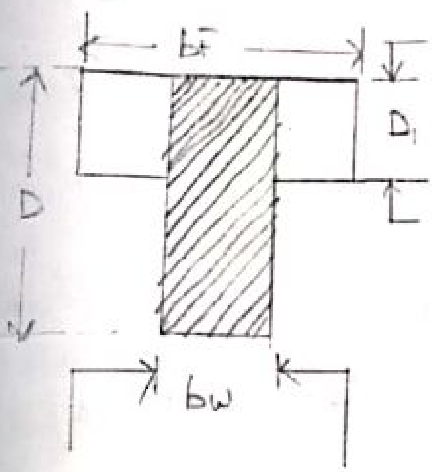
STRUCTURAL DESIGN

CL-5 ANALYSIS AND DESIGN of T. BEAM (LSM)

FLANGED BEAMS :- A.



A Tee beam (or) Lu beam can be considered of a rectangular beam ($b_w \times D$) and a flange of area ($b_f - b_w$) $\times D_f$

position of N-axis :-

The neutral axis may be
(a) Inside the flange (b) lies at the bottom of the flange. (c) lies in the web.

(1) Let us assume the N-A lies at the bottom of the flange ∴ Total compression = F_{tc} = 0.36 f_{ck} b_f D_f.

Total Tension = F_{ts} = 0.87 f_y A_{st}

(2) If ; $F_{tc} > F_{ts}$; N-A lies is flange

$F_{tc} = F_{ts}$; N-A lies at bottom of flange

$F_{tc} < F_{ts}$; N-A lies in the web.

Case-1 :- If N-A lies in the flange ($x_u < D_f$)

(a) for a single reinforced beam.

(i) total compression = Total Tension.

$$0.36\, f_{ck}\, b_f\, x_u = 0.87\, f_y\, A_{st}.$$

$$\therefore x_u = [(0.87\, f_y\, A_{st}) / 0.36\, f_{ck}\, b_f]$$

Scanned with CamScanner

(ii) calculation of mu :-

(i) for UR section :- $m_u = 0.87 \cdot f_y A_{st} (d - 0.42 x_u)$

$M_u = 0.36 f_{ck} b_f (x_u)(d - 0.42 x_u)$

(ii) for Bal ~~reinf~~ (OR) section:-

m_u limit $= 0.36 f_{ck} b_f\, x_{u\max} (d - 0.42 x_{u\max})$

m_u, limit $= 0.87 f_y A_{st\,lim} (d - 0.42 x_{u\max})$

(b) for a doubly Reinforced beam :-

(i) $(m_u - m_{u\,limit}) = f_{sc} A_{sc} (d - d')$

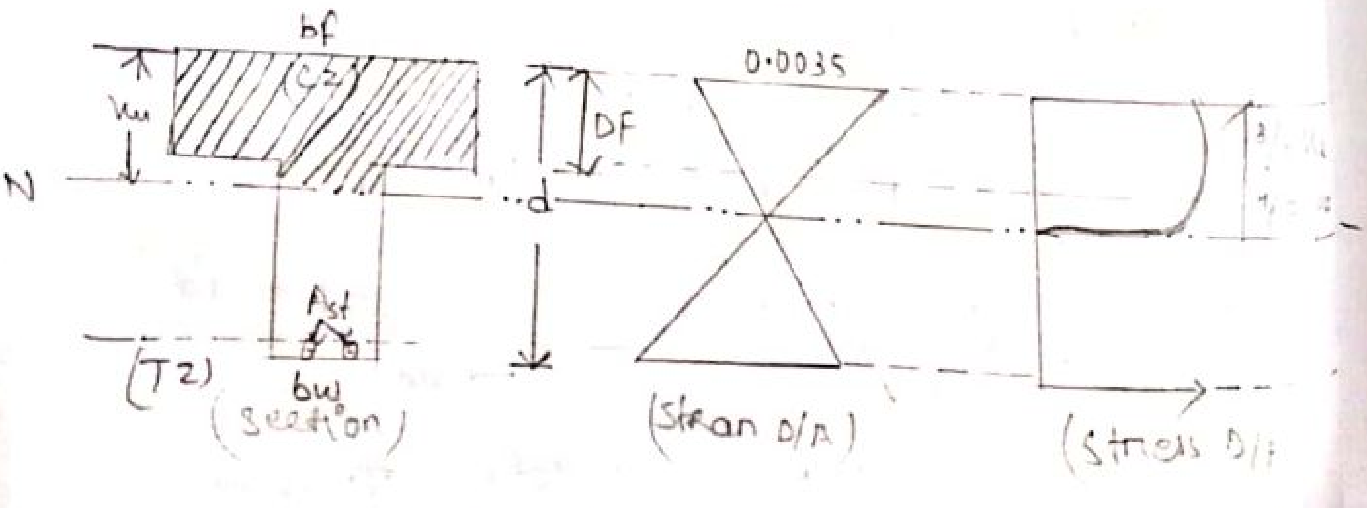
(ii) $A_{st1} = A_{st.lim}$

(iii) $A_{st2} = \left[\dfrac{A_{sc} f_{sc}}{0.87 f_y}\right]$

(iv) $A_{st} = (A_{st1} + A_{st2})$

case -2 :- N-A lies in web ($x_u > D_f$)

(a) section is a balanced one :-



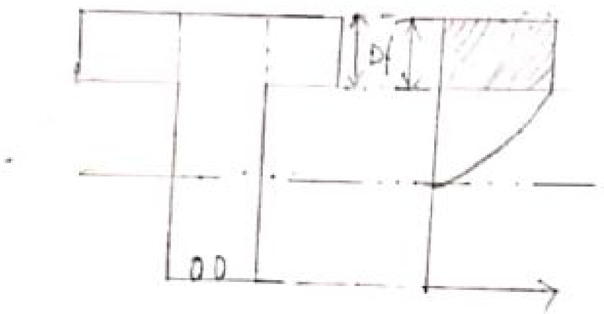
Note :-

(1) If ms is used ; the stress are unform in concrete upto a depth 3/7 x_u 0.53d = 0.227 (d).

(2) If fe 415 is used ; 3/7 (0.48d) = 0.206 (d).

(3) If fe 500 is used ; 3/7 (0.46d) = 0.197 (d)

4 (1) is $\left(\frac{D_f}{d}\right) \leq 0.2$ then the depth of flange (D_f) is less then the depth the rectangular ratio of stres block



Total tension
= 0.87 $f_y A_{st}$

Total compression $= 0.36 f_{ck} b_w x_{umax} + 0.446 f_{ck} (b_f - b_w) D_f$.

$$M_u]_{lim, T} = \left[\left\{0.36 f_{ck} b_w x_{umax} (d - 0.42 x_{umax})\right\} + \left\{(b_f - b_w) D_f \times 0.446 f_{ck} \left(d - \frac{D_f}{2}\right)\right\}\right]$$

$$A_{st}]_{lim} = \frac{0.36 f_{ck} b_w x_{umax} + 0.446 f_{ck} (b_f - b_w) D_f}{0.87 (f_y)}$$

(II) if $\left(\frac{D_f}{d}\right) > 0.2$;

The rectangular portion of stress block in this case is assumed to be equals to y_f.

