REINFORCED CONCRETE-II

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A Handbook on

Reinforced Concrete-ii

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RETAINING WALL

COLUMN

DESIGN OF COLUMN (USD)

COMPRESSION + BENDING (WSD)

QUESTION SOLVE (COLUMN)

YIELD LINE THEORY

FOOTING

FLAT SLAB

FLAT PLATE

মো: রবিউল ইসলাম
রাজশাহী প্রকৌশল ও প্রযুক্তি বিশ্ববিদ্যালয়
পুরকৌশল বিভাগ
রোল নং: ১৩০১১০.

CE- 3217, Reinforced Concrete - II

Retaining WALL

Q. Define retaining wall and classify it.

Answer:-

Retaining wall: Retaining wall are used to hold back masses of earth or other loose material where conditions make it impossible to let those masses assume their natural slopes.

Types of retaining wall: The most common retaining wall are three types.

(I) Gravity retaining wall.

(II) Cantilever retaining wall.

(III) counter fort retaining wall.

(I) Gravity Retaining wall :-

(a) It resists all forces with the help of its own weight

(b) It is used for small or low height of backfill.

(c) ~~It is~~ usually reinforced are not used in the Gravity Retaining wall.

(d) It's height is generally 8 to 10 ft.

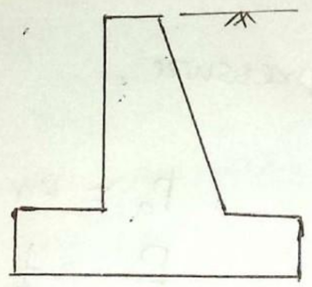


Fig. Gravity retaining wall.

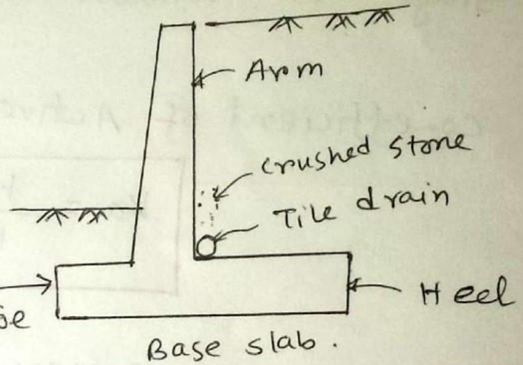


Fig. Cantilever retaining wall.

(II) Cantilever Retaining wall:-

(a) If the height of backfill is of moderate to high hight, cantilever retaining wall is used.

(b) It's height is generally more than 12 ft. and in between 12 - 20 ft.

(c) Reinforced are used in the cantilever retaining wall.

(III) Counter-fort retaining wall :

(a) To reduce span length so that bulking may not occur. counter-fort retaining wall is used.

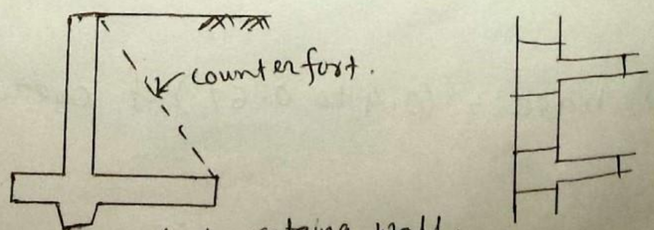


Fig. counter-fort retaing wall.

Forces on retaining wall:

Earth pressure at rest: If there is no movement of retaining structure the pressure on the structure at this stage is known as earth pressure at rest.

Active earth pressure: If the wall moves away from the soil, the pressure exerts on the structure at this stage is known as active earth pressure.

co-efficient of Active earth pressure,

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

$$p_a = K_a \gamma h$$

$$P_a = \frac{1}{2} K_a \gamma h^2$$

Passive earth pressure: If the wall moves toward the soil, the pressure exerts on the structure at this stage is known as passive earth pressure.

co-efficient of passive earth pressure,

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

$$P_p = \frac{1}{2} K_p \gamma h^2$$

Surcharged এর ক্ষেত্রে সূত্র :-

$$p_s = K_a \gamma h_s \qquad h_s = \frac{W_s}{\gamma_s}$$

$$P_s = K_a \gamma h_s * h_o$$

h_o = overall depth/height.

⇒ Length of the base = (0.4 to 0.67) of overall height.

Mode of failure:-

(a) The structure may be unstable.

(b) These may be structural fault.

External stability:-

(I) Sliding

(II) Over turning. (tilting)

(III) Sinking or bearing failure.

Internal stability:-

(I) Shear failure.

(II) Flexure failure.

⇒ Stability Test:

→ Cheque against sliding:-

$$FS = \frac{W \times f + P_p}{P_a + P_s} \geq 1.5$$

W = total load

f = frictional co-efficient

P_a = active earth pressure.

P_s = Active earth pressure due to surcharge

P_p = passive earth pressure.

