As, $\tau_{ve} > \tau_c$, in addition to the longitudinal reinforcement provided, transverse reinforcement is also to be provided.

Consider that transverse steel comprises of 8 mm diameter, 2 legged vertical stirrups,

$$A_{sv} = 2 \times \frac{\pi}{4} \times 8^2 = 10 \text{ mm}^2$$

$$\text{Spacing of stirrups, } S_v = \frac{0.87 f_y A_{sv}}{\dfrac{T_u}{b_1 d_1} + \dfrac{V_u}{2.5 d_1}}$$

where, b_1 = c/c distance between corner bars along width of beam

$= 550 - 25 - 25 - 16 = 484$ mm

d_1 = c/c distance between corner bars along depth of beam

$= 750 - 25 - 25 - 10 - 8 = 682$ mm

$$\therefore \quad S_e = \frac{0.87 \times 415 \times 100}{\dfrac{50 \times 10^6}{484 \times 682} + \dfrac{130 \times 10^3}{2.5 \times 682}} = 158.5 \text{ mm}$$

The spacing of vertical stirrups shall not be more than,

$$S_v = \frac{0.87 f_y A_{sv}}{(\tau_{ve} - \tau_c) b} = \frac{0.87 \times 415 \times 100}{(0.705 - 0.35)550} = 185 \text{ mm}$$

The spacing of stirrups should not exceed

$S_v < x_1$ = shorter dimension of stirrup

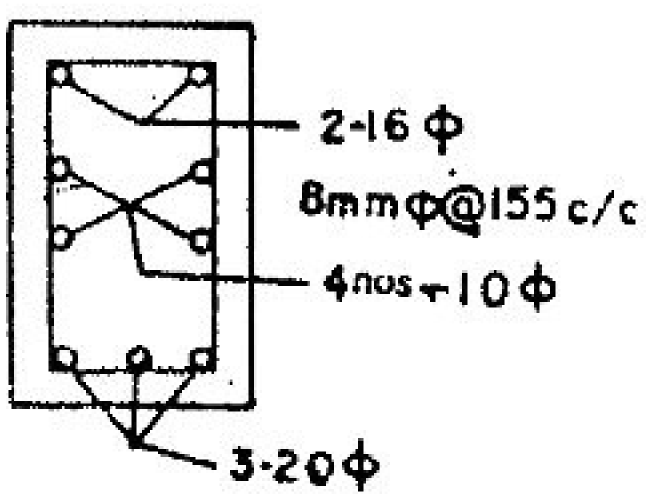
$= 484 + 16 + 8 = 508$ mm

$$S_v < \frac{x_1 + y_1}{4}$$

$$= \left(\frac{\text{Shorter+Longer dimension of stirrups}}{4}\right)$$

$$= \frac{508 + (682 + 8 + 10 + 8)}{4}$$

$= 304$ mm



d = 750 – 25 – 10 = 715

$S_v < 0.75\,d$; *i.e.* $0.75 \times 715 = 536.25$ mm

$S_v <$ 450 mm

Hence provide 8 mm diameter 2 legged stirrups. @ 155 mm c/c.

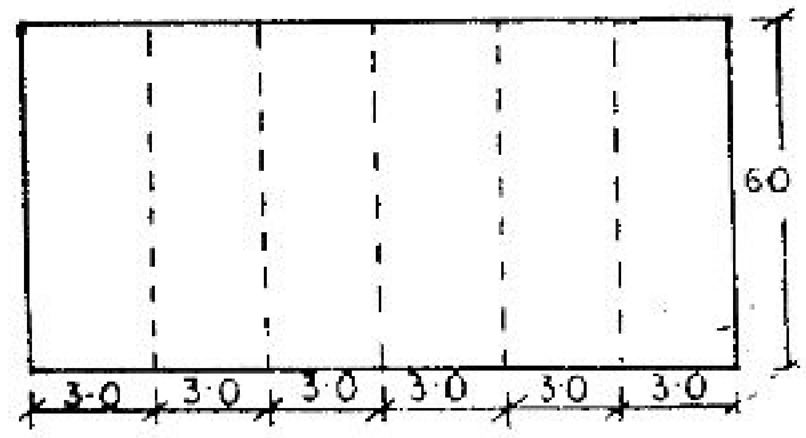
19. A hall ofclear dimensions 6.0 m × 18.0 m is covered by a reinforced slab 100 mm thick supported on T-beams spaced at 3 m. centres. Walls are 450 mm thick. Rib of the beam is restricted to 300 mm. Live load on the slab is 4.0 kN/m² and floor finished weight 1.0 kN/mm². Concrete mix used is M15. $\sigma_{cbc} = 5$ N/mm², $\sigma_{st} = 140$ N/mm² and m = 19. Design a T-beam for flexure and shear. Draw a neat longitudinal section and cross-section at mid span of the beam and show reinforcement details.

Solution: $d_{eff} = [300 + 100 - 25 - 10] = 365$ mm

Effective span of beam is lesser of

(*i*) clear span + $d_{eff} = 6 + 0.45 - 0.365$

$= 6.815$ m



(*ii*) $$l_{c/c} = 6.0 + 2 \times \frac{0.45}{2} = 6.45 \text{ m}$$

Width of T-beam is lesser of,

$$b_f = \frac{l_0}{6} + b_w + 6\,D_f \text{ (assume } b_w = 300 \text{ mm)}$$

$$b_f = \frac{6450}{6} + 300 + 6 \times 100 = 1995 \text{ mm}$$

or b_f = c/c distance = 3000 mm

$\therefore$ $b_f = 1975$ mm

Loads

Dead load of slab = 0.10 × 25 = 2.5 kN/m²

Floor finish = 1.0 kN/m²

Live load = 4.0 kN/m²

Total load = 7.5 kN/m²

Load from slab on beam per meter run

= 7.5 × 3 = 22.5 kN/m

Self weight of beam per meter run

= 0.3 × 0.3 × 25 = 2.25 kN/m

Total load per meter run of beam

= (22.5 + 2.25) = 24.75 kN/m

Maximum bending moment,

$$M = \frac{wl^2}{8} = \frac{24.75 \times 6.45}{8} = 128.71$$

Depth of balance neutral axis,

$$x_{bal} = \frac{1}{1+\frac{\sigma_{st}}{m\sigma_{cbc}}} (\because k_{bal} = 0.4)$$

$$d = \frac{1}{1+\frac{140}{19\times 5}} \times 365 = 147.55 \text{ mm}$$

As neutral lies in the web, moment of resistance for balanced section,

$$MR = \frac{1}{2}.b_f\, \sigma_{cbc} d\left(d - \frac{x}{3}\right)$$

$$= \frac{1}{2} \times 1975 \times 5 \times 365\left(365 - \frac{147.55}{3}\right)$$

$$= 569 \times 10^6 \text{ N/mm}$$

$$= 569 \text{ kN-m}$$

Since M < MR; so the section is to be designed as an under-reinforced section,

$$M = \sigma_{st}.A_{st}\left(d - \frac{x_{bal}}{2}\right) \text{ (since } x_{bal} = 147.55)$$

$$\therefore \quad A_{st} = \frac{128.7\times 10^6}{140\times\left(365 - \frac{147.55}{3}\right)}$$

$$= 2911 \text{ mm}^2 = 29.1 \text{cm}^2$$

Provide 25 mm diameter bars :

$$\text{Number of bars required} = \frac{29.1}{\frac{\pi}{4}(2.5)^2} = 5.9 \simeq 6$$

