REINFORCED CONCRETE - II

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A Hand-note On

REINFORCED CONCRETE - II

CE 317

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Topics

Tied column
Circular/spiral column
Flat plate
Flat slab
Two way slab
Two way slab - different form
Retaining wall
Squre footing
Combined footing

CE-317

Tied Column

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090063

USD method:

* $C = C_b = d \times \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_y}$ $\quad \varepsilon_u = 0.003$

$$\varepsilon_y = \frac{f_y}{E_s}$$

* $a = 0.85\, C_b = \beta_1 C_b$

* $f_s' = \varepsilon_u E_s \frac{c - d'}{c} \leq 60 \text{ ksi}$

* $C = 0.85 f_c' a b$

* $P_b = 0.85 f_c' a b + A_s' f_s' - A_s f_s$

$= 0.85 f_c' a b + A_s' f_y - A_s f_s$ [For balanced failure, $f_s' = f_y$]

* $M_b = 0.85 f_c' a b \left(\frac{h}{2} - \frac{a}{2}\right) + A_s' f_s' \left(\frac{h}{2} - d'\right) + A_s f_s \left(d - \frac{h}{2}\right)$

* $e_b = \frac{M_b}{P_b}$

* If $e > e_b$ then tension failure will occur, $f_s = f_y$

If $e < e_b$ then compression " " "

If $e = e_b$ then balanced, $f_s' = f_y$

Ex-8.1 : (a) A 12×20 in column is reinforced with 4#9 bars, one in each corner, $f_c' = 4000$ psi, $f_y = 60$ ksi. Determine the load P_b, moment M_b and corresponding eccentricity e_b for balanced failure. $d' = 2.5''$

Solution: $b = 12''$ $h = 20''$ $d = 20 - 2.5 = 17.5''$ $A_s = 2 \text{ in}^2$ $A_s' = 2 \text{ in}^2$

$$c_b = \frac{\epsilon_u}{\epsilon_u + \epsilon_y} d \qquad \epsilon_y = \frac{f_y}{E_s} = \frac{60}{29000} = 0.0021$$

$$= \frac{0.003}{0.003 + 0.0021} \times 17.5$$

$$= 10.3 \text{ in}$$

$$\boxed{a = \beta_1 c} = 0.85 \times 10.3 = 8.76 \text{ in}$$

$$f_s' = \epsilon_u E_s \frac{c - d'}{c} = 0.003 \times 29000 \times \frac{10.3 - 2.5}{10.3}$$

$$= 65.88 \text{ ksi}$$

But, $f_s' \leq 60$ ksi

$$P_b = 0.85 f_c' ab + A_s' f_s' - A_s f_s$$ [for balanced failure, $f_s = f_y$] ***

$$= 0.85 \times 4 \times 8.76 \times 12 + 2 \times 60 - 2 \times 60$$

$$= 357 \text{ kips}$$

$$M_b = 0.85 f_c' ab \left(\frac{h}{2} - \frac{a}{2}\right) + A_s' f_s' \left(\frac{h}{2} - d'\right) + A_s f_s \left(d - \frac{h}{2}\right)$$

$$= 0.85 \times 4 \times 8.76 \times 12 (10 - 4.38) + 2 \times 60 \times (10 - 2.5) + 2 \times 60 \times (17.5 - 10)$$

$$= 3808 \text{ in-kips}$$

$$e_b = \frac{M_b}{P_b} = \frac{3808}{357} = 10.67 \text{ in} \quad \text{(Ans).}$$

(b) The load and moment for a representative point in the tension failure region of the interaction curve.

Solution:

Let, $c = 5$ in $\quad a = \beta_1 c = 0.85 \times 5 = 4.25$

$$f_s' = \epsilon_u E_s \frac{c-d'}{c} = 0.003 \times 29000 \times \frac{5-2.5}{5}$$
$$= 43.5 \text{ ksi}$$

$$P_n = 0.85 f_c' a b + A_s' f_s' - A_s f_s$$
$$= 0.85 \times 4 \times 4.25 \times 12 + 2 \times 43.5 - 2 \times 60 \qquad \overset{***}{\boxed{f_s = f_y}}$$
$$= 140 \text{ kips}$$

$$M_n = 0.85 f_c' a b \left(\frac{h}{2} - \frac{a}{2}\right) + A_s' f_s' \left(\frac{h}{2} - d'\right) + A_s f_s \left(d - \frac{h}{2}\right)$$
$$= 0.85 \times 4 \times 4.25 \times 12 \times (10 - 2.12) + 2 \times 43.5 \times (10 - 2.5) + 2 \times 60 \times (17.5 - 10)$$
$$= 2916 \text{ in-kips}$$

$$e = \frac{M_n}{P_n} = \frac{2916}{140} = 20.83 \text{ in} > e_b = 10.67 \text{ in}$$

$\therefore$ $e > e_b$ (Tensile failure will occur)

(c) Determine the load and moment for a representative point in the compression failure region.

Solution:

Let, $c = 18''$

$a = \beta_1 c = 0.85 \times 18 = 15.3$ in

$f_s = \epsilon_u E_s \times \boxed{\frac{d-c}{c}}$ *** [since stress in the steel at the left side]

For tension steel

$= 0.003 \times 29000 \times \frac{17.5-18}{18}$

$= -2.42$ ksi

$f_s' = \epsilon_u E_s \times \boxed{\frac{c-d'}{c}}$ → For compression steel

$= 0.003 \times 29000 \times \frac{18-2.5}{18}$

$= 74.92$ ksi

But, $f_s' \leq 60$ ksi

$P_n = 0.85 f_c' ab + A_s' f_s' - A_s f_s$

$= 0.85 \times 4 \times 15.3 \times 12 + 2 \times 60 - 2 \times (-2.42)$

$= 749.08$ kips

$M_n = 0.85 f_c' ab \left(\frac{h}{2} - \frac{a}{2}\right) + A_s' f_s' \left(\frac{h}{2} - d'\right) + A_s f_s \left(d - \frac{h}{2}\right)$

$= 0.85 \times 4 \times 15.3 \times 12 (10 - 7.65) + 2 \times 60 \times (10 - 2.5) + 2(-2.42)(17.5 - 10)$

$= 2330.66$ in-kips $= 194.22$ ft-kips

$e = \frac{M_n}{P_n} = \frac{2330.66}{749.08} = 3.11$ in

(d) Determine the axial load strength for zero eccentricity.

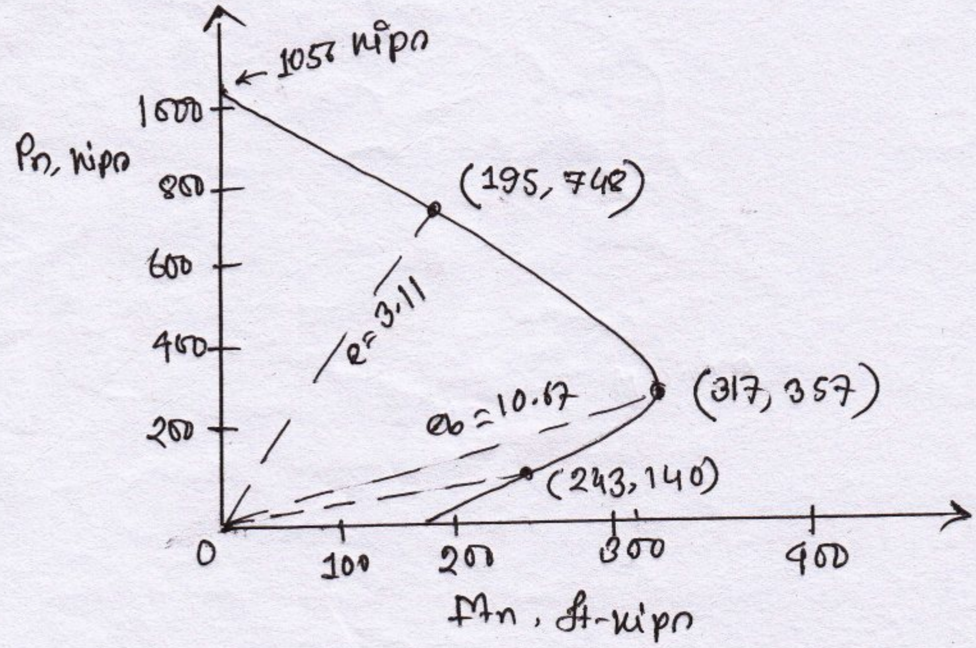
Solution:

$e = 0 \quad c = \infty$

for this case,

$$P_o = 0.85 f_c' A_c + A_{st} f_y$$
$$= 0.85 f_c' bh + A_{st} f_y$$
$$= 0.85 \times 4 \times 12 \times 20 + 4 \times 60$$
$$= 1056 \text{ kips}$$

(e) sketch the strength interaction diagram for the column.



CE-317

Column
(Circular/Spiral)

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☐ Design of circular column: (USD method)

i) Ultimate load, $\boxed{P_u} = 1.2 \times D.L + 1.6 \times L.L$

ii) ρ_g না দেয়া থাকলে 2% ধরে নিতে হবে, $\rho_g = 0.02$

iii) $P_u = \phi \alpha \left[0.85 f_c' A_c + A_{st} f_y\right]$

$= \phi \alpha \left[0.85 f_c' (A_g - A_{st}) + A_{st} f_y\right]$

এখানে,

$\phi = 0.7$ for spiral column, $\phi = 0.65$ for tied column

$\alpha = 0.85$ " " " , $\alpha = 0.80$ for " "

$\boxed{A_g} = ?$ বাহির করবো

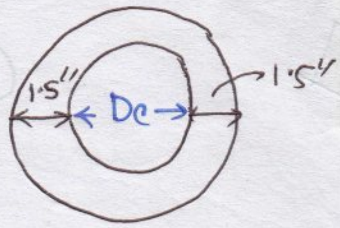
iv) $A_g = \frac{\pi}{4} D^2$ এখান থেকে, $\boxed{D}$ বাহির করতে হবে

v) (iii) নং থেকে $\boxed{A_{st}}$ বাহির করবো

এখানে, কমপক্ষে 6টা বার use করতে হবে (spiral column)

" 4টা " " " " (tied ")

vi) Spiral steel;



Assuming cover = 1.5''

$\therefore$ core cutter diameter, $D_c = D - 2\times 1.5$

Core concrete area, $A_c = \frac{\pi}{4} \times D_c^2$

vii) By ACI code, Minimum spiral steel ratio,

$$\rho_{s1} = 0.45 \frac{f_c'}{f_y}\left(\frac{A_g}{A_c} - 1\right)$$

$$\rho_{s2} = \frac{4 A_{sp}}{S D_c}$$

✻

✻ এখানে, S এর মান এমন বসাতে হবে যেন, $\rho_{s2} > \rho_{s1}$ হয়।

USD method

07, 10

Prob-01: Design a circular column to support a dead load of 250 kips and live load of 150 kips using $f_c' = 3$ ksi and $f_y = 50$ ksi.

Solution:

i) $P_u = 1.2 \times D.L + 1.6 \times L.L$

$= 1.2 \times 250 + 1.6 \times 150$

$= 540$ kips

ii) Let, $\rho_g = 2\% = 0.02$ $\quad A_{st} = \rho_g A_g = 0.02 A_g$

iii) A_g $\quad P_u = \varphi \alpha [0.85 f_c' (A_g - A_{st}) + A_{st} f_y]$

$\Rightarrow 540 = 0.7 \times 0.85 [0.85 \times 3 \times (\cancel{0.02}\, A_g - A_{st}) + A_{st} \times 50]$

$\Rightarrow \cancel{907.56 = 0.051 - 2.55\, A_{st} + 50 A_{st}}$

$\therefore \cancel{A_{st}} =$

$\Rightarrow 907.56 = [0.85 \times 3 \times (\cancel{0.02}\, A_g - 0.02 A_g) + 0.02 A_g \times 50]$.

$\therefore A_g = 259.38$ in^2

iv) $A_g = \frac{\pi}{4} d^2$

$\Rightarrow 259.38 = \frac{\pi}{4} D^2$

$\therefore D = 18.17'' \simeq 18''$

$A_g = \frac{\pi}{4} D^2 = \frac{\pi}{4} \times (18)^2 = 254.47$ in^2

v)

$$P_u = \phi\alpha\left[0.85 f_c' \cdot (A_g - A_{st}) + A_{st} f_y\right]$$

$$\Rightarrow 540 = 0.7 \times 0.85\left[0.85 \times 3 \times (254.47 - A_{st}) + A_{st} \times 50\right]$$

$$\Rightarrow 907.56 = 648.9 + 47.45 A_{st}$$

$$\therefore A_{st} = 5.45 \text{ in}^2$$

$\therefore$ Use 7 #8 bar.

vi) Assuming cover $= 1.5''$

core cutter diameter $D_c = D - 2 \times 1.5''$

$$= 18 - 3 = 15''$$

$$\therefore A_c = \frac{\pi}{4} D_c^2 = \frac{\pi}{4} \times (15)^2 = 176.72 \text{ in}^2$$

vii)

$$\rho_{s1} = 0.45 \frac{f_c'}{f_y}\left(\frac{A_g}{A_c} - 1\right)$$

$$= 0.45 \times \frac{3}{50}\left(\frac{254.47}{176.72} - 1\right)$$

$$= 0.0119$$

$$\rho_{s2} = \frac{4A_s}{s D_c} = \frac{4 \times 0.11}{3 \times 15} = 0.009 \quad \left[\text{Taking, } s = 3''\right]$$

$$\rho_{s1} > \rho_{s2}$$

$$\rho_{s2} = \frac{4 \times 0.11}{2.25 \times 15} = 0.013 \quad \therefore \rho_{s2} > \rho_{s1}$$

$\therefore$ Use #3 bar @ $2.25''$ c/c. (Ans)

USD method

Prob-02: Design a circular spiral column to support a dead load of 200 kips and live load of ~~...~~ 175 kips. Use $f_c' = 3$ ksi, $f_y = 60$ ksi.

Solution:

$P_u = 1.2 \times D.L + 1.6 \times L.L$

$= 1.2 \times 200 + 1.6 \times 175$

$= 520$ kips

Let, $\rho_g = 2\% = 0.02$ $\quad A_{st} = \rho_g A_g = 0.02 A_g$

$P_u = \phi \alpha \left[0.85 f_c' (A_g - A_{st}) + A_{st} f_y\right]$

$\Rightarrow 520 = 0.7 \times 0.85 \times \left[0.85 \times 3 \times (A_g - 0.02 A_g) + 0.02 A_g \times 60\right]$

$\Rightarrow 873.95 = 3.699 A_g$

$\therefore A_g = 236.27 \text{ in}^2$

$A_g = \frac{\pi}{4} D^2$

$\Rightarrow 236.27 = \frac{\pi}{4} D^2 \quad \therefore D = 17.34'' \simeq 17.5''$

$A_g = \frac{\pi}{4} \times (17.5)^2 = 240.53 \text{ in}^2$

$520 = 0.7 \times 0.85 \times \left[0.85 \times 3 \times (240.53 - A_{st}) + A_{st} \times 60\right]$

$\Rightarrow 873.95 = 613.35 + 57.45 A_{st}$

$\therefore A_{st} = 4.54 \text{ in}^2$

$\therefore$ Use 6 #8 bar.

Let, $cover = 1.5''$

$\therefore$ Core cutter diameter, $D_c = D - 2\times1.5$

$= 17.5 - 3 = 14.5''$

$$A_c = \frac{\pi}{4} D_c^2 = \frac{\pi}{4} \times (14.5)^2$$

$$= 165.13 \ in^2$$

$$\rho_{s1} = 0.45 \frac{f_c'}{f_y}\left(\frac{A_g}{A_c} - 1\right)$$

$$= 0.45 \times \frac{3}{60}\left(\frac{240.53}{165.13} - 1\right)$$

$$= 0.0103$$

$$\rho_{s2} = \frac{4A_{sp}}{sD_c} = \frac{4\times0.11}{3\times14.5} = 0.0101$$

$\therefore \rho_{s1} > \rho_{s2}$

$$\rho_{s2} = \frac{4\times0.11}{2.75\times14.5} = 0.0110$$

$\therefore \rho_{s2} > \rho_{s1}$

$\therefore$ Use #3 bar @ $2.75''$ c/c

(Ans)

☐ Design of circular column: (WSD method)

i) $P = D.L + L.L$

ii) $A_{st} = \rho_g A_g$ (ρ_g না দেওয়া থাকলে ২% assume করতে হবে)

iii) $P = 0.25 f_c' A_g + A_{st} f_s$

$A_g = ?$

iv) $A_g = \frac{\pi}{4} D^2 \qquad D = ?$

v) $A_g = \frac{\pi}{4} D^2$

vi) Load carried by concrete, $P_c = 0.25 f_c' A_g$

vii) " " " steel, $P_s = \cancel{A} \cdot P - P_c$

viii) $P_s = A_{st} f_s$

$A_{st} = ?$

ix) Assuming, cover $= 1.5''$

$D_c = D - 2 \times 1.5$

$S_{max} = \frac{D_c}{6}$

According to ACI, $S_{max} = 3''$

x) $\rho_{s1} = 0.45 \frac{f_c'}{f_y} \left(\frac{D^2}{D_c^2} - 1 \right)$

xi) $\rho_{s2} = \frac{4 A_{sp}}{S D_c}$

$\therefore \rho_{s2} > \rho_{s1}$ (OK)

WSD method (Circular ~~Spring~~ Column)

05

Prob-01: Design completely a spiral column to support total 450 kips load, $\rho_g = 0.02$, $f_c' = 3500$ psi, $f_y = 60000$ psi, Use WSD method;

Solution:

$$P = 450 \text{ kips}$$

$$A_{st} = \rho_g A_g = 0.02 A_g$$

$$P = 0.25 f_c' A_g + A_{st} f_s$$

$$\Rightarrow 450 = 0.25 \times 3.5 \times A_g + 0.02 A_g \times 0.4 \times 60$$

$$\therefore A_g = 332.10 \text{ in}^2$$

$$A_g = \frac{\pi}{4} D^2$$

$$\Rightarrow D = \sqrt{\frac{4A_g}{\pi}} = \sqrt{\frac{4 \times 332.10}{3.1416}} = 20.56''$$

Taking $D = 20''$ $\quad A_g = \frac{\pi}{4} \times (20)^2 = 314.16 \text{ in}^2$

Load carried by concrete, $P_c = 0.25 f_c' A_g = 0.25 \times 3.5 \times 314.16$
$= 274.89$ kips

Load carried by steel, $P_s = P - P_c = 450 - 274.89 = 175.11$ kips

Now, $\quad P_s = A_{st} f_s$

$$\Rightarrow 175.11 = A_{st} \times 0.4 \times 60$$

$\therefore A_{st} = 7.3 \text{ in}^2$ $\quad \therefore$ Use 8 #9 bars.

Spacing of spiral steel:

Assuming, cover = 1.5''

Core cutter diameter = $D_c = D - 2 \times 1.5$

$= 20 - 2 \times 1.5$

$= 17''$

$$S_{max} = \frac{D_c}{6} = \frac{17}{6} = 2.83 \simeq 2.75''$$

According to ACI, $S_{max} = 3''$

$\therefore$ Use Spacing = 2.75''

Check:

$$\rho_{s1} = 0.45 \frac{f_c'}{f_y} \left(\frac{D^2}{D_c^2} - 1 \right)$$

$$= 0.45 \times \frac{3.5}{60} \times \left(\frac{20^2}{17^2} - 1 \right)$$

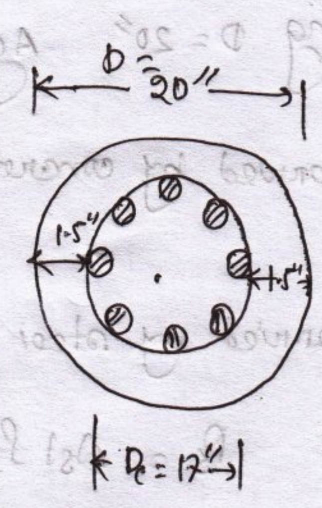
$$= 0.01$$

$$\rho_{s2} = \frac{4 A_{sp}}{s D_c} = \frac{4 \times 0.11}{2.75 \times 17} = 0.009$$

$$\rho_{s2} = \frac{4 \times 0.2}{2.75 \times 17} = 0.017$$

$\therefore \rho_{s2} > \rho_{s1}$.

Use #4 bar @ 2.75'' c/c.



(Ans)

04, 03, 00

Prob-02: Design circular spiral column to support a concentric working load of 500 kips using $f_c' = 3$ ksi, $f_y = 60$ ksi $\rho_g = 0.03$.

Solution:

$P = 500$ kips

$A_{st} = \rho_g A_g = 0.03 A_g$

$P = 0.25 f_c' A_g + A_{st} f_s$

$\Rightarrow 500 = 0.25 \times 3 A_g + 0.03 A_g \times 0.4 \times 60$

$\therefore A_g = 340.14 \text{ in}^2$

$A_g = \frac{\pi}{4} D^2 \Rightarrow D = \sqrt{\frac{4 A_g}{\pi}} = \sqrt{\frac{4 \times 340.14}{3.1416}}$

$= 20.81''$

Taking, $D = 21''$ $\quad A_g = \frac{\pi}{4} \times (21)^2 = 346.36 \text{ in}^2$

Load carried by concrete,

$P_c = 0.25 f_c' A_g = 0.25 \times 3 \times 346.36 = 259.77$ kips

Load carried by steel,

$P_s = P - P_c = 500 - 259.77 = 240.23$ kips

$P_s = A_{st} f_s$

$\Rightarrow 240.23 = A_{st} \times 0.4 \times 60$

$\therefore A_{st} = 10 \text{ in}^2$ Use 10 #9 bars

Spacing of spiral steel,

Assuming, cover = 1.5"

Core cutter diameter, $D_c = D - 2 \times 1.5$

$= 21 - 3$

$= 18"$

$$S_{max} = \frac{D_c}{6} = \frac{18}{6} = 3"$$

According to ACI, $S_{max} = 3"$

We may provide, $S = 3"$

Check;

$$\rho_{s_1} = 0.45 \frac{f_c'}{f_y}\left(\frac{D^2}{D_c^2} - 1\right)$$

$$= 0.45 \times \frac{3}{60}\left(\frac{21^2}{18^2} - 1\right)$$

$$= 0.0081$$

$$\rho_{s_2} = \frac{4 A_{sp}}{s D_c} = \frac{4 \times 0.2}{3 \times 18} = 0.015$$

$\therefore \rho_{s_2} > \rho_{s_1}$. (ok).

090063

9

Flat Plate

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i) **Thickness:**

Ln = Interior panel এর বড় length – Length of column

a) $t = 5''$

b) $t = \dfrac{Ln(800 + 0.005 f_y)}{36000}$

ii) **Load calculation:** $D.L = \dfrac{t}{12} \times 150$ L.L

$\omega = 1.2 \times D.L + 1.6 \times L.L$

iii) **Punching shear check:** $d = t - 0.75 - \dfrac{4}{8 \times 2}$

$Vu = \{\text{Interior panel area} - (\text{column} + d)\ \text{area}\} \times \omega$

$V_{all} = 4\phi\sqrt{f_c'}\ b_o d$
↓
perimeter

$\therefore V_{all} > Vu$ (OK)

iv) **Moment calculation:**

$d_l = t - 0.75 - \dfrac{4}{8 \times 2}$ ← c.c

$d_s = d_l - \dfrac{4}{8}$

Long $M_o = 0.125\ \omega l_2 l_n^2$
l_2 ↓ short length

M_o – short $= 0.125\ \omega l_2 l_n^2$
l_2 ↓ long length

v) Depth check:

$$\rho = \rho_{max} = 0.75\rho_b$$

$$= 0.75 \times 0.85\beta_1 \times \frac{f_c'}{f_y} \times \frac{87}{87+f_y}$$

$$R = \rho f_y \left(1 - 0.59\rho \frac{f_y}{f_c'}\right)$$

$$d = \sqrt{\frac{M}{\phi R b}} \longrightarrow b = \frac{s}{2} \rightarrow \text{shorter length}$$

$$a = \frac{A_s f_y}{0.85 f_c' b}$$

vi) Table :

vii) working diagram:

06 | 08

Problem : A 19'×22' interior panel of a flat plate floor is to carry LL = 130 psf. Design the panel assuming column diameter = 20". f_c' = 3 ksi f_y = 60 ksi.