Where $y_f = (0.16 x_u + 0.65 D_f)$ but $> D_f$

Total Tension = 0.87 $f_y A_{st}]_{lim}$.

Total compression $= 0.36 \cdot f_{ck} b_w x_{umax} + 0.446 f_{ck} (b_f - b_w) y_f$

$$M_u]_{lim,T} = 0.36 f_{ck} b_w x_{umax}(d - 0.42 x_{umax}) + 0.446 f_{ck}(b_f - b_w) \times y_f \left(d - \frac{y_f}{2}\right)$$

$$A_{st}]_{lim} = \left[\frac{0.36 f_{ck} b_w x_{umax} + 0.446 f_{ck}(b_f - b_w) y_f}{0.87 f_y}\right]$$

(b) Section is under reinforced :-

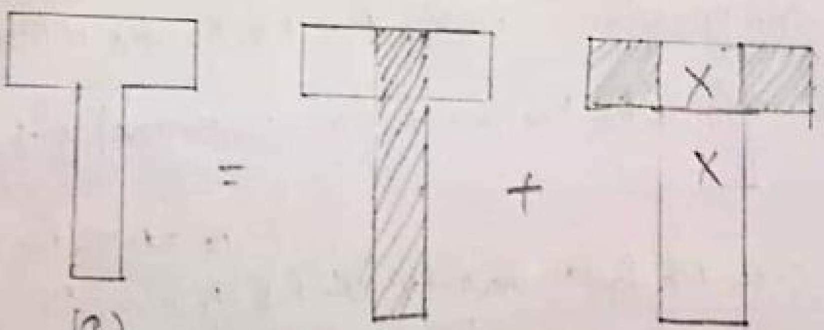
(I) $D_f \le 3/7\, x_u$:-

Total tension $= 0.87 f_y A_{st}$

Total compression $= 0.36 f_{ck} b_w x_u + 0.446 f_{ck} (b_f - b_w) D_f$

$x_u = ?$

$$M_u = [0.36 f_{ck} b_w x_u (d - 0.42 x_u) + 0.446 f_{ck}(b_f - b_w)] \times \left[D_f \left(d - \frac{D_f}{2}\right)\right]$$



(a) (b) (c)

for given m_u, The steel area A_{st} for sec (a)

$= (A_{st1}$ for (b) $+ A_{st2}$ for c)

$\therefore M_u = M_u]_1 + M_u]_2$

& $A_{st} = A_{st}]_1 + A_{st}]_2$

for (c) Section :-

$$A_{st}]_2 = \left[\frac{0.446 f_{ck} (b_f - b_w) D_f}{0.87 f_y}\right]$$

$$Mu]_2 = 0.446\, f_{ck} (b_f - b_w)\, D_f \left[d - \frac{D_f}{2}\right]$$

for (b) section):-

$$mu]_1 = m_u - mu]_2$$

$$P_t]_1 = \frac{50 f_{ck}}{f_y} \left[1 - \sqrt{1 - R_u}\right]$$

$$R_u = \left[\frac{4.6}{f_{ck}} \cdot \frac{Mu]_1}{bd^2}\right]$$

$$\therefore Ast]_1 = \frac{P_t]_1\, bd}{100}$$

(II) $D_f > 3/7\, x_u$:-

* Here we have to take y_f is stead of D_f

(i) Total tension $= 0.87 f_y\, A_{st}$

(2) Total compression $= 0.36 f_{ck}\, b_w\, x_u + 0.446 f_{ck} (b_f - b_w) y_f$

$$y_f = (0.15 x_u + 0.65 D_f)$$

(3) Equating ① and ② $x_u = ?$

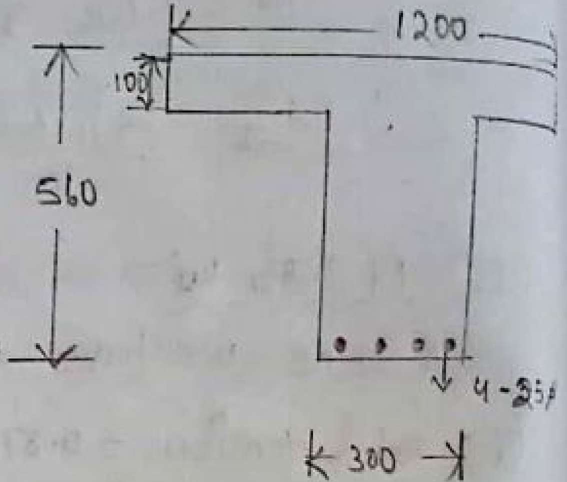
(4) $m_u = \left[0.36 f_{ck}\, b_w\, x_u (d - 0.42 x_u) + 0.446 f_{ck} (b_f - b_w)\right] \times y_f \left[d - \frac{y_f}{2}\right]$

Problem :-

A tee beam of effective flange width 1200 mm thickness of slab 100 MM, width of rib 300 MM and effective depth of 560 MM is reinforced with 4 no. 25 diameter bars. Calculate the factored moment of resistance. The materials are m20 grade concrete and HYSD reinforcement of grade fe 415.

Solution :-

given data :-



$d = 560$ MM

$b_f = 1200$ MM

$d_f = 100$ MM

$d_w = 300$ MM

$f_{ck} = 20$

$f_y = 415$

$A_{st} = 4 \times 491 = 1964$ MM^2

$F_{tc} = 0.36\, f_{ck} \cdot b_f\, D_f$

$= 0.36 \times 20 \times 1200 \times 100 \times 10^{-3}$

$= 864$ KN

$F_{ts} = 0.87\, f_y\, A_{st}$

$= 0.87 \times 415 \times 1964 \times 10^{-3}$

$= 709$ KN

$F_{tc} > F_{ts}$

∴ Neutral axis lies in flange.

Equating for forces

Total compression = total tension.

$0.36\, f_{ck}\, b_f\, x_u = 0.87 \cdot f_y\, A_{st}$

$0.36 \times 20 \times 1200\, x_u = 0.87 \times 415 \times 1964$

$8640\, x_u = 709102$

$x_u = 82.02$ MM < 100 MM

$x_{u\,max} = 0.48 \cdot d = 0.48 \times 560 = 268.8$ MM.

$x_u < x_{u\,max}$.

Section is under-reinforced.

$M_u = 0.87\, f_y\, A_{st}\, (d - 0.42\, x_u)$

$= 0.87 \times 415 \times 1964\, (560 - 0.42 \times 82.07) \times 10^{-6} =$

$= 372.65$ KNm.

Alternatively

$M_u = 0.36\, f_{ck}\, b_f\, x_u\, (d - 0.42\, x_u)$

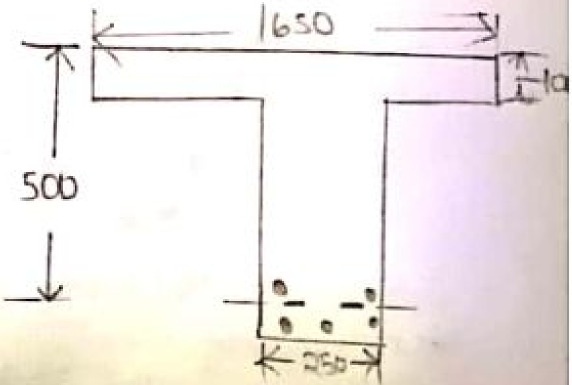
$= 0.36 \times 20 \times 1200 \times 82.07\, (560 - 0.42 \times 82.07) \times 10^{-6}$

$= 372.65$ KNm.