Maximum shear force at 'd' from face of support

$$= \frac{wl}{2} - w\,(0.365 + 0.15)$$

$$= \frac{24.75\times 6.45}{2} - 24.75\,(0.355 + 0.225)$$

$$= 65.21 \text{ kN}$$

$$\text{Percentage of steel provided} = \frac{100A_{st}}{b_f d}$$

$$= \frac{100\times 6\times 4.91}{197.5\times 36.5} = 0.41$$

For $p = 0.41, \quad \tau_c = 0.26$

$\therefore$ Shear force for which shear reinforcement to be provided,

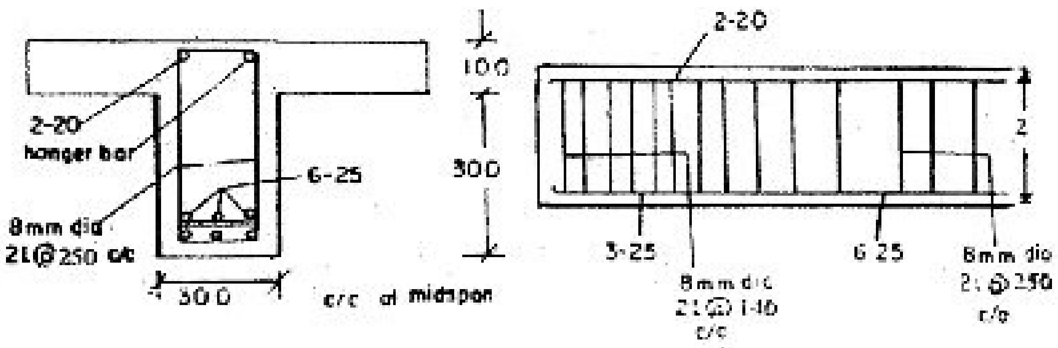
$$V_s = 65.21 \times 10^3 - 0.26 \times 300 \times 365$$
$$= 36740 \text{ N}$$

$\therefore$ Spacing of 8 mm diameter, two legged vertical stirrups,

$$S_v = \frac{\sigma_{sv}.A_{sv}.d}{V_s} = \frac{140\times 2\times\frac{\pi}{2}\times(8)^2\times 365}{36740}$$

$$= 140 \textbf{ mm}$$

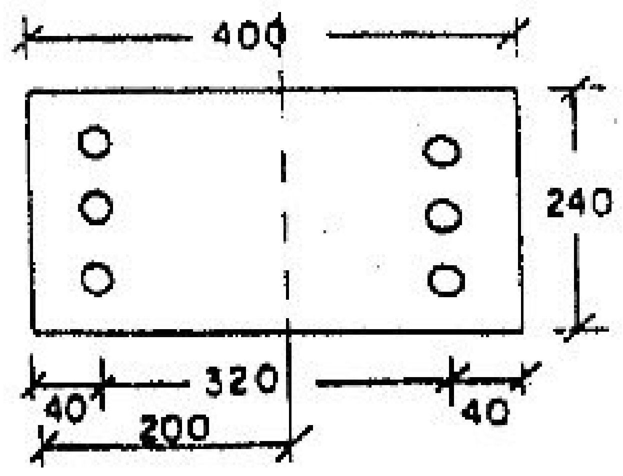
Provide 8 mm diameter, 2 legged stirrups @ 140 mm c/c at support and 8 mm diameter, 2 legged stirrups @ 250 mm c/c at centre.



COMPRESSION MEMBER

21. A rectangular column 240 mm × 400 mm overall is reinforced with 6 numbers of 16 mm bars. The bars are equally distributed on the short faces with an effective cover of 40 mm. Determine the eccentricity at which a load of 400 kN should act so that the maximum stress in concrete is limited to 7 N/mm², Assume m = 13.

Solution:



Area of steel,

$$A_s = 6 \times \frac{\pi}{4} \times 16^2$$
$$= 1206.37 \text{ mm}^2$$

Equivalent area of column

$$A_{eq} = 240 \times 400 + (1.5 \times 13 - 1) \times 1206.37$$
$$= 118317.85 \text{ mm}^2$$

Equivalent moment of inertia of columns section,

$$I_{eq} = \frac{1}{12} \times 240 \times 400^3 + (1.5 \times 13 - 1) \times 1206.37(200 - 40)^2$$

$$= 1.851 \times 10^9 \text{ mm}^4$$

Direct stress,

$$\sigma_{cc,cal} = \frac{P}{A_{eq}} = \frac{400\times 10^3}{11837.85} = 3.3807 \text{ N/mm}^2$$

$$\text{Bending stress, } \sigma_{cbc,cal} = \frac{P.e}{I_{eq}} \times 0.5 \text{ D}$$

$$= \frac{400\times 10^3\times e}{1.851\times 10^9}\times\frac{400}{2} = 0.0432\, e \text{ N/mm}^2$$

where, e = maximum eccentricity.

For maximum stress in concrete to reach a value 7 N/mm²,

$$\sigma_{cc,cal} + \sigma_{cbc,cal} = 7$$

or $$3.3807 + 0.0432\,e = 7$$

$$e = 83.78 \text{ mm}$$

Hence maximum eccentricity at which load of 400 kN can be safely loaded = 83.78 mm.

22. Calculate by working stress method the main reinforcement and ties required for a column of effective length 4.5 m to carry a service load of 550 kN including self weight of column. The size of column is 300 × 300 mm. The allowable stresses in direct compression in concrete and steel are 5 MPa and 190 MPa, respectively. The expression for the reduction coefficient as recommended by IS-456 with usual definition of notations is :

$$C_r = 1.25 - \frac{l_{ef}}{48b}$$

Solution: Given,

$L_{eff} = 4.5$ m, $\quad b = d = 300$ mm

$\sigma_{ccl} = 5$ N/mm², $\sigma_{sc} = 190$ N/mm²,

$$\frac{l_{eff}}{b} = \frac{4.5\times10^3}{300} = 15 > 12$$

Hence, given column is a long column

Reduction coefficient,

$$C_r = 1.25 - \frac{l_{eff}}{48b}$$

$$= 1.25 - \frac{4500}{48\times300} = 0.9375$$

Axial load = 550 kN

Axial load, $P = \sigma_{cc}A_c + \sigma_{sc}A_{sc}$

where, A_c = cross-section area of concrete

$= 300 \times 300 = 90000$ mm²

$\therefore \quad 550 \times 10^3 = 5 \times 90{,}000 + 190 \times A_{sc}$

$A_{sc} = 526.32$ mm²

Minimum cross-sectional area of longitudinal reinforcement

$$= 0.8\% = \frac{0.8}{100} \times 300 \times 300 = 720 \text{ mm}^2$$

Provide 4 bars of 16 mm diameter.

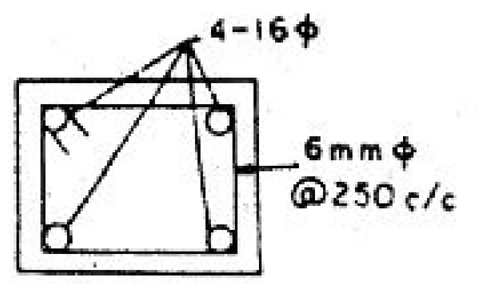
Lateral Ties

Diameter of lateral tie shall not be less than $\frac{\phi_1}{4}$

i.e. $\frac{16}{4}$ *i.e. 4* mm.

and nor less than 5 mm

Hence, provide 6 mm lateral Ties



Pitch

Pitch < 300 mm (lateral dimension of column)

$< 16\phi_l$, *i.e.* $16 \times 16 = 256$ mm

$< 48\phi_t$, *i.e.* $48 \times 6 = 288$ mm

Hence, provide 6 mm ties at 250 mm c/c spacing.

23. Determine the cross-sectional dimensions and the reinforcement for an axially loaded column for the following data :

Collapse load = 2500 kN,

Unsupported length = 4m

End condition : both the ends are pinned Concrete grade : M20,

Steel grade : Fe 415

Adopt a square section for the columns.

Solution: It is assumed that the column is short and that the minimum eccentricity does not exceed 0.05 times the lateral dimension.

