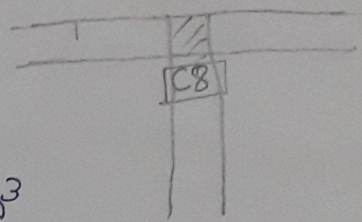
Name- Tushar Bhakat, Roll- 200104113

Group - 9

Column design



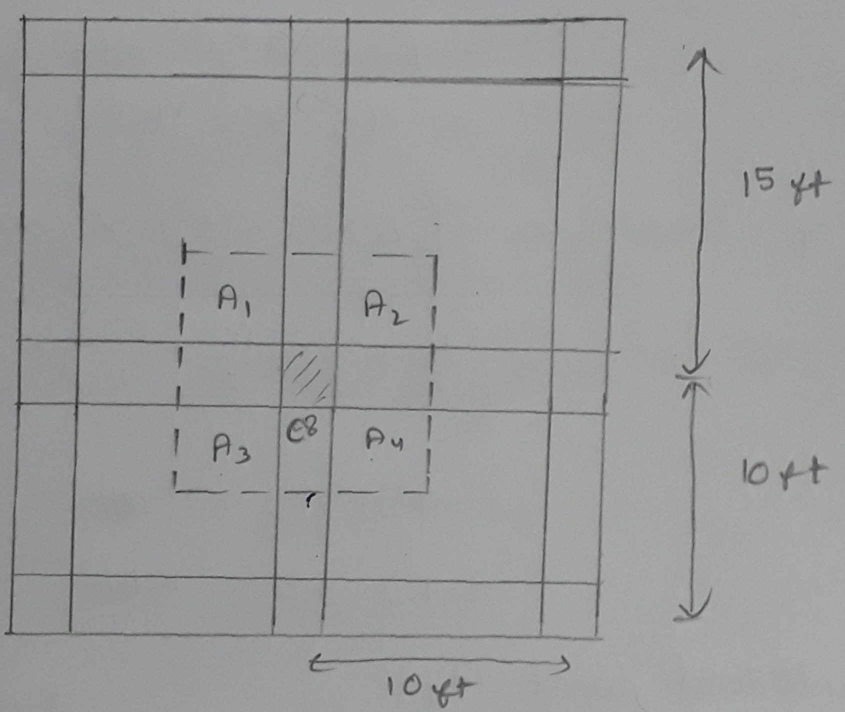
Step 1 :- Load Calculation :-

Unit weight of concrete = 25 KN/m^3

Super imposed DL = 3.5 KN/m^2.

Wall load = 7.50 KN/m^2 $\quad$ f_y = 500 MPa

LL = 2.75 KN/m^2



$$\text{Area} = A_1 + A_2 + A_3 + A_4$$

$$= \left\{\left(5 \times \frac{15}{2}\right) + \left(5 \times \frac{15}{2}\right) + (5 \times 5) + (5 \times 5)\right\}$$

$$= 125 \text{ ft}^2$$

$$= 11.6125 \text{ m}^2$$

Self wt. of the slab

= (Area) × (Unit wt. of concrete) × (Slab th.)

$= 11.6125 \times 25 \times 125 \times 10^{-3}$

= 36.29 KN

Superimposed dead load = Area × 3.5 = 40.6..

~~= 127.015 KN~~

~~= 40.644 KN~~

LL = 11.6125 × 2.75

= 31.934 KN

Total wall load = (wall length) × (thickness load)

$= \left(\frac{10}{2} + \frac{10}{2} + \frac{10}{2} + \frac{10}{2}\right) ft \times 7.5$ KN/m

~~= 13.935 KN~~

= 45.72 KN.

Self wt. of beam = (L×B×H) × unit wt. of concrete

$= \left(\frac{10}{2} + \frac{10}{2} + \frac{10}{2} + \frac{10}{2}\right) \times 0.3048 \times \frac{250 \times 350 \times 25}{10^6}$

~~= 43.75 KN~~

= 13.335 KN.

~~Total~~ factored load Wu

= 1.5 × (Self wt. of slab + Superimposed DL + LL + total wall load + Self wt. of beam).

= 1.5 × (36.29 + ~~127.015~~ 40.644 + 31.934 + 45.72 + 13.335)

= 251.8845 KN.

