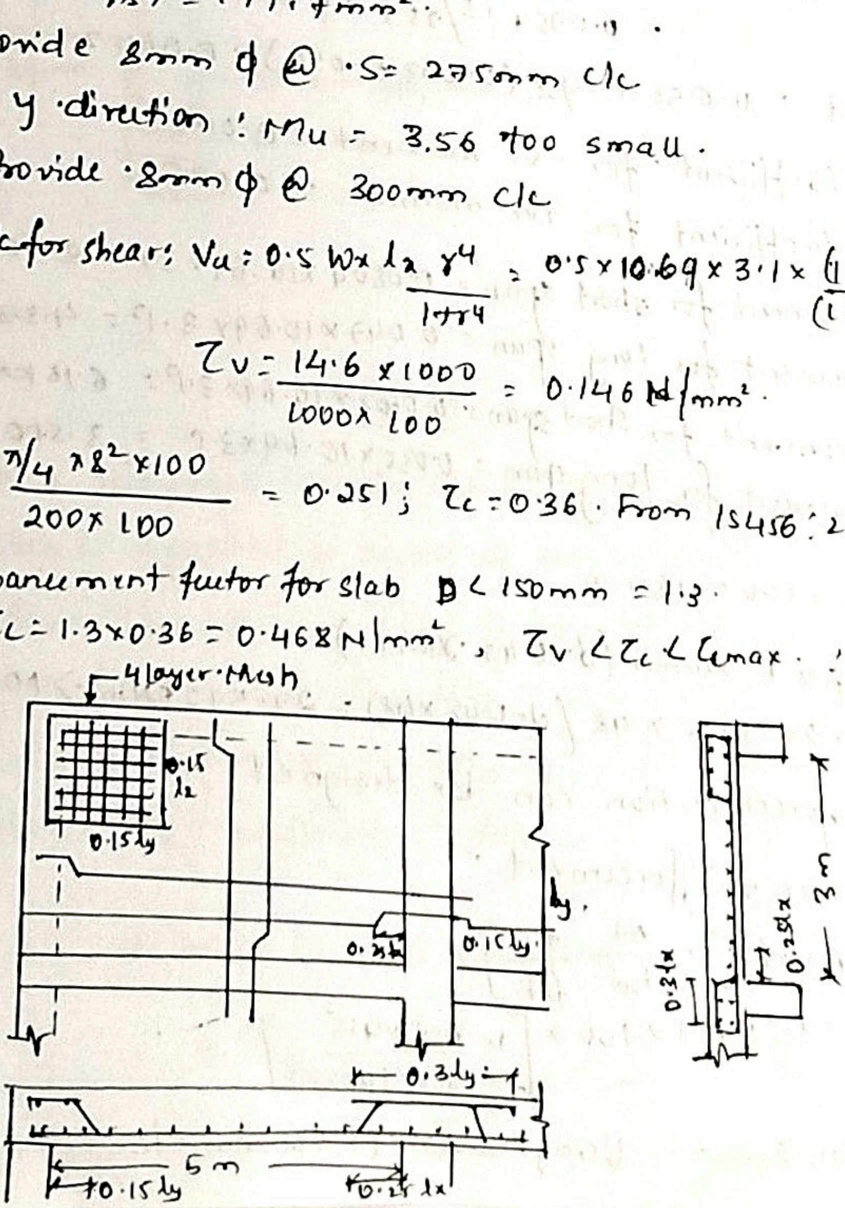
Check for shear: $V_u = \frac{0.5\,w\times l_x\,r^4}{1+r^4} = 0.5\times10.69\times3.1\times\frac{(1.65^4)}{(1+1.65^4)} = 14.6$ kN.

$$\tau_v = \frac{14.6\times1000}{1000\times100} = 0.146\,N/mm^2.$$

$P_t = \frac{\pi/4\times8^2\times100}{200\times100} = 0.251;\ \tau_c = 0.36$. From IS456:2000 table 19.

Enhancement factor for slab D < 150mm = 1.3.

$\tau_c = 1.3\times0.36 = 0.468\,N/mm^2$, $\tau_v < \tau_c < \tau_{cmax}$ ∴ Hence Safe 03M



Design a dog legged stair for an office building in a room measuring 2.8m × 5.8m clear vertical distance between the floor is 3.6m. The width of flight is to be 1.25m. Assume live load of 3 kN/m². Use M20 concrete and Fe415 grade steel. Assume the stairs are supported on 230mm at the outer edges of landing stairs. Sketch the reinforcement details.