Solution:

$$a = \sqrt{\frac{\pi}{4}D^2} = \sqrt{\frac{\pi}{4}\times(20)^2} = 17.72'' = 1.48'$$

$$L_n = 22 - 1.48 = 20.52' = 246''$$

i) $t = 5''$

ii) $$t = \frac{L_n(800 + 0.005 f_y)}{36000} = \frac{246(800 + 0.005\times 60000)}{36000}$$

$$= 7.5''$$

Let us take, $t = 12''$

Load calculation:

$$D.L = \frac{t}{12}\times 150 = \frac{12}{12}\times 150 = 150 \text{ psf}$$

$$\therefore W = 1.2\times D.L + 1.6\times L.L = 1.2\times 150 + 1.6\times 130$$

$$= 388 \text{ psf}$$

Punching shear check:

$$d = 12 - 0.75 - \frac{4}{8\times 2} = 11''$$

$$V_d = \left\{19\times 22 - \frac{\pi}{4}\left(\frac{20+11}{12}\right)^2\right\}\times 388\times 10^{-3} = 160.2\text{ k}$$

$$V_{allowable} = 4\phi\sqrt{f_c'}\times b_o d$$

$$= 4\times 0.75\times\sqrt{3000}\times\pi(20+11)\times 11$$

$$= 176 \text{ k}$$

$\therefore$ $V_{allowable} > V_d$ (OK)

Moment calculation :

$$d_l = 12 - 0.75 - \frac{4}{8\times 2} = 11''$$

$$d_s = 11 - \frac{4}{8} = 10.5''$$

$$M_o - long = 0.125\, \omega\, l_2\, l_n^2 = 0.125 \times 388 \times 19 \times (22-1.48)^2 \times 10^{-3}$$
$$= 388 \text{ k-ft}$$

$$M_o\text{- short} = 0.125 \times 388 \times 22 \times (19-1.48)^2 \times 10^{-3} = 328 \text{ k-ft}$$

Depth check :

$$\rho = \rho_{max} = 0.75\, \rho_b$$
$$= 0.75 \times 0.85 \times \beta_1 \frac{f_c'}{f_y} \times \frac{87}{87+f_y}$$
$$= 0.75 \times 0.85 \times 0.85 \times \frac{3}{60} \times \frac{87}{87+60}$$
$$= 0.016$$

$$R = \rho f_y \left(1 - 0.59 \rho \frac{f_y}{f_c'}\right)$$
$$= 0.016 \times 60 \left(1 - 0.59 \times 0.016 \times \frac{60}{3}\right) = 0.779 \text{ ksi}$$

$$d = \sqrt{\frac{M}{\phi R b}} = \sqrt{\frac{388 \times 12}{\cancel{0.75 \times 0.7}\ 0.9 \times 0.779 \times 9.5 \times 12}}$$
$$\boxed{b = \frac{s}{2} = \frac{19}{2}}$$
$$= 7.63''$$

$\therefore$ Provided, $d = 11''$ i.e. (ok)

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{A_s \times 60}{0.85 \times 3 \times b} = 23.53 \frac{A_s}{b}$$

Direction	Longer $\left(M_n = \frac{M_u}{\phi} = \frac{388}{0.9} = 431 \text{ k-ft}\right)$				Shorter $\left(M_n = \frac{328}{0.9} = 364 \text{ k-ft}\right)$			
Strip	column (9.5′)		Middle (9.5′)		column (9.5′)		Middle (12.5′)	
Sign	−	+	−	+	−	+	−	+
K	0.49	0.21	0.16	0.14	0.49	0.21	0.16	0.14
$M_n = m_n K$ k-ft	211.19	90.51	68.96	60.34	178.36	76.44	58.24	50.96
d(in)	11	11	11	11	10.5	10.5	10.5	10.5
$b(in) = \frac{s}{2}$	114	114	114	114	114	114	$L - \frac{s}{2} =$ 150	150
$a = 23.53 \frac{A_s}{b}$	$0.21 A_s$	$0.21 A_s$	$0.21 A_s$	$0.21 A_s$	$0.21 A_s$	$0.21 A_s$	$0.16 A_s$	0.16
$A_s = \frac{M_n}{\cancel{\phi} f_y (d - \frac{a}{2})}$	4.47	1.86	1.41	1.23	3.94	1.65	1.25	1.09
$A_{s\,min} = 0.0018bt$	2.46	2.46	2.46	2.46	2.46	2.46	3.24	3.2
No of #4 bars	23	19	13	13	20	13	17	17
$S_{av} = \frac{b}{n}$	4.96	~~11.4~~ 8.8	~~14.25~~ 8.8	~~16.29~~ 8.8	5.7	~~12.67~~ 8.8	~~16.29~~ 8.8	~~12~~ 8
$S_{max} = 24$ in	24	24	24	24	24	24	24	24

∴ $S_{av} < S_{max}$ for all cases

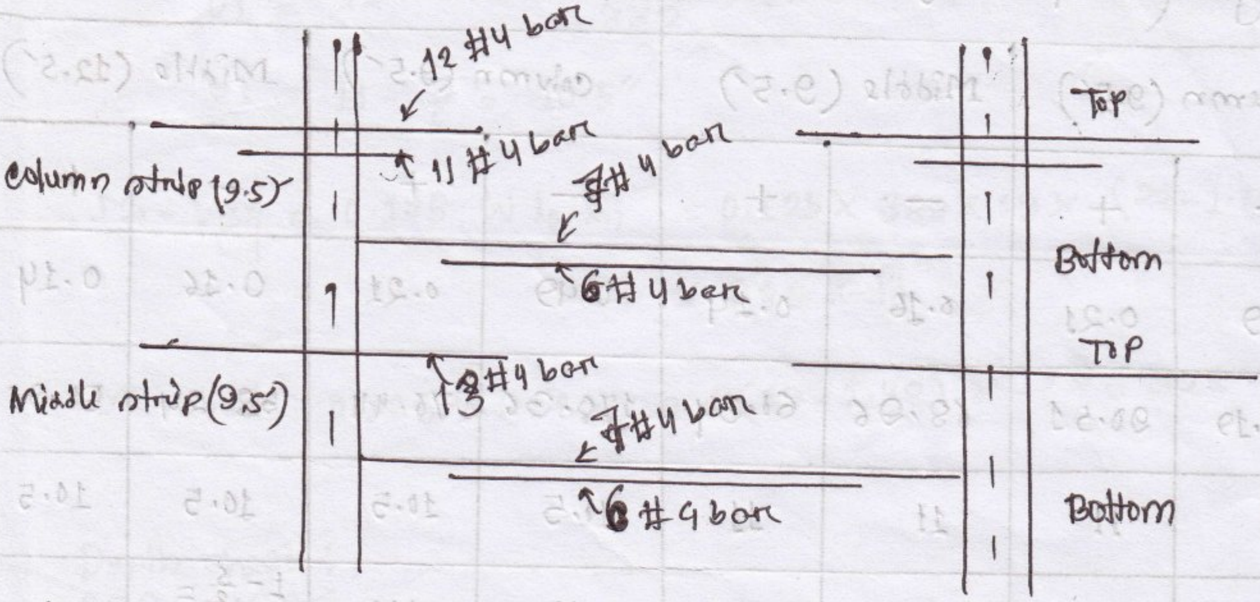
∴ No need to use # 3 bar,

Section	Longer ($M_n = \frac{M_u}{\phi} = \frac{388}{0.9} = 431$ k-ft)				Shorter ($M_n = \frac{328}{0.9} = 364$ k-ft)			
Strip	column (9.5')		Middle (9.5')		column (9.5')		Middle (12.5')	
Sign	−	+	−	+	−	+	−	+
K	0.49	0.21	0.16	0.14	0.49	0.21	0.16	0.14
$=M_n K$ k-ft	211.19	90.51	68.96	61.34	178.36	76.44	58.24	50.96
d(in)	11	11	11	11	10.5	10.5	10.5	10.5
$b(in) = \frac{s}{2}$	114	114	114	114	114	114	$l - \frac{s}{2} =$ 150	150
$= 23.53 \frac{A_s}{b}$	$0.21 A_s$	$0.21 A_s$	$0.21 A_s$	$0.21 A_s$	$0.21 A_s$	$0.21 A_s$	$0.16 A_s$	$0.16 A_s$
$\frac{M_n}{\phi f_y (d - \frac{a}{2})}$	4.47	1.86	1.41	1.23	3.94	1.65	1.25	1.09
$_{min} = 0.0018bt$	2.46	2.46	2.46	2.46	2.46	2.46	3.24	3.24
of #4 bars	23	19	13	13	20	13	17	17
$_{av} = \frac{b}{n}$	4.96	~~11.4~~ 8.8	~~14.25~~ 8.8	~~16.29~~ 8.8	5.7	~~12.67~~ 8.8	~~16.29~~ 8.8	~~19~~ 8.8
$_{max} = 2t$ in	24	24	24	24	24	24	24	24

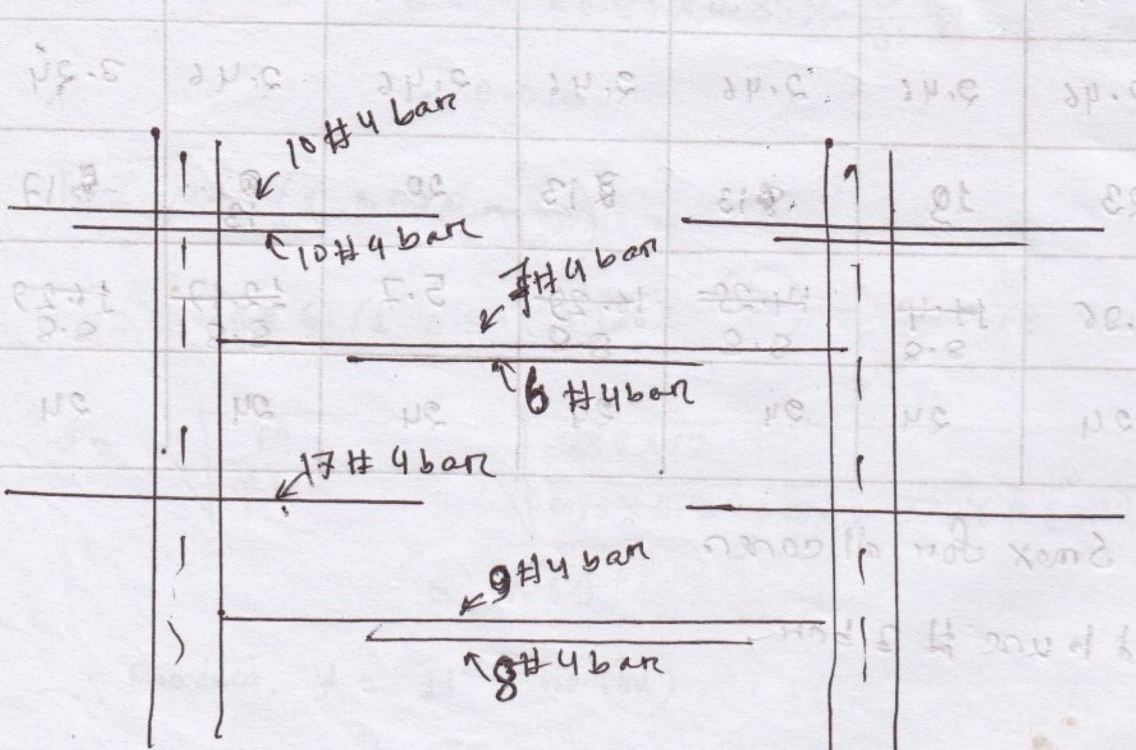
∴ $S_{av} < S_{max}$ for all cases

∴ No need to use # 3 bar.

Reinforcement details:



Longer direction



Shorter direction

Prablum : A 18′×20′ interior panel of a flat plate floor carries LL = 60 psf, DL = 100 psf (including wt). Design the panel assuming column size 15″×15″. Take $f_c' = 4000$ psi, $f_y = 60000$ psi. c.c = 0.75″

Solution:

$L_n = 20 - \frac{15}{12} = 18.75'$

I) Thickness:

a) $t = 5''$

b) $t = \frac{L_n(800 + 0.005 f_y)}{36000} = \frac{18.75\,(800 + 0.005 \times 60000) \times 12}{36000}$

$= 6.88''$

Let us take, $t = 7''$

II) Load calculation:

$w = 1.2 \times D.L + 1.6 \times L.L$

$= 1.2 \times 100 + 1.6 \times 60 = 216$ psf

III) Punching shear check:

$d = 7 - 0.75 - \frac{4}{8 \times 2} = 6''$

$V_u = \left\{ 18 \times 20 - \frac{(15+6)(15+6)}{144} \right\} \times 216 = 77.10$ K

$V_{all} = 4\phi \sqrt{f_c'}\, b_o d = \frac{4 \times 0.75 \times \sqrt{4000} \times 4 \times (15+6) \times 6}{1000}$

$= 95.63$ K

$\therefore$ $V_{all} > V_u$ (OK)

iv) Moment calculation:

$$d_l = 7 - 0.75 - \frac{4}{8\times 2} \approx 6''$$

$$d_s = 6 - \frac{4}{8} = 5.5''$$

$$M_{long} = 0.125\, w\, l_2 l_n^2$$

$$= 0.125 \times 216 \times 18 \times (20 - 1.25)^2 \times 10^{-3} = 171 \text{ k-ft}$$

$$M_{short} = 0.125 \times 216 \times 20 \times (18 - 1.25)^2 \times 10^{-3} = 152 \text{ k-ft}$$

v) Depth check:

$$\rho = \rho_{max} = 0.75 \rho_b$$

$$= 0.75 \times 0.85\, \beta \times \frac{f_c'}{f_y} \times \frac{87}{87 + f_y}$$

$$= 0.75 \times 0.85 \times 0.85 \times \frac{4}{60} \times \frac{87}{87 + 60}$$

$$= 0.0213$$

$$R = \rho f_y \left(1 - 0.59 \rho \frac{f_y}{f_c'}\right)$$

$$= 0.0213 \times 60000 \times \left(1 - 0.59 \times 0.0213 \times \frac{4}{60}\right)$$

$$= 1276.93 \approx 1.28$$

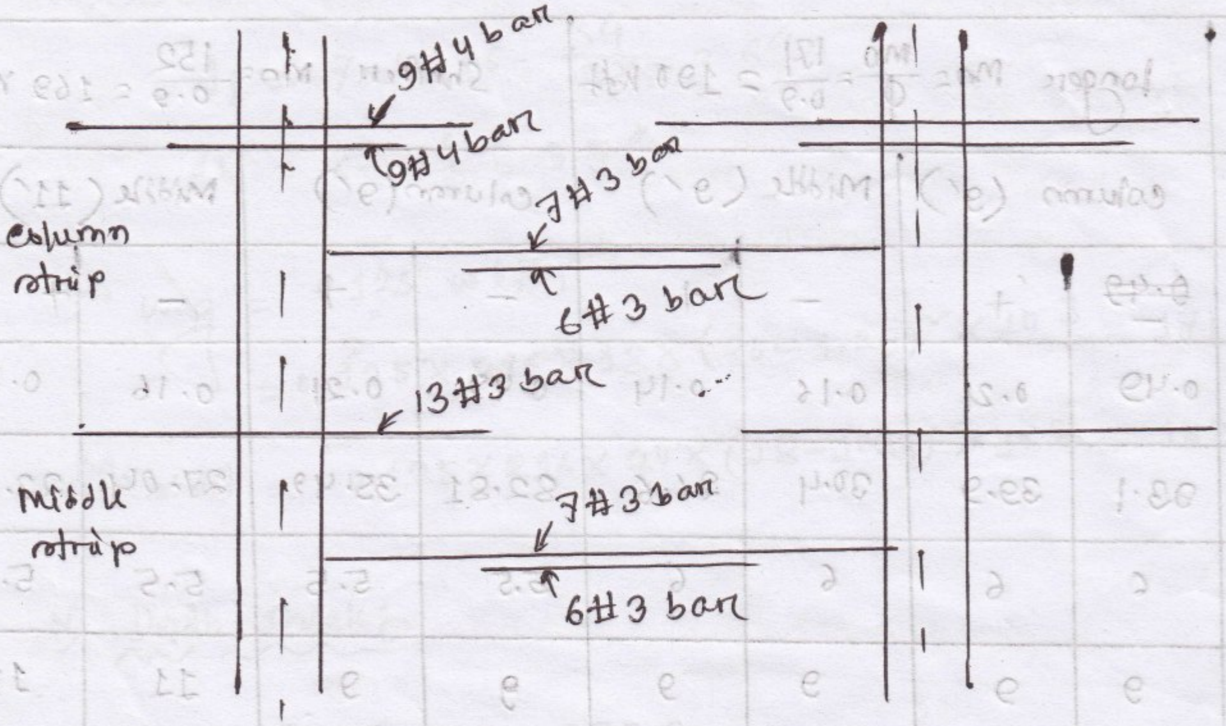
$$d = \sqrt{\frac{M}{\phi R b}} = \sqrt{\frac{171 \times 12}{0.9 \times 1.28 \times 9 \times 12}} \qquad \left[b = \frac{S}{2} = \frac{18}{2} = 9' \right]$$

$$\approx 3.52''$$

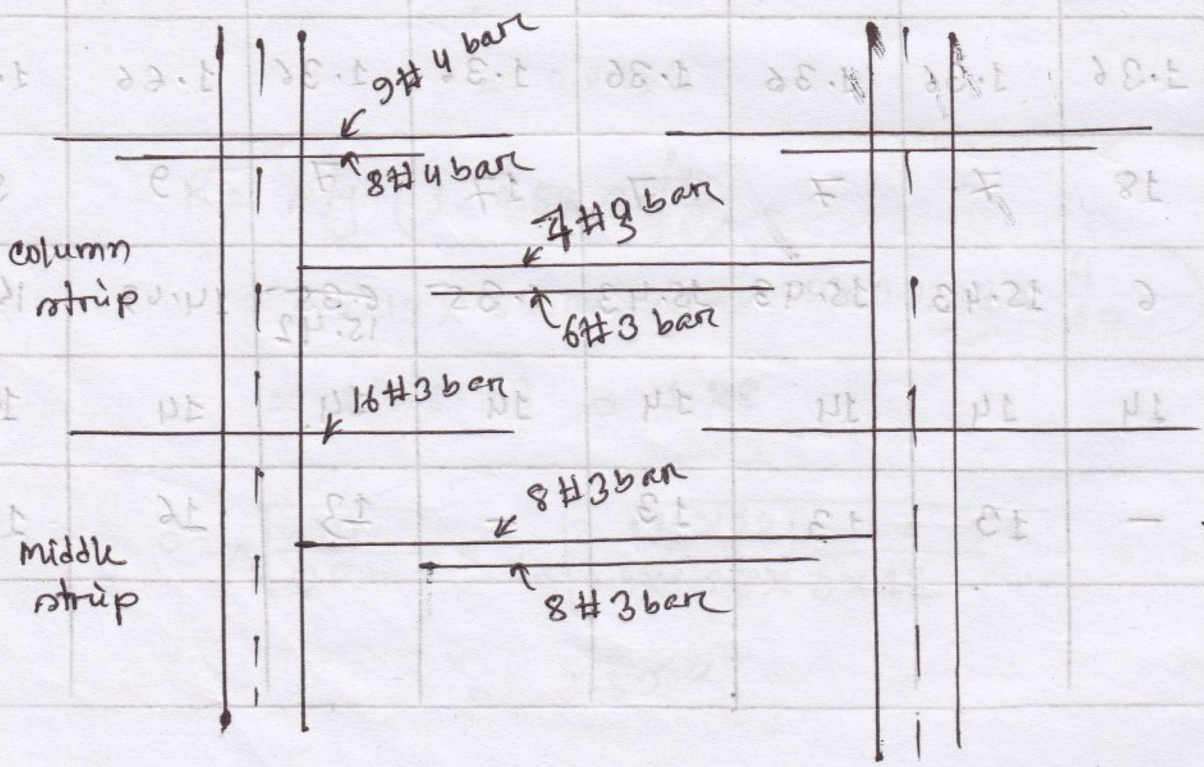
$\therefore$ Provided $d = 6''$

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{A_s \times 60}{0.85 \times 4 \times b} = 17.65 \frac{A_s}{b}$$

Direction	Longer $M_n = \frac{M_o}{\phi} = \frac{171}{0.9} = 190$ k-ft				Shorter $M_n = \frac{152}{0.9} = 169$ k-ft			
Strip	Column (9')		Middle (9')		Column (9')		Middle (11')	
Sign	~~0.49~~ −	+	−	+	−	+	−	+
K	0.49	0.21	0.16	0.14	0.49	0.21	0.16	0.14
$M_n = K M_n$ k-ft	93.1	39.9	30.4	26.6	82.81	35.49	27.04	23.66
d (in)	6	6	6	6	5.5	5.5	5.5	5.5
b (ft)	9	9	9	9	9	9	11	11
$a = 17.65 \frac{A_s}{b}$	$0.16A_s$	$0.16A_s$	$0.16A_s$	$0.16A_s$	$0.16A_s$	$0.16A_s$	$0.13A_s$	$0.13A_s$
$A_s = \frac{M_n}{\phi f_y (d - \frac{a}{2})}$	3.41	1.38	1.04	0.91	3.33	1.34	1	0.89
$A_{s\,min} = 0.0018\,bt$	1.36	1.36	1.36	1.36	1.36	1.36	1.66	1.66
Using #4 No bars n	18	7	7	7	17	7	9	9
$S_{av} = \frac{b}{n}$	6	15.43	15.43	15.43	6.35	~~6.35~~ 15.42	14.67	14.67
$S_{max} = 2t$	14	14	14	14	14	14	14	14
Using #3 No of bar	−	13	13	13	−	13	16	16



Longer direction



Shorter direction

10

Problem-01 :

Design a 17 ft by 19 ft interior panel of a flat plate floor system to support a LL of 100 psf exclusive its own weight using $f_c' = 4$ ksi $f_y = 60$ ksi. Assume 18 in square column.