Problem:-

A ~~tee~~ beam as shown in fig is subjected to a factored moment of 400 KNM Design the steel reinforcement for flexure. The materials are M20 grade concrete HYSD reinforcement of grade fe415.

Solution:-



given data:-

$b_f = 1650$ MM

$b_w = 250$ MM

$D_f = 100$ MM

~~d~~ = 500 MM

$f_e = 415$

$f_y = 20$ MM

we assume here that the section will be singly reinforced.

Alternatively, we may first find out $M_{u,limit}$ for the section.

$$\frac{b_f}{b_w} = \frac{1650}{250} = 6.6$$

$$\frac{D_f}{d} = \frac{100}{500} = 0.2$$

$$\frac{M_{u,limit}}{f_{ck} b_w d^2} = 0.588$$

$$M_{u,limit} = 0.588 \times 20 \times 250 \times 500^2 \times 10^{-6} = 735 \text{ kNm.}$$

$$M_u < M_{u,limit}.$$

$$z = d - \frac{D_f}{2} = 500 - 50 = 450 \text{ MM}$$

Approximate $A_{st} = \frac{400 \times 10^6}{0.87 \times 415 \times 450} = 2462 \text{ MM}^2$

Provide 5-25 # $A_{st} = 5 \times 491 = 2455 \text{ mm}^2$

The approximate design is now checked.

To find Level arm

$$F_{tc} = 0.36 f_{ck} b_f D_f = 0.36 \times 20 \times 1650 \times 100 \times 10^{-3} = 1188$$

$$F_{ts} = 0.87 f_y A_{st} = 0.87 \times 415 \times 2455 \times 10^{-3} = 886.4 \text{ kN.}$$

$$F_{tc} > F_{ts}.$$

Neutral axis lies in flange

$$0.36 f_{ck} b_f x_u = 0.87 f_y A_{st}$$

$$0.36 \times 20 \times 1650 x_u = 0.87 \times 415 \times 2455$$

$$x_u = 74.61 \text{ MM} < 100 \text{ MM}$$

$$x_{u,max} = 0.48 d = 0.48 \times 500 = 240 \text{ MM}$$

$$x_u < x_{u,max}$$

section is under-reinforced.

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

$$= 0.87 \times 415 \times 2455 (500 - 0.42 \times 74.61) \times 10^{-6}$$

$$= 415.4 \text{ KNM} > 400 \text{ KNM.}$$

Alternatively.

$$M_u = 0.36 f_{ck} b_f x_u (d - 0.42 x_u)$$

$$= 0.87 \times 20 \times 1650 \times 74.61 (500 - 0.42 \times 74.61) \times 10^{-6}$$

$$= 415.4 \text{ kNm.}$$

Problem:-

Data:-

$d = 560$, $b_f = 1200$ MM

$b_w = 300$ MM

$D_f = 100$ MM

$A_{st} = 5$ of 25 mm $\phi = 5 \times \left(\frac{\pi \times 25^2}{4}\right) = 2455$ mm^2

M20 grade of concrete.

ϕ HYSD fe 415 - steel

Requirement:-

factored moment $M_U = ?$

Solution:-

(A) To find position of N-A

$F_{tc} = 0.36 f_{ck} b_f D_f = 0.36 \times 20 \times 1200 \times 100 \times 10^{-3}$

$F_{ts} = 0.87 f_y A_{st} = 0.87 \times 415 \times 2455 = 886.4$ KN

$F_{tc} < F_{ts}$ $\therefore$ N-A Lies in WEB

(B) Assume $D_f > 3/7 x_u$.

$\therefore y_s = 0.15 x_u + 0.65 D_f = 0.15 x_u + 0.65 (100)$

$y_s = 0.15(x_u) + 65$

(C) Total compression

$= 0.36 f_{ck} b_w x_u + 0.446 f_{ck} (b_f - b_w) y_f$

$= 0.36 (20) (300) x_u + 0.446 f_{ck} (1200 - 300) y_f$

$= 0.36(20)(300) x_u + 0.446 f_{ck} (1200 - 300)(0.15 x_u + 65)$

$= 3364 \cdot 2 (x_u) + 521820.$

(D) Total tension

$= 0.87 f_y A_{st} = 0.87 (415)(2455) = 886378$

Equating (C) and (D) we get.

$x_u = 108.36\,MM$

$\therefore \frac{3}{7} x_u = 3(108.36)/7 = 46.44\,mm < D_f$

(F) $x_{u\,max} = 0.48(d) = 0.48(560)$ safe $= 268.8\,Mm$

$\therefore x_u < x_{u\,max}$

(F) $y_f = 0.15 \times 108.36 + 65 = 81.25\,MM$

$\therefore M_u = 0.36\, f_{ck}\, b_w\, x_u\, (d - 0.42\, x_u) + 0.446\, f_{ck}\, (b_f - b_w) f_y \left(d - \frac{y_f}{2}\right)$

$$= \left\{\left[0.36(20)(300)(108.36)(560 - 0.42 \times 108.36)\right] + \left[0.446(20)(1200 - 300)\,81.25\left(560 - \frac{81.25}{2}\right)\right]\right\} \times 10^{-6}$$

$= 459.2\,KN \cdot mt$

Ans. The factored moment

$M_u = 459.2\,KN\text{-}mt.$

Ch-6 Slabs :- ANALYSIS AND DESIGN OF SLAB AND STAIR CASE (LSM)

Introductory :-

Slabs are plate elements having the depth much smaller in its span and width.

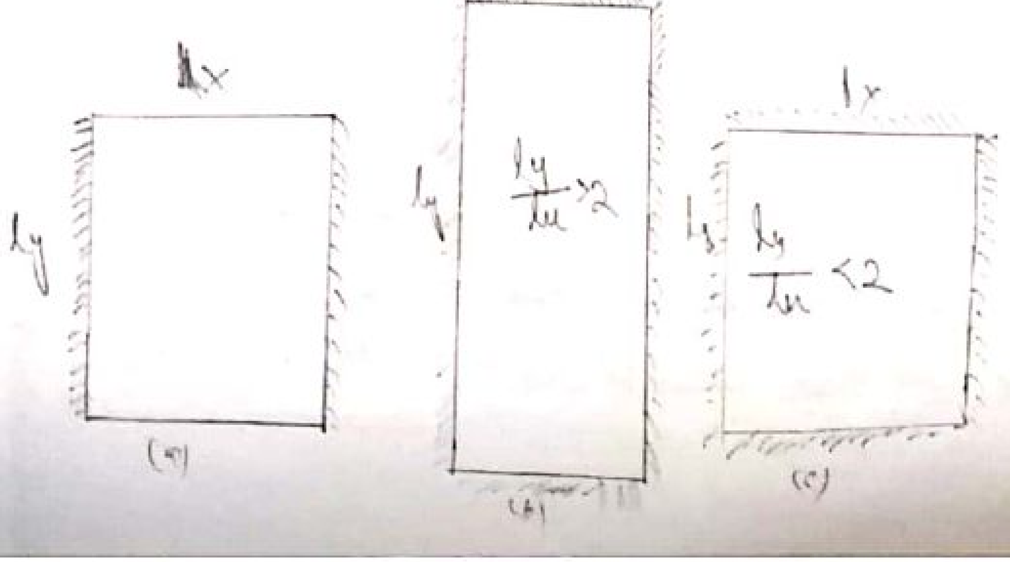
They usually carry a uniformly disstributed load and form the or roof of the building. Like beams, slabs also may be simply supported. cantilever continuous depending upon the support condition. They are classified according the system of supports used as under.