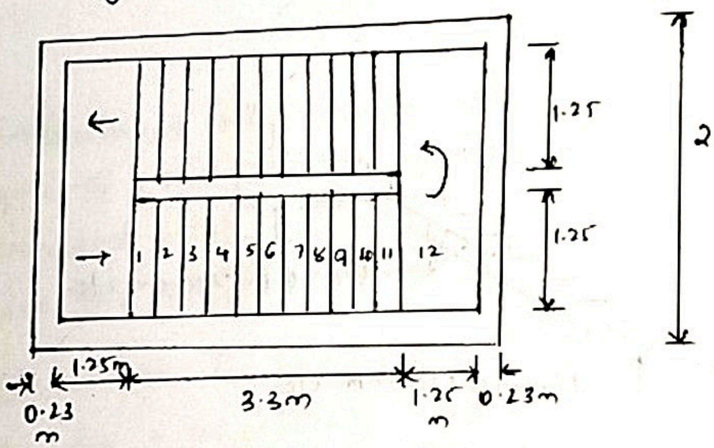
R = 150mm, F to F height = 3.6m. Height of each flight = 1.8m.

No. of Risers = 1800/150 = 12 No.

No of Treads = 12−1 = 11 Nos. 02M

width of Stair = 1.25m

Minimum Landing width = 1.25 m T = 300mm.



For threads 11 we need length = 11×0.3 = 3.3m

Eff Span: c/c distance between walls = 6.03m 02M

Loads: Thickness of waist slab = 1/20 to 1/25 = 300mm to 240mm. 01M

Use t = 250mm D = 280mm.

weight of waist slab = $0.28\sqrt{1+(150/300)^2} \times 25$ = 7.83 kN/m

weight of steps = $\frac{1}{2} \times \frac{(0.15 \times 0.25)}{0.25} \times 25$ = 1.875 kN/m

In going portion FF. Let us take = 0.795 kN/m

Total D.L = 10.5 kN/m

Landing portion D.L = 0.25×1×25 = 6.25 kN/m

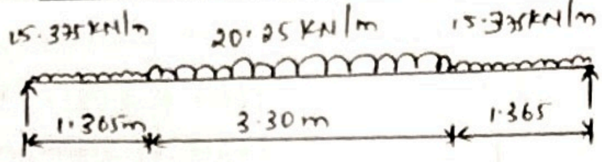
F.F = 1.00 kN/m

= 7.25 kN/m

Live load = 3.00 kN/m

Factored load on going = (10.5 + 3)×1.5 = 20.25 kN/m

Landing = (7.25+3)×1.5 = 15.375 kN/m 04M



$R_A = R_B = 54.40$ kN

$M_u = 54.40 \times \frac{6.03}{2} - 15.375 \times 1.365\left(\frac{6.03 - 1.365}{2}\right) - \frac{20.25 \times 3.3^2}{8} = 87.5 kN\text{-}m$

$M_{ulim} = 0.138 f_{ck} bd^2 = 0.138 \times 20 \times 1000 \times 250^2 = 172.5 \times 10^6$ N-mm.

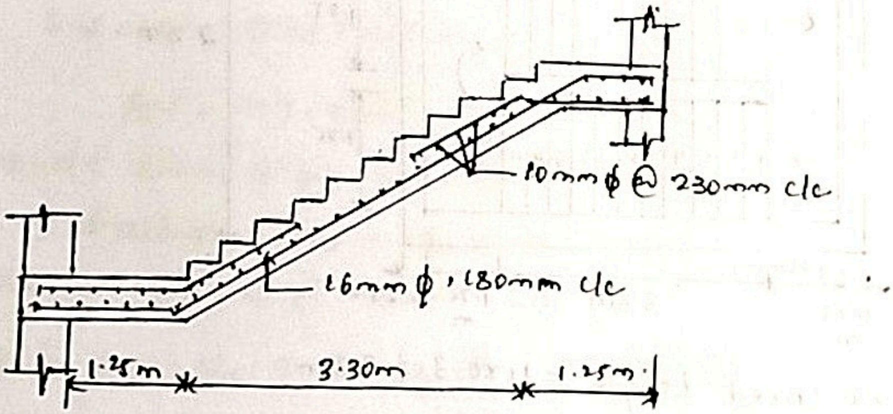
Which is greater than actual M_u 02M

Reinforcement: $87.5 \times 10^6 = 0.87 \times 415 A_{st}\left[1 - \frac{A_{st}}{1000 \times 250} \times \frac{415}{20}\right] 250$

$A_{st} = 1063 \text{ mm}^2$. Using 16mm ϕ spacing 180 mm c/c

Distribution Steel $= \frac{0.12}{100} \times 1000 \times 280 = 336 \text{mm}^2$ 03M.