Solution:

$$\cancel{a = \sqrt{\frac{\pi}{4} D^2}} = \cancel{\sqrt{\frac{\pi}{4} \times}}$$

$$L_n = 19 - \frac{18}{12} = 17.5' = 210''$$

1) Thickness:

a) $t = 5''$

b) $$t = \frac{L_n (800 + 0.005 f_y)}{36000} = \frac{210 (800 + 0.005 \times 60000)}{36000}$$

$$= 6.11'' \cong 7''$$

Let, thickness $t = 7''$

ii) Load calculation:

$$D.L = \frac{t}{12} \times 150 = \frac{7}{12} \times 150 = 87.5 \text{ psf}$$

$$w = 1.2 \times 87.5 + 1.6 \times 100 = 265 \text{ psf}$$

iii) Punching shear check:

$$d = 7 - 0.75 - \frac{4}{8 \times 2} = 6''$$

$$V_u = \left\{ 17 \times 19 - \frac{(18+6)(18+6)}{144} \right\} \times 265 \times 10^{-3} = 84.54 k$$

$$V_{all} = 4\phi \sqrt{f_c'}\, b_o d = \frac{4 \times 0.75 \times \sqrt{4000} \times 4 (18+6) \times 6}{1000} = 109.29 k$$

$\therefore$ $V_{all} > V_u$ (OK)

10

iv) Moment calculation:

$d_l = 7 - 0.75 - \frac{4}{8\times2} = 6''$

$d_s = 6 - \frac{4}{8} = 5.5''$

$M_{o\text{-long}} = 0.125 \times w \times l_2 \times l_n^2 = 0.125 \times 265 \times 17\,(19-1.5)^2 \times 10^{-3} = 172.46$ k-ft

$\simeq 173$ k-ft

$M_{o\text{-short}} = 0.125 \times 265 \times 19 \times (17-1.5)^2 \times 10^{-3} = 151.21$ k-ft

$\simeq 152$ k-ft

v) Depth check:

$\rho = \rho_{max} = 0.75\,\rho_b$

$= 0.75 \times 0.85\,\beta \times \frac{f_c'}{f_y} \times \frac{87}{87+f_y}$

$= 0.75 \times 0.85 \times 0.85 \times \frac{4}{60} \times \frac{87}{87+60}$

$= 0.021$

$R = \rho f_y \left(1 - 0.59\,\rho \frac{f_y}{f_c'}\right) = 0.021 \times 60 \times \left(1 - 0.59 \times 0.021 \times \frac{60}{4}\right)$

$= 1.03$

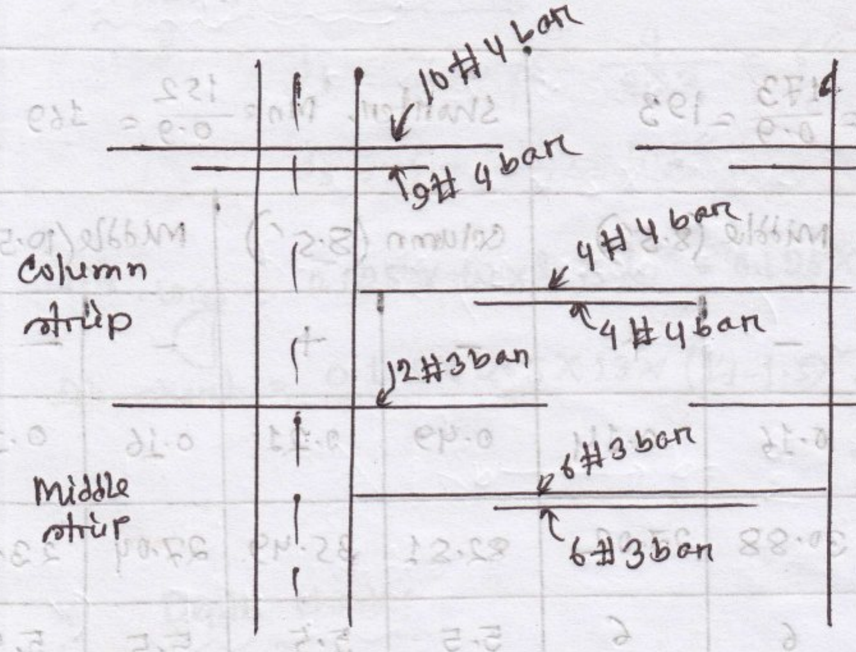
$d = \sqrt{\frac{m}{\phi R b}} = \sqrt{\frac{173 \times 12}{0.9 \times 1.03 \times 8.5 \times 12}} = 4.69''$

$b = \frac{S}{2} = \frac{17}{2} = 8.5''$

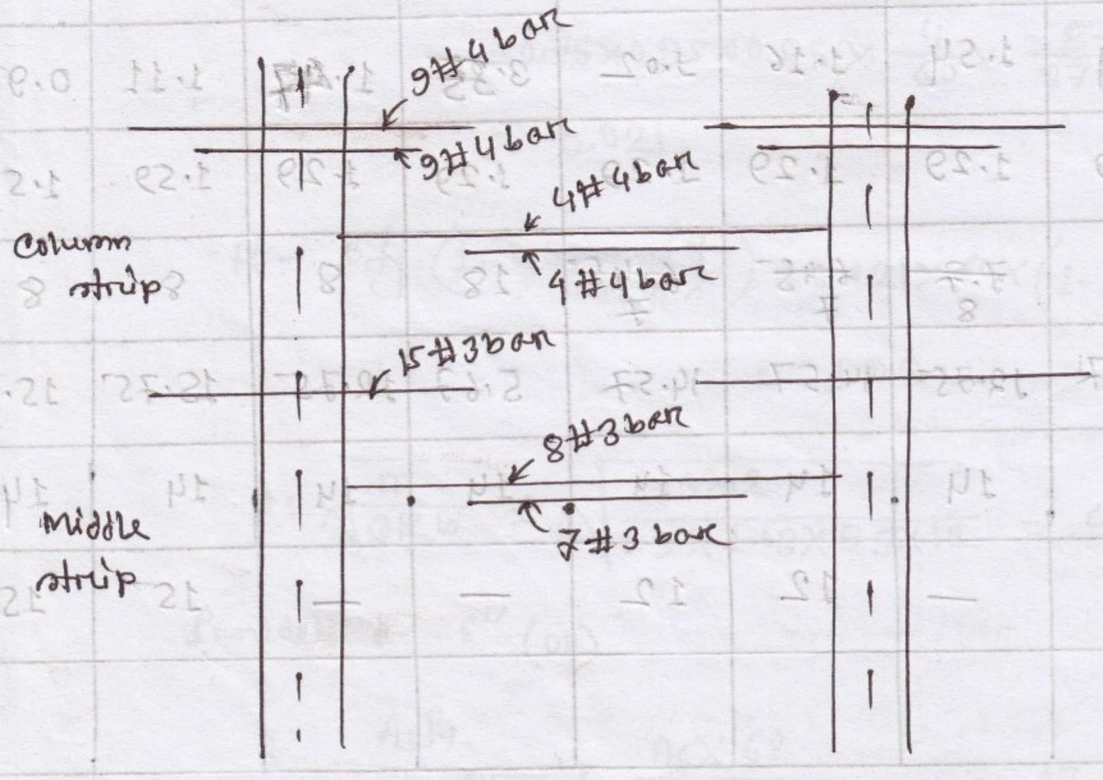
Provided, $d = 6''$ (ok)

$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{A_s \times 60}{0.85 \times 4 \times b} = 17.65\,\frac{A_s}{b}$

Direction	Longer, $M_n = \frac{M_o}{\phi} = \frac{173}{0.9} = 193$				Shorter, $M_n = \frac{152}{0.9} = 169$			
Strip	Column (8.5′)		Middle (8.5′)		Column (8.5′)		Middle (10.5′)	
Sign	—	+	—	+	—	+	—	—
k	0.49	0.21	0.16	0.14	0.49	0.21	0.16	0.14
$M_n = kM_n$ k-ft	94.57	40.53	30.88	27.02	82.81	35.49	27.04	23.66
d (in)	6	6	6	6	5.5	5.5	5.5	5.5
b (in)	102	102	102	102	102	102	126	126
$a = 17.65 \frac{A_s}{b}$	$0.17A_s$	$0.17A_s$	$0.17A_s$	$0.17A_s$	$0.17A_s$	$0.17A_s$	$0.14A_s$	$0.14A_s$
$A_s = \frac{M_n}{0.85\phi f_y (d - \frac{a}{2})}$	3.71	1.54	1.16	1.02	3.35	1.47	1.11	0.97
$A_{smin} = 0.0018bt$	1.29	1.29	1.29	1.29	1.29	1.29	1.59	1.59
sing #4 No of bar $= \frac{A_s}{A_b}$ n	19	~~7.7~~ 8	~~6.45~~ 7	~~6.45~~ 7	18	8	8	8
$S_{av} = \frac{b}{n}$	5.37	12.75	14.57	14.57	5.67	12.75	15.75	15.75
$S_{max} = 2t$	14	14	14	14	14	14	14	14
sing #3 No of bar	—	—	12	12	—	—	15	15



Longer direction

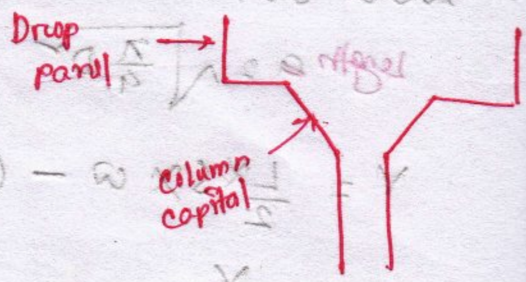


Shorter direction

FLAT SLAB

{AHSAN 090063}

* Flat slab with drop panel:



i) Thickness:

a) $t \geq 4''$

b) $t = \frac{L}{40}$

c) $t = 0.024L\left(1 - \frac{2c}{3L}\right)\sqrt{\frac{w}{\frac{f'_c}{2000}}} + 1 = t_2$

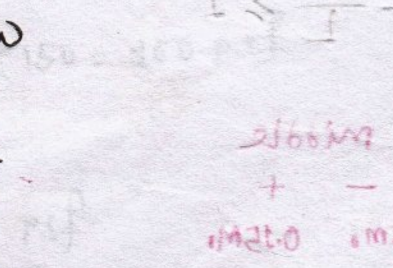
$c =$ dia $= 0.15L$ to $0.25L$ (এই range এর ভিতরে নিতে হবে)

$w = D.L + L.L \qquad D.L = \frac{t}{12} \times 150$

$t_1 \leq 1.5\, t_2$

ii) Punching shear check:

a) Around column capital



within drop, $d_1 = t_1 - 1.5$ → clear cover

outside ", $d_2 = t_2 - 1.5$

$V = S \times L \times w - \frac{\pi}{4}\left(c + \frac{d_1}{12}\right)^2 \times w$

$v = \frac{V}{b_o d} = \frac{V}{\pi(c \times 12 + d_1) \times d_1}$

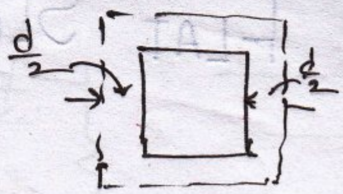
$v_c = 2\sqrt{f'_c} \qquad v_c > v$ (ok)

iii) Around drop panel:

$\nu =$

$\nu = \frac{V}{A}$

$\nu_c > \nu$



Drop panel $\geq \frac{L}{3}$

iv) Beam shear check:

Length $a = \sqrt{\frac{\pi}{4} D^2}$

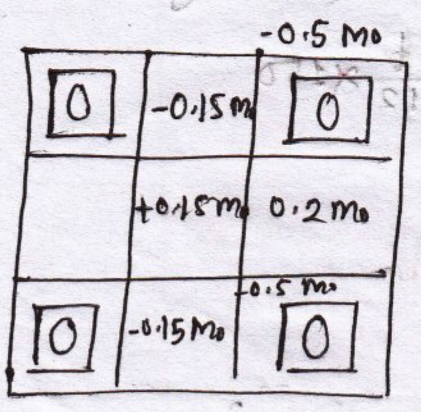
$V = \frac{L}{2} \times S \times \omega - \left(\frac{a}{2} + \frac{d_2}{12}\right) \times S \times \omega$

short direction length

$\nu = \frac{V}{bd}$

$\nu_c = 1.1\sqrt{f_c'} > \nu$ (OK)

v) Moment calculation:



$M_o = 0.09\, WLF \left(1 - \frac{2c}{3L}\right)^2$

ω = Total panel load

$F = 1.15 - \frac{c}{L} \geq 1$

Column		Middle	
−	+	−	+
5Mo	0.2Mo	0.15Mo	0.15Mo

v) Depth check:

a) where drop panel is present

$d_1 = \sqrt{\frac{M}{Rb}}$ b = 75% of drop panel width

$d_{eff} = t_1 => d_1$

b) where drop panel is absent

$d_2 = \sqrt{\frac{M}{Rb}}$

$d_{eff} = t_2 > d_2$ (OK)

vi) Reinforcement calculation:

a) Column strip,

$A_s(-) = \frac{M}{f_s j d}$

$A_s(+) =$

b) Middle strip:

$A_s(+) = A_s(-) = \frac{M}{f_s j d}$

06 04

Problem 1:

Spacing of column = 22′. Diameter of column capital at the intersection of column and drop panel = 4′. Size of drop panel = 8′×8′. Live load = 200 psf. f_s = 20000 psi f_c' = 3000 psi. Design flat slab.

Solution:

$$n = \frac{E_s}{E_c} = \frac{29\times10^6}{57000\sqrt{3000}} = 9.28 \simeq 9$$

$$r = \frac{f_s}{f_c} = \frac{20}{0.45\times3} = 14.81$$

$$k = \frac{n}{n+r} = \frac{9}{9+14.81} = 0.38$$

$$j = 1-\frac{k}{3} = 0.87$$

$$R = \frac{1}{2}f_c jk = \frac{1}{2}\times0.45\times3000\times0.87\times0.38 = 223.16$$

1) Thickness calculation:

a) $t \geq 4''$

b) $t = \frac{L}{40} = \frac{22\times12}{40} = 6.6'' \simeq 8''$

c) $t = 0.024\,L\left(1-\frac{2c}{3L}\right)\sqrt{\frac{w}{\frac{f_c'}{2000}}} + 1$

c = dia = 0.15 L to 0.25 L (range এর ভিতরে 1টা ধরতে হবে)

$= 4'$

$$D.L = \frac{t}{12}\times150 = \frac{8}{12}\times150 = 100 \text{ psf}$$

$L.L = 200$ psf

$w = 100+200 = 300$ psf

$\therefore t = 7.56'' \simeq 8'' = t_2$

$t_1 \leq 1.5t_2 = 12'' \simeq 11''$

$\therefore t_1 = 11''$ $t_2 = 8''$

ii) Punching shear check:

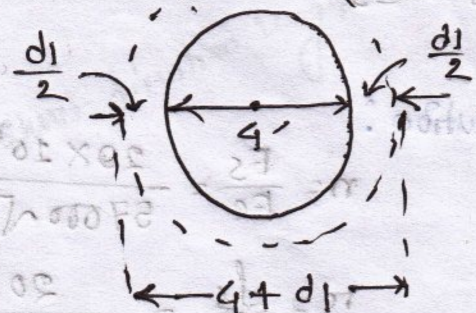
a) Around column capital:

within drop, $d_1 = t_1 - \text{cover}$

$= 11 - 1.5 = 9.5''$

outside drop, $d_2 = t_2 - 1.5 =$

$= 8 - 1.5 = 6.5''$



$V = 22 \times 22 \times 300 - \frac{\pi}{4}\left(4 + \frac{d_1}{12}\right)^2 \times 300$

$= 145200 - 5409.85$

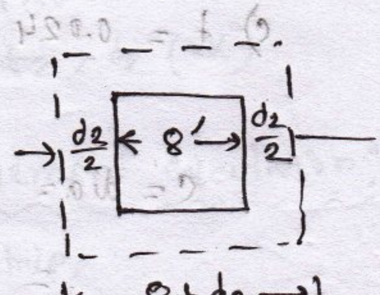
$= 139.79^K$

shear stress, $v = \frac{V}{A} = \frac{139.79}{\pi \times (4\times12+9.5)\times 9.5}$ $\pi d L = A = b_0 d$

$= 81.46$ ~~psf~~ psi

$v_c = 2\sqrt{f_c'} = 2\sqrt{300} = 109.54$ psf > 81.46 psf (ok)

b) Around drop panel:



$V = 22 \times 22 \times 300 - \left(8 + \frac{d_2}{12}\right)^2 \times 300$

$= 145200 - 21888.02$

$= 123.31^K$

$v = \frac{V}{A} = \frac{123.31 \times 10^3}{4(8\times12+6.5)\times 6.5} = 46.27$ psi

$\therefore v_c = 109.54$ psi $> v$.

(ok).

e) <u>Beam shear check:</u>

$$a = \sqrt{\frac{\pi}{4} D^2} = \sqrt{\frac{\pi}{4}(4)^2} = 3.54'$$

$$V = 22 \times \frac{22}{2} \times 350 - \left(\frac{3.54}{2} + \frac{6.5}{12}\right) \times 22 \times 350$$

$$= 57.34^k$$

$$v = \frac{V}{bd} = \frac{57.34 \times 10^3}{22 \times 12 \times 6.5} = 33.41 \text{ psi}$$

$$v_c = 1.1\sqrt{f_c'} = 1.1\sqrt{3000} = 60.25 \text{ psi} > v \quad \text{(OK)}$$

<u>Moment calculation:</u>

Column strip	Middle strip	Column strip
	$-0.15 M_o$	$-0.5 M_o$
	$+0.15 M_o$	$0.2 M_o$
	$-0.15 M_o$	$-0.5 M_o$
$\frac{l}{3}$	$\frac{l}{3}$	$\frac{l}{3}$

	-33303.6	-111012
	$+33303.6$	44404.8
	-33303.6	-111012

$$M_o = 0.09\, WLF \left(1 - \frac{2c}{3L}\right)^2$$

W = Total load on panel $= 22 \times 22 \times 350 = 145200$ lb

$L = 22'$

$$F = 1.15 - \frac{c}{L} \geq 1 = 1.15 - \frac{4}{22} = 0.97 \qquad \therefore F = 1$$

$$M_o = 0.09 \times 145200 \times 22 \times 1 \times \left(1 - \frac{2 \times 4}{3 \times 22}\right)^2$$

$$= 222024 \text{ lb-ft}$$

v) Depth check:

a) Depth check of column strip where drop panel is present

$$d_1 = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{111012 \times 12}{223 \times 72}}$$

$$= 9.1''$$

$b = 75\%$ of drop panel width

$= 0.75 \times 8 \times 12$

$= 72''$

$d_{eff} = \cancel{9.5}\ 11 - 1.5 = 9.5'' > d_1$ (OK)

b) Depth check of slab where drop panel is absent

$$d_2 = \sqrt{\frac{44404.8 \times 12}{223 \times 11 \times 0.75 \times 12}}$$

$$= 4.9''$$

$d_{eff} = 8 - 1.5 = 6.5'' > d_2$ (OK)

vi) Reinforcement calculation:

a) Reinforcement of column strip:

$$A_s(-) = \frac{M}{f_s j d} = \frac{111012 \times 12}{20000 \times 0.87 \times 9.5} = 8.02 \text{ in}^2$$

$= 19$ #6 bars

$$A_s(+) = \frac{44404.8 \times 12}{20000 \times 0.87 \times 6.5} = 4.69 \text{ in}^2$$

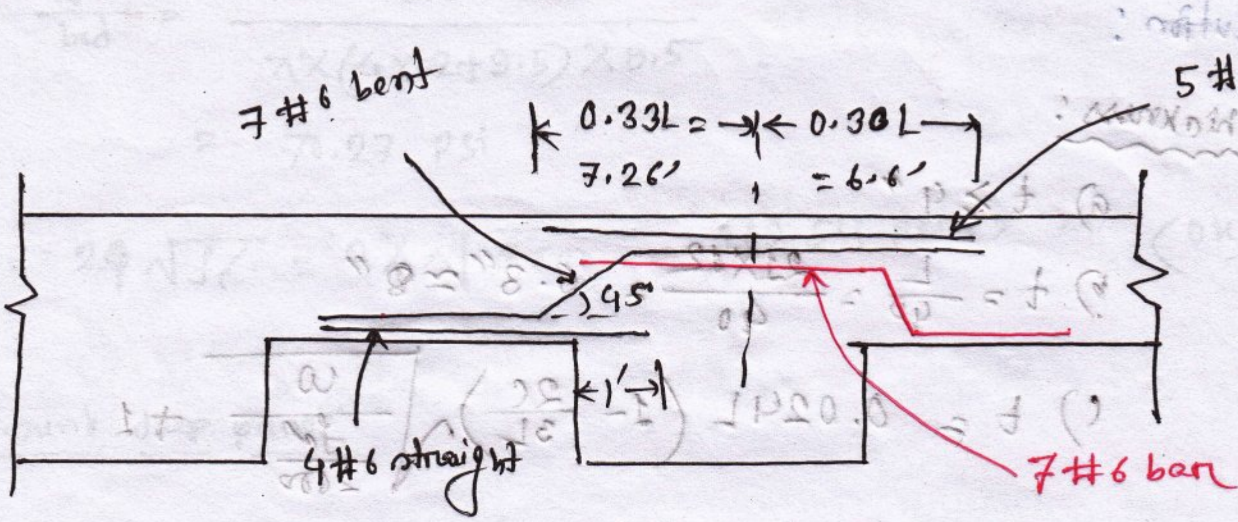
$= 11$ #6 bars

07

b) Reinforcement of middle strip:

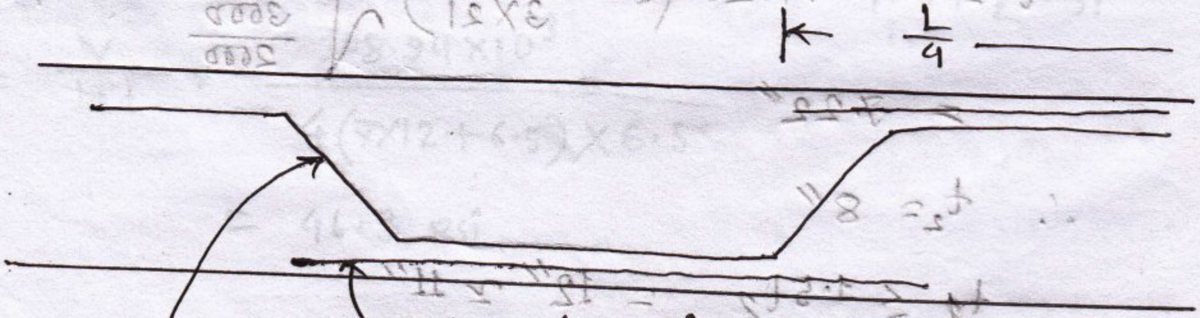
$$A_s(+) = A_s(-) = \frac{33\,303.6 \times 12}{20000 \times 0.87 \times 6.5} = 3.52\ in^2$$

$$= 12 \# 5 \text{ bar}$$



33% bar will be straight = 11 × 0.33 = 3.63 ≃ 4 bar straight

$\boxed{\text{column strip}}$



50% bar straight, 50% bent

$\boxed{\text{Middle strip}}$

07 | 05

Problem-02:

A 20′ X 21′ interior panel of a flat slab system with drop panel is to designed to carry a total load of 300 psf. Show the detail reinforcement $f_c' = 3$ ksi $f_y = f_s = 24$ ksi.