Now, $$P_u = 0.4f_{ck}A + 0.67f_yA_{sc}$$
$$= 0.4f_{ck}(A_g - A_{sc}) + 0.67f_yA_{sc}$$
$$= 0.4f_{ck}A_g + (0.67f_y - 0.4f_{ck})A_{sc}$$
$$= 0.4f_{ck}A_g + (0.67f_y - 0.4f_{ck})pA_g$$
$$= \{0.4f_{ck} + (0.67f_y - 0.4f_{ck})p\}A_g \qquad ...(i)$$

where $p = \dfrac{A_{sc}}{A_g}$

If the longitudinal reinforcement be kept at 0.8% of the sectional area, then $p = 0.008$. Substituting all the known values in equation (*i*), we get

$$2500 \times 10^3 = \{0.4 \times 20 + (0.67 \times 415 - 0.4 \times 20) \times 0.008\}A_g$$

or $$A_g = 246.06 \times 10^3 \text{ mm}^2$$

Length of the side of column = $\sqrt{A_g}$ = 496 mm

Adopt a section of 500 mm × 500 mm in dimension.

Effective length of column = 1.0 × 4 = 4m

As the column is square, the slenderness ratio

$$\frac{l_{ex}}{D} = \frac{l_{ey}}{b} = \frac{4\times10^3}{500} = 8.0 < 12$$

Therefore, the assumption of the column being short is justified. Longitudinal reinforcement

$$A_{sc} = 0.008 \times 496 \times 496 = 1968.13 \text{ mm}^2$$

Provide 18 mm diameter bars, 8 numbers

Now, $0.05D = 0.05 \times 0.05 \times 500 = 25$ mm

Minimum eccentriciy for the column

$$= \frac{\text{Unsupported length}}{500} + \frac{\text{Lateral dimension}}{30}$$

$$= \frac{4 \times 1000}{500} + \frac{500}{30}$$

$$= 24.67 \text{ mm} < 0.05\,D$$

Therefore, the assumption that the minimum eccentricity does not exceed 0.05 times the lateral dimensions is justified.

The longitudinal bars are proposed to be placed with their centres 50 mm away from the edge of the section. This will ensure a concrete cover of 41 mm on the bars, as against the minimum requirement of 40 mm.

Diameter of transverse reinforcement or lateral ties

$\not< \frac{1}{4} \times$ Diameter of longitudinal bar,

i.e. $\frac{1}{4} \times 16 = 4$ mm.

$\not<$ 5 mm

Provide 6 mm diameter bars for lateral ties.

Pitch of the ties

$\not>$ the least lateral dimension, *i.e.* 500 mm

$\not>$ 16 $\times$ diameter of longitudinal bar,

i.e. $16 \times 18 = 288$ mm

$\not>$ 48 $\times$ diameter of lateral ties,

i.e. $48 \times 6 = 288$ mm

Adopt a pitch of 280 mm c/c for the ties. The corner bars are effectively tied in two directions by the outer closed tie. However, the c/c distance between the corner bars along a side is 400 mm which is greater than 48 times the diameter of the tie, *i.e.* $48 \times 6 = 288$ mm. Therefore, open ties cannot be provided to tie the inner bars for which an additional closed tie is, therefore, provided to tie them effectively in two directions. Thus a pair of ties are provided at a section as shown in the figure.

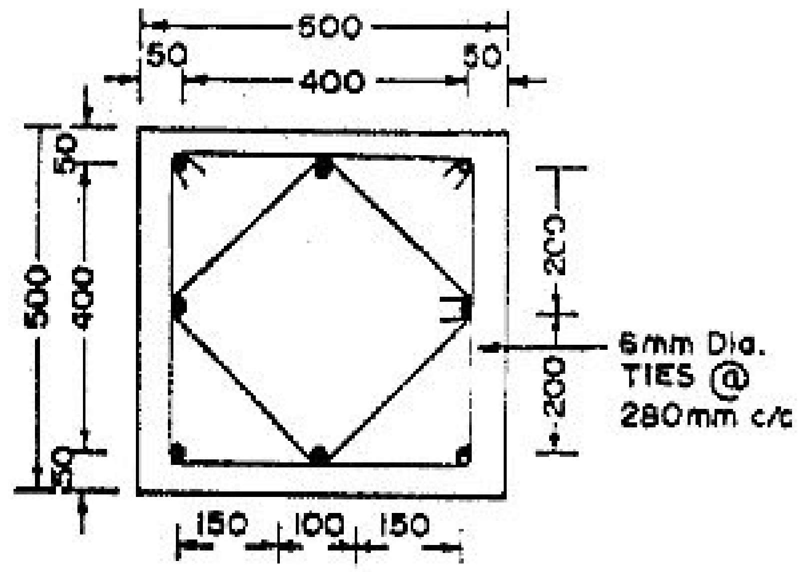
23. Redesign the column of above example keeping a circular section instead of square. All the data is same.

Solution: It is assumed that the column is short and that the minimum eccentricity does not exceed 0.05 times the lateral dimension.

Keeping $p = 0.008$ and on substituting the known values in this equation for a helically reinforced column, we have

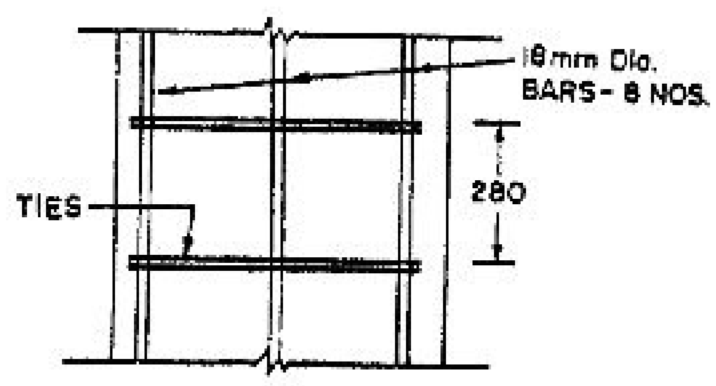
$$2500 \times 10^3 = 1.05\,\{0.4 \times 20 + (0.67 \times 415 - 0.4 \times 20) \times 0.008\}\,A_g$$

or $A_g = 234.34 \times 10^3 \text{ mm}^2$



SECTIONAL PLAN

Fig.(a)



LONGITUDINAL VIEW

Fig. (b)

Outer diameter of the column section,

$$D_0 = \sqrt{\frac{4 \times 234.34 \times 10^3}{\pi}}$$

$$= 546.24 \text{ mm}$$

Adopt $D_0 = 550$ mm

Longitudinal reinforcement.

$$A_{sc} = 0.008 \times 234.24 \times 10^3$$

$$= 1874.72 \text{ mm}^2$$

16 mm diameter bars 10 numbers may be provided. The bars may be placed with their centres 50 mm away from the edge of the section. This will ensure a concrete cover of 42 mm on the bars as against the minimum requirement of 40 mm.

Diameter of the helical reinforcement bar

$$\not< \frac{1}{4} \times 16 \textit{ i.e. } 4 \text{ mm}$$

$$\not< 5 \text{ mm}$$

Provide 6 mm diameter bar for the helical reinforcement.

Core diameter for the column,

$$D_c = 550 - 2 \times 50 + 16 + 2 \times 6$$

$$= 478 \text{ mm}$$

Effective length of column $= 1.0 \times 4 = 4$ m

$$\text{Slenderness ratio} = \frac{\text{Effective length}}{\text{Core diameter}}$$

$$= \frac{4 \times 1000}{478} = 8.37 < 12$$

Therefore, the assumption that the column is short is justified.

Now, $0.05\,D_0 = 0.05 \times 550 = 27.5$ mm

Minimum eccentricity for the column

$$e_{\min} = \frac{\text{Unsupported length}}{500} + \frac{\text{Lateral dimension}}{30}$$

$$= \frac{4+1000}{500} + \frac{500}{30}$$

$$= 26.33 \text{ mm} < 0.05\,D_0$$

Therefore, the assumption that the minimum eccentrity does not exceed 0.05 times the lateral dimension is justified.