1. One-way spanning slabs
2. Two-way spanning slabs
3. flat slabs supported directly on columns without beams
4. Grid slabs
5. circular and other shapes.
6. Ribbed and waffle slabs.

These are briefly discussed as follows.

1. One-way spanning slabs :-

The slabs supported on two opposite supports is a one-way spanning slab. In short, a slab which transfers its load on one of the set of two opposite edge supports is called one-way slab.



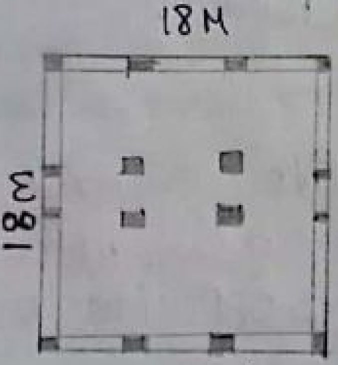
The slab where $\frac{l_y}{l_x} \geq 2$, is called one-way slab provided that it is suppoted on all four edges. is one-way by vintome of Provision of supports although $l_y > 2l_x$:

② Two-way spanning slab:-

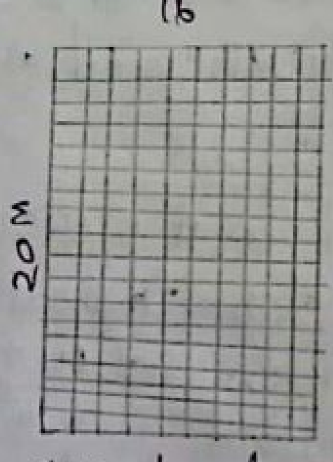
From the above discussion, it is clear that if the slab is supported on all four edges and if $l_y \leq 2l_x$, the tendency of the slab it is bend in both directions are called two-way spanning slabs.

(i) The slab shall be supported on all four edges.

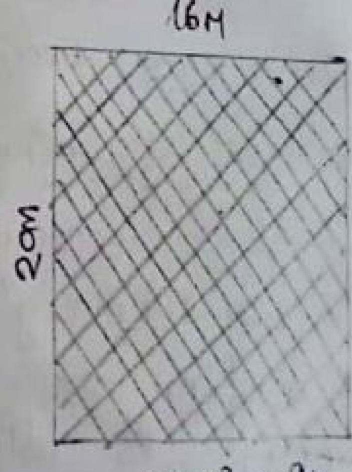
(ii) $l_y < 2l_x$ or $\frac{l_y}{l_x} < 2$



(a) Flat slab



(b) Rectangular grid



(c) Diagrid

Thickness of slab

$$\frac{\text{span}}{\text{effective depth}} = $$

for cantilever - 7

simply supporte - 20

simply supported continuous - 26

* For one way slab modification factor = 1.5

Effective span of a slab:- Page No - 34 (Clause No - 22.2)

(i) clean span + effective depth or
center to center distance.

Reinforcement for slab :- (Page No - 48 clause No - 26.5.2.1)

(i) min^m reinforcement for mild steel should n't be less than (K) 0.15% of gross area.

(ii) min^m reinforcement for ~~tor~~ steel should't be less than (K') 0.12% of gross area.

Diameter of the main bar :- (Page No - 48, clause No - 26.5.22)

→ The dia of bar shall n't be exceed thickness of slab
The dia of main bar may be 8mm or 10MM when Fe415 steel is used.

The dia may be 10mm or 12MM when Fe 250 steel is used.

max^m spacing of main bar :- (Page No-46 (26.3.3 B))

(i) spacing of main bar may n't exceed
(a) 3 times of the effective depth of the slab
(b) 300MM } which is small

min^m spacing of main bar :-
spacing of bar shall n't be less than 75 MM.

Distribution reinforcement:-

minm spacing length:-

The steel area not less than

(a) 0.12% of grass sectional of area of fe 415 steel.

(b) 0.15% grass sectional area for fe 20 steel

maxm spacing of distubution bar

Shall not exceed

(a) 5 times of the effective depth

(b) 300 MM

Diameter of the distrubution bar:-

→ Generally 8MM for fe415 steel

→ 6mm to 8mm for fe250 steel

Problem:-

A simply supported one-way slab of effective spa 4 m is supported on masonry walls of 230 Mm thickness design the slab. Take live equal to 2.5 KN/m2 and floor finish equal to 1 KN/m2. The meterials are m20 grad concrete and HYSD reinforcement of grade fe 415.

Solution:-

Assuming 0.35 per cent steel. a trial depth can be found out by using deflection criteria

Service stress = 0.58 fy = 0.58× 415 = 240 N/mm²

modification factor 8-2 is 1-4.

Permissible $\frac{span}{d}$ ratio $= 1.4 \times 20 = 28$

$d_{required} = \frac{4000}{28} = 142.9$ mm

$D = 142.9 + 15$ (cover) $+ 5$ (assume to # bar)

$= 162.9$ mm.

Assume an overall depth = 170 mm

self weight $= 0.17 \times 25 = 4.25$ kN/m^2

floor finish $= 1.00$ kN/m^2

Live load $= 2.50$ kN/m^2

7.75 kN/m^2

factored ~~mom~~ load $= 7.75 \times 1.5 =$ ~~16~~ 11.6 kN/m

maximum moment $= 11.6 \times \frac{4^2}{8} = 23.2$ kN/m.

maximum shear $= 11.6 \times \frac{4}{2} = 23.2$ kN.

Design for flexure.

$d = 170 - 15 - 5 = 150$ mm

$\frac{M_u}{bd^2} = \frac{23.2 \times 10^6}{1000 \times 150 \times 150} = 1.03$

$P_t = 0.299$.

$A_{st} = \frac{0.299 \times 1000 \times 150}{1000} = 449$ mm^2.

Provide 10 mm # @ 170 mm c/c = 462 mm^2.

Half the bars are bent out $0.1\, l = 400$ mm and remaining bars provide 231 mm^2 area.