Total factored load = 5 × 251.8845

= 1259.4225 KN

≃ 1260 KN

Step 2 : Moment Calculation.

~~w (kN/m)~~

$w = W_{wall} + W_{beam}$.

In x-direction.

$w_{x_1} = w_{x,wall} + W_{x_1 beam}$

$= 7.5 + \left(25 \times \frac{250 \times 350}{10^6}\right)$

$= 9.6875$ kN/m.

$M_{x_1} = \frac{1}{12} \times w_{x_1} \times (10 \times 0.3048)^2$

$= \frac{1}{12} \times 9.6875 \times (3.048)^2$

$= 7.5$ kNm

$w_{x_2} = 9.6875$

$M_{x_2} = \frac{1}{12} \times w_{x_2} \times (10 \times 0.3048)^2$

$= 7.5$ kNm

$\therefore M_x = 0$

In y-direction,

$w_{y_1} = w_{y,wall} + w_{y_1 beam}$

$= 9.6875$ kN/m

$M_{y_1} = \frac{1}{12} \times w_{y_1} \times (10 \times 0.3048)^2$

$= 7.5$ kNm

$w_{y_2} = 9.6875$ kN/m

$M_{y_2} = \frac{1}{12} \times 9.6875 \times (15 \times 0.3048)^2 = 16.875$ kNm.

$\therefore M_y = M_{y_2} - M_{y_1} = 16.875 - 7.5$

$= 9.375$ KNm.

Let us assume 400 mm × 400 mm column

Limiting moment calculation,

$$M_{u,lim} = 0.36 f_{ck} b d^2 \frac{x_{u,lim}}{d}\left(1 - 0.42 \frac{x_{u,lim}}{d}\right)$$

$b = 250$, $d = 350$

$$M_{u\,lim} = 0.36 \times 20 \times 250 \times (350)^2 \times 0.46 \left(1 - 0.42 \times 0.46\right)$$

~~= 90.85 KNm~~

$= 81.834$ KNm

$$V_u = 1.4\left(\frac{M_u^{As} + M_u^{Bh}}{h_{sr}}\right)$$

$$= 1.4 \times \left(\frac{81.834 + 81.834}{3}\right)$$

$= 76.378$ KN

Step 3 : Nominal C.C = 40 for M20 grade (Cl. 2.6.4.2.1)

Calculate unsupported length = 3000 - beam depth

= 3000 - 378

= 2622 mm

column dimension 400 × 400

$l_{eff} = 0.65 \times 2622 = 1704.3$

$\frac{l_{eff}}{D} = \frac{1704.3}{400} = 4.26 < 12$ short column.

3) Minimum eccentricity,

$e_{min\ x-y} = \frac{l_{unsupp}}{500} + \frac{D}{30}$ (Cl. 25.4 IS 456-2000)

$= \frac{2622}{500} + \frac{300}{30}$

$= 15.24 < 20$ ok.

$e_{min\ x-x} = e_{min-yy} = 20$ mm.

Step 4 :- Minimum BM,

x- direction,

$M_{u min\ x-y} = P_u \cdot e_{min\ xx}$

$= \cancel{1260}\ 1500 \times 20 \times 10^{-3}$

$\cancel{25.2\ KNm}$

$= 30$ KNm

Similarly,

$M_{u min\ y-y} = \cancel{25.2\ KNm}$ 30 KNm.

$M_{u min\ x-x} > M_x\ (0)$

$M_{u min\ y-y} > M_y\ (9.375$ KNm)

so we have to consider only axial loading by considering only 90% of Pu by condition of minimum eccentricity.

Step 5 :- Calculating area of steel

$$1260 = 0.4 f_{ck} A_c + 0.67 f_y A_s$$

$$= 0.4 f_{ck} (A_g - A_s) + 0.67 f_y A_s$$

$$A_g = BD = (0.4)^2 = 0.16\ m^2$$

$$\therefore \overset{1500}{\cancel{1260}} \times 10^3 = 0.4 \times 20 \times (160000 - A_s) + 0.67 \times 500 \times A_s$$

$$\Rightarrow \overset{1500}{\cancel{1260}} \times 10^3 = 1280000 - 8A_s + 335 A_s$$