Pitch of the helical reinforcement

$\not>$ 75 mm

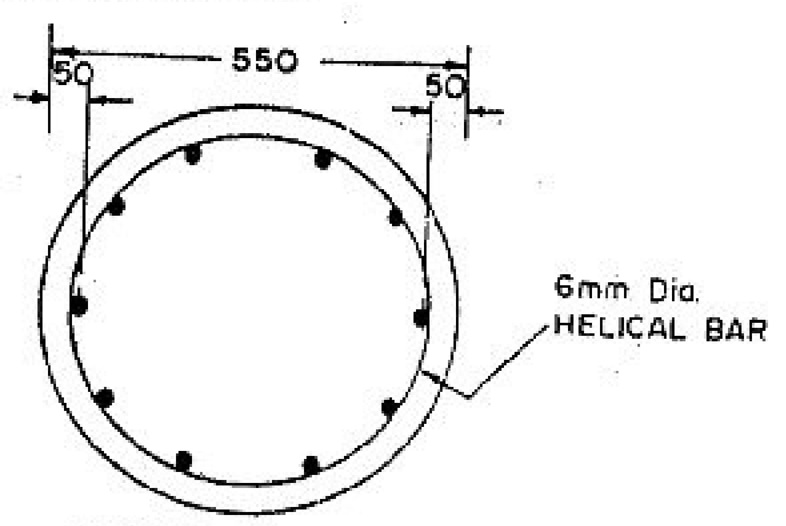
$\not>$ $\frac{1}{6}$ times the core diameter,

i.e. $\frac{1}{6} \times 478 = 79.67$ mm

$\not<$ 25 mm

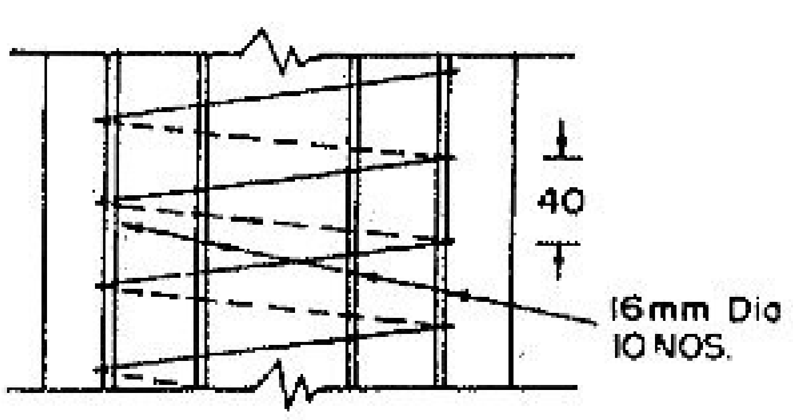
$\not<$ 3 × diameter of helical bar,

i.e. $3 \times 6 = 18$ mm



SECTIONAL PLAN

Fig. (a)



LONGITUDINAL VIEW

Fig. (b)

As a trial keep the pitch = 40 mm.

Diameter of helix = 478 – 6 = 472 mm

Length of helical bar in one pitch

$$= \sqrt{(\pi \times 472)^2 + 40^2}$$

$$= 1483.4 \text{ mm}$$

Volume of helical reinforcement in one pitch

$$= 1483.4 \times 28.27$$

$$= 41935.7 \text{ mm}^3$$

$$\text{Volume of the core in one pitch} = \frac{\pi}{4} \times 478^2 \times 40$$

$$= 7178053 \text{ mm}^3$$

$$\frac{\text{Volume of helical reinforcement}}{\text{Volume of the core}} = \frac{41935.7}{7178053}$$

$$= 5.842 \times 10^{-3}$$

$$0.36\left(\frac{A_g}{A_{co}} - 1\right)\frac{f_{ck}}{f_y} = 0.36\left(\frac{550^2}{478^2} - 1\right) \times \frac{20}{415}$$

$$= 5.62 \times 10^{-3}$$

Evidently,

$$\frac{\text{volume of helical reinforcement}}{\text{volume of the core}} \not< 0.36\left(\frac{A_g}{A_{co}} - 1\right)\frac{f_{ck}}{f_y}$$

Hence, the pitch of 40 mm for the helical reinforcement bar is alright.

24. A circular RC column of M 15 grade concrete and 300 mm. diameter has 8 number of 12 mm. diameter, Fe 415 grade steel bars as longitudinal reinforcement. The length of the column is 7 m. If its ends are effectively held in position and restrained against rotation, determine the strength of the column.

Solution: As per IS 456:1978, the effective length of column effectively held in position and restrained against rotation at both ends

$$I_{eff} = 0.65\, l$$

where, l is length of column

$\therefore \quad l_{eff} = 0.65 \times 7 = 4.55$ m

Given,

diameter of column = 300 mm

$\therefore \quad \frac{l_{eff}}{D} = \frac{4550}{300} = 15.17 > 12$, hence column is

$\therefore$ Load carrying capacity of the column,

$$P = C_r[\sigma_{cc}.A_c + \Sigma_{sc}.A_{sc}]$$

where,

$$C_r = \text{reduction factor} = 1.25 - \frac{l_{eff}}{48b}$$

$$= 1.25 - \frac{4550}{48 \times 300} = 0.934$$

$\sigma_{cc} = 4$ N/mm^2

$\sigma_{sc} = 190$ N/mm^2

$A_c = (A_g - A_{sc})$

$= \dfrac{\pi}{4}(300)^2 - 8 \times \dfrac{\pi}{4} \times (12)^2 = 69781$ mm^2

$A_{sc} = 8 \times \dfrac{\pi}{4} \times (12)^2 = 904.8$ mm^2

$P = 0.934[4 \times 69781 + 190 \times 904.8]$

$= 421263.84$ N

$=$ **421.26** kN

25. A column of dimensions 200 × 300 mm has an effective length of 3 m. It is reinforced with 6 HYSD bars of 20 mm diameter. Determine the safe load the column can carry using working method of design. Grade of concrete is M 20.

Solution: Slenderness ratio of column

$$= \frac{3000}{20} = 15 > 12$$

So, it is a long column.

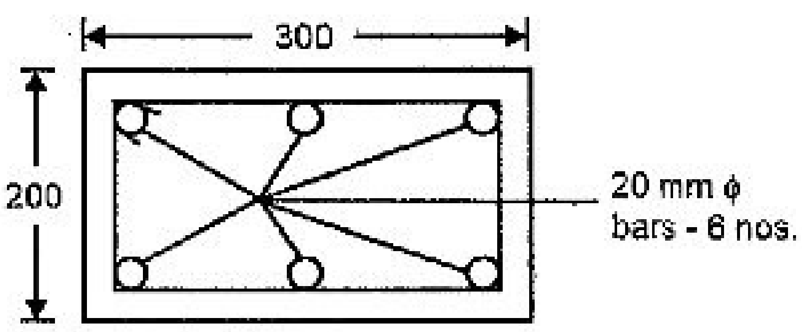
Reduction factor, $C_r = 1.25 - \dfrac{l_{eff}}{48b}$

$$= 1.25\ \frac{3000}{48 \times 200}$$

$$= 0.9375$$

Load carrying capacity of column,

$$P = C_r(\sigma_{cc}.A_c + \sigma_{sc}.A_{sc})$$



where, $\sigma_{sc} = 5$ N/mm^2

$\sigma_{sc} = 190$ N/mm^2

$A_{sc} = 6 \times \dfrac{\pi}{4} \times (20)^2 = 1884$ mm^2

$A_c = (A_g - A_{sc})$

$= (200 \times 300 - 1984) = 58116$ mm^2

$P = 0.9375[5 \times 58116 + 190 \times 1884]$

$= 608 \times 10^3$ N

$= 608$kN

7. Check the safely of a rectangular RC column 250 × 350 mm reinforced with 4 bars of 16 mm diameter on each of its short faces and subjected to an axial load of 450 kN and a bending moment of 15 kN. m acting about its major axis. Can the column carry any more axial load? If so how much more?

Concrete used is M20 grade for which permissible stresses are:

in bending compression = 7 N/mm^2,

in direct compression = 5 N/mm^2,

Modular ratio = 19.