$\frac{100 A_s}{bD} = \frac{100 \times 231}{1000 \times 170} = 0.136 > 0.12$

Distribution steel $= \frac{0.12}{100} \times 1000 \times 170 = 204$ mm^2.

maximum spacing $= 5 \times 100 = 800$ or 450 mm. ie. 450 mm

~~check for shear~~. provide 8 mm @ 230 mm c/c = 217 mm^2

For bars at support . $d = 150$ mm

$A_s = 231$ mm2

$$\frac{100 A_s}{bD} = \frac{100 \times 231}{1000 \times 150} = 0.154$$

$\tau_c = 0.28$ N/mm2

for 150mm thick slab

. $k = 1.26$

$k\tau_c = 1.26 \times 0.28 = 0.353$ N/mm2

Actual shear stress $= \frac{24 \times 10^3}{1000 \times 150} = 0.16$ N/m3 $< k\tau_c$ ... Safe

Check for development length.

consider $L_0 = 8\#$ for continuing bars

$A_s = 231$ mm2

Assume . $M_{u1} = 0.87 f_y A_{st} (d - 0.42 x_u)$

$x_u = x_{u\,max} = 0.48\, d$.

$M_{u1} = 0.87 \times 415 \times 231 \times (150 - 0.42) \times 0.53 \times 1.50 \times 10^{-6}$

$= 9.73$ kN/m

Note :- Different formulae are used for calculations mul. In different worked examples. Note that the lever arm of balanced section is assumed as actual lever arm. This is conservative, however used for speedy calculation. If the check is not satisfied . one may exactly find out the value of mul. Refer to example 10-2

$V_u = 24$ kN

$$1.3 \frac{M_1}{V} + L_0 \geq L_d$$

$$1.3 \times \frac{9.73 \times 10^6}{24 \times 10^3} + 8\# \geq 47\#$$

39 $\phi \geq 527$

$\phi \leq 13.51$ mm

check for deflection

Basic $\frac{\text{span}}{d}$ ratio = 20

$$P_t = \frac{100 \times 462}{1000 \times 150} = 0.308$$

Service stress $= 0.58 \times 415 \times \frac{449}{462} = 234$ N/mm²

modification factor = 1.42

permissible $\frac{\text{span}}{d}$ ratio = 20×1.42 = 28.4

actual $\frac{\text{span}}{d}$ ratio $= \frac{4000}{150} = 26.66 < 28.4$.

check for cracking.

maximum spacing permitted for main reinforcement

= 3×160 = 480 MM or 300 MM ie, 300 mm.

Actual spacing = 170 MM

maximum spacing permitted for secondary reinforcement

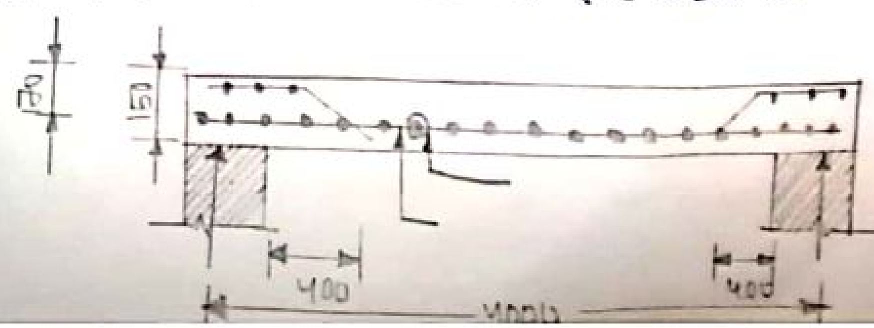
= 5×160 = 800 MM or 450 MM. i.e, 450 MM

Actual spacing = 230 MM.

for tying the bent bars at top. Provide 8 mm ϕ @ 230 mm c/c.

simply supported and cantilever slabs.

Sketch: The cross-section of the slab is.



STAIRS:-

Stair slabs:-

→ The staircase is used to give an acess to different floors of a building. ~~there~~

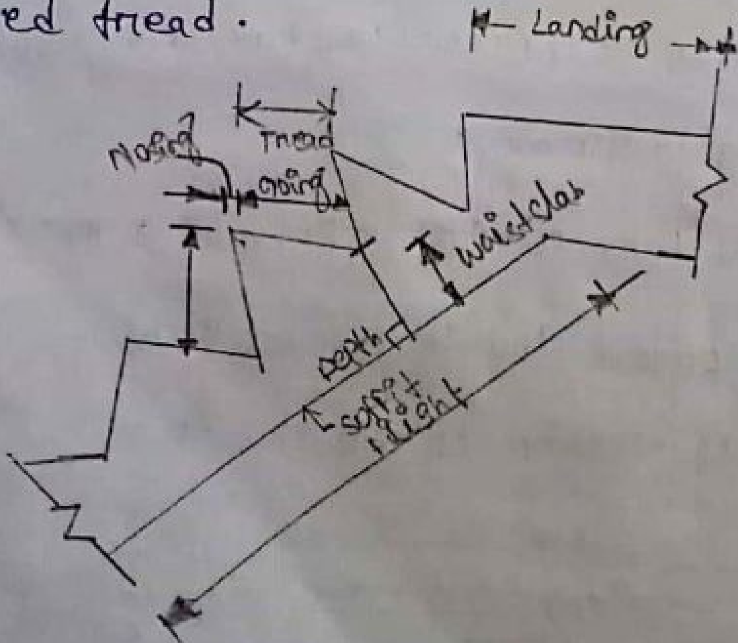
→ The inclined slab of a stair is known as flight of stair while the straight portion other than the floor level is known as the landing.

→ while going on flight, one travels vertically. The landing is provided midway either to turn the position and/or to relax while going up.

→ The vertical height of the step is known as rise and the available horizontal distance on a step is known as tread.

→ Tread consists of going and nosing.

→ The net horizontal distance used in plan is known as going and additional nosing is provided to get the required tread.



The stairs are grouped into following two types according to their use.

(1) Private stairs

(2) common stairs.

Private stair:-

is for the use of one family and common stair is for the use of more than one family.

common stairs:-

of a commercial building, theatre halls, school buildings etc.

classification of stairs:-

there are many types of staircases provided in building. structurally speaking. the types of staircases are two.

(1) spanning longitudinaly e.g. between floor beams of one floor to other floor or one floor to landing beam

(2) spanning in transverse direction each step is spanning betn two parallel beams or cantilevered from one beam or wall.

According to arrangement of stair, some popular stairs are described below.

(1) Straight stair:-

This is a long narrow staircase that may or may no have landing. These stairs are popular in building. where the stairs are kept outside the building.

~~Example~~.