Solution:

i) Thickness:

a) $t \geq 4''$

b) $t = \frac{L}{40} = \frac{21 \times 12}{40} = 6.3'' \simeq 8''$

c) $t = 0.024L\left(1 - \frac{2c}{3L}\right)\sqrt{\frac{w}{\frac{f_c'}{2000}}} + 1$

$c = 0.15L$ to $0.25L$

$\simeq 4'$

~~$D.L = \frac{t}{12} \times 150 = \frac{8}{12} \times 150$~~ $w = 300$ psf

$t = 0.024 \times 21\left(1 - \frac{2 \times 4}{3 \times 21}\right)\sqrt{\frac{300}{\frac{3000}{2000}}} + 1$

$= 7.22''$

$\therefore t_2 = 8''$

$t_1 \leq 1.5 t_2 = 12'' \simeq 11''$

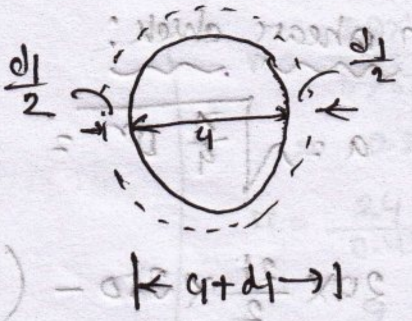
ii) Punching shear check:

a) Around column capital:

$d_1 = 11 - 1.5 = 9.5''$

$d_2 = 8 - 1.5 = 6.5''$

$$V = 20 \times 21 \times 300 - \frac{\pi}{4}\left(4 + \frac{9.5}{12}\right)^2 \times 300$$



$$= 126000 - 5409.85$$

$$= 120.59 \text{ K}$$

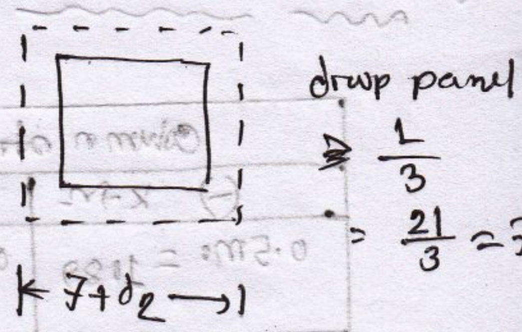
$$v = \frac{V}{b_o d} = \frac{120.59 \times 10^3}{\pi \times (4 \times 12 + 9.5) \times 9.5}$$

$$= 70.27 \text{ psi}$$

$$V_c = 2\sqrt{f'_c} = 2 \times \sqrt{3000} = 109.54 \text{ psi} > v \quad \text{(ok)}$$

b) Around drop panel:

$$V = 20 \times 21 \times 300 - \left(7 + \frac{d_2}{12}\right)^2 \times 300$$

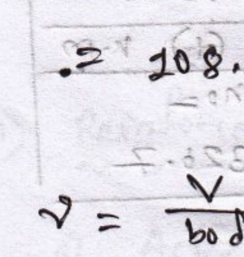


$$= 126000 - 17063.02$$

$$= 108.94 \text{ K}$$

$$v = \frac{V}{b_o d} = \frac{108.94 \times 10^3}{4(7 \times 12 + 6.5) \times 6.5}$$

$$= 46.3 \text{ psi}$$



$$v_c = 109.54 \text{ psi} > v.$$

(ok)

iv) **Beam shear check:**

$$a = \sqrt{\frac{\pi}{4} D^2} = \sqrt{\frac{\pi}{4} \times 4^2} = 3.54'$$

$$V = 20 \times \frac{21}{2} \times 300 - \left(\frac{3.54}{2} + \frac{86.5}{12}\right) \times 20 \times 300$$

$= 63000 - 13870$

$= 49.13$ K

$$v = \frac{V}{b_o d} = \frac{49.13 \times 10^3}{20 \times 12 \times 6.5} = 31.49 \text{ psi}$$

$V_c = 1.1 \sqrt{f_c'} = 1.1 \sqrt{3000} = 60.25$ psi $> v$. (ok)

v) **Moment calculation:**

Column strip		Middle strip	
(−) k-m	(+) k-m	(−) k-m	(+) k-m
$0.5 M_o = 1189$	$0.2 M_o = 435.6$	$0.15 M_o = 326.7$	$0.15 M_o = 326.7$

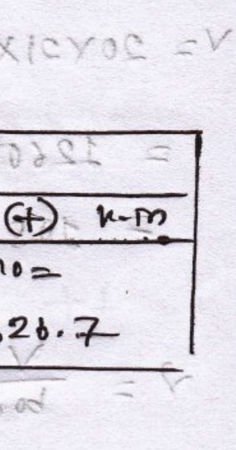
$$M_o = 0.09 \; WLF \left(1 - \frac{2c}{3L}\right)^2$$

$L = 21'$

$W = 20 \times 21 \times 300 = 126000$ lb

$F = 1.15 - \frac{c}{L} \geq 1 = 1.15 - \frac{4}{21} = 0.96$

$\therefore F = 1$



$$M_o = 0.09 \times 126000 \times 21 \times 1 \times \left(1 - \frac{2 \times 4}{3 \times 21}\right)^2$$

$= 181.5$ K-ft

$= 2178$ K-m

v) Depth Check:

Drop panel present

$$d_1 = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{1089 \times 10^3}{204 \times 63}}$$

$= 9.21$ in

$d_{eff} = 11 - 1.5 = 9.5$ in $> d_1$ (ok)

$$n = \frac{29 \times 10^6}{57000\sqrt{3000}} = 9.28 \simeq 9$$

$$r = \frac{24}{0.45 \times 3} = 17.78$$

$$k = \frac{9}{9 + 17.78} = 0.34$$

$j = 0.89$

$R = \frac{1}{2} f_c j k$

$= 204.26$

$b = 75\%$ of drop panel width

$= 0.75 \times 7 \times 12$

$= 63$ in

b) Drop panel absent:

$$d_2 = \sqrt{\frac{326.7 \times 103}{204 \times 0.75 \times 11 \times 12}}$$

$= 4.02$ in

$d_{eff} = 8 - 1.5 = 6.5'' > d_2$ (ok)

vi) Reinforcement calculation:

Column strip a)

$$A_s(-) = \frac{M}{f_s j d} = \frac{1089}{24 \times 0.89 \times 9.5} = 5.37 \text{ in}^2$$

$= 13$ #6 bar

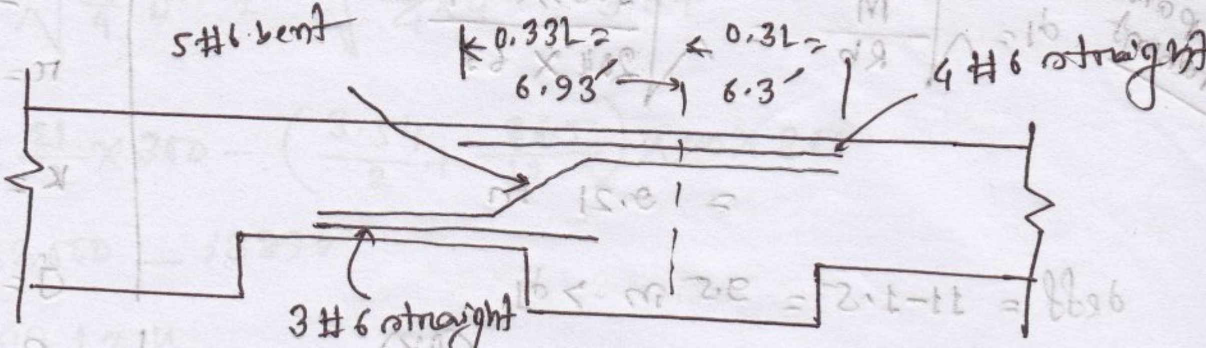
$$A_s(+) = \frac{435.6}{24 \times 0.89 \times 6.5} = 3.14 \text{ in}^2$$

$= 8$ #6 bar

Middle strip:

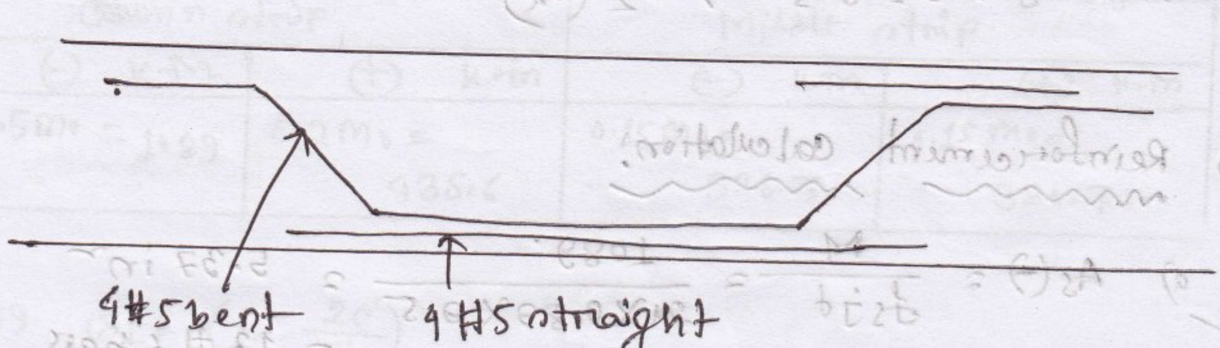
$$A_s(+) = A_s(-) = \frac{326.7}{24 \times 0.89 \times 6.5} = 2.35 \text{ in}^2$$

$= 8$ #5 bar



33% bar will be straight = ~~3~~ 0.33 X 8 = 2.64 ≈ 3 bar straight

Column strip



50% will be straight bar = 50% bent

Middle strip

5. $f_c' = 3000$ psi $\quad f_s = 24000$ psi $\qquad$ 19'×21'

i) $t \geq 4''$

ii) $t = \frac{L}{40} = \frac{21 \times 12}{40} = 6.3 \simeq 8''$

iii) $t = 0.024 L \left(1 - \frac{2c}{3L}\right) \times \sqrt{\frac{w}{\frac{f_c'}{2000}}} + 1$

$c = 0.15L$ to $0.25L \qquad w = 290\#$

$= 0.15 \times 21 \qquad 0.25 \times 21$

$= 3.15' \qquad = 5.25'$

$c = 4'$

$t = 0.024 \times 21 \times \left(1 - \frac{2 \times 4}{3 \times 21}\right) \times \sqrt{\frac{290 \times 2000}{3000}} + 1$

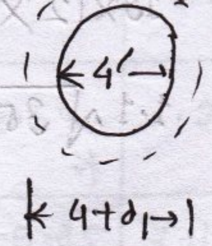
$= 0.44 \times 13.90 + 1$

$= 7.12'' \simeq 8''$

$t_2 = 8'' \qquad t_1 \leq 1.5 t_2 = 12'' \simeq 11''$

Punching shear check:

i) Around column capital



$d_1 = 11 - 1.5 = 9.5''$

$d_2 = 8 - 1.5 = 6.5''$

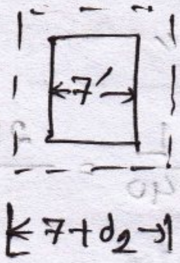
$V_u = 19 \times 21 \times 290 - \frac{\pi}{4}\left(4 + \frac{9.5}{12}\right)^2 \times 290$

$= 115710 - 5229.52 = 110480.48$

$v = \frac{110480.48}{\pi(4 \times 12 + 9.5) \times 9.5} = 64.38$

$V_c = 2\sqrt{3000} = 109.54$ psi $> v$ (ok)

ii) Around drop panel:



$$V = 19 \times 21 \times 290 - \left(7 + \frac{6.5}{12}\right)^2 \times 290$$

$$= 115710 - 16494.25$$

$$= 99215.75$$

$$v = \frac{99215.75}{4 \times (7 \times 12 + 6.5) \times 6.5} = 42.17 \text{ psi}$$

$v_c = 2.\ 109.54$ psi $> v$ (ok)

iii) Beam Shear Check: $a = \sqrt{\frac{\pi}{4}(4)^2} = 3.54'$

$$V = \frac{21}{2} \times 19 \times 290 - \left(\frac{3.54}{2} + \frac{6.5}{12}\right) \times 19 \times 290$$

$$= 57855 - 12737.28$$

$$= 45117.72$$

$$v = \frac{45117.72}{19 \times 12 \times 6.5} = 30.44 \text{ psi}.$$

$V_c = 1.1\sqrt{3000} = 60.25$ psi $> v$. (ok)

$$M_0 = 0.09\, WLF \left(1 - \frac{2C}{3L}\right)^2 \qquad F = 1.15 - \frac{C}{L} = 1$$

$$W = 19 \times 21 \times \cancel{300} = \cancel{119700}$$
$$290 = 115710 \text{ lb}$$

$$M_L = 0.09 \times \underset{115710}{\cancel{119700}} \times 21 \left(1 - \frac{2\times4}{3\times21}\right)^2$$

*

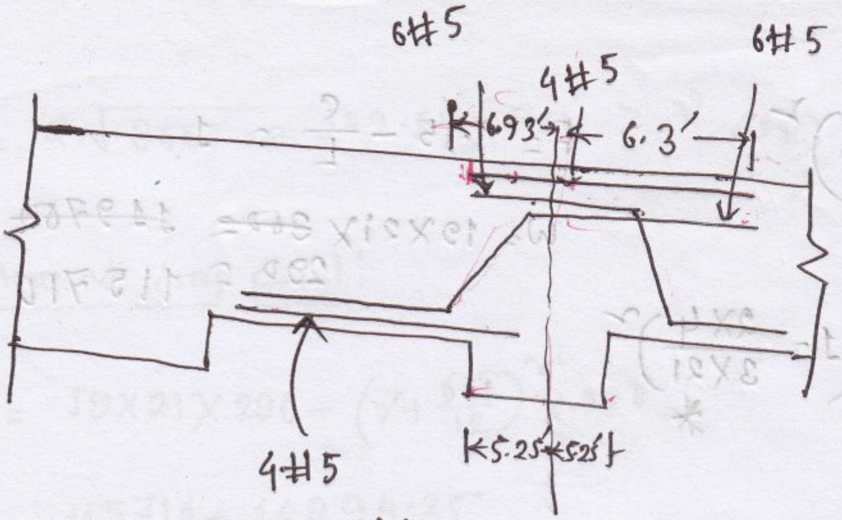
$$= 2000 \text{ k-in}$$

$$M_S = 0.09 \times 115710 \times 19 \left(1 - \frac{2\times4}{3\times19}\right)^2$$

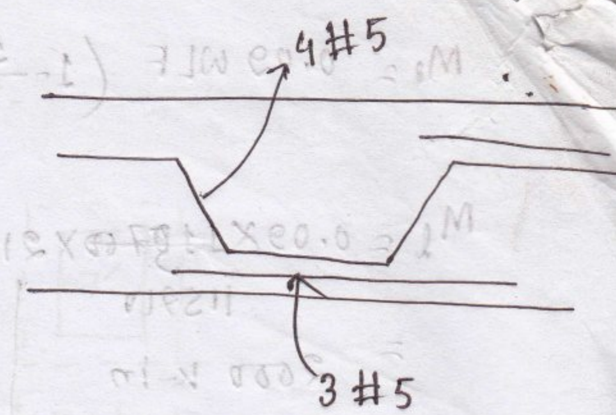
*

$$= 1754.65 \text{ k-in}$$

Direction	Longer ($\overset{M}{2000}$)				Shorter ($M = 1754.65$)			
Strip	Column		middle		Column		middle	
Sign	(−)	(+)	(−)	(+)	(−)	(+)	(−)	(+)
K	0.5	0.2	0.15	0.15	0.5	0.2	0.15	0.15
d (in)	9.5	6.5	6.5	6.5	9.5	6.5	6.5	6.5
b (in)	$0.75 \times 7 \times 12 = 63$	$0.75 \times \frac{19}{2} \times 12$ 85.5	85.5	85.5	63	94.5	94.5	94.5
$M = M \times K$ (k-in)	1000	400	300	300	877.33	350.93	263.20	263.2
= R	204.26	204.26	204.26	204.26	204.26	204.26	204.21	204.26
$d = \sqrt{\frac{M}{Rb}}$	8.8	4.79	4.14	4.14	8.26	4.26	3.69	3.69
$A_s = \frac{M}{f_s j d}$ (in²)	4.93	2.88	2.16	2.16	4.42	2.59 *	1.94	1.94
Bar	16#5	10#5	7#5	7#5	15#5	9#5	7#5	7#5

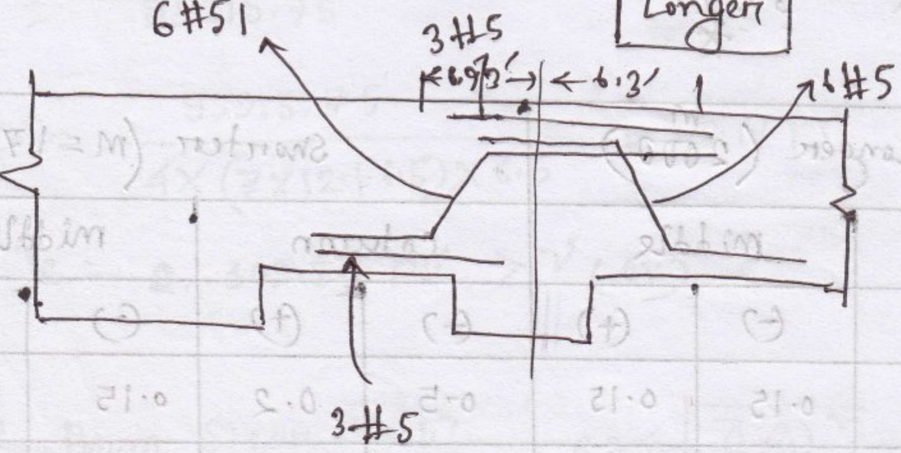


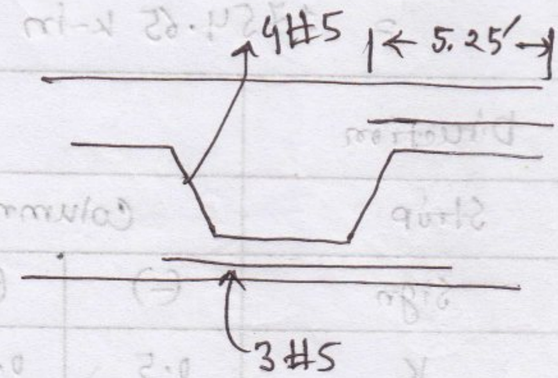
Column strip



Middle strip

Longer





Shorter

(4) (1)

CE-317

Two Way Slab

AHSAN
090063

WSD method

i) **Dimension check:** A = short direction
B = Long "

$\frac{B}{A} \le 2$ then two way slab

ii) **Thickness:** $t = \frac{\text{Perimeter}}{180} = \frac{2(A+B) \times 12}{180}$

iii) **Load calculation:** $D.L = \frac{t}{12} \times 150$ psf

L.L না দেয়া থাকলে 40 psf ধরতে হবে

$T.L = D.L + L.L$

iv) **Moment calculation:** $m = \frac{A}{B}$

কোন case এ পড়েছে দেখতে হবে এবং

$C_{A\,neg}$, $C_{B\,neg}$, $C_{A\,D.L}$, $C_{B\,D.L}$, $C_{A\,L.L}$, $C_{B\,L.L}$

At continuous end: $(-)\,M_{A\,neg} = C_{A\,neg}\, w L_A^2$

$(-)\,M_{B\,neg} = C_{B\,neg}\, w L_B^2$

At mid span: ~~(+) MAneg~~ $M_A(+) = C_{A\,L.L}\, w L_A^2 + C_{A\,D.L}\, w L_A^2$

$M_B(+) = C_{B\,L.L}\, w L_B^2 + C_{B\,D.L}\, w L_B^2$

v) Depth Check:

$$d = \sqrt{\frac{M}{Rb}} \qquad b = 12''$$

$d_{actual} = \cancel{t} - 1$

$\therefore d_{actual} > d$ (OK)

vi) Reinforcement calculation:

In short direction: $(+)\ A_s = \frac{M}{f_s j d}$

$A_{s\,min} = 0.0018\ bt \quad (f_y = 60\text{ksi})$

$= 0.0020\ bt \quad (f_y = 40 \text{ or } 50 \text{ ksi})$

A_s কয়টা নিয়ে spacing বের করতে হবে $S = ?$

$S_{max} = 3t$

$(-)\ A_s = \frac{M}{f_s j d}$

$A_{s\,min} = 0.0018\ bt$

$S = \frac{0.11 \times 12}{A_s(+)}$

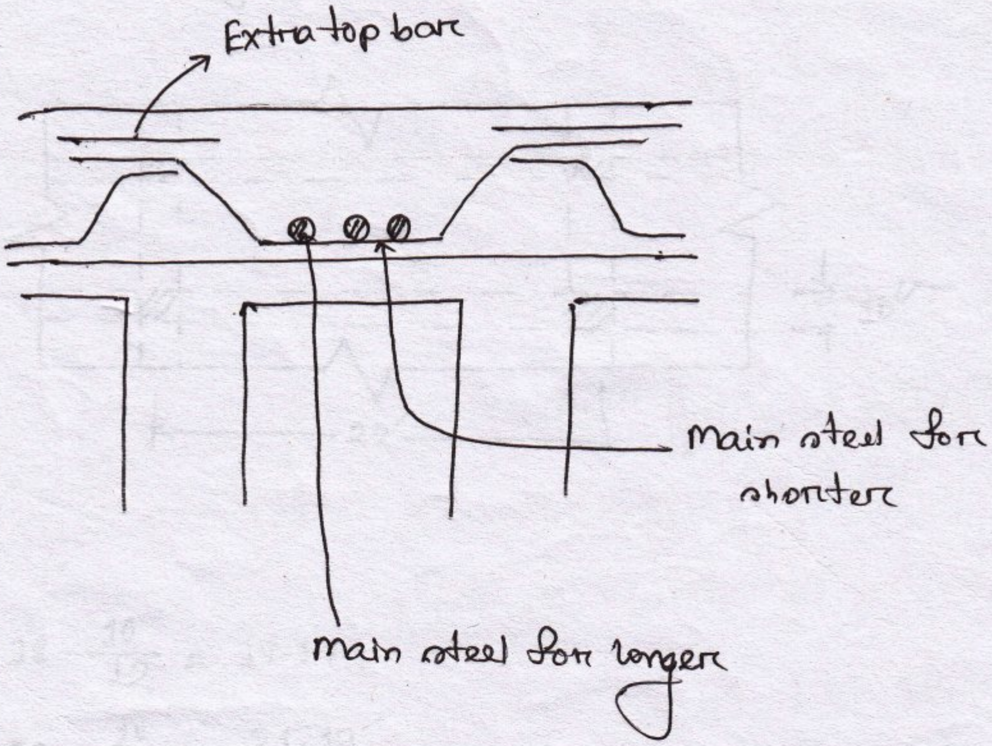
Additional steel $= A_s(-) - A_s(+)$

In longer direction:

Same ভাবে বের করতে হবে

vii) Working diagram:

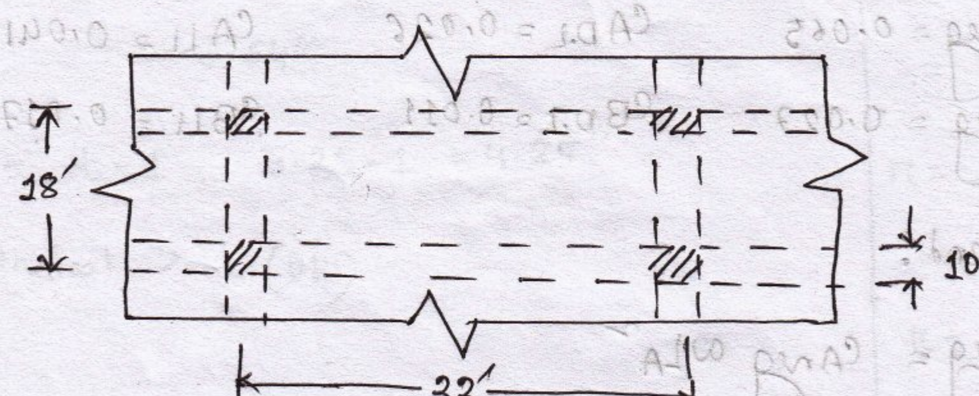
Shorter direction:



WSD method

02

Q. The partial plan of a building is shown. Design the slab with LL=80psf using WSD. $f_c' = 3$ ksi, $f_y = 50$ ksi and beam width 10"



Solution :

i) $A = 18 - \frac{10}{12} = 17.17'$

$B = 22 - \frac{10}{12} = 21.17'$

$\frac{B}{A} = \frac{21.17}{17.17} = 1.23 < 2$, So, two way slab

ii) Thickness calculation :

$$t = \frac{2(17.17 + 21.17) \times 12}{180} = 5.11'' \simeq 5.25''$$

iii) Load calculation :

$$D.L = \frac{t}{12} \times 150 = \frac{5.25}{12} \times 150 = 65.63 \text{ psf}$$

$$L.L = 80 \text{ psf}$$

$$\therefore T.L,\ \omega_T = D.L + L.L = 65.63 + 80$$

$$= 145.63 \text{ psf}$$

iv) Moment calculation:

$$m = \frac{17.17}{21.17} = 0.8$$ and case-2

from table:

$C_{A\,neg} = 0.065$ $\quad$ $C_{A\,D.L} = 0.026$ $\quad$ $C_{A\,LL} = 0.041$

$C_{B\,neg} = 0.027$ $\quad$ $C_{B\,D.L} = 0.011$ $\quad$ $C_{B\,LL} = 0.017$

At continuous end:

$(-) M_{A\,neg} = C_{A\,neg}\, \omega L_A^2$

$= 0.065 \times 145.63 \times (17.17)^2$

$= 2790.65$ lb-ft

$(-) M_{B\,neg} = C_{B\,neg}\, \omega L_B^2$

$= 0.027 \times 145.63 \times (21.17)^2$

$= 1762.20$ lb-ft

At mid span:

$M_A(+) = C_{A\,D.L}\, \omega_D L_A^2 + C_{A\,L.L}\, \omega_L L_A^2$

$= 0.026 \times \cancel{145.63}^{\,65.63} \times (17.17)^2 + 0.041 \times \cancel{145.63}^{\,80} \times (17.17)^2$

$= \cancel{2876}$ 1470 lb-ft

$M_B(+) = C_{B\,D.L}\, \omega_D L_B^2 + C_{B\,L.L}\, \omega_L L_B^2$

$= 0.011 \times 65.63 \times (21.17)^2 + 0.017 \times 80 \times (21.17)^2$

$= 933.05$ lb-ft

v) Depth check:

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{2790.65 \times 12}{223.16 \times 12}}$$

$= 3.54''$

$d_{actual} = t - 1 = 5.25 - 1 = 4.25''$

$\therefore d_{actual} > d$ (OK)

$n = \frac{E_s}{E_c} = \frac{29 \times 10^6}{57000\sqrt{f_c'}}$

$= \frac{29 \times 10^6}{57000\sqrt{3000}}$

$= 9.28 \simeq 9$

$r = \frac{f_s}{f_c} = \frac{20000}{0.45 \times 3000}$

$= 14.81$

$k = \frac{n}{n+r} = \frac{9}{9+14.81}$

$= 0.38$

$j = 1 - \frac{k}{3} = 0.87$

$R = \frac{1}{2} f_c j k$

$= \frac{1}{2} \times 0.45 \times 3000 \times 0.87 \times 0.38$

$= 223.16$ lb

vi) Reinforcement calculation:

Short:

(+) $A_s = \frac{M}{f_s j d} = \frac{1470 \times 12}{20000 \times 0.87 \times 4.25}$

$= 0.24$ in^2

$A_{s\,min} = 0.0020\, bt$

$= 0.0020 \times 12 \times 5.25 = 0.13$ in^2

#3 bar, $S = \frac{0.11 \times 12}{0.24} = 5.5''$ c/c

$\therefore$ Use #3 bar @ 5.5'' c/c

$S_{max} = 3t = 3 \times 5.25 = 15.75''$ c/c

(−) $A_s = \frac{M}{f_s j d} = \frac{2790.65 \times 12}{20000 \times 0.87 \times 4.25} = 0.45$ in^2

$S = \frac{0.11 \times 12}{A_s(+)} \Rightarrow 5.5 = \frac{0.11 \times 12}{A_s(+)}$

$\therefore A_s(+) = 0.24$ in^2

∴ Additional steel $= 0.45 - 0.24 = 0.21\ in^2$

$S = \dfrac{0.11 \times 12}{\cancel{0.21}\ 2 \times 0.11} = \cancel{6.29 \simeq 6.25'' c/c}\ 6'' c/c$

Use 2#3 bar @ ~~6.25" c/c~~ 6" c/c

In longer direction:

$$As(+) = \frac{933.05 \times 12}{20000 \times 0.87 \times 4.25} = 0.15\ in^2$$

$$As_{min} = 0.0020\ bt = 0.0020 \times 12 \times 5.25 = 0.13\ in^2$$

Use #3 bar, $s = \dfrac{0.11 \times 12}{0.15} = 8.8 \simeq 8.75'' c/c$

$$As_{max} = 3t = 3 \times 5.25 = 15.75'' c/c$$

Use #3 bar @ 8.75" c/c

$$As(-) = \frac{1762.20 \times 12}{20000 \times 0.87 \times 4.25} = 0.29\ in^2$$

$$S = \frac{0.11 \times 12}{As(+)} \Rightarrow 8.75 = \frac{0.11 \times 12}{As(+)}$$

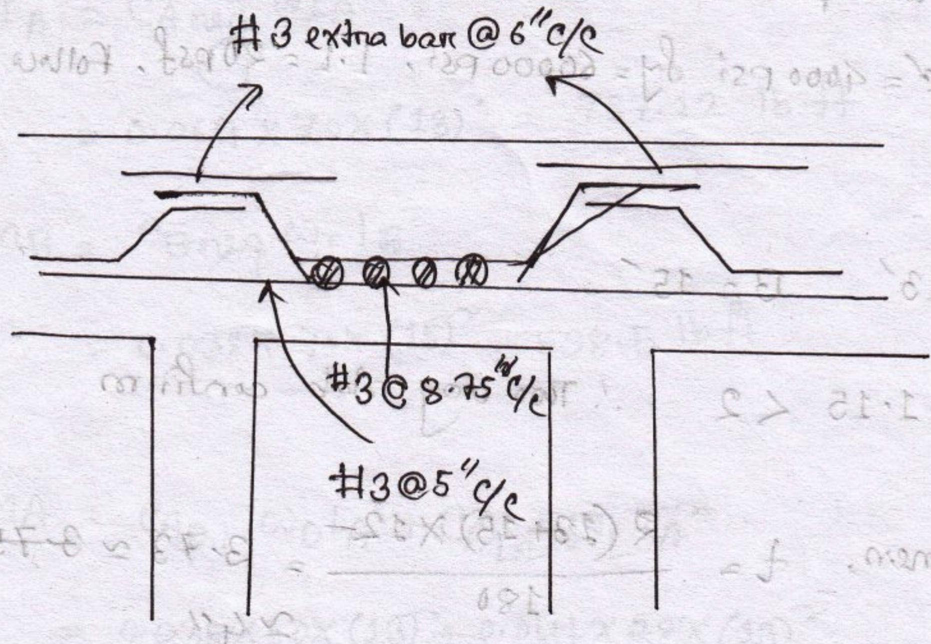
$$\therefore As(+) = 0.15\ in^2$$

∴ Additional steel $= 0.29 - 0.15 = 0.14\ in^2$

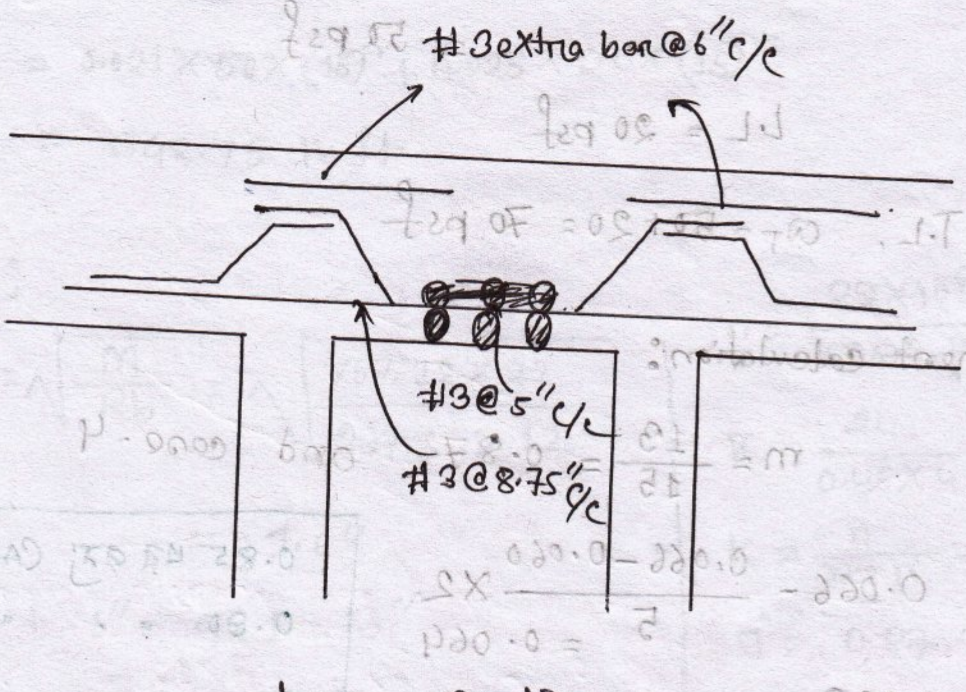
$S = \dfrac{0.11 \times 12}{0.14} = \cancel{9.43 \simeq 9.5'' c/c}$

Use 2#3 bar @ 6" c/c

Reinforcement details :



Shorter direction



Longer direction

Q. Design a slab panel of size 15′ X 13′. The panel is a corner panel of a group. Use $f_c' = 4000$ psi $f_y = 60000$ psi. L.L = 20 psf. Follow CASD method.

Solution:

i) $A = 13'$ $\quad B = 15'$

$$\frac{B}{A} = 1.15 < 2$$ ∴ Two way slab confirm.

ii) Thickness, $$t = \frac{2(13+15) \times 12}{180} = 3.73 \simeq \cancel{3.75}'' \simeq 4''$$

iii) Load calculation:

$$D.L = \frac{t}{12} \times 150 = \frac{\cancel{3.75}\ 4}{12} \times 150 = 50 \text{ psf}$$

$$LL = 20 \text{ psf}$$

$$\therefore T.L, \quad \omega_T = 50 + 20 = 70 \text{ psf}$$

iv) Moment calculation:

$$m = \frac{13}{15} = 0.87 \quad \text{and case - 4}$$

$$C_{A\,neg} = 0.066 - \frac{0.066 - 0.060}{5} \times 2 = 0.064$$

$$C_{B\,neg} = 0.038$$

$$C_{A\,DL} = 0.035 \qquad C_{A\,LL} = 0.041$$

$$C_{B\,DL} = 0.021 \qquad C_{B\,LL} = 0.025$$

0.85 এর জন্য $C_{A\,neg} = 0.066$

0.90 " " " $= 0.060$

$$C_{A\,neg} = 0.066 - \frac{\overset{}{0.066 - 0.060}}{\underbrace{5}_{\left(\substack{0.85-0.90 \\ 5\text{টা ঘর}}\right)}} \times \underbrace{2}_{\left(\substack{0.85-0.87 \\ 2\text{টা ঘর}}\right)}$$

At continuous end,

$(-)M_A = C_{A\,neg}\, \omega_T L_A^2$

$= 0.064 \times 70 \times (13)^2 = 757.12$ lb-ft

$(-)M_B = C_{B\,neg}\, \omega_T L_B^2$

$= 0.038 \times 70 \times (15)^2 = 598.5$ lb-ft

At mid span,

$(+)M_A = C_{A\,D.L}\, \omega_D L_A^2 + C_{A\,L.L}\, \omega_L L_A^2$

$= 0.035 \times 50 \times (13)^2 + 0.041 \times 20 \times (13)^2$

$= 434.33$ lb-ft

$(+)M_B = C_{B\,D.L}\, \omega_D L_B^2 + C_{B\,L.L}\, \omega_L L_B^2$

$= 0.021 \times 50 \times (15)^2 + 0.025 \times 20 \times (15)^2$

$= 348.75$ lb-ft

v) Depth check:

$d = \sqrt{\dfrac{M}{Rb}} = \sqrt{\dfrac{757.12 \times 12}{297.54 \times 12}}$

$= 1.6''$

$d_{actual} = t - 1'' = 4 - 1 = 3''$

$\therefore d_{actual} > d$ (ok)

$n = \dfrac{29 \times 10^6}{57000\sqrt{4000}} = 8$

$r = \dfrac{24}{0.45 \times 4} = 13.33$

$k = \dfrac{n}{n+r} = 0.38$

$j = 0.87$

$R = \frac{1}{2} f_c j k$

$= 0.5 \times 0.45 \times 4000 \times 0.87 \times 0.38$

$= 297.54$

vi) Reinforcement Calculation:

Short direction:

(+) $A_s = \frac{M}{f_s j d} = \frac{434.33 \times 12}{24000 \times 0.87 \times 3} = 0.083 \text{ in}^2$

$A_{s\,min} = 0.0018\, bt = 0.0018 \times 12 \times 4 = 0.086 \text{ in}^2$

#3 bar, $S = \frac{0.11 \times 12}{0.086} = 15.35 \simeq 15.5''$

$S_{max} = 3t = 3 \times 4 = 12''$ c/c

$\therefore$ Use #3 bar @ 12" c/c

(−) $A_s = \frac{757.12 \times 12}{24000 \times 0.87 \times 3} = 0.15 \text{ in}^2$

$A_{s\,min} = 0.086 \text{ in}^2$

$\therefore A_s = 0.15 \text{ in}^2$

$S = \frac{0.11 \times 12}{A_s(+)} \Rightarrow 12 = \frac{0.11 \times 12}{A_s(+)}$

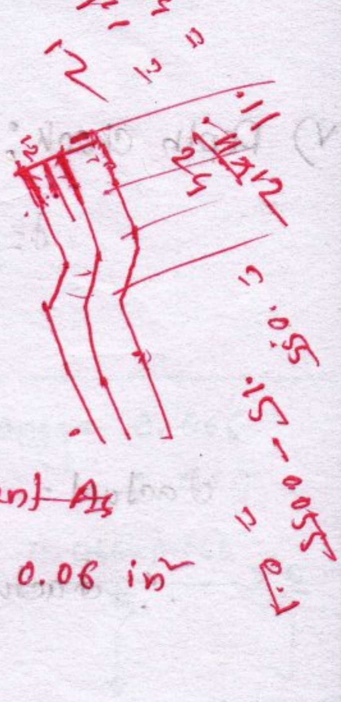
$\therefore A_s(+) = 0.11 \text{ in}^2$

Additional steel $= 0.15 - 0.11 = 0.04 \text{ in}^2$

Use #4 bar, .. $S = \frac{0.11 \times 12}{0.04} = 33''$ c/c

$S_{max} = 3t = 3 \times 4 = 12''$ c/c

Use #3 bar @ 12" c/c



Assuming alternating crank of positive reinforcement ~~A_s~~

$A_s(-)$ available from crank $= \frac{0.11 \times 12}{2 \times 12} = 0.06 \text{ in}^2$

$\therefore A_s(-)$ required $= 0.15 - 0.06 = 0.09 \text{ in}^2$

$S = \frac{0.11 \times 12}{0.09} = 14.67 \simeq 14.75''$ c/c

$S_{max} = 3t = 3 \times 4 = ~~12''~~$ c/c

Long direction:

$$(+)\ A_s = \frac{348.75 \times 12}{24000 \times 0.87 \times 3} = 0.06\ in^2$$

$$A_{s\,min} = 0.0018\ bt = 0.086\ in^2$$

$$S_{max} = 3t = 3 \times 4 = 12''\ c/c$$

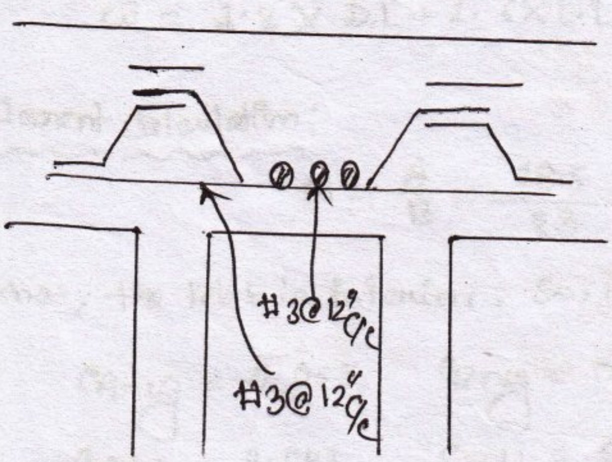
Use #3 bar@ 12" c/c

$$(-)\ A_s = \frac{598.5 \times 12}{24000 \times 0.87 \times 3} = 0.11\ in^2$$

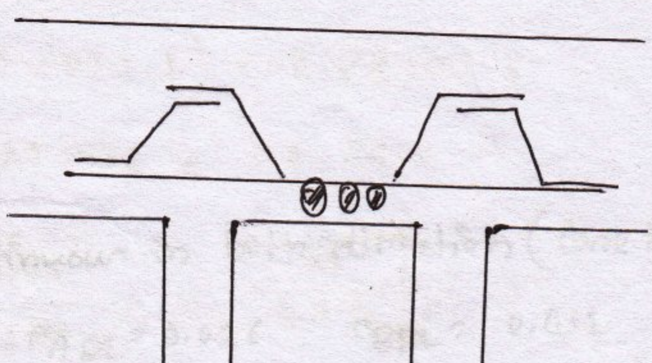
$$s = \frac{0.11 \times 12}{A_s(+)} \Rightarrow 12 = \frac{0.11 \times 12}{A_s(+)}$$

$$A_s(+) = 0.11$$

vii) Reinforcement details:



Shorter direction



CE-317

USD METHOD

06, 07

Prob-01: A 19.5' by 24' interior panel of a floor slab supports a LL of 125 psf. Dimensions are centre to centre of beam lines. Beam width is 12". Design the slab by USD, using $f_c' = 3500$ psi and $f_y = 50000$ psi.

Solution:

$$A = 19.5 - \frac{12}{12} = 18.5'$$

$$B = 24 - \frac{12}{12} = 23'$$

$$\frac{B}{A} = \frac{23}{18.5} = 1.24 < 2 \quad \therefore \text{ Two way slab confirmed}$$

Thickness:

$$t = \frac{2(A+B)}{180} = \frac{2(18.5+23)\times 12}{180} = \cancel{0.46\times}\ 5.53 \approx 5.75''$$

Load calculation:

$$D.L = \frac{t}{12}\times 150 = \frac{5.75}{12}\times 150 = 71.88 \text{ psf}$$

$$L.L = 125 \text{ psf}$$

$$\omega = 1.2\times D.L + 1.6\times L.L = 1.2\times 71.88 + 1.6\times 125 = 286.26 \text{ psf}$$

Moment calculation:

$$m = \frac{A}{B} = \frac{18.5}{23} = 0.80$$

Since, the slab is interior. So, it is continuous in both direction (Case 2)

$CA_{neg} = 0.065$ $CB_{neg} = 0.027$ $CA_{DL} = 0.026$ $CB_{DL} = 0.011$

$CA_{LL} = 0.041$ $CB_{LL} = 0.017$

$$M_A(-) = CA_{neg}\,\omega L_A^2 = 0.065\times 286.26\times (18.5)^2 = 6368.21 \text{ lb-ft}$$

$$M_B(-) = CB_{neg}\,\omega L_B^2 = 0.027\times 286.26\times (23)^2 = 4088.65 \text{ lb-ft}$$

$$M_A(+)_{DL} = CA_{DL}\,\omega_{DL}\,L_A^2 = 0.026\times 71.88\times (18.5)^2 = 639.62\ \cancel{\text{lb-ft}}\times 1.2 = 767.54 \text{ lb-ft}$$

$$M_A(+)_{LL} = CA_{LL}\,\omega_{LL}\,L_A^2 = 0.041\times 125\times (18.5)^2 = 1754.03\ \cancel{\text{lb-ft}}\times 1.6 = 2806.45 \text{ lb-ft}$$

~~C_{BDL} =~~

$M_B(+)_{DL} = C_{B_{DL}}\, \omega_{DL}\, l_B^2 = 0.011 \times 71.88 \times (23)^2 = 418.27 \times 1.2$ lb-ft $= 501.92$ lb-ft

$M_B(+)_{LL} = C_{B_{LL}}\, \omega_{LL}\, l_B^2 = 0.017 \times 125 \times (23)^2 = 1124.13 \times 1.6$ lb-ft $= 1798.9$ lb-ft

$M_A(+) = M_A(+)DL + M_A(+)L.L$

$= 767.54 + 2806.45 = 3574$ lb-ft

$M_B(+) = M_B(+)DL + M_B(+)L.L$

$= 501.92 + 1798.61 = 2300.53$ lb-ft

Depth Check:

$$p = 0.75\, p_b = 0.75 \times 0.85\, \beta_1 \frac{f_c'}{f_y} \times \frac{87}{87 + f_y}$$

$$= 0.75 \times 0.85 \times \frac{3.5}{50} \times \frac{87}{87+50} \times \boxed{0.85}$$

$$= 0.028$$

$$R = pf_y\left(1 - 0.59\, p\frac{f_y}{f_c'}\right)$$

$$= 0.028 \times 50 \times \left(1 - 0.59 \times 0.028 \times \frac{50}{3.5}\right)$$

$$= 1.07 \text{ ksi} = 1069 \text{ psi}$$

$$d = \sqrt{\frac{M}{\phi R b}} = \sqrt{\frac{6368.21 \times 12}{0.85 \times 1069 \times 12}} = 2.65''$$

$$d_{short} = t - cover - \frac{\phi}{2} = 5.75 - \frac{3}{4} - \frac{3}{8 \times 2} \quad (\text{Assuming \#3 bar})$$

$$= 4.81''$$

$$d_{long} = d_{short} - \phi = 4.81 - \frac{3}{8} = 4.44''$$

$\therefore d <$ deffective (OK)

...g direction:

$$\cancel{M_B(+)} \quad A_s(+) = \frac{M_B(+)}{\phi f_y (d - \frac{a}{2})} = \frac{2300.53 \times 12}{0.85 \times 50000 (4.44 - 0.7 A_s)}$$

$$\Rightarrow 4.44 A_s(+) - 0.7 A_{s(+)}^2 = 0.65$$

$$\therefore A_s(+) = 0.15 \text{ in}^2$$

$$s = \frac{0.11 \times 12}{0.15} = 8.8 \simeq 8.75''.$$

$$A_s(-) = \frac{4088.65 \times 12}{0.85 \times 50000 \times (4.44 - 0.7 A_s)}$$

$$\Rightarrow 4.44 A_s(-) - 0.7 A_s^2(-) = \cancel{0.096} \; 1.15$$

$$\therefore A_s(-) = \cancel{0.02 \text{ in}^2} \; 0.27 \text{ in}^2$$

$$s = \frac{0.11 \times 12}{A_s(+)} \Rightarrow A_s(+) = \frac{0.11 \times 12}{8.75} = 0.15 \text{ in}^2$$