Use 10mm @ 230 mm c/c



03M.

Module 5

9. a. Design a short column of size 450mm x 600mm. The column is subjected to a factored load of 3000kN. Use M20 concrete and Fe415 grade steel.

Size. 450mm x 600mm $P_u = 3000$ kN. M20, Fe415 steel, $f_{ck} = 20 N/mm^2$, $f_y = 415 N/mm^2$ 01M

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

$A_c = A_g - A_{sc} = 450 \times 600 - A_{sc} = 27 \times 10^4 - A_{sc}$.

$3000 \times 1000 = 0.4 \times 20 \times (27 \times 10^4 - A_{sc}) + 0.67 \times 415 \times A_{sc}$

$A_{sc} = 3110.54 \text{ mm}^2$ 02M.

Provide 8Nos. 25mm ϕ. A_{sc} provided $= 3926.99 \text{ mm}^2$ 01M

% of steel = $\frac{100 \times 3926.99}{450 \times 600} = 1.45\% > Asc_{min} (0.8\%) < 4\%$ 01M

Design of lateral ties : spacing Diameter greater of

(a) 6mm (b) 1/4 of 25 = 6.25 mm say 8mm.

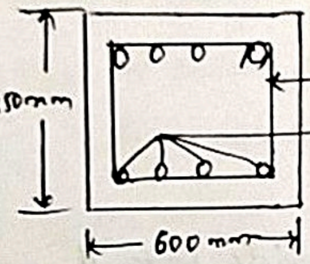
Provide 8mm ϕ lateral ties.

Spacing : (a) Least lateral diameter b = 450mm.

(b) 16× diameter of main bar = 16×25 = 400mm

(c) 300mm

Provide 8mm ϕ lateral ties at 300mm c/c 04M



01M

9b Design the reinforcement for a column of size 300mm × 400mm having an effective length of 2.5m. Moment about the major axis of the column is 100 kNm and axial load of 800 kN. Use M25 concrete and Fe500 steel provide the reinforcement on two sides.

$D = 400mm$, $b = 300mm$, $P = 800 \times 10^3 N$. $M_u = 100 \times 10^6 Nmm$

$f_y = 500 N/mm^2$, $f_{ck} = 25 N/mm^2$. $l_{eff} = 2500mm$.

step 1: Check for slenderness ratio. $\frac{l_{eff}}{D} = \frac{2500}{400} = 6.25 < 12$ 01M

$\frac{l_{eff}}{b} = \frac{2500}{300} = 8.33 < 12$, Hence it is short column.

step 2: Check for eccentricity.

$P_u = 800 \times 10^3 \times 1.5 = 1200 \times 10^3 N.$

$M_u = 100 \times 10^6 \times 1.5 = 150 \times 10^6 Nmm$

Eccentricity, $e = \frac{M_u}{P_u} = \frac{150 \times 10^6}{1200 \times 10^3} = 125mm.$

$e_{min} = \frac{l}{500} + \frac{D}{30} \Rightarrow \frac{2500}{500} + \frac{400}{30} = 18.33mm \geq 20mm.$

Hence it is designed as short column subjected to an axial force and bending. 01M.

Assume effective cover $d' = 40mm$.

Step 3: $\frac{P_u}{f_{ck} bd} = \frac{1200 \times 10^3}{25 \times 300 \times 400} = 0.40$

$\frac{M_u}{f_{ck} bd^2} = \frac{150 \times 10^6}{25 \times 300 \times 400} = 0.125$

Using SP16, chart 36 $f_y = 500 \, N/mm^2$ $d'/d = 0.10$

$P/f_{ck} = 0.07$ from chart.

Step 4: $P = 0.07 \times f_{ck} = 0.07 \times 25 = 1.75\% < 4\%$ Maximum Ok 01M.