(2) Dog-legged stair:-

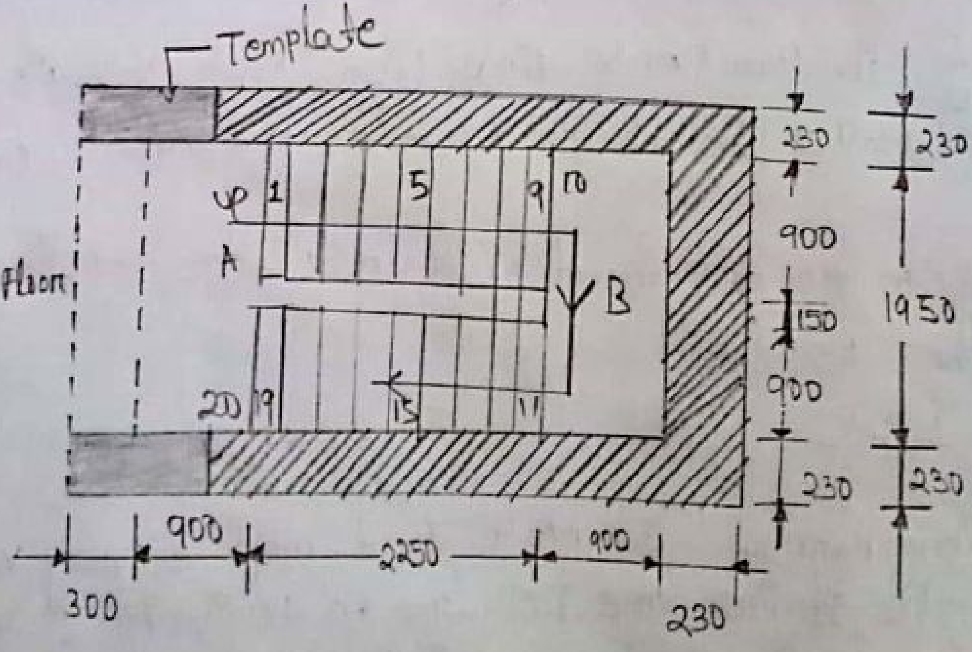
This consists of two separate opposite flights as The clear distance betⁿ two flights in plan may be zero to 150 MM. landing is provided where the two flights meet.

DESIGN PROCEDURE:-

1. Assume the thickness of waist slab and landing
2. Design for Landing with checks.
3. Design for (waist slab) flight. with checks.
4. sketch (Details (of Reinforcement)

Problem:-

The arrangement of a dog-legged staircase in a residential building is shown in fig. Rise of step is 160 mm and tread is 250 MM. Nosing is not provided. The materials are grade M20 concrete and HYSD reinforcement of grade Fe 415. Design the staircase.



Step-1

① Assumed thickness of waist slab as 150 MM

LANDING 'A' & 'B' DESIGN :-

(i) Self load $0.16 \times 25 \, kN/m^3 = 3.75 \, kN/m^2$

(ii) L. Load $3 \, kN/m^2 = 3.00 \, kN/m^2$

(iii) F. finish @ $1 \, kN/m^2 = 1.00 \, kN/m^2$

Total load $P = 7.75 \, kN/m^2$

$\therefore$ factored load $= w_u (P \times 1.5) = (7.75 \times 1.5)$

$= 11.63 \, kN/m^3$

Effective span $= (d + l) = (130 + 1950)$

$= 2080 \, MM = 2.08 \, mt$

$\approx 2.1 \, mt$

$$\left[d = (D - c_c - \phi/2) = (150 - 15 - 5) = 130 \, MM \right]$$

consider a 1mt width

$$m_u = \frac{w_u (l_e^2)}{8} = \frac{11.63 \times (2.1)^2}{8} = 6.41 \, kN.mt$$

Check for 'd' $= \sqrt{m_u / Q_{ub}}$

$P_t = 0.127$ from formula

$$\therefore A_{st} = \frac{0.127 \times 1000 \times 120}{100} = 152 \, mm^2$$

$$min^m \text{ steel} = \frac{0.12}{100} \times b \times d = \frac{0.12}{100} \times 1000 \times 120$$

$= 180 \, mm^2$

Providing 8mm ϕ spacing $= 270 \, mm \, c/c$

max^m spacing $= 3 \times 120 = 360 \, MM$

Check for shear (τ_v) :-

$$V_u = \frac{2.1}{2} \times 11.63 = 12.21 \text{ kN}$$

$$\tau_u = \frac{12.21 \times 10^3}{1000 \times 120} = 0.102 \text{ N/mm}^2$$

$\tau_c = 0.28 \text{ N/mm}^2$

$\tau_v < \tau_c$ Hence safe

Check for deflection :-

$$\frac{\text{Span}}{d} = 20$$

$$\frac{100 A_{st}}{bd} = 0.154$$

service stress $= 0.58 \times f_y \times \frac{A_{st}\, R}{A_{st}\, P} = 198 \text{ N/mm}^2$

modification factor $= 1.8$

$\therefore \frac{\text{Span}}{d}$ permitted $= 1.8 \times 20 = 36$

Actual ratio $= \frac{2100}{120} = 17.5 < 36$ safe.

Design of Height :-

Loads :-

inclined length of slab for one step

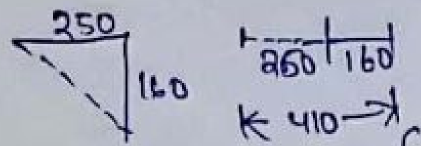
$= \sqrt{250^2 + 160^2}$ 295.8 MM

D = 150 mm

$\therefore$ self load $\frac{296.8}{250} \times 0.15 \times 25$ kN/m^3

$$\left\{ \frac{\sqrt{T^2 + R^2}}{T} \right\} \times D \times f_u$$

floor finish $= \frac{410}{250} \times 1$



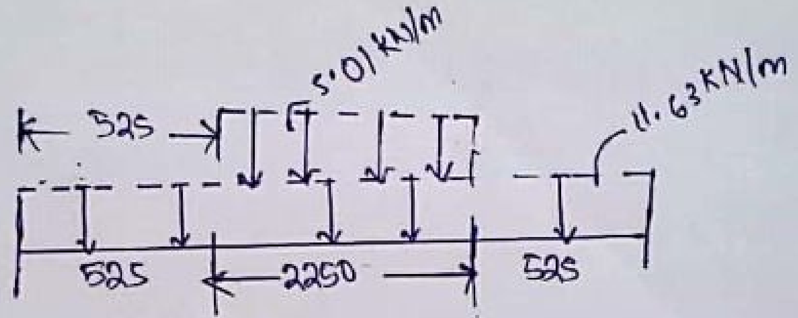
wt of step $= \left\{\dfrac{\frac{0+160}{2}}{1000}\right\} \times 25 = 2.0\ kN/m^2$

L.L $= \longrightarrow = 3.0\ kN/m^2$

$11.09\ kN/m^2$

$\therefore P_u = 11.09 \times 1.5 = 16.64\ kN/m^2$

load / unit length $= 16.64$ kN/mt.



$R_A = R_m = 24.83$ kN

Total $= 16.464$ kN/m.

Ch-7 COLUMNS :- DESIGN OF AXIALLY LOADED COLUMNS AND FOOTINGS (LSM)

Introduction :-

A compression member whose effective length is more then 3 times of it's lateral dimension then it is called as a column.

If the length is less than 3 times its lateral dimenssion then it is called as pedestal.

(1) column are designed with reinforcements.

(2) Pedestals are designed with out reinforcements (or) minimum reinforements.

(3) columns are usually takes axial loads and moments.