Additional steel $= A_s(-) - A_s(+)$

$= 0.27 - 0.15 = 0.12 \text{ in}^2$

Use 1 ~~#4 bar~~ extra top bar

Shear Check:

$$V_d = 1.15 \frac{wl}{2} = 1.15 \times \frac{286.26 \times 23}{2} = 3785.79 \text{ lb}$$

$$V_{allowable} = 2\phi\sqrt{f_c'}\, bd = 2 \times 0.75 \times \sqrt{3500} \times 12 \times 4.44$$

$$= 4728.13 \text{ lb}$$

$\therefore V_d < V_{allowable}$ (ok)

Steel Calculation:

Short direction:

$$As(+) = \frac{M}{\phi f_y\left(d - \frac{a}{2}\right)} = \frac{3574 \times 12 \times 10^{-3}}{0.85 \times 50 \times \left(4.81 - \frac{a}{2}\right)} \quad -(1)$$

$$a = \frac{As f_y}{0.85 f_c' b} = \frac{As \times 50}{0.85 \times 3.5 \times 12} \Rightarrow \frac{a}{2} = 0.7\, As(+)$$

$$(1) \Rightarrow \quad As(+) = \frac{3574 \times 12 \times 10^{-3}}{0.85 \times 50 \times (4.81 - 0.7 As)}$$

$$\Rightarrow 4.81\, As(+) - 0.7\, As^2(+) = \cancel{1009.13}\ 1.01$$

$$\therefore As(+) = 0.22\, in^2$$

$$S = \frac{0.11 \times 12}{0.22} = 6''$$

$$As(-) = \frac{6368.21 \times 12}{0.85 \times 50000 \times (4.81 - 0.7 As)}$$

$$\Rightarrow 4.81\, As(-) - 0.7\, As^2(-) = 1.8$$

$$\therefore As(-) = 0.4\, in^2$$

Now,

$$S = \frac{0.11 \times 12}{As(+)} \Rightarrow As(+) = \frac{0.11 \times 12}{6} = 0.22\, in^2$$

$$\therefore \text{Additional steel} = As(-) - As(+) = 0.4 - 0.22 = 0.18\, in^2$$

Use 2 #3 top bar

nd check:

$$u_d = \frac{V_d}{\Sigma o \left(d - \frac{a}{2}\right)} = \frac{3785.79}{2 \times \frac{3}{8} \times \frac{12}{8.75} \times (4.44 - 0.7 \times 0.22)}$$

$$= 546.7 \text{ psi}$$

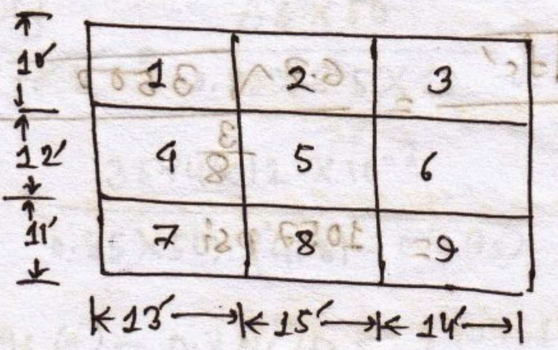
$$u_{allowable} = \frac{6.7\sqrt{f_c'}}{D} = \frac{6.7\sqrt{3500}}{\frac{3}{8}}$$

$$= 1057 \text{ psi}$$

$\therefore$ $u_d < u_{allowable}$. (OK).

Prob-02: The floor slab of a building as shown in figure is supported by beams at each grid line. Design panel 2 for LL of 75 psf using $f_c' = 3000$ psi $f_y = 50000$ psi, beam width 15".

Solution:



$$A = 10 - \frac{15}{12} = 8.75'$$

$$B = 15 - \frac{15}{12} = 13.75'$$

Thickness:

$$t = \frac{2(A+B) \times 12}{180} = \frac{2(8.75 + 13.75) \times 12}{180} = 3"$$

Load calculation:

$$D.L = \frac{t}{12} \times 150 = \frac{3}{12} \times 150 = 37.5 \text{ psf}$$

$$L.L = 75 \text{ psf}$$

$$\omega = 1.2 \times D.L + 1.6 \times L.L = 1.2 \times 37.5 + 1.6 \times 75 = 165 \text{ psf}$$

Moment calculation:

$$m = \frac{A}{B} = \frac{8.75}{13.75} = 0.64 \quad \text{and case-8}$$

$$C_{A\,neg} = 0.080 - \frac{0.080 - 0.074}{5} \times 4 = 0.0752$$

$$C_{B\,neg} = 0.024 - \frac{0.024 - 0.018}{5} \times 1 = 0.0228$$

$$C_{A\,DL} = 0.048 - \frac{0.048 - 0.044}{5} \times 4 = 0.0448$$

$$C_{BDL} = 0.009 - \frac{0.009 - 0.007}{5} \times 1 = 0.0086$$

$$C_{ALL} = 0.065 - \frac{0.065 - 0.059}{5} \times 4 = 0.0602$$

$$C_{BLL} = 0.011 - \frac{0.011 - 0.009}{5} \times 1 = 0.0106$$

$$M_{A(-)} = C_{Aneg}\, w L_A^2 = 0.0752 \times 165 \times (8.75)^2 = 949.99 \text{ lb-ft}$$

$$M_{B(-)} = C_{Bneg}\, w L_B^2 = 0.0228 \times 165 \times (13.75)^2 = 711.25 \text{ lb-ft}$$

$$M_{A(+)DL} = C_{AD.L}\, w_{D.L} L_A^2 = 0.0448 \times 37.5 \times (8.75)^2 = 128.63 \text{ lb-ft}$$

$$M_{A(+)LL} = C_{ALL}\, w_{L.L} L_A^2 = 0.0602 \times 75 \times (8.75)^2 = 345.68 \text{ lb-ft}$$

$$M_{B(+)DL} = C_{BD.L}\, w_{D.L} L_B^2 = 0.0086 \times 37.5 \times (13.75)^2 = 60.97 \text{ lb-ft}$$

$$M_{B(+)L.L} = C_{BL.L}\, w_{L.L} L_B^2 = 0.0106 \times 75 \times (13.75)^2 = 150.30 \text{ lb-ft}$$

$$M_{A(+)} = 128.63 + 345.68 = 474.31 \text{ lb-ft}$$

$$M_{B(+)} = 60.97 + 150.30 = 211.27 \text{ lb-ft}$$

Depth check:

$$\rho = 0.75\,\rho_b = 0.75 \times 0.85 \times \frac{f_c'}{f_y} \times \frac{87}{87 + f_y}$$

$$= 0.75 \times 0.85 \times \frac{3}{50} \times \frac{87}{87 + 50}$$

$$= 0.0243$$

$$R = \rho f_y \left(1 - 0.59 \rho \frac{f_y}{f_c'}\right)$$

$$= 0.0243 \times 50 \left(1 - 0.59 \times 0.0243 \times \frac{50}{3}\right)$$

$$= 0.925 = 925 \text{ psi}$$

$$d = \sqrt{\frac{M}{\phi R b}} = \sqrt{\frac{949.99 \times 12}{0.85 \times 925 \times 12}} = 0.98''$$

$d_{short} = t - cover - \frac{\phi}{2} = 3 - \frac{3}{4} - \frac{3}{2\times8} = 2.06''$

$d_{long} = d_{short} - \phi = 2.06 - \frac{3}{8} = 1.685''$

$\therefore d < d_{effective}$ (OK)

Steel Calculation:

Short

$$As(+) = \frac{M}{\phi f_y (d - \frac{a}{2})} = \frac{474.31\times12}{0.85\times50\times(2.06 - \frac{a}{2})} = \frac{133.92}{(2.06 - \frac{a}{2})} \quad (1)$$

$$a = \frac{As f_y}{0.85 f_c' b} = \frac{As(+)\times50}{0.85\times3\times15} = 1.31\,As$$

$\frac{a}{2} = 0.66\,As(+)$

$$(1) \Rightarrow \quad As(+) = \frac{133.92}{(2.06 - 0.66\,As)}$$

$\Rightarrow 2.06\,As(+) - 0.66\,As(+)^2 = 133.92$

$\therefore As(+) = 1.56\ in^2$

$$S = \frac{0.11\times12}{1.56} =$$

(7)

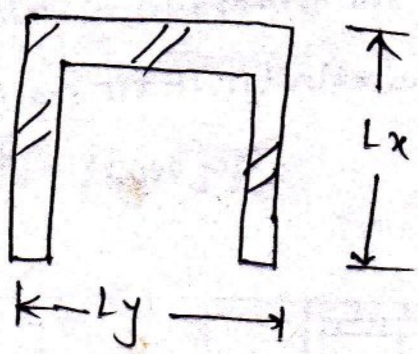
Two way Slab

Different form

~~Cantilever~~

*

~~Simple supported~~



$$r = \frac{Ly}{Lx}$$

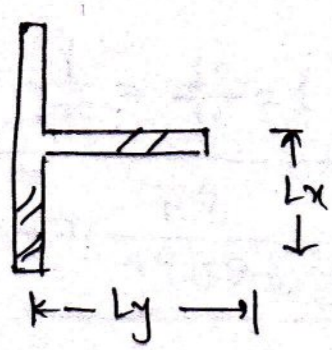
$$W_x = w \frac{r^4}{r^4 + 9.6}$$

$$W_y = w - W_x$$

$$M_{max} = \frac{1}{2} W_x L_x^2$$

$$M_{max} = \frac{1}{8} W_y L_y^2$$

*



$$W_y = \frac{w}{1 + r^4}$$

$$W_x = w - W_y$$

* **Isolated simple support rectangular:**

$$\frac{W_a}{W_b} = \left(\frac{L_b}{L_a}\right)^4$$

L_a = Short direction
L_b = Long "

$$W = W_a + W_b$$

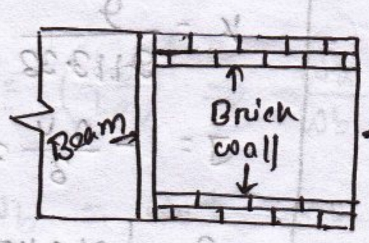
$$M_a = \frac{1}{8} W_a L_a^2 \qquad M_b = \frac{1}{8} W_b L_b^2$$

* **Isolated simple support square:**

$$W_a = W_b \qquad M_{max} = \frac{1}{16} W L^2$$

2011

Q.5. Desig a balcony slab 10 ft by 10 ft to support a L.L of 100 psf. The slab is supported on brick walls on two opposite sides and on a beam on the third side as shown in fig below. Assume $f_c' = 3000$ psi and $f_s = 18000$ psi.

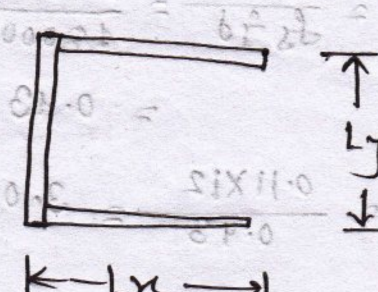


Solution:

$L_x = 10'$ $L_y = 10'$

$r = \frac{L_y}{L_x} = \frac{10}{10} = 1$

$$\boxed{w_x = w \cdot \frac{r^4}{r^4 + 9.6}}$$ (formula) class lecture



$= w \frac{1}{1+9.6} = 0.094\ w$

$D.L = \frac{t}{12} \times 150$

$t = \frac{2(A+B) \times 12}{180} = \frac{2(10+10) \times 12}{180} = 2.67''$

$\simeq 4''$

$D = \frac{4}{12} \times 150 = 50$ psf

$L.L = 100$ psf

$\therefore\ w = 150$ psf

$w_x = 0.094 \times 150 = 14.1$ psf

$w_y = w - w_x = 150 - 14.1 = 135.9$ psf

$M_x = \frac{1}{2} w_x L_x^2 = \frac{1}{2} \times 14.1 \times (10)^2 = 705$ lb-ft

$M_y = \frac{1}{8} w_y L_y^2 = \frac{1}{8} \times 135.9 \times (10)^2 = \cancel{6795 \text{ lb-ft}}\ 1698.75$ lb-ft

$d = \sqrt{\frac{M}{Rb}}$

$= \sqrt{\frac{\cancel{6795 \times 12}\ 1698.75 \times 12}{\cancel{261} \times 12 \times 234.9}}$

$= \cancel{5.10''}\ 2.69''$

$d_{eff} = t - 1 = 4 - 1 = 3'' > d$

(OK)

$n = \frac{29 \times 10^6}{57000\sqrt{3000}} = 29$

$r = \frac{18}{0.45 \times 3} = 13.33$

$k = \frac{9}{9 + 13.33} = 0.4$

$j = 1 - \frac{0.4}{3} = 0.87$

$R = \frac{1}{2} \times 0.45 \times 3000 \times 0.87 \times 0.4$

$= \cancel{261.13}\ 234.9$ lb

Reinforcement calculation: (Short direction)

$A_s = \frac{M}{f_s j d} = \frac{1698.75 \times 12}{18000 \times 0.87 \times 3}$

$= 0.43 \text{ in}^2$

$S = \frac{0.11 \times 12}{0.43} = 3.06 \simeq 3''$ c/c

$A_{s\,min} = 0.0020 \times bt = 0.0020 \times 12 \times 4 = 0.096 \text{ in}^2$

Reinforcement calculation for $M = 705$ lb-ft

$d_{long} = d_{short} - \phi$

$\cancel{d_{short} = d_{long} - \phi}$

$= 3 - \frac{3}{8}$

$= 2.63''$

$A_s = \frac{705 \times 12}{18000 \times 0.87 \times 2.63} = \cancel{0.18 \text{ in}^2}\ 0.21$

$S = \frac{0.11 \times 12}{\cancel{0.18}\ 0.21} = \cancel{7.33 \simeq 7.5'' \text{ c/c}}\ 6.29$

$\simeq 6.3''$ c/c

$A_{s\,min} = 0.096 \text{ in}^2$

2010

6. (b) Design a two-way isolated slab panel of size 12 ft by 18 ft. Assume
(i) All sides are supported on simple supports (ii) D.L = 30 psf (iii) L.L = 80 psf
(iv) $f_c' = 4$ ksi (v) $f_y = 60$ ksi

Solution:

$l_a = 12'$ $\quad L_b = 18'$

$$\frac{w_a}{w_b} = \left(\frac{L_b}{L_a}\right)^4 \Rightarrow \frac{w_a}{w_b} = \left(\frac{18}{12}\right)^4 \Rightarrow w_a = 5.06\, w_b$$

$$w = w_a + w_b = \cancel{w_a +}\ 5.06 w_b + w_b = 6.06\, w_b$$

i) Self weight $= \frac{t}{12} \times 150$ $\qquad t = \frac{2(12+18)\times 12}{180} = 4''$

$= \frac{4}{12} \times 150 = 50$ psf

ii) D.L = 30 psf $\qquad$ iii) L.L = 80 psf

$w = 50 + 30 + 80 = 160$ psf

$160 = 6.06\, w_b$ $\quad \therefore w_b = 26.40$ psf

$\therefore w_a = 133.58$ psf

$$M_a = \frac{w_a L_a^2}{8} = \frac{133.58 \times (12)^2}{8} = 2404.44 \text{ lb-ft}$$

$$M_b = \frac{26.40 \times (18)^2}{8} = 1069.2 \text{ lb-ft}$$

Depth check:

$$d = \sqrt{\frac{M}{Rb}}$$

$$= \sqrt{\frac{2404.44 \times 12}{297.54 \times 12}}$$

$= 2.84''$

$d_{eff} = t - 1 = 4 - 1 = 3'' > d$ (ok)

$$n = \frac{29 \times 10^6}{57000\sqrt{4000}} = 8.04 \approx 8$$

$$r = \frac{24}{0.45 \times 4} = 13.33$$

$$k = \frac{8}{8 + 13.33} = 0.38$$

$j = 0.87$

$R = 297.54$

Reinforcement Calculation: (Short direction)

$$A_s = \frac{M}{f_s j d} = \frac{2404.44 \times 12}{24000 \times 0.87 \times 3}$$

$$= 0.46 \text{ in}^2$$

$$S = \frac{0.11 \times 12}{0.46} = 2.87 \simeq 2.75'' \text{ C/C}$$

$$A_{smin} = 0.0018\,bt = 0.0018 \times 12 \times 4 = 0.09 \text{ in}^2$$

Using #3 bar @ 2.75" c/c

Long direction:

$$A_s = \frac{1069.2 \times 12}{24000 \times 0.87 \times 2.63}$$

$$d_{long} = d_{short} - \phi = 3 - \frac{3}{8} = 2.63''$$

$$= 0.23 \text{ in}^2$$

$$S = \frac{0.11 \times 12}{0.23} = 5.74 \simeq 5.75'' \text{ c/c}$$

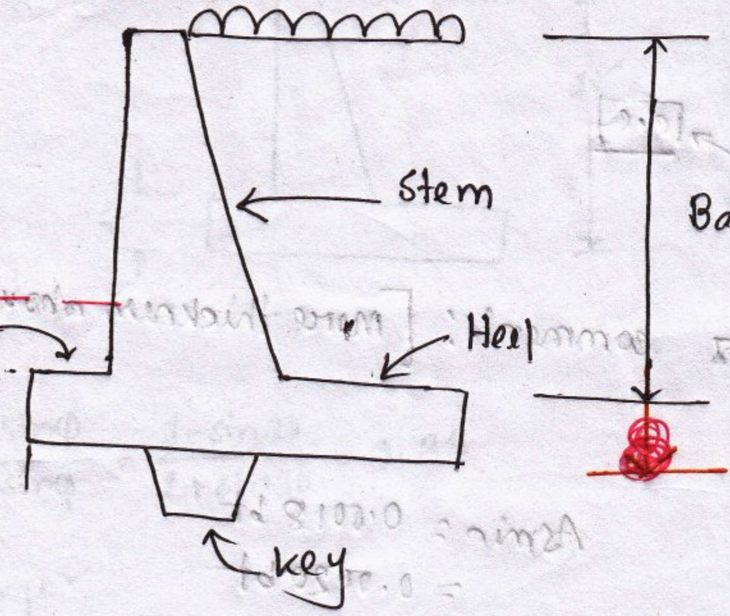
$$A_{smin} = 0.0018\,bt = 0.09 \text{ in}^2$$

Using #3 bar @ 5.75" c/c

⑥

Retaining Wall

AHSAN
090063



$$K_a = \frac{1-\sin\phi}{1+\sin\phi} \qquad K_p = \frac{1}{K_a}$$

$$h_s = \frac{W_a}{\gamma_s}$$ → surcharge load

Externel stability check করতে বললে:

i) F.S against overturning $= \dfrac{M_R}{M_O} > 1.5$ (ok) → Resisting moment

ii) F.S " sliding $= \dfrac{W \times f}{P_R} > 1.5$ (ok)

< 1.5 হলে key provide করতে হবে

iii) Soil pressure:

$$a = \frac{M_R - M_O}{W}$$ [দেখতে হবে middle third এ আছে কিনা]

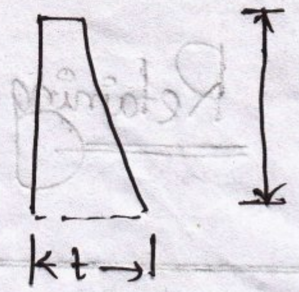
$$e = \frac{b}{2} - a \qquad M = W \times e \qquad A = 1 \times b \qquad I = \frac{1 \times b^3}{12}$$

$$P = \frac{W}{A} \pm \frac{Mc}{I}$$

Design of stem:

P = Vmax =

M =



Depth check: $d = \sqrt{\frac{M}{Rb}}$

$d_{eff} = t - 3''$ → $\boxed{c.c}$

$d_{eff} > d$ (OK)

* $d_{eff} < d$ হয় তাহলে comment: [More thickness should be provided]

Reinforcement Calculation:

$$A_s = \frac{M}{f_s j d} \qquad A_{s\,min} = 0.0018\,bt$$
$$= 0.0020\,bt$$

Development length:

$$L_d = \frac{f_s \phi}{4 u_d}$$

$$u_d = \frac{V_{max}}{\Sigma_0 j d} = \frac{V_{max}}{\Sigma_0 \times \phi \times \frac{b}{spacing} \times j d}$$

Shear check:

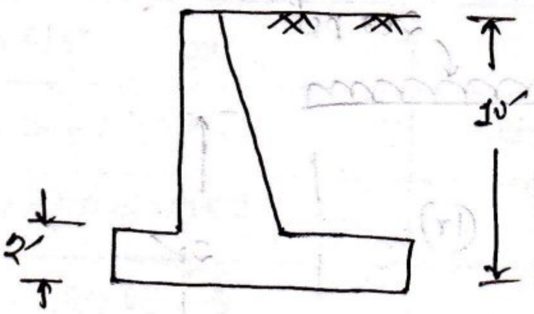
$$V_d = \frac{V_{max}}{bd} \qquad V_c = 1.1\sqrt{f_c'} \qquad V_c > V_d \text{ (OK)}$$

Design of heel:

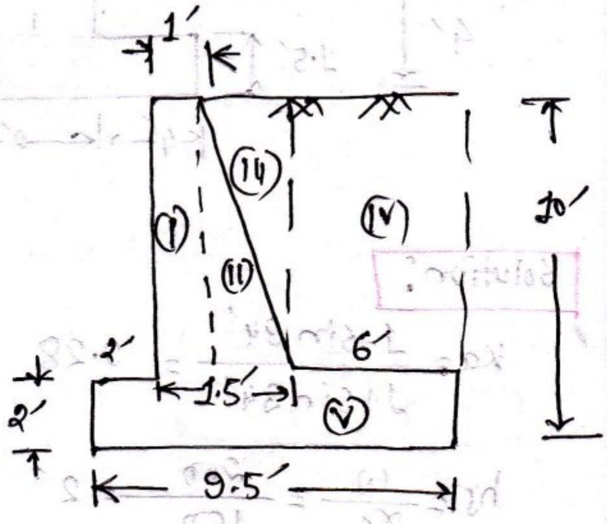
Design of toe

2007

8. (a) Calculate the total vertical load of retaining wall that will be safe against sliding F.S = 2 $f = 0.5$ unit weight of soil = 120 pcf, $\phi = 30^\circ$



Solution:



$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30}{1+\sin 30} = 0.33$$

$$P_a = \frac{1}{2} K_a \gamma H^2$$
$$= \frac{1}{2} \times 0.33 \times 120 \times (10)^2$$
$$= 1980 \text{ lb}$$

$$M_o = 1980 \times \frac{10}{3} = 6600 \text{ lb-ft}$$

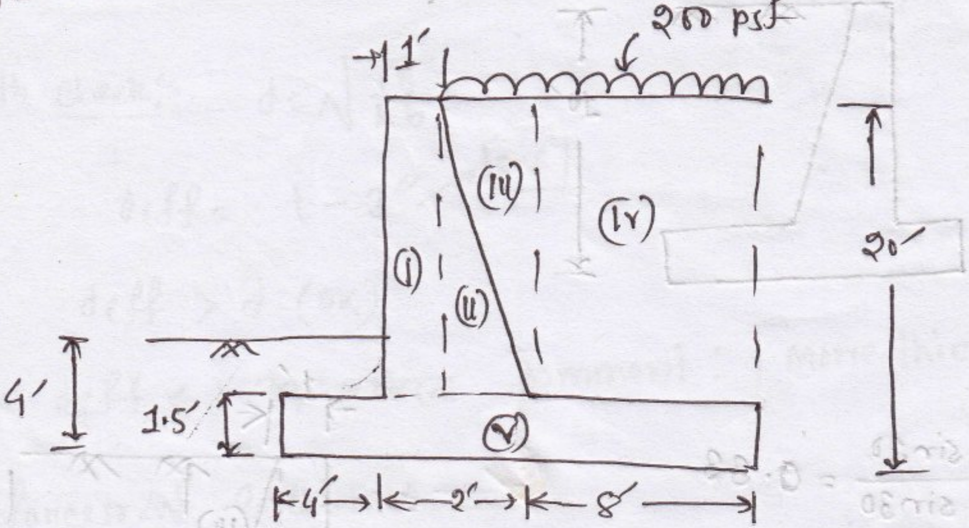
Section	W (lb)	Moment arm (ft)	Moment lb-ft
(I)	$8 \times 1 \times 150 = 1200$	2.5	3000
(II)	$\frac{1}{2} \times 0.5 \times 8 \times 151 = 300$	$3 + \frac{0.5}{3} = 3.17$	951
(III)	$\frac{1}{2} \times 0.5 \times 8 \times 120 = 240$	$3 + \frac{2 \times 0.5}{3} = 3.33$	799.2
(IV)	$6 \times 8 \times 120 = 5760$	$3.5 + \frac{6}{2} = 6.5$	37440
(V)	$9.5 \times 2 \times 150 = 2850$	4.75	13537.5
	$\Sigma W = 10350$		$\Sigma M_R = 55727.7$

$\therefore$ F.S against sliding $= \frac{W \times f}{P_a} = \frac{10350 \times 0.5}{1980} = 2.61 > 2$ (OK)

Ans: 10350 lb

2007

8.(b) Design the heel of the retaining wall shown in fig below. Assuming $\phi = 34°$, $f = 0.5$ unit weight of soil = 100 pcf, $q_a = 3$ ksf, $f_c' = 3$ ksi, $f_y = 60$ ksi.