Cover to centres of reinforcing bar = 40 mm.

As per code, section is safe if,

$$\frac{\sigma_{cc,\,cal}}{\sigma_{cc}} + \frac{\sigma_{cbc,\,cal}}{\sigma_{cbc}} \le 1.0$$

where $\sigma_{cc,cal}$ and $\sigma_{cbc,cal}$ are the calculated direct and bending compressive stresses in concrete respectively and σ_{cbc} are permissible direct and bending compressive stresses in concrete respectively.

Equivalent area of column

$$A_c + 1.5\text{ m } A_{sc}$$

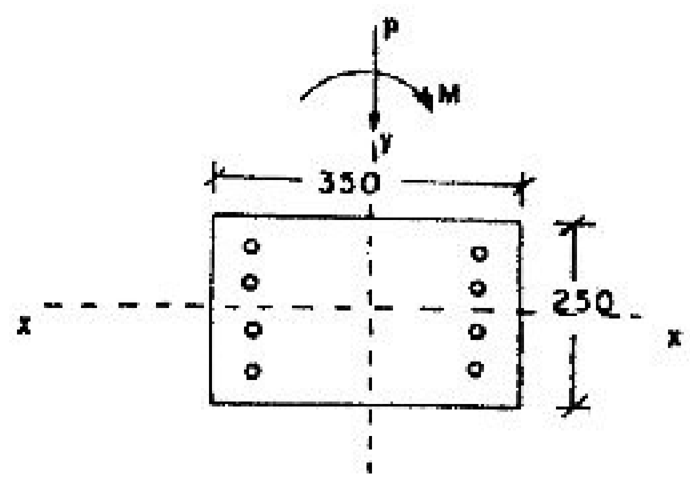
where, A_g is the area of concrete and A_{sc} is the area of steel in the column.

Solution: Given,

$P = 450$ kN

$M_x = 15$ kNm

M 20 mix



Equivalent area of column,

$$A_{eq} = A_c + 1.5\text{ m } A_{sc}$$

where, $A_{sc} = 4 \times \dfrac{\pi}{4} \times 16^2 \times 2$

$= 1608.5$ mm

But, $A_c = A_g - A_{sc}$

where, A_g = gross sectional area of column

$= (250 \times 350) = 87500$ mm^2

$\therefore \quad A_{eq} = A_g - A_{sc} + 1.5\text{ m}A_{sc}$

$A_g + (1.5\text{ m} - 1)A_{sc}$

$= 87500 + (1.5 \times 19 - 1) \times 1608.5$

$= 131733.75$ mm^2

Moment of inertia about axis of bending,

$$I_{eq} = \frac{1}{12} \times 250 \times 350^3 + (1.5 \times 19 - 1) \times 1608.5 \times (350 - 40)^2$$

$$= 5.144 \times 10^9 \text{ mm}^4$$

We know,

$$\sigma_{cc,\,cal} = \frac{P}{A_{eq}} = \frac{450\times10^3}{131733.75} = 3.416 \text{ N/mm}^2$$

and $$\sigma_{cbc,\,cal} = \frac{M}{Z} = \frac{M}{I_{eq}}\times\frac{350}{2}$$

$$= \frac{15-10^6}{5.144\times10^9}\times\frac{350}{2} = 0.51 \text{ N/mm}^2$$

As per codal provision, column can safely carry the given loads if

$$\frac{\sigma_{cc.cal}}{\sigma_{cc}} + \frac{\sigma_{cbc.cal}}{\sigma_{cbc}} \le 1$$

$$\frac{3.416}{5} + \frac{0.510}{7} = 0.756 \le 1$$

hence the column is safe.

Now, let P be the maximum load that can be carried by the same column, then

$$\sigma_{cc,\,cal} = \frac{P}{A_{eq}} = \frac{P}{131733.75} \text{ N/mm}^2$$

For the limiting case,

$$\frac{\sigma_{cc,cal}}{\sigma_{cc}} + \frac{\sigma_{cbc,cal}}{\sigma_{cbc}} = 1$$

or $$\frac{P}{131733.75\times5} + \frac{0.51}{7} = 1$$

$$P = 610680 \text{ N} = 610.68 \text{ kN}$$

So,extra load that can be carried by the column $= 610.68 - 450 = 160.68$ kN

8. Design a square section column using M-15 grade concrete and mild steel bars to carry an axial load (P) of 30,000 kg. Effective length of column $(l_{eff}) = 4.0$ m.

Assume $\sigma_{cc} = 40$ kg/cm² and $\sigma_{st} = 1300$ kg/cm² respectively,

As per IS code, $P = C_r(A_c\sigma_{cc} + \sigma_{sc}A_{sc})$

where, A_c and A_{sc} are areas of cross-section of concrete and steel respectively.

$$C_r = \text{reduction factor} = 1.25 - \frac{l_{eff}}{48b} \ngtr 1.0$$

b = lateral dimension of column.

Draw the arrangement for longitudinal and lateral reinforcement.

Solution:

Adopt a square column of size 250 mm × 250 mm

$$\frac{l_{eff}}{b} = \frac{4000}{250} = 16 > 12,$$

so the column is slender column.

Given, $P = 30{,}000 \text{ kg} = 300 \text{ kN},$

$$\sigma_{cc} = 4 \text{ N/mm}^2,\quad \sigma_{sc} = 130 \text{ N/mm}^2$$

$$C_r = 1.25 - \frac{4000}{48\times250} = 0.917$$

$$P = C_r(\sigma_{cc}A_c + \sigma_{sc})$$

$$= C_r[\sigma_{cc}.A_q + (\sigma_{sc} - \sigma_{cc})A_{sc}]$$

or $$300\times10^3 = 0.91.7\,[4\times250\times250 + (130-4)A_{sc}]$$

$\therefore$ $$A_K = 603.9 \text{ mm}^2 \approx 604 \text{ mm}^2$$

Providing 16 mm diameter longitudinal bars

$$\text{Number of bars required} = \frac{604}{\frac{\pi}{4}(10)^2} = 3.005$$

Provide 4 number of 16 mm ϕ bars

Lateral ties

$$\text{Diameter of tie} \nless \frac{\phi_{main}}{4},\ i.e.\ \frac{16}{4} = 4 \text{ mm}$$

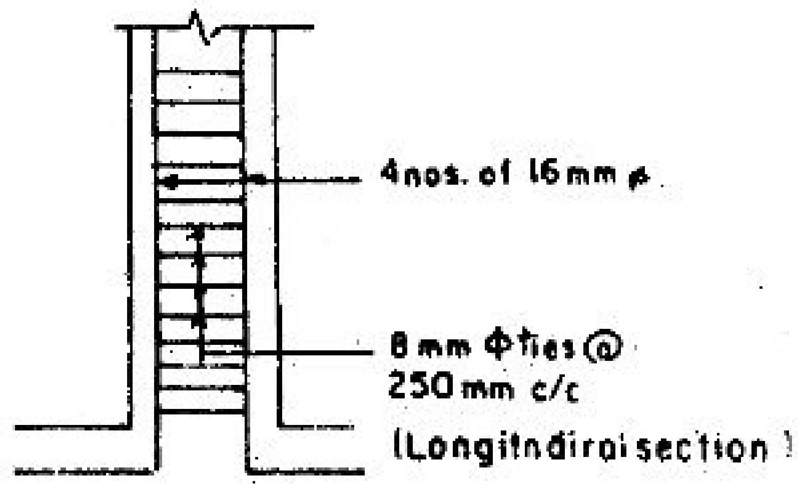
$$\nless 5 \text{ mm}$$

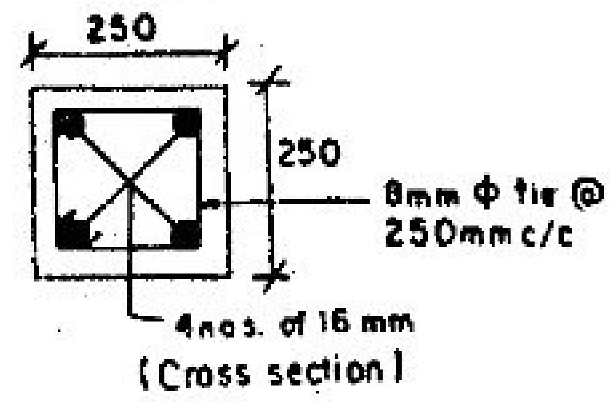
Provide 6 mm ϕ as lateral ties.