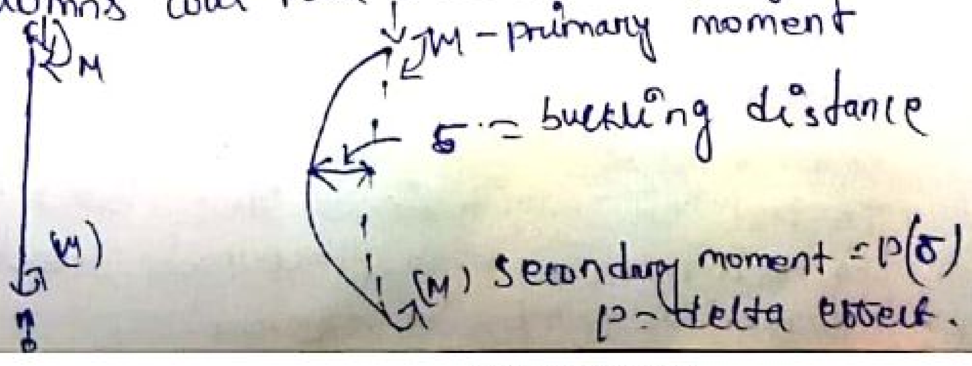
(4) shapes of columns are usualy square, rectangular circular, tee, Ell or cruciform (swastik)

Classification :-

(i) Long and short column (2) Braced column and un-braced (3) No-sway & way col^m (4) Tied spiral and composite col^m.

Long and short column :-

under a compressive load all columns have a tendency to buckle the colums with small length will fail primerily due to material failure where as the long columns will fail due to buckling.



Short colume:-

If slenderness ratio l/r and ley 12 then it is called as short column.

Reinforcement requirements:-

IS-456-2000 ; cl-26.53 Page-48.

Minimum eccentricity:-

The criteria regarding minimum eccentricity is at in clause 25.4 of IS-456-2000. Accordingly, all column shall be designed for num eccentricity equal to the unsupported length of column/500 plus lateral axis subject to a minimum of 20 MM. where biaxial bending is considered sufficient to ensure that eccentricity exceeds the minimum about one axis at time.

The columns may be clasified based on different crit as follows:-

(i) Braced and unbraced columns
(2) No-sway and sway columns
(3) Tied, spiral and composite columns
(4) short and Long columns.

Braced and unbraced columns:-

The columns in a building are classified as brace or unbraced according to the method applied to provi lateral stability of the building.

composite columns:-

~~under the action of a compressive load, all column~~

~~have a tendency to buckle and~~

Insted of longitudinal steel bars, it the column is reinforced with structural steel shape. it is known as composite columns.

No-sway and sway columns:-

The columns is the given floor in the given direction are classified as no-sway and sway columns depending on the amount of the sway under the action of horizontal forces. Accordingly, the columns of the floor having limited value of the sway are called no-sway columns.

Tied columns:-

a typical tied column where the transverse ties are used as lateral restraint. The function of the transverse ties are as follows.

(i) when the load commences on the column, they support the longitudinal ~~the column~~ bars so that they are not buckled, so that the load carrying capacity of the column is increased.

(ii) when horizontal forces like wind and earthquake are acting on column, these reinforcement also resist the shear force.

(iii) They support the longitudinal bars from being displaced during the construction time.

② Spiral columns:-

when a continuous bar or heavy wire is wrapped around the longitudinal bars in the form of a helical spiral. the column is referred to as a spiral column. Although, the spirals can be used for all shapes of the columns. they are particularly used for circular columns.

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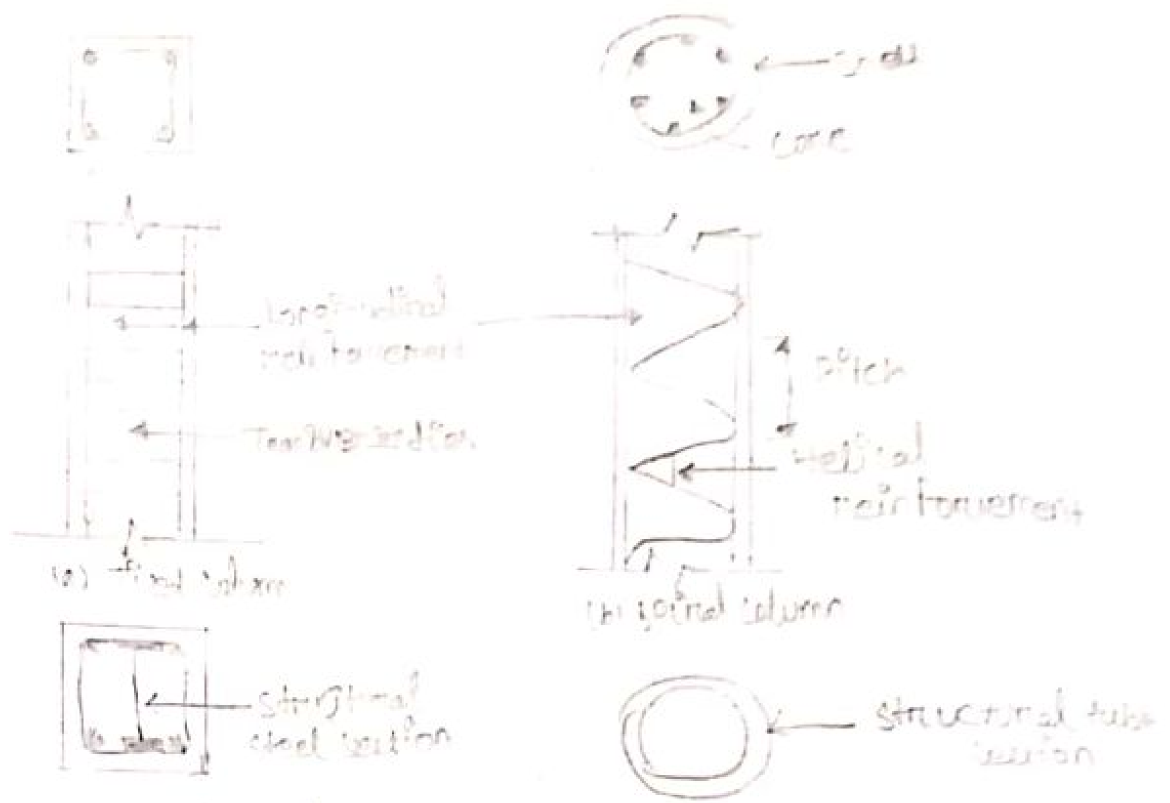


(a) tied column
(b) spiral column
(c) composite column with I section
(d) composite column with steel tube

Longitudinal :-

(1) C/S Area of longitudinal Rein. ≮ 0.8% of gross area of colm

(2) C/S Area of longitudinal Rein ≯ 6.0% " "

(3) minimum No of bars (i) for ▭ or ▱ → 4 nos
(ii) for ○ → 6 NOS.

(4) min^m ϕ of bars = 12 mm

(5) For helical reinforced col^n min^m No of longitudinal steel = 6. NOS.

(6) spacing of longitudinal bars ≯ 300 MM
(cracking Reqm)

(B) Transverse :-

usual

(C) Cover

(c) Cover

$c \not< 40\,MM$
$\not< \phi$

Tiebars

$\phi\ max = 10\,MM$

$\phi\ min = 6\,MM$

for columns of $b \times D = 200 \times 200$ (or) less

Cover = 25 MM

$\phi\ max = 12\,MM$

ECCENTRICITY (e):- IS : 456 : 2000 ; CT : 25.4 Pg - 42

(1) All column should be designed for minimum e'min

$$e_{min} = \left[\frac{\text{unsupported}}{600} + \frac{\text{Latral dimantion}}{30}\right] \not< 20\,MM$$

Cl (39) Page - 70-72

(2) If calculated e min is > 20 mm then e min should be considered.