Solution:

$$K_a = \frac{1 - \sin 34'}{1 + \sin 34'} = 0.28$$

$$h_s = \frac{\omega}{\gamma_s} = \frac{200}{100} = 2'$$

$P_1 = K_a \gamma h_s = 0.28 \times 100 \times 2 = 56$ lb

$P_2 = K_a \gamma H = 0.28 \times 100 \times 20 = 560$ lb

$P_a = 56 \times 20 + \frac{1}{2} 560 \times 20 = \cancel{12320 \text{ lb}}\ 6720$ lb

$$M_a = 56 \times 20 \times \frac{20}{2} + \frac{1}{2} \times 560 \times 20 \times \frac{20}{3}$$

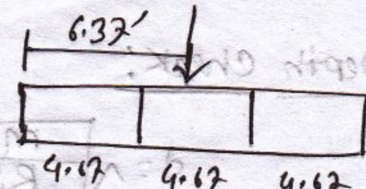
$= 48533.33$ lb-ft

Section	W (lb)	Moment arm (ft)	Moment MR lb.ft
(i)	$1\times18.5\times150 = 2775$	$4+\frac{1}{2} = 4.5$	12487.5
(ii)	$\frac{1}{2}\times1\times18.5\times150 = 1387.5$	$5+\frac{1}{3} = 5.33$	7400
(iii)	$\frac{1}{2}\times1\times18.5\times100 = 925$	$5+\frac{2}{3} = 5.67$	5241.67
(iv)	$8\times18.5\times100 = 14800$	$6+\frac{8}{2} = 10$	148000
(v)	$1.5\times14\times150 = 3150$	$\frac{14}{2} = 7$	22050
	$\Sigma W = 23037.5$		$\Sigma = 195179.17$

(i) F.S against overturning $= \dfrac{MR}{Mo} = \dfrac{195179.17}{48533.33} = 4.02 > 1.5$

(ii) $a = \dfrac{MR - Mo}{W} = \dfrac{195179.17 - 48533.33}{23037.5} = 6.37'$

which will be on middle portion



$e = \dfrac{b}{2} - a = \dfrac{14}{2} - 6.37 = 0.63$

$M = \Sigma W \times e = 23037.5 \times 0.63 = 14513.63$ lb.ft

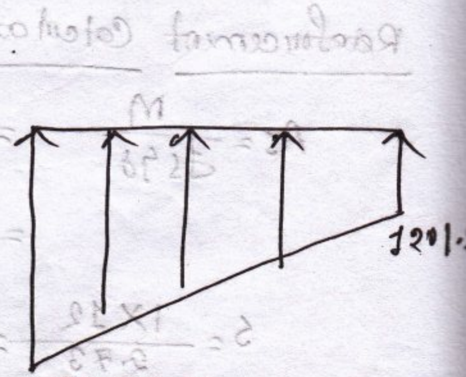
$A = b\times1 = 14\ ft^2$

$I = \dfrac{1\times(14)^3}{12} = 228.67\ ft^4$

$P_E = \dfrac{\Sigma W}{A} \pm \dfrac{Mc}{I}$

$P_1 = \dfrac{23037.5}{14} + \dfrac{14513.63\times\frac{14}{2}}{228.67}$

$= 1645.54 + 444.29 = 2089.83$ lb ksf



$P_2 = 1645.54 - 444.29 = 1201.25$ lb ksf

(iv) F.S against sliding $= \frac{\omega \times f}{P_a} = \frac{23037.5 \times 0.5}{6720}$

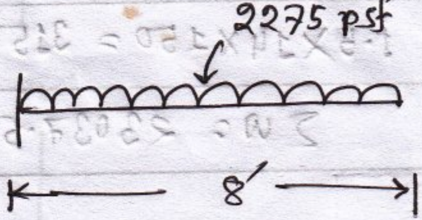
$= 1.71 > 1.5$ (OK)

Design of heel;

i) Self weight $= 1.5 \times 150 = 225$ psf

ii) Soil " $= 18.5 \times 8 \times 100 = 1850$ psf

iii) Surcharge $= 200$ psf



$\omega = 2275$ psf

$M_{max} = \frac{\omega L^2}{2} = \frac{2275 \times (8)^2}{2} = 72800$ lb-ft

$V_{max} = \frac{\omega L}{2} = \frac{2275 \times 8}{2} = 9100$ lb

Depth check;

$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{72800 \times 12}{204.26 \times 12}}$

$= 18.88''$

1.5×12

$d_{effective} = 18 - 3 = 15'' < d$ (Not OK)

$\therefore$ more thickness should be provided

$n = \frac{29 \times 10^6}{57000\sqrt{3000}}$

$= 9.28 \simeq 9$

$r = \frac{24}{0.45 \times 3} = 17.78$

$k = \frac{9}{9 + 17.78} = 0.34$

$j = 1 - \frac{k}{3} = 0.89$

$R = \frac{1}{2} f_c j k$

$= \frac{1}{2} \times 0.45 \times 3000 \times 0.89 \times 0.34$

$= 204.26$ lb

Reinforcement Calculation;

$A_s = \frac{M}{f_s j d} = \frac{72800 \times 12}{24000 \times 0.89 \times 15}$

$= 2.73$ in^2

$S = \frac{1 \times 12}{2.73} = 4.39 \simeq 4.5''$ c/c

$A_{s\,min} = 0.0018\,bt = 0.0018 \times 12 \times 18 = 0.39$ in^2

$S = \frac{0.31 \times 12}{0.39} = 9.53 \simeq 9.5''$ c/c

Development length:

$$L_d = \frac{f_s \phi}{4 u_d}$$

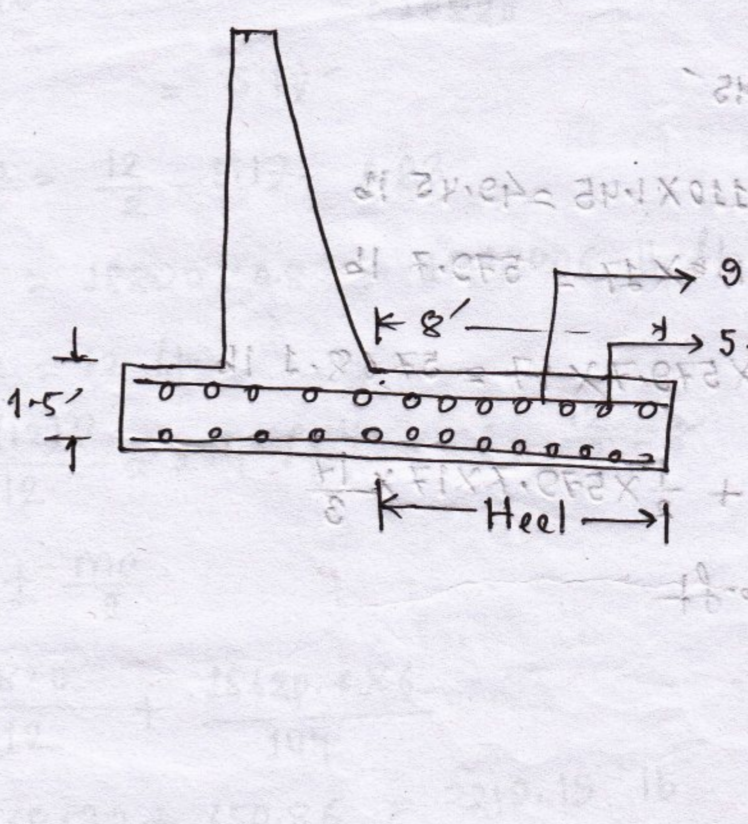
$$u_d = \frac{V_{max}}{\Sigma_o\, j\, d} = \frac{9100}{\pi \times \frac{9}{8} \times \frac{12}{4.5} \times 0.89 \times 15} = 72.32$$

$$L_d = \frac{24000 \times \frac{9}{8}}{4 \times 72.32} = 93.34''$$

Shear Check:

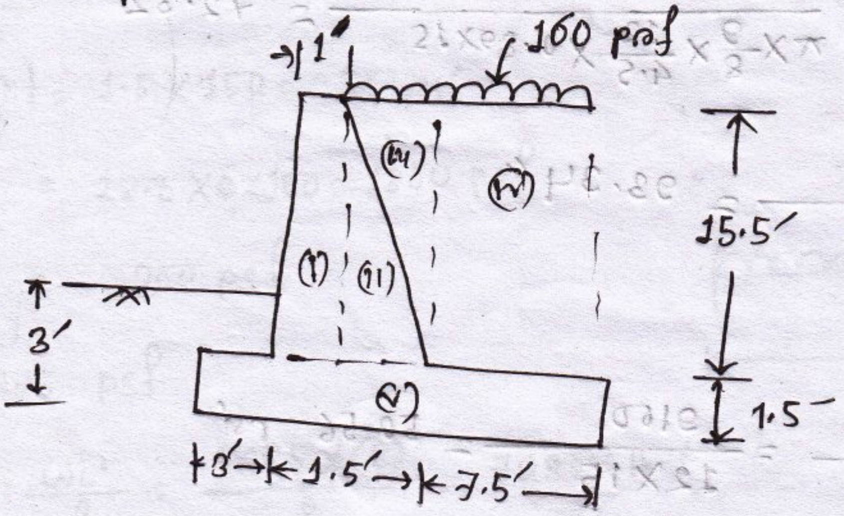
$$V_u = \frac{V_{max}}{bd} = \frac{9100}{12 \times 15} = 50.56 \text{ psi}$$

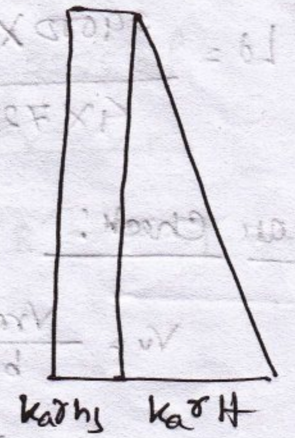
$$V_c = 1.1\sqrt{3000} = 60.25 \text{ psi} > V_u \text{ (ok)}$$



2006 | 05 | 04 | 03

6.(b) Design the stem of the retaining wall shown in the figure below. Necessary data are $\phi = 32^\circ$, $f = 0.4$, unit weight of soil $= 110$ pcf, $f_c' = 3000$ psi and $f_y = 60000$ psi





Solution:

$$K_a = \frac{1-\sin 32^\circ}{1+\sin 32^\circ} = 0.31$$

$$h_s = \frac{w}{\gamma_s} = \frac{160}{110} = 1.45'$$

$$P_1 = K_a \gamma h_s = 0.31 \times 110 \times 1.45 = 49.45 \text{ lb}$$

$$P_2 = K_a \gamma H = 0.31 \times 110 \times 17 = 579.7 \text{ lb}$$

$$P_a = 49.45 \times 17 + \frac{1}{2} \times 579.7 \times 17 = 5768.1 \text{ lb}$$

$$M_a = 49.45 \times 17 \times \frac{17}{2} + \frac{1}{2} \times 579.7 \times 17 \times \frac{17}{3}$$

$$= 35067.74 \text{ lb-ft}$$

Section	W (lb)	Moment arm (ft)	Moment lb-ft M_R
(i)	$1 \times 15.5 \times 150 = 2325$	3.5	8137.5
(ii)	$\frac{1}{2} \times 0.5 \times 15.5 \times 150 = 581.25$	$4 + \frac{0.5}{3} = 4.17$	2421.88
(iii)	$\frac{1}{2} \times 0.5 \times 15.5 \times 110 = 426.25$	$4 + \frac{2 \times 0.5}{3} = 4.33$	142.08
(iv)	$7.5 \times 15.5 \times 110 = 12787.5$	$4.5 + \frac{7.5}{2} = 8.25$	105496.88
(v)	$12 \times 1.5 \times 150 = 2700$	$\frac{12}{2} = 6'$	16200
	$\Sigma W = 18820$		$\Sigma M_R = 132398.34$

1) F.S against overturning $= \frac{M_R}{M_0} = \frac{132398.34}{\cancel{18820}\ 35067.74}$

$= \cancel{7 > 1.5}\ 3.78 > 1.5$ (ok)

4) $a = \frac{M_R - M_0}{W} = \frac{132398.34 - 35067.74}{18820}$

$= 5.17$

$e = \frac{b}{2} - a = \frac{12}{2} - 5.17 = 0.83$

$M = W \times e = 18820 \times 0.83 = 15620.6$ lb-ft

$A = 1 \times 12 = 12\ ft^2$

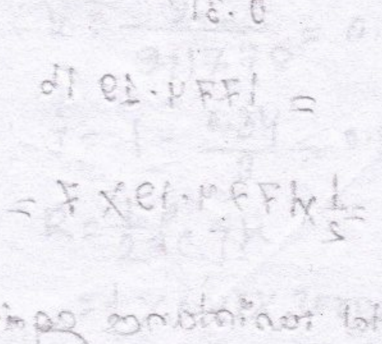
$I = \frac{1 \times (12)^3}{12} = 144\ ft^4 \qquad C = \frac{12}{2} = 6$

$P_t = \frac{W}{A} \pm \frac{MC}{I}$

$P_1 = \frac{18820}{12} + \frac{15620.6 \times 6}{144}$

$= 1568.33 + 650.86 = 2219.19$ lb

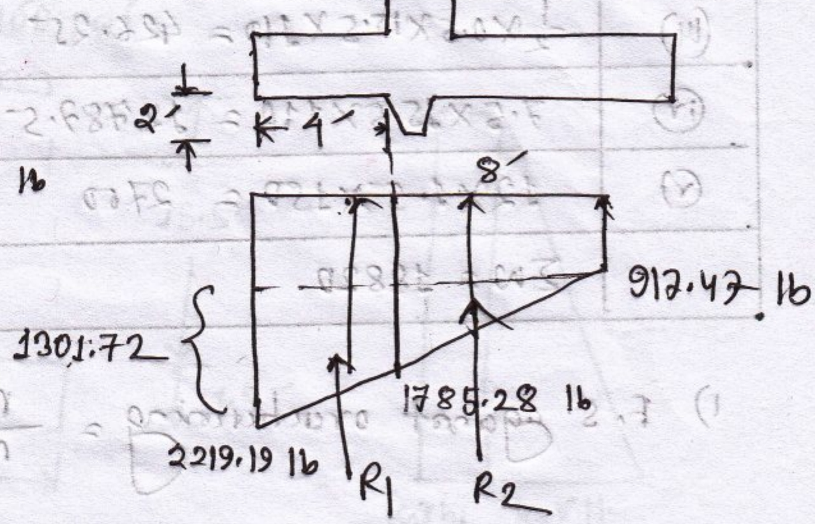
$P_2 = 1568.33 - 650.86 = 917.47$ lb



iii) F.S against sliding $= \frac{W_1 \times f}{P_a} = \frac{18820 \times 0.4}{5768.1} = 1.31 < 1.5$

(Not ok)

The resistance provided does not give safety against sliding. Therefore, key should be provided.



$R_1 = \frac{1}{2} \times 4 \times (2219.19 + 1785.28) = 8008.94$ lb

$R_2 = \frac{1}{2} \times 8 \times (1785.28 + 917.47)$

$= 10811$ lb

$\frac{1301.72}{12} = \frac{x}{8}$

$x = 867.81$

$p = \frac{1}{\cancel{2}} \times \ k_p \gamma H = \frac{1}{K_a} \gamma H$

$= \frac{1}{0.31} \times 110 \times 5$ → $\boxed{\text{বাম পাশের soil + key এর height}}$

$= 1774.19$ lb

$P_P = \frac{1}{2} \times 1774.19 \times 7 = 6209.68$ lb

$f_1 = \tan 0.4$

$= 0.62$

Total resistance against sliding $= \cancel{R_1 f_1 \times R_2 f_2 + P_P}$

$= R_1 f_1 + R_2 f_2 + P_P$

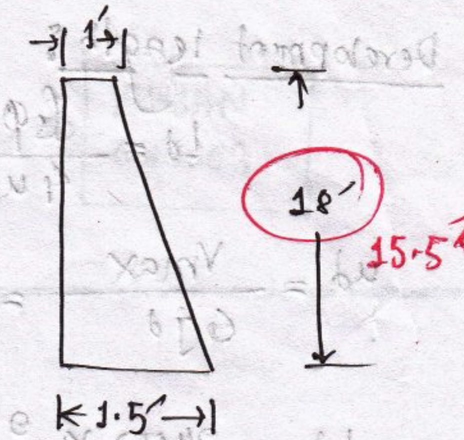
$= 8008.94 \times 0.62 + 10811 \times 0.4 + 6209.68$

$= 15499.62$ lb

$\therefore$ F.S against sliding $= \frac{15499.62}{\cancel{18820}\ 5768.1} = 2.69 > 1.5$

(OK)

Design of stem:



$$P_a = 49.45 \times \overset{K_a\gamma H}{\boxed{18}} + \frac{1}{2}\times 0.3 \times 110 \times \boxed{18\times 18}$$

$$= 6236.1 \text{ lb} = V_{max}$$

$$M = 49.45 \times \boxed{18 \times \frac{18}{2}} + \underbrace{\frac{1}{2}\times 0.3\times 110\times \boxed{18\times 18}}_{\boxed{\frac{1}{2}K_a\gamma H^2}} \times \frac{18}{3}$$

$$= 8010.9 + 32076 = 40086.9 \text{ lb-ft}$$

Depth check:

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{40086.9\times 12}{204.26\times 12}}$$

$$= 14''$$

$$d_{eff} = 1.5\times 12 - 3 = 15'' > d \quad \text{(ok)}$$

$$n = \frac{29\times 10^6}{57000\sqrt{3000}} \simeq 9$$

$$n = \frac{24}{0.45\times 3} = 17.78$$

$$k = \frac{9}{9+17.78} = 0.34$$

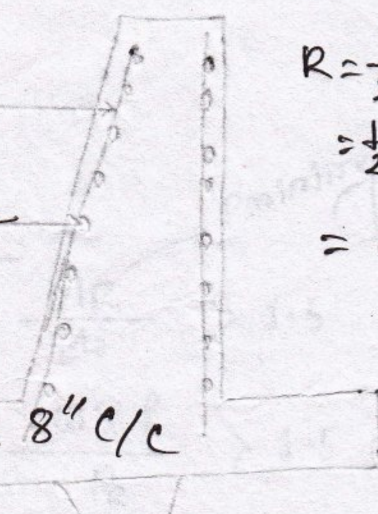
$$j = 1 - \frac{0.34}{3} = 0.89$$

$$R = \frac{1}{2} f_c j k$$

$$= \frac{1}{2}\times 0.45\times 3000\times 0.89 \times 0.34$$

$$= 204.26 \text{ lb}$$

Reinforcement calculation:



$$A_s = \frac{M}{f_s j d} = \frac{40086.9\times 12}{24000\times 0.89\times 15}$$

$$= 1.50 \text{ in}^2$$

using #9 bar, $s = \frac{1\times 12}{1.50} = 8'' \text{ c/c}$

$$A_{s\,min} = 0.0018\,bt = 0.0018\times 12\times 18 = 0.39 \text{ in}^2$$

using #5 bar, $s = \frac{0.31\times 12}{0.39} = 9.54 \simeq 9.50'' \text{ c/c}$

Development length:

$$L_d = \frac{f_s \phi}{4 u_d}$$

$$u_d = \frac{V_{max}}{\Sigma_0 j d} = \frac{6236.1}{\pi \times \frac{9}{8} \times \frac{12}{8} \times 0.89 \times 15} = 88.11 \text{ psi}$$

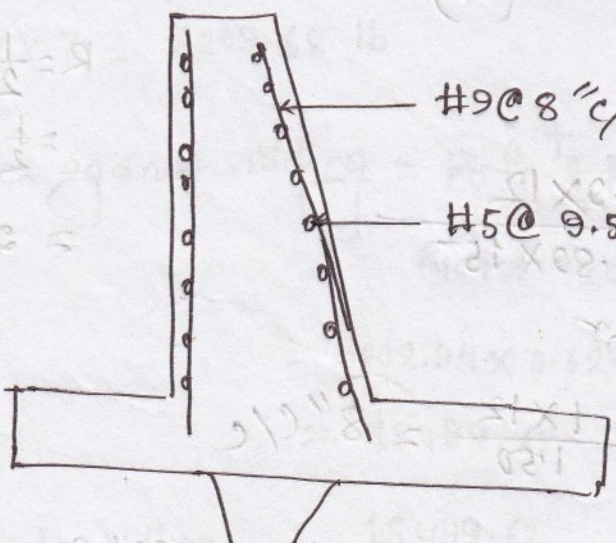
$$L_d = \frac{24000 \times \frac{9}{8}}{4 \times 88.11} = 76.61''$$

Shear Check:

$$v_d = \frac{V_{max}}{bd} = \frac{6236.1}{12 \times 15} = 34.65 \text{ psi}$$

$$v_c = 1.1\sqrt{3000} = 60.25 \text{ psi}$$

$\therefore v_c > v_d$ (ok)



③

Footing-1

Square Footing

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i) Area = $\dfrac{\text{Service Load}}{q_o} = \dfrac{D.L + L.L}{q_a - \gamma_s \times D}$

γ_s = unit weight of soil = 125 ~~pcf~~ lb ft^{-3}

D = depth of footing $\longrightarrow$ না দেয়া থাকলে 5 ft ধরতে হবে

q_a = allowable bearing pressure

ii) $q_u = \dfrac{1.2 \times D.L + 1.6 \times L.L}{\text{Area}}$

iii) Punching shear check:

V_u = Area of footing × q_u − Area of column perimeter × q_u

$V_c = 4\phi\sqrt{f_c'}\, bd$ $\quad \boxed{\phi = 0.75}$

$V_u = V_c \rightarrow d = ?$

iv) Beam shear check:

$V_u =$

$V_c = 2\phi\sqrt{f_c'}\, bd$ $\quad \boxed{\phi = 0.75}$

$V_c > V_u$ (OK)

v) Moment check:

$M_u = \dfrac{wL^2}{12}$

$A_s = \dfrac{M_u}{\phi f_y (d - \frac{a}{2})}$ $\quad \boxed{\phi = 0.9}$ $\qquad a = \dfrac{A_s f_y}{0.85 f_c' b}$ $\qquad A_s = ?$

$A_{s\,min} = \dfrac{200}{f_y\,(\text{psi})} bd$ $\qquad h = d + 1.5 d_b + 3''$

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①

3. (b) A column 15"×15" carries a service live load of 140 kips and a service dead load of 180 kips. The allowable soil bearing pressure is 4000 psf. Desing a square footing using $f_c' = 3500$ psi and $f_y = 50$ ksi.