Spacing of ties $\ngtr 16\times\phi_{main}$ *i.e.* $16\times16 = 256$ mm

$\ngtr 48\times\phi_{tie}$, *i.e.* $48\times6 = 288$ mm

$\ngtr$ least lateral dimension, *i.e.* 250 mm.





TWO WAYSLAB AND FOOTING

1. A slab is simply supported over a clear opening in plan 6 m × 4.5 m. The corners of the slab are free to lift up. There is a 50 mm thick plain concrete floor finish on the top of the slab. Live load is 3 kN/m.². Concrete grade: M15 and steel grade: Fe 415. Design the slab.

Solution: To estimate the self weight and the effective span of slab, assume its overall thickness = 180 mm and its effective depth = 160 mm.

Then the loads on the slab will be as follows :

Self weight of slab $= 0.18 \times 1 \times 1 \times 25$

$= 4.5$ kN/m²

Floor finish, $\dfrac{0.05 \times 1 \times 1 \times 24 = 1.2kN/m^2}{\text{Total dead load DL} = 5.7kN/m^2}$

Live load, LL = 3.0 kN/m²

Design load on slab, $w = 1.5 \times 4.7 + 1.5 \times 3.0$

$= 13.05$ kN/m²

Effective span, $l_x = 4.5 + 0.16 = 4.66$ m

$l_y = 6.0 + 0.16 = 6.16$ m

Ratio $\dfrac{l_y}{l_x} = \dfrac{6.16}{4.66} = 1.32 < 2.0.$

Hence, the slab should be designed as a two-way slab.

As the slab is simply supported and the corners are free to lift up, therefore

$$a_x = 0.0942;$$

$$a_y = 0.0542$$

Mid-span bending moments per unit width are

$$M_x = a_x w l_x^2$$
$$= 0.0942 \times 13.05 \times 4.66^2$$
$$= 26.7 \text{ kN-m/m}$$
$$M_y = a_y w l^2$$
$$= 0.0542 \times 13.05 \times 4.66^2$$
$$= 15.36 \text{ kN-m/m}$$

Effective depth will be based on the consideration of flexural strength as well as on the criterion of control of deflection.

Flexural strength consideration

The larger bending moment is along the shorter span viz. 26.7 kN/m.

For a balanced design.

$0.36 \times 150.48\,(1 - 0.42 \times 0.48) \times 1000 = 26.7 \times 10^6$

$\therefore \quad d = 113.6$ mm

Area of tension reinforcement,

$$A_{st} = \frac{26.7 \times 10^6}{0.87 \times 415(1 - 0.42 \times 0.48) \times 113.6}$$

$= 815.35$ mm²/m

Control of deflection

Along the short span

$$\frac{100A_{st}}{bd} = \frac{100 \times 815.35}{1000 \times 113.6} = 0.718$$

From IS : 456, : Modification factor = 1.05

Basic value of span/effective depth ratio = 20.

$\therefore$ Permitted value of span/effective depth ratio

$= 20 \times 1.05 = 21.$

Actual value of span/effective depth ratio

$$= \frac{4660}{113.6} = 41 >> \text{permitted value.}$$

Hence, the effective depth should be increased.

Try an effective depth = 165 mm.

Then, tension reinforcement along short span is obtained as

$$0.87 \times 415A_{st}\left(165 - \frac{415A_{st}}{15 \times 1000}\right) = 26.7 \times 10^6$$

$\therefore \; A_{st}\delta = 488.16$ mm²/m

$$\frac{100A_{st}}{bd} = \frac{100 \times 488.16}{1000 \times 165} = 0.296$$

From IS : 456, Modification factor = 1.43

Permitted value of span/effective depth ratio

$$= \frac{4660}{165} = 28.2 < \text{permitted value (just).}$$

Therefore, an effective depth of 15 mm satisfies the criterion of deflection.

Hence, adopt an effective depth for the reinforcement in shorter direction to be 165 mm.

Along the short span provide 8 mm dia. bars = 100 mm c/c.

Then, for reinforcement along longer direction, effective depth = 165 – 8 = 157 mm.

Adopt overall thickness of slab to be 185 mm.

Along the longer span, bending moment

= 15.36 kN-m/m.

Tension reinforcement is calculated as

$$0.87 \times 415A_{st}\left(\frac{415A_{st}}{15 \times 1000}\right) = 15.36 \times 10^6$$

$\therefore \; A_{st} = 285.4$ mm²

Provide 8 mm dia. bars – 175 mm c/c.

The slab will now be checked for shear.

Load distribution along the shorter span is given by

$$w_x = \frac{wr^4}{1 + r^4}$$
$$= \frac{13.05 \times 1.32^4}{1 + 1.32^4} = 9.82 \text{ kN/m}^2$$

Critical section occurs an effective depth away from the face of support. The design shear force at this section is

$$V_u = \left(\frac{1}{2} \times 4.5 - 0.165\right) \times 9.82$$

$= 20.48$ kN/m

Area of reinforcement per m width of slab

$$= \frac{50.26 \times 1000}{100}$$

$$= 502.6 \text{ mm}^2/\text{m}$$

$$\frac{100A_{st}}{bd} = \frac{100 \times 502.6}{1000 \times 165}$$

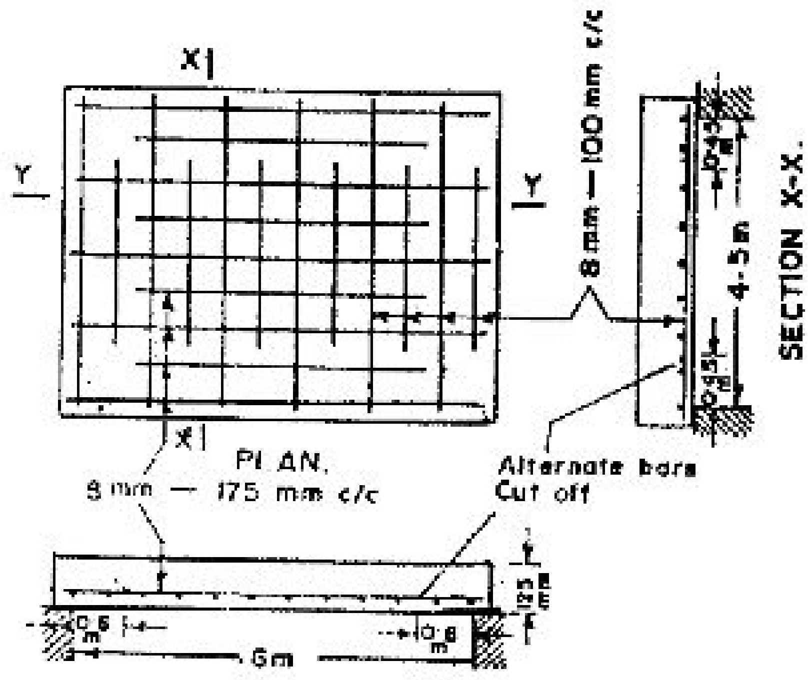
$$= 0.305$$

From IS : 456, design shear strength of concrete,

$$\tau_c = 0.374 \text{ N/mm}^2$$

From IS : 456, for a slab thickness of 185 mm, coefficient

$$k = 1.23$$



SECTION Y-Y.