* Note of (1) and (2)
we should take the max value of [e min calculated and 20MM]

(3) minimum 'e' ≤ 0.05 (D)

for ms - Design stress at 0.002 strain = 0.87 fy

Fe 415 - Design " = 0.79 fy

Fe 500 - " " = 0.75 fy .

* But code adopts critical value = 0.78 fy.

Pure Axial load carrying capcity of column :-

$$P_{uz} = 0.446\ f_{ck}\ A_c + 0.75\ f_y\ A_{sc} \quad (\text{Exact})$$

$$= 0.45\ f_{ck}\ A_c + 0.75\ f_y\ A_{sc} \quad (\text{Approx})$$

[Ac → Area of conc. Asc → Area of steel] considering the eccentricity min.

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

* [i.e this load carrying capacity of col^m reduces by 10%]

However $e_{min} > 0.05(D)$, the Design should be taken for moments.

$$P_u = 0.4 f_{ck}\left[A_g - \frac{PA_g}{100}\right] + 0.67 f_y\left(\frac{PA_g}{100}\right)$$

A_g = gross area of c/s of col^m

P = % of reinforcement

Ex:- 1

Design data:-

$b \times D = 230 \times 350$ MM, $P_u = 1500$ KN, $l = 3.2$ m, $e = e_{min}$

Requirements:-

$M_u]_x = ?$ $M_u]_y = ?$

Solution:-

(i) $e_x = \frac{3200}{500} + \frac{350}{30} = 18.06$ mm < 20 MM

$\therefore e_x = 20$ MM; $0.05D = 0.05 \times 350 = 17.5$ mm $< e_x$

(2) $e_y = \frac{3200}{500} + \frac{230}{30} = 14.06$ mm < 20 MM

$\therefore e_y = 20$ MM; $0.05b = 0.05 \times 230 = 11.6 < e_y$.

$\therefore M_{ux} = M_{uy} = 1500 \times 0.02 = 30$ kN·mt.

Ex:-2 Design data:-

$b \times D = 400 \times 400$ MM

$A_{sc} = 4 \times \left(\frac{\pi \times 25^2}{4}\right) = 1964$ mm^2

$e_{min} = 0.05 \times D = 0.05 \times 400 = 20$ m

M20, Fe 415 grade.

solution

$A_{sc} = 19.64 mm^2$, $A_c = (400 \times 400) - A_{sc}$

$= (160000 - 1964)$

$= 158036\ mm^2$

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

$= 0.4 (20)(158036) + 0.67 (415)(1964)(10^{-3})$

$= 1810.4$ kN

$P_u = 1810.4$ kN

Ex-3

Data

Short col^m $P_u = 1900$ kN , square col^m

Assume ; $l_{min} < 0.05D$

Grades M20 , Ms

Requirment :- $A_{sc} = ?$ $A_c = ?$ size of col^m = ?

solution:-

* 1. Assume % of steel = 0.8% of Ag

$\therefore A_{sc} = \frac{0.8}{100} [A_g] = 0.008 A_g$

$A_c = \left[A_g - \frac{0.8}{100} A_g\right] = 0.992 A_g$

* 2 $P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$

$1900 \times 10^3 = 0.4 \times 20 \times 0.992 A_g + 0.67 \times 250 \times 0.0$

$\therefore A_g = 204830\ mm^2$

$\therefore b (or) D = \sqrt{A_g}$

$= \sqrt{204830} = 453$ mm

Adopt ; $b \times D = 450 \times 450$ mm.

3 Determine Asc from Pu and Ag calculated

$$1900 \times 10^3 = 0.4 (20)(450 \times 450 - Asc) + 0.67 \times 250 \times$$

$\therefore Asc = 1756 \ mm^2$

Provide 6 Nos of 20 mm ϕ (Ast 20 = 314 mm²)

$$\therefore Asc = 6 \times \frac{\pi \times 20^2}{4} \quad 21884 \ mm^2$$

Here the spacing becomes more than 300mm

$\therefore$ provide 4-20mm ϕ and 4 of 16 mm ϕ (ast = 201 mm

$Asc = 4(314) + 4(201) = 2060 \ mm^2$

lateral ties = use 6 mm ϕ bars.

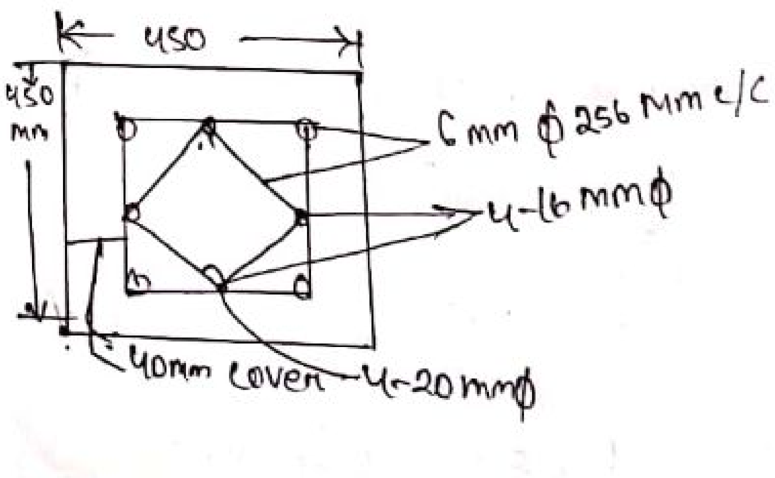
spacing at lateral ties

minimum of (1) minimum of b/D = 450

(2) 16 ϕ min = 16×16 = 256 mm

(3) 300 mm.

$\therefore$ provide spacing 256 mm c/c.



Types of footings :-

Some of the common footing usual in general building construction are as follow.

(1) continuous wall footing
(2) Isolated footing
(3) combined footing
(4) strap footing
(5) strip footing
(6) Raft foundation
(7) pile foundation.

(1) continuous wall footing :-

A footing that supports a continus long masonry or R.C.C wall is known as continuous footing.

(2) Isolated footing :-

An individual footing under a single column is known as isolated footing.

(3) combined footing :-

A footing that supports a group of columns is known as combined footing.

(4) Strap footing :-

It a combined footing is required due to site conditions. but the distance between the columns is large. a strap footing is used for economy refer.

(5) strip footing :-

If a number of footings in a line are to be combined a strip footing is used. Differential settlement can be minimized by using such footings.

(6) Raft Foundation :-

A single slab or a slab beam footing that covers the entire stratum beneath the entire area of the super-structure is known as a mat or raft footing

⇒ when safe bearing capacity of soil is low and colum carry heavy loads, then footings of a group of colum or all the columns in structure are combined to form a raft foundation.

(7) Pile Foundation :-

If good soil is available at a higher depth (more then 3 m) below the ground level. pile foundat are economical. piles transfer the loads from colums to the hard soil by end bearing and to the surrounding soil by friction.