Solution:

$$\text{Area} = \frac{\text{Service load}}{q_o} = \frac{(140+180)\times 10^3}{4000 - 125\times 5}$$ [Assume, depth of footing = 5 ft]

$= 94.81\ ft^2$

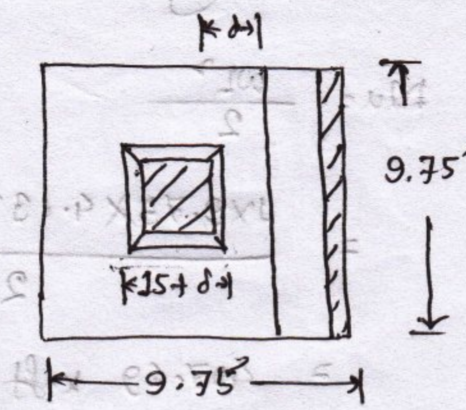
$= 9.75' \times 9.75'$

$\simeq$

$$q_{ud} = \frac{1.2\times 180 + 1.6\times 140}{(9.75)^2} = 4.63\ \text{ksf}$$

Punching shear check:

$$V_u = 9.75\times 9.75\times 4.63 - \left(\frac{15+d}{12}\right)^2 \times 4.63$$



$V_c = 4\phi\sqrt{f_c'}\, b d$

$$= \frac{4\times 0.75\times\sqrt{3500}}{1000}\times\{4(15+d)\}\times d$$

$= 0.71\,(15+d)d$

At equilibrium, $V_u = V_c$

$\Rightarrow 440.14 - (225 + 30d + d^2)\times 0.03 = 10.65d + 0.71d^2$

$\Rightarrow 440.14 - 6.75 - 0.9d - 0.03d^2 = 10.65d + 0.71d^2$

$\Rightarrow 0.74d^2 + 11.55d - 433.39 = 0$

$\therefore d = 17.62'' \simeq 18''$

ii) Beam shear check:

$$V_u = \left(\frac{9.75}{2} - \frac{15}{2\times12} - \frac{18}{12}\right) \times 9.75 \times 4.63$$

$$= 124.14 \text{ kip}$$

$$V_c = 2\phi\sqrt{f_c'}\, bd$$

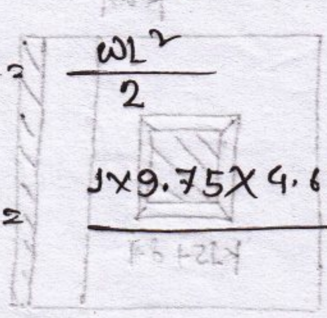
$$= \frac{2\times0.75\times\sqrt{3500}\times(9.75\times12)\times18}{1000}$$

$$= 186.89 \text{ kip}$$

$$\therefore V_c > V_u \quad \text{(ok)}$$

iii) Moment check:

Taking moment of 1' strip about the face of column



$$M_u = \frac{wL^2}{2}$$

$$= \frac{1\times9.75\times4.63\times\left(\frac{9.75}{2} - \frac{15}{2\times12}\right)^2}{2}$$

$$= 407.69 \text{ k-ft}$$

$$A_s = \frac{M_u}{\phi f_y \left(d - \frac{a}{2}\right)} \quad \cdots (1)$$

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{A_s\times50}{0.85\times3.5\times9.75\times12} = 0.14\,A_s$$

$$(1) \Rightarrow A_s = \frac{407.69\times12}{0.9\times50\times\left(18 - \frac{0.14A_s}{2}\right)}$$

$$\Rightarrow 18A_s - 0.07A_s^2 = 108.72 \qquad \therefore A_s = 6.79 \text{ in}^2$$

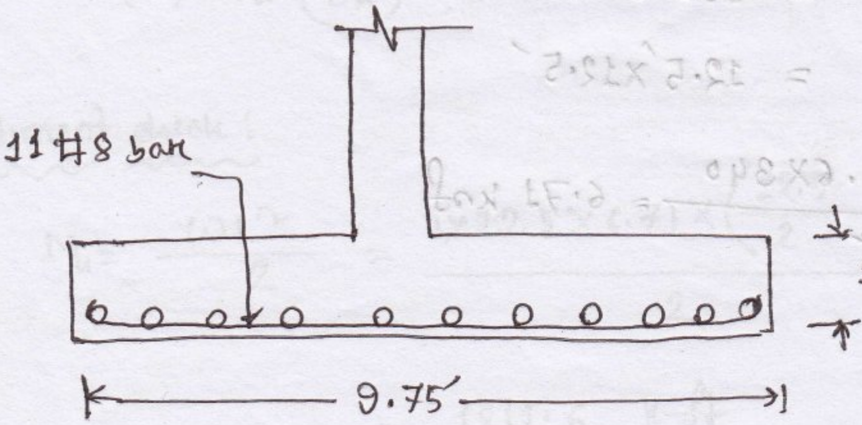
$$As_{min} = \frac{200}{f_y} bd$$

$$= \frac{200}{50000} \times 9.75 \times 12 \times 18 = 8.42 \text{ in}^2$$

So, 8.42 in^2 has to be provided.

Use 11 #8 bar (8.69 in^2)

$$h = d + 1.5\, d_b + 3'' = 18 + 1.5 \times \frac{8}{8} + 3 = 22.5'' \simeq 23''$$



06

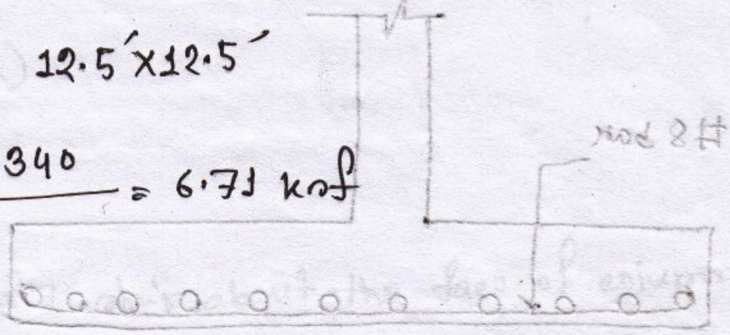
4. An interior column of a building carries a service dead load of 420 ki[ps] and a service live load of 340 kips. The column size is 21"X21". Design a square footing to support this column load. Allowable soil bearing pressure is 5.5 ksf. Assume, $f_c' = 3$ ksi and $f_y = 60$ ksi.

Solution:

$$\text{Area} = \frac{\text{Service load}}{q_0} = \frac{(420+340)\times10^3}{5500-125\times5}$$ [Assume, depth of footing = 5']

$$= 155.9\ ft^2$$

$$= 12.5' \times 12.5'$$

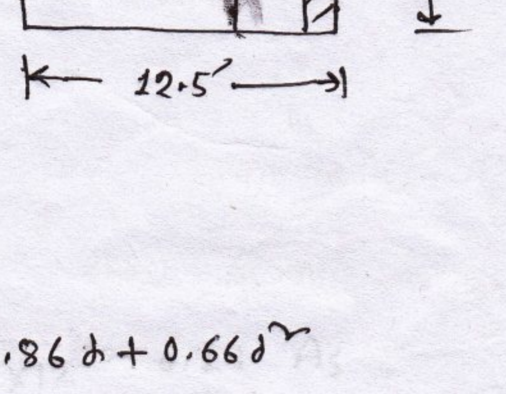


$$q_u = \frac{1.2\times420 + 1.6\times340}{(12.5)^2} = 6.71\ ksf$$

① Punching shear check:

$$V_u = 12.5\times12.5\times6.71 - \left(\frac{21+d}{12}\right)^2 \times 6.71$$

$$= 1048.44 - 0.05(21^2 + 42d + d^2)$$



$$V_c = 4\phi\sqrt{f_c'}\, bd$$

$$= \frac{4\times0.75\times\sqrt{3000}\times\cancel{12.5\times12}\,4(21+d)\times d}{1000}$$

$$= 0.66(21+d)d$$

At equilibrium, $V_u = V_c$

$$\Rightarrow 1048.44 - 22.05 - 2.1d - 0.05d^2 = 13.86d + 0.66d^2$$

$$\Rightarrow 0.71d^2 + 15.96d - 1026.39 = 0$$

$$\therefore d = 28.41 \simeq 29''$$

(ii) Beam shear check:

$$V_u = \left(\frac{12.5}{2} - \frac{21}{2\times12} - \frac{29}{12}\right)\times 12.5\times 6.71$$

$$= 248.13 \text{ kips}$$

$$V_c = 2\phi\sqrt{f_c'}\,bd$$

$$= \frac{2\times0.75\times\sqrt{3000}\times12.5\times12\times29}{1000}$$

$$= 357.39 \text{ kips}$$

$\therefore V_c > V_u$ (OK)

(iii) Moment check:

$$M_u = \frac{\omega L^2}{2} = \frac{1\times12.5\times6.71\times\left(\frac{12.5}{2} - \frac{21}{2\times12}\right)^2}{2}$$

$$= 1211.6 \text{ k-ft}$$

$$A_s = \frac{M_u}{\phi f_y\left(d - \frac{a}{2}\right)} \quad \text{--- (1)}$$

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{A_s\times60}{0.85\times3\times12.5\times12} = 0.16\,A_s$$

(1) ⇒

$$A_s = \frac{1211.6\times12}{0.9\times60\times(29 - 0.08A_s)}$$

$$\Rightarrow 29A_s - 0.08A_s^2 = 269.24$$

$$\therefore A_s = 9.53 \text{ in}^2$$

$$A_{s\,min} = \frac{200}{f_y} bd$$

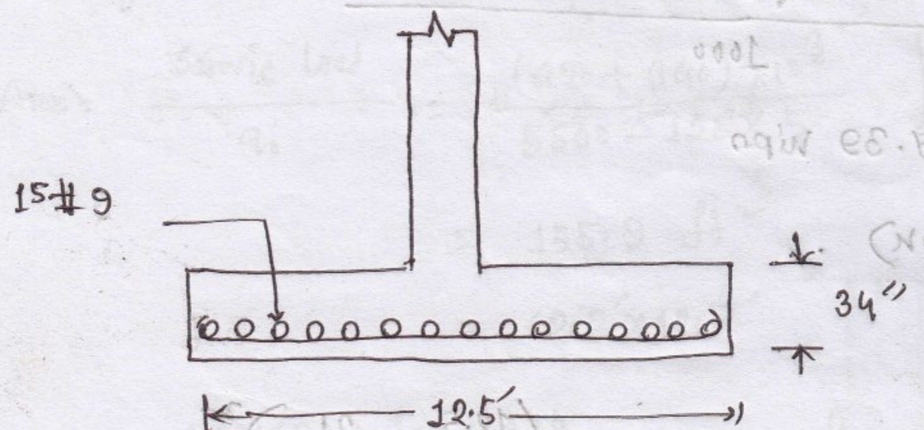
$$= \frac{200}{60000} \times 12.5 \times 12 \times 29 = 14.5 \text{ in}^2$$

Use 15 #9 bar (15 in^2)

$\therefore h = d + 1.5 d_b + 3'' = 29 + 1.5 \times \frac{9}{8} + 3$

$= 33.69'' \simeq 34''$

$\therefore$



②

Combined Footing

Footing-2:

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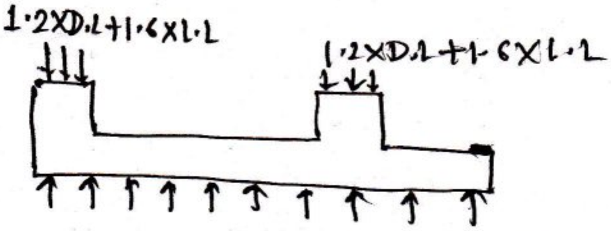
i) $\text{Area} = \dfrac{D.L + L.L}{q_o} = \dfrac{D.L + L.L}{q_a - \gamma_s \times D - \text{surcharge}}$

ii) $x = ?$ (Moment নিবো p point এ) $L = ?$ $B = ?$

iii) $q_{ud} = \dfrac{1.2 \times D.L + 1.6 \times L.L}{\text{Area}}$



iv) SFD, BMD

$V_{max} = ?$ $M_{max} = ?$

$q_{ud} \times B = w_u$

v) Beam shear check:

$V_{ud} = V_{max} - \dfrac{d}{12} w_u$

$V_c = 2\phi\sqrt{f_c'}\, bd$ $\boxed{V_{ud} = V_c} \rightarrow d = ?$

vi) Punching shear check:

Exterior column

$V_{ud} = (1.2 \times D.L + 1.6 \times L.L)$ — Upward soil pressure

$V_c = 4\phi\sqrt{f_c'}\, bd$

vii) Steel:

$A_s = \dfrac{M_u}{\phi f_y (d - \frac{a}{2})}$ $a = \dfrac{A_s f_y}{0.85 f_c' b}$

$A_s = \dfrac{3\sqrt{f_c'}}{f_y} bd$

$A_s = \dfrac{200}{f_y} bd$

VIII) Design of transverse beam under interior column ?

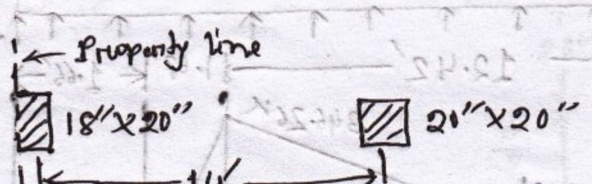
$d = d - 1 = ?$

$b = d +$ Interior column এর $b = ?$

$$A_s = \frac{250}{f_y} bd$$

10 | 08

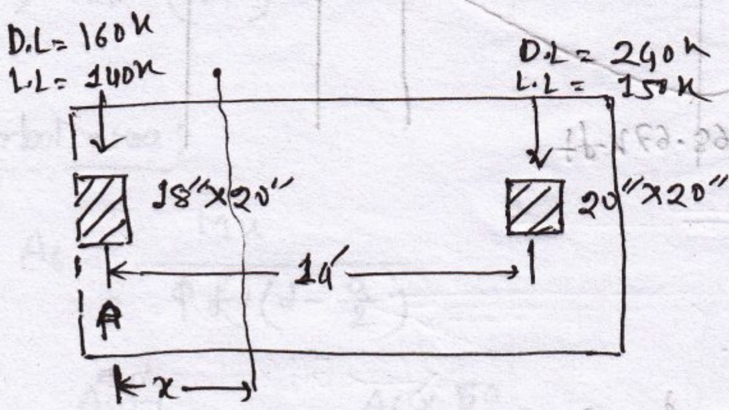
Q-2: Find dimensions of a rectangular combined footing to support two columns as shown in the figure. The exterior side of the exterior column touches the property line. The centre to centre distance of columns is 14 ft. The exterior column with 18X20 in. size carries an axial dead load of 160 kip and a live load of 140 kip. The interior column with 20 in square in size carries an axial load of 240 kip and a live load of 150 kip. The depth of footing is 5 ft. Use $f_c' = 4$ ksi, $f_y = 50$ ksi and $q_a = 5$ ksf.



Solution:

$$\text{Area} = \frac{\text{column load}}{q_e} = \frac{(160+140+240+150)\times 10^3}{5000 - 125\times 5}$$

$$= 157.71 \ ft^2$$



$\Sigma M_p = 0,$

$390 \times 14 = 690 \times x \quad \therefore x = 7.91'$

$\therefore$ Distance from extreme left side $= 7.91 + \frac{18}{2\times 12} = 8.66'$

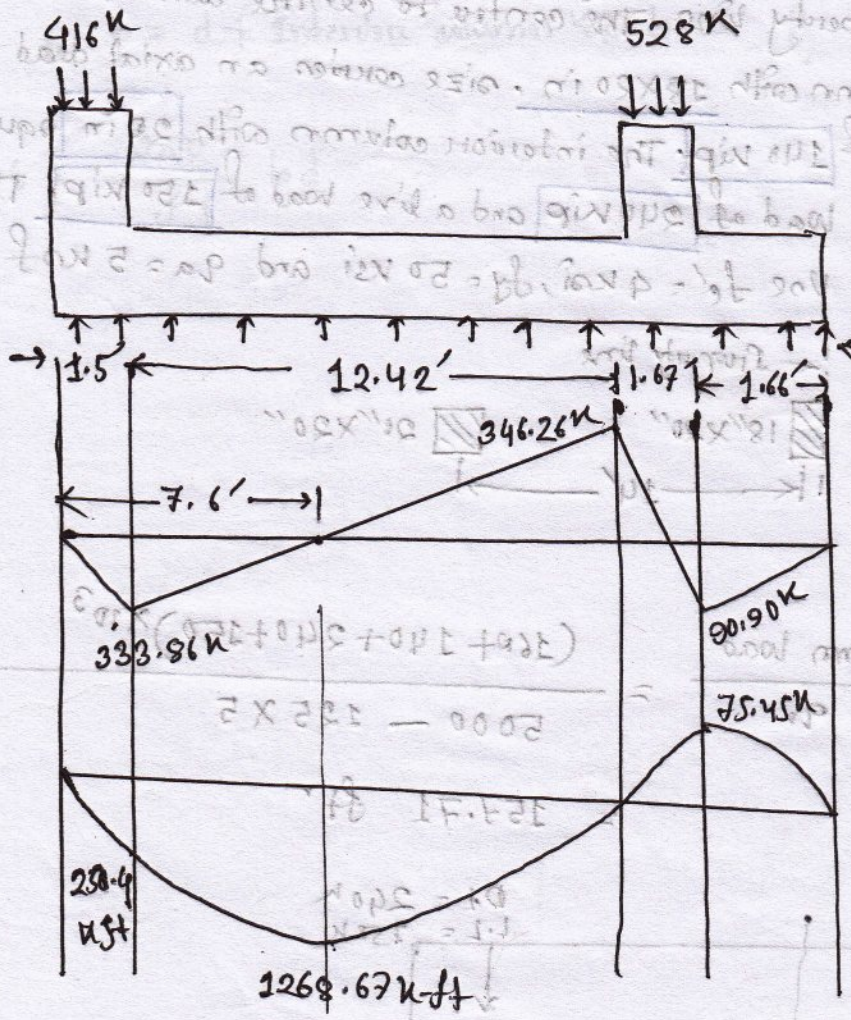
$\therefore$ Length of footing $= 8.66\times 2 = 17.32' \simeq 17.25'$

Area = Length X width

$\Rightarrow \quad 157.71 = 17.25 \times \text{width}$

$\therefore$ width $= 9.14' \simeq 9.25'$

$$q_{ud} = \frac{1.2(160+240)+1.6(140+150)}{17.25 \times 9.25} = 5.92 \text{ ksf}$$



$5.92 \times 9.25 = 54.76 \text{ kft}^{-1} = w_u$

$$\frac{333.86}{x} = \frac{346.26}{12.42-x}$$

$\therefore x = 6.10'$

1) <u>Beam shear check:</u>

$$V_{ud} = V_{max} - \frac{d}{12} w_u$$

$$= 346.26 - \frac{d}{12} \times 54.76 = 346.26 - 4.56d$$

$$V_c = 2\phi\sqrt{f_c'}\, bd$$

$$= \frac{2 \times 0.75 \times \sqrt{4000} \times 9.25 \times 12 \times d}{1000}$$

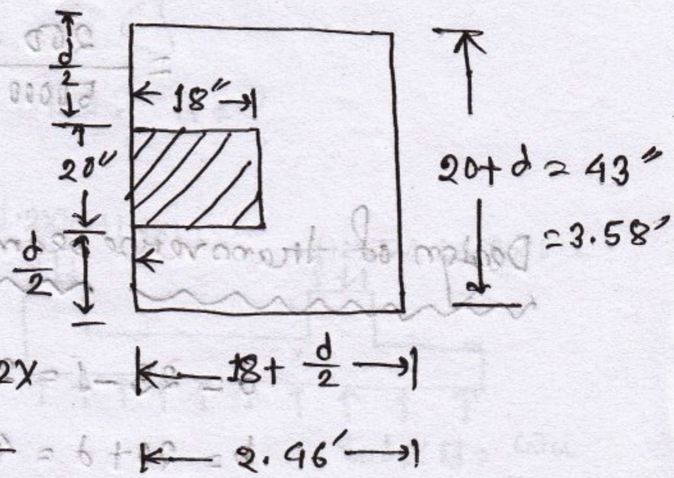
$$= 10.53d$$

$V_{ud} = V_c$

$\Rightarrow 396.26 - 4.56\,d = 10.53\,d$

$\therefore d = 22.95'' \simeq 23''$

(ii) Punching sheare check:



$V_{ud} = 416 - 2.46 \times 3.58 \times 5.92$

$= 363.86^{K}$

$V_c = 4\phi\sqrt{f_c'}\, b_0 d$

$$= \frac{4 \times 0.75 \times \sqrt{4000} \times (2 \times 2.46 + 3.58) \times 12 \times 23}{1000}$$

$= 445.12^{K}$

$\therefore V_c > V_{ud}$ (OK)

(iii) Required steel area:

$$A_s = \frac{M_u}{\phi f_y (d - \frac{a}{2})}$$

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{A_s \times 50}{0.85 \times 4 \times 0.25 \times 12} = 0.13\, A_s$$

$$A_s = \frac{1268.67 \times 12}{0.9 \times 50 \times (23 - 0.065\, A_s)}$$

$\Rightarrow 23\, A_s - 0.065\, A_s^2 = 338.31$

$\therefore A_s = 15.38\ in^2$

$\rho bd = 0.0035 \times 9.25 \times 12 \times 23$

$= 8.94 \text{ in}^2$

12#8 bar

Minimum required $= \dfrac{3\sqrt{f_c'}}{f_y} bd$

$= \dfrac{3 \times \sqrt{4000}}{50000} \times 9.25 \times 12 \times 23 = 9.69 \text{ in}^2$

But not less than,

$A_s = \dfrac{200}{f_y} bd$

$= \dfrac{200}{50000} \times 9.25 \times 12 \times 23 = 10.21 \text{ in}^2$

$= 13\#8$

Design of transverse beam under interior column:

$d = 23 - 1 = 22''$

$b = 20 + d = 42''$

$\rho = \dfrac{60}{3}$

$A_s = \dfrac{200}{f_y} bd = \dfrac{200}{50000} \times 42 \times 22$

$= 3.7 \text{ in}^2$

$= 5\#8$