Permitted shear stress in slab concrete $= k\tau_c$,

$$= 1.23 \times 0.374$$

$$= 0.46 \text{ N/mm}^2$$

Shear strength of concrete in slab

$$= 0.46 \times 1000 \times 165$$

$$= 75.9 \times 10^3 \text{ N/m}$$

$$= 75.9 \text{ kN/m} > 20.48 \text{ kN/m},$$

i.e. shear force per m width of the slab.

Hence the slab is safe in shear.

Alternate bars are terminated at 0.1 of span from the face of support. The continuing bars should extend into the support for a distance $\not< l_d/3$.

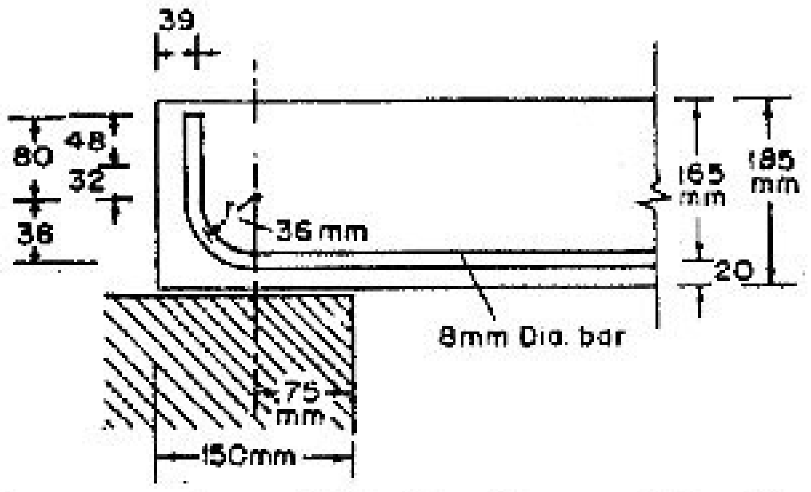
Development length of 8 mm bar in tension,

$$l_d = \frac{8 \times 0.87 \times 415}{4 \times 1.0 \times 1.6}$$

$$= 451.2 \text{ mm}$$

$$\therefore \quad \frac{l_d}{3} = \frac{451.2}{3} = 150.4 \text{ mm}$$

Let the bearing of the slab at the edges be 150 mm. Then for adequate anchorage of bars into the supports a standard 90□ bgnd may be provided.



Anchorage value of the bar beyond the face of support

= 75 + Anchorage value of the bend + 48 mm

= 155 mm which is > 150.4 mm. Hence, O.K.

At the support,

$$A_{st} = \frac{1}{2} \times 50.26 \times 1000/100$$

$$= 251.3 \text{ mm}^2/\text{m}.$$

M_1 = bending strength of slab with A_{st} = 251.3 mm²/m.

$$= 0.87 \times 415 \times 251.3\left(165 - \frac{415 \times 251.3}{15 \times 1000}\right)$$

$$= 14.34 \text{ kN-m/m}$$

V = shear force at the support

$$= \frac{1}{2} \times 4.66 \times 13.05 = 30.41 \text{ kN/m}$$

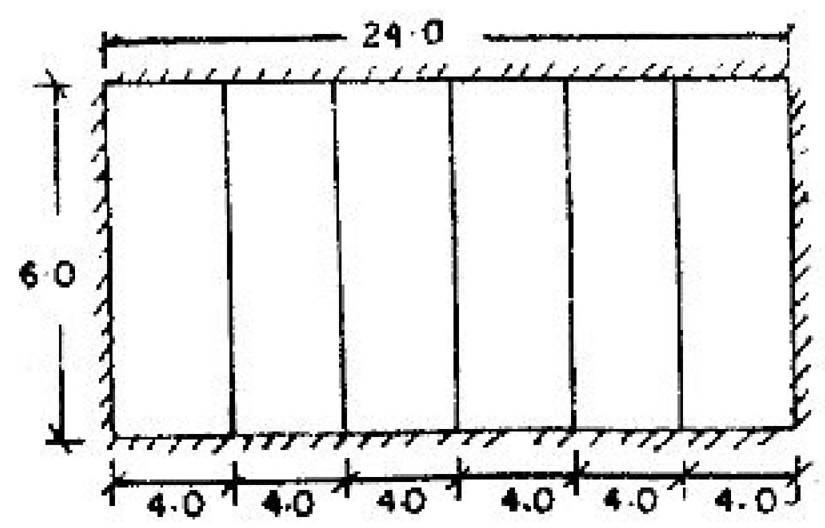
Anchorage of bar beyond the centre of support,

$$l_0 = 8 \times 8 + 48 = 112 \text{ mm}$$

$$\frac{M_1}{V} + l_0 = \frac{14.34 \times 10^6}{30.41 \times 10^3} \times 1.3 + 112 = 725 \text{ mm}$$

Since $l_d > \frac{M_1}{V} + l_0$ hence, the diameter of the tension reinforcement bar is alright from the point of view of anchorage and need not be reduced.

2. A hall 6 m × 24 m is to be covered by a slab and beam system. Beams are located at 4 m intervals and supported on walls all around of 350 mm thickness. Live load on the roof is 2 kN/m² and floor finish is 0.2 kN/m². Using working stress design and adopting M15 concrete and mild steel, design the slab in the end panel only.



Solution: Adopt beams of width 350 mm

Assume a slab thickness of 125 mm

Loads

Self weight of slab = $0.125 \times 15 = 3.125$ kN/m^2

Floor finish = 0.2 kN/m^2

Total dead load = 3.225 kN/m^2

Live load = 2.0 kN/m^2

Total load on slab = (3.225 + 2.0) = 5.225 kN/m^2

Effective span, $l_x = 4.0 - 0.35 + 0.10 = 3.75$ m

$l_y = 6.0 - 0.35 + 0.10 = 5.75$ m

$$\text{Ratio } \frac{l_y}{l_x} = \frac{5.75}{3.75} = 1.53 < 2,$$

hence design as two way slab.

The end panel which is to be designed is continuous in one long edge and discontinuous on all three edges.

	Short span coefficient	*Long span coefficient*
Negative moment at continuous edge	0.0845	–
Positive moment at mid span	0.0645	0.043

$$M_x \text{ at support} = a_{x1} w . l_x^2 = 0.0845 \times 5.325 \times 3.75^2 = 6.33 \text{ kNm/m}$$

$$M_y \text{ at midspan} = a_y w . l_x^2 = 0.0645 \times 5.325 \times 3.75^2 = 4.83 \text{ kNm/m}$$

$$M \text{ at midspan} = a_{y1} w . l_x^2 = 0.043 \times 5.325 \times 3.75^2 = 3.22 \text{ kNm/m}$$

Given

$\sigma_{cbc} = 5$ N/mm^2, $\sigma_{st} = 140$ N/mm^2

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

$$k = \frac{1}{1 + \frac{\sigma_{st}}{m\sigma_{cbc}}} = 0.4$$

$$j = 1 - \frac{k}{3} = 0.867$$

$$Q = \frac{1}{2}\sigma_{cbc} jk = \frac{1}{2} \times 5 \times 0.867 \times 0.4 = 0.867$$

We know,

$Qbd^2 = M$

$$d = \sqrt{\frac{M}{Qb}} = \sqrt{\frac{6.33 \times 10^6}{1000 \times 0.867}}$$

= 85.45 mm < 125 mm (adopted).

Hence O.K.