$P = \frac{100 A_{sc}}{bD} = A_{sc} = \frac{1.75 \times 300 \times 400}{100} = 2100 \, mm^2$ 01M

Provide 4-25mm and 2, 20mm ϕ Bars

$A_{sc} = 2591.79 \, mm^2$ 01M.

Step 5: Design of lateral ties.

diameter (a) $1/4 \times 25 = 6.25mm$ (b) 6mm provide 8mm ϕ.

spacing of lateral ties.

(a) least lateral dimension b = 300mm

(b) $16 \times 20 = 320mm$ (c) 300mm.

Provide 8mm ϕ @ 300mm c/c

03M.

10. Design a square footing to carry a column load of 1200kN from a 400mm square column. The bearing capacity of soil is 120 kN/m². Use M20 concrete and Fe415 grade steel. sketch the reinforcement details.

$P = 1200kN$; $b = D = 400mm$. $SBC = 120 kN/m^2$. $f_{ck} = 20 \, N/mm^2$

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

Step ①: Load: Column Axial load = 1200kN.

Self weight of footing = $0.1 \times 1200 = 120$ kN.

Total load = 1320kN. 01M

Step ②: Area required = $\frac{1320}{120} = 11 m^2$ 01M

Step ③: For Square footing $L = B = \sqrt{11} = 3.31$ m. Provide 3.5×3.5 m.

Area of footing provided = $3.5m \times 3.5m = 12.25$ Sqm 01M.

Step ④: Net upward pressure $q = P/A = 1200/12.25 = 97.96 \, kN/m^2$.

$< 120 kN/m^2$ SBC of soil. OK 01M.

Step ⑤: Bending moment (M_u)

$M = q \times B \times \frac{1}{2}\left(\frac{L-D}{2}\right)^2 = 97.96 \times 3.5 \times 1/2\left(\frac{3.5-0.4}{2}\right)^2 = 411.86$ kNm

Factored Moment $M_u = 1.5 \times 411.86 = 617.8$ kNm — 01M.

Step ⑥: Effective depth required : $M_u = M_{ulim}$

$M_{ulim} = 0.138 f_{ck} b d_f^2$ for Fe 415 grade

$$d_f = \sqrt{\frac{617.8 \times 10^6}{0.138 \times 20 \times 3500}} = 252.89 \text{ mm}$$ — 01M.

Now increase depth of footing (d_f) 1.75 to 2 times.
due to shear reinforcement considerations.

$d_f = 1.75 \times 252.89 = 442.56$ .. say 500mm.

By assuming 50mm effective cover $d' = 50$mm.

$D_f = d_f + d' = 500 + 50 = 550$mm

Step ⑦: Area of steel $M_u = 0.87 f_y A_{st} d_f \left[1 - \frac{A_{st} f_y}{f_{ck} B D_f}\right]$.

$$617.8 \times 10^6 = 0.87 \times 415 \times A_{st} \times 500\left[1 - \frac{A_{st} \times 415}{20 \times 3500 \times 500}\right]$$

$A_{st} = 3573.63 \text{ mm}^2$.

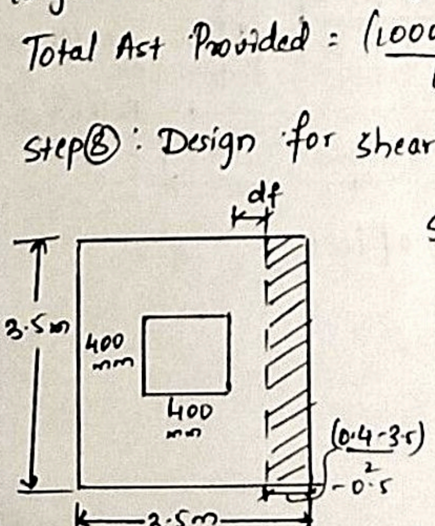
A_{st}/metre : $= \frac{3573.63}{3.5} = 1021.03 \text{ mm}^2$. — 02M.

Assume 12mm ϕ bars @ spacing $= \frac{1000 \times 113.09}{1021.03} = 110.75$mm — 01M.

Provide 12 mm ϕ bars @ 100mm c/c along shorter longer direction both ways. — 01M.

Total A_{st} Provided $= \frac{(1000 \times 113.09)}{100} \times 3.5 = 3958.15 \text{ mm}^2$

Step ⑧: Design for shear one way.