$$\Rightarrow A_s = \frac{220000}{327} = 672.78\ mm^2$$

Take 16 ϕ bars,

$$A_\phi = \frac{\pi}{4}(16)^2 = 201\ mm^2$$

No. of bars $= \cancel{1280}\ \frac{672.78}{201} = 3.34$

A provided $= 8 \times 201 = \cancel{804\ mm}\ 1608\ mm^2$

Min reinforcement $= 0.8\ \%$ of A_g

$$= 0.8 \times (400)^2$$

$$= 1280\ mm^2$$

A provided > 1280

Tie bars :-

Diameter of tie bar, $\phi_t = \left.\begin{matrix} \frac{\phi_{main}}{4} \\ 6mm \end{matrix}\right\} max$

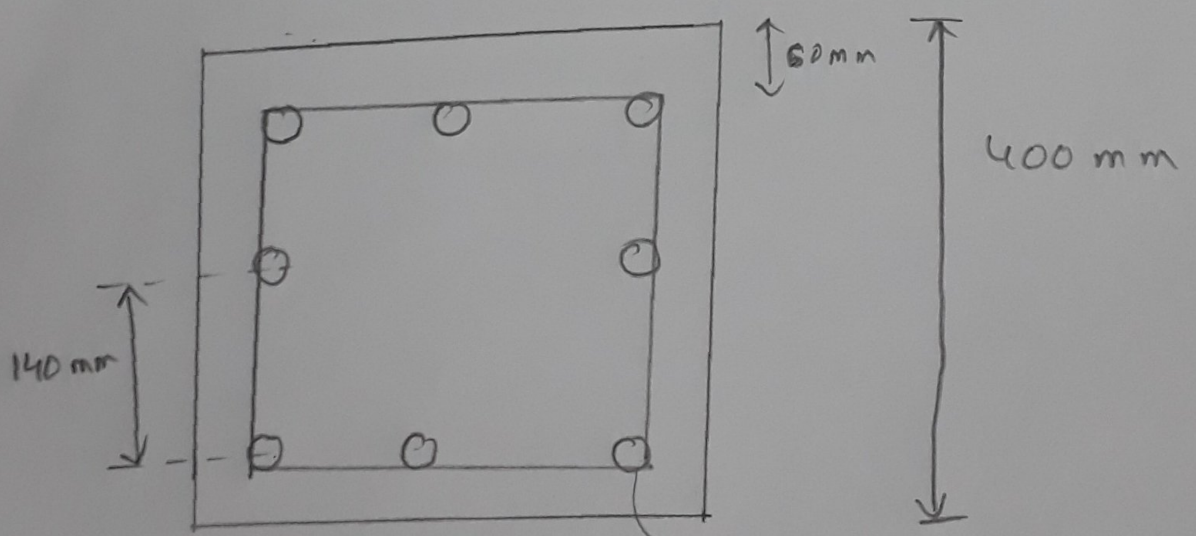
provided $= 6mm\ \phi$ tie bars $= \left.\begin{matrix} 4mm \\ 6m \end{matrix}\right\} min$

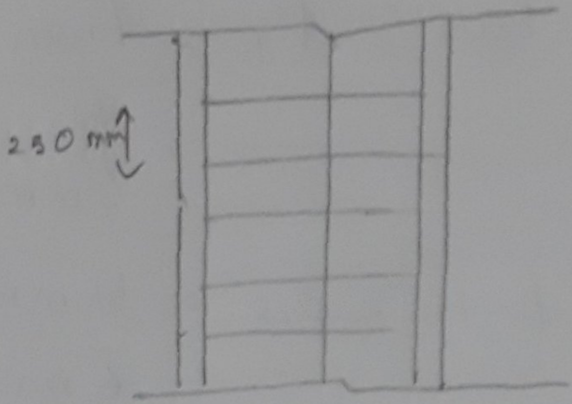
Spacing of tie bars :-

$min \left\{\begin{matrix} LLD = 400\ mm \\ 16\ \phi\ main = 16 \times 16 = 256\ mm \\ 300\ mm. \end{matrix}\right.$

Provided $= 256$ mm.

Detailing :-





front view

clear cover = 44 mm

effective cover

$= 44 + 8 + \frac{16}{2}$

$= 60$ mm

Spacing b/w main bars

= 140 mm.