$d = (125 - 15 - 6) = 104$ mm

Area of steel required for continuous edge,

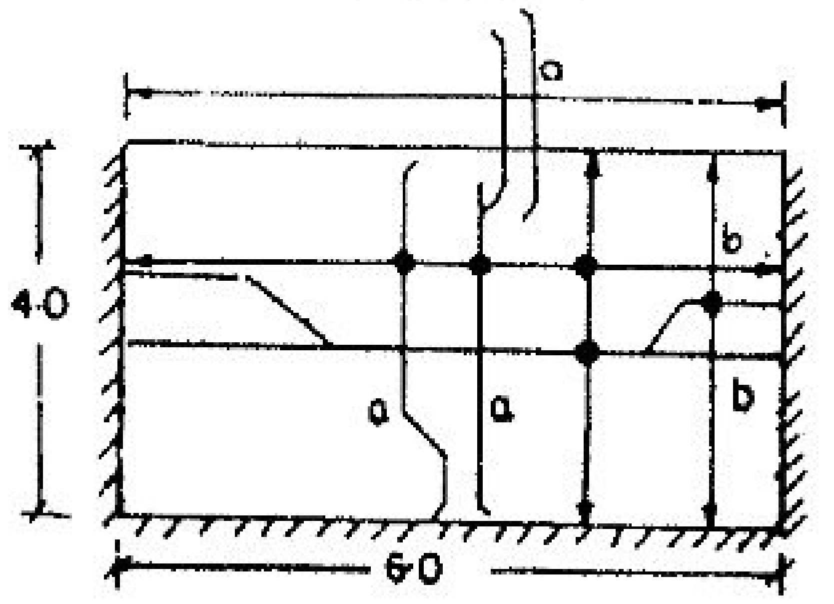
$$A_{st_1} = \frac{M_{x1}}{\sigma_{st} jd} = \frac{6.33 \times 10^6}{140 \times 0.867 \times 104} = 501.4 \text{ mm}^4$$

Providing 10 mm dia bar,

$$\text{Spacing} = \frac{1000 \times 78.54}{501.4} = 156.6 \text{ mm}$$

Area of steel required at midspan in longer direction,

$$A_{st_2} = \frac{M_{x2}}{\sigma_{st} jd} = \frac{4.83 \times 10^6}{140 \times 0.867 \times 104} = 382.6 \text{ mm}^2$$



Providing 10 mm dia bar

$$\text{Spacing} = \frac{1000 \times 78.54}{382.6} = 205.27 \text{ mm}$$

a = 10 mm dia @ 400 mm c/c.

Area of steel required at midspan in longer direction

$$A_{st3} = \frac{M_y}{\sigma_{st} jd} = \frac{3.22 \times 10^6}{140 \times 0.867 \times (104 - 5 - 4)} = 279.2 \text{ mm}^2$$

Providing 8 mm dia bars

$$\text{Spacing} = \frac{1000 \times 50.26}{279.2} = 179.9 \text{ mm}$$

b = 8 mm dia @ 350 mm c/c.

Additional steel required at continuous edge

$$= 501.4 - \frac{1000 \times 78.54}{400} = 305.05 \text{ mm}^2$$

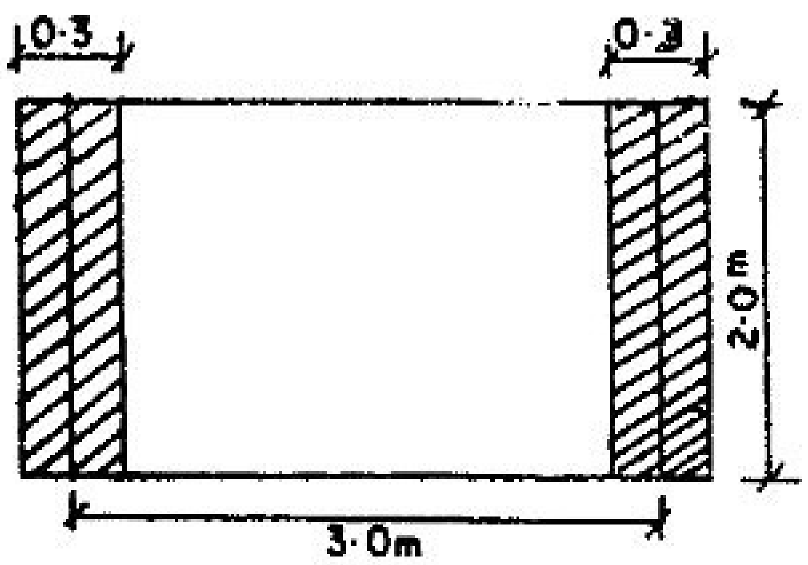
Providing 10 mm dia bars,

$$\text{Spacing} = \frac{1000 \times 78.54}{305.05} = 257 \text{ mm}$$

Provide additional steel @ 10 mm @ 250 mm c/c.

3. A rectangular R.C. slab 2 m. × 3 m. is simply supported along shorter edges such that clear distance between the supporting walls is 2.7 m. The slab is 15 cm thick and reinforced with 16 mm dia mild steel bars spaced at 25 cm centre to centre at effective cover of 25 mm along longer edges and with 10 mm dia bars along shorter edges spaced at 25 cm centre to centre. Concrete used is M15 grade for which permissible stresses in bending, shear (nominal) and bond are 50, 3 and 6 kg/cm² respectively. Permissible tensile stress in mild steel = 1400 kg/cm². Modular ratio = 19. Calculate the maximum safe intensity of load that the slab can carry in addition to its self weight.

Solution: Span of slab is lesser of



(*a*) clear span + effective depth

$$= 3.0 - 0.3 + 0.125 = 2.825 \text{ m}$$

(*b*) c/c of support = 3.0 m

so, effective span of slab is 2.825 m

The main steel provided is 16 mm dia @ 250 mm c/c

$$A_{st} = \frac{1000 \times \frac{\pi}{4} \times (16)^2}{250} = 804.25 \text{ mm}^2$$

∴ Moment capacity,

$$M = A_{st}.\sigma_{st} jd$$

Given $m = 19$, $k = \dfrac{1}{1 + \dfrac{\sigma_{st}}{m\sigma_{cbc}}} = 0.4$,

$$j = 1 - \frac{0.4}{3} = 0.867$$

$$\therefore \quad M = 804.25 \times 140 \times 0.867 \times 125$$

$$= 12.2 \times 10^6 \text{ Nmm}$$

$$= 12.2 \text{ kNm per meter width}$$

But, maximum bending moment $= \dfrac{w.l^2}{8}$

where, w = total load coming on slab per m².

$$\therefore \quad \frac{w.l^2}{8} = 12.2$$

$$\text{or} \quad \frac{12.2 \times 8}{(2.825)^2} = 12.232 \text{ kN/m}^2$$

Self weight of 150 mm, thick slab

$$= 0.15 \times 15 = 3.75 \text{ kN/m}^2$$

∴ Safe intensity of load that can be carried by the given slab

$$= 12.232 - 3.75 = 8.482 \text{ kN/m}^2$$

4. Design an isolated square footing of uniform thickness for a R.C.C. column 400 mm x 400 mm in cross section. Unfactored load transmitted by column to fooling is DL = 450 kN and LL = 300 kN. Allowable bearing pressure on soil = 180 kN/m*. Depth of foundation below G.L =1m. Unit weight of soil = 20 kN/m². The main reinforcement in column is 16 mm dia. bars - 8 Nos. Concrete grade : M 20 for column and M 15 for the footing.

Steel grade : Fe 415.

Solution: Area of footing

Unfactored load transmitted by column

$$= DL + LL = 450 + 300 = 750 \text{ kN}.$$

Estimated weight of footing slab

$$= 1 \times 1 \times 0.36 \times 25 = 9 \text{ kN/m}^2$$

Weight of soil above the footing slab

$$= 1 \times 1 \times 0.64 \times 20 = 12.8 \text{ kN/m}^2$$

Total, $w_s = 21.8$ kN/m²

Allowable bearing pressure on soil = 180 kN/m²

Hence, area of footing, $A \geq \dfrac{750}{180 - 21.8}$

i.e. 4.74 m²

Length of side $\geq \sqrt{4.74}$ *i.e.* 2.177 m.

Adopt a square footing of size 2.18 m × 2.18 m.

Net upward soil reaction

Factored load transmitted by the column

$$P_u = 1.5 \text{ DL} + 1.5 \text{ LL}$$

$$= 1.5 \times 450 + 1.5 \times 300$$

$$= 1125 \text{ kN}.$$

Net upward soil reaction,

$$q = \frac{1125}{2.18 \times 2.18} = 236.72 \text{ kN/m}^2$$