SF at critical section

$V = q \times B \times \left[\left(\frac{L-D}{2}\right) - d_f\right]$

$= 97.96 \times 3.5\left[\left(\frac{3.5-0.4}{2}\right) - 0.5\right]$.

$V = 360$ kN

$V_u = 1.5 \times 360 = 540$ kN — 01M

Nominal shear stress $\tau_v = \frac{V_u}{B \cdot D_f} = \frac{540 \times 10^3}{3500 \times 500} = 0.308 \text{ N/mm}^2$

$P_t = \frac{100 \times A_{st}}{b\,d_f} = \frac{100 \times 3958.15}{3500 \times 100} = 0.22\%$, By table 19 — 02m

$\tau_c = 0.36 - \left[\frac{0.25 - 0.15}{0.36 - 0.28}\right] 0.03 = 0.336\ N/mm^2$

Permissible shear stress = $k\,\tau_c$, $k = 1$ => $0.336\ N/mm^2$ — 01M.

$k\tau_c > \tau_v$ Design is safe.

Two way shear: $a_0 = D + d_f$ and $b_0 = b + d_f = 0.9m$.

$V = q \times [A_{ppox} - a_0 \times b_0] = 97.96[12.26 - 0.9 \times 0.9] = 1120.66\ kN$. — 01M.

and $V_u = 1.5 \times 1120.66 = 1681\ kN$

$\tau_v = \frac{1681 \times 1000}{2(900+900)500} = 0.93\ N/mm^2$ and $\tau_c = 0.25\sqrt{20} = 1.11\ N/mm^2$

Permissible shear stress = $k_s\,\tau_c$. $k_s = 1$ = $k_s\,\tau_c = 1.11\ N/mm^2$.

which is greater than τ_v hence safe — 01M

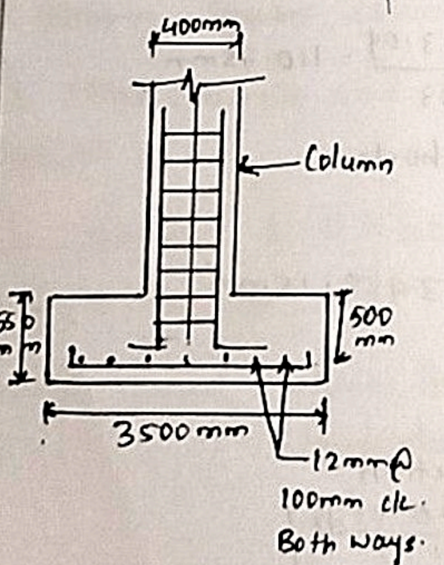
Step ⑨: Development length $L_d = \frac{\phi\,\sigma_s}{4\,\tau_{bd}}$

$\sigma_s = 0.87 f_y$. $\tau_{bd} = 1.2$ for M_{20} ∴ $L_d = \frac{0.87 \times 415}{4 \times 1.2 \times 1.6} \times 12$.

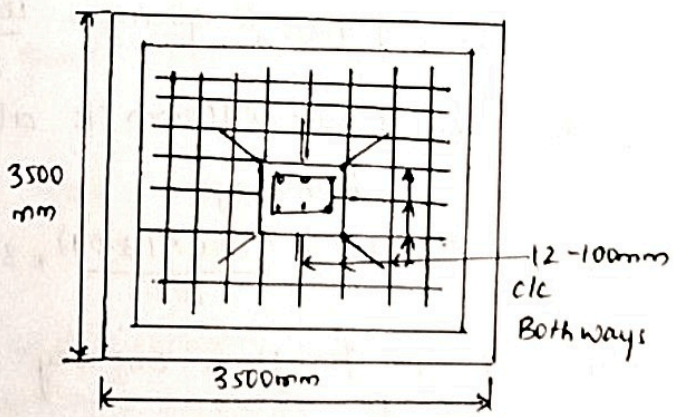
$L_d = 564\ mm$, Providing 50mm side cover,

Length available = $\frac{1}{2} \times (3500 - 400) - 50 = 1500mm > L_d$

Hence OK. — 01M.



L/s of footing



Plan of Footing

03M.

Laxmi G Hattihodi

HEAD
Dept of Civil Engg
KLS V.D.I.T, Haliyal

Dean Academic
Dr. Gururaj Hatti