আগে P_p বাদে checque করে দেখতে হবে।

→ Overturning:-

$$M_o = P_a \times \frac{h}{3} + P_s \times \frac{h}{2}$$

$$M_R = \Sigma (W \times \bar{x})$$

$$FS = \frac{M_R}{M_o} \geq 1.5$$

→ Bearing failure:-

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

$\sigma_1 = ?$

$\sigma_2 = ?$

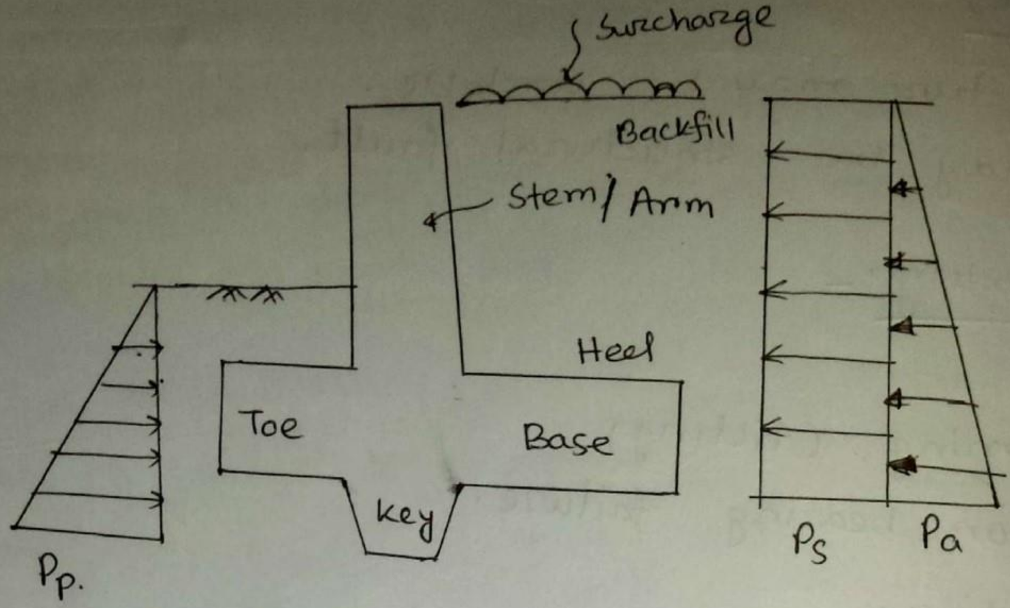
$$a = \frac{M_R - M_o}{W}$$

$$e = \frac{B}{2} - a$$

$$M = W \times e$$

$$I = \frac{1 \times B^3}{12}$$

$A = 1 \times B$, $C = \frac{B}{2}$



Function of key:

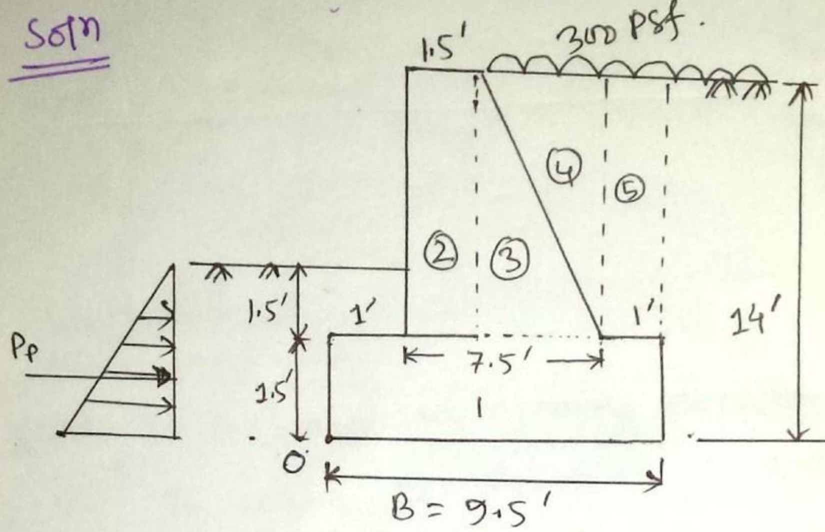
(I) To increase the sliding resistance.

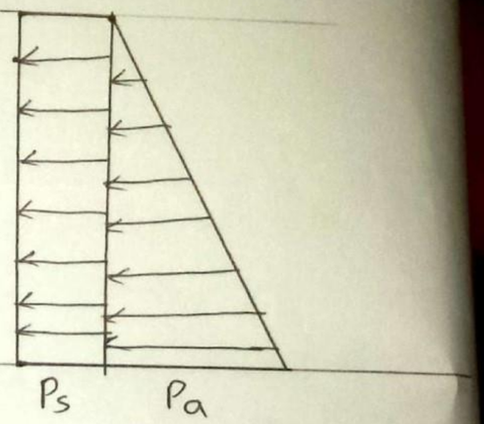
(II) To serve the function of housing the reinforcement of stem

(III) To increase frictional resistance.

Problem 01: Design a gravity retaining wall to retain a 11' backfill. Assume unit weight of soil 120 pcf. Angle of internal friction $\phi = 30°$. co-efficient of friction between concrete and soil at the base 0.5. Allowable bearing capacity of the soil 5000 psf. Assume reasonable data if not given.

Soln





Let us consider, overall height, $H = 11' + 3' = 14'$

width of breadth, $B = \frac{2}{3} H = \frac{2}{3} \times 14 = 9.33' \sim 9.5'$

$$k_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30°}{1+\sin 30°} = \frac{1}{3}$$

$$k_p = \frac{1}{k_a} = 3$$

$$p_a = k_a \gamma H = \frac{1}{3} \times 120 \times 14 = 560 \text{ psf}$$

Active earth pressure,

$$P_a = \frac{1}{2}(k_a \gamma H) H = \frac{1}{2} \times 560 \times 14 = 3920 \text{ lb.}$$

$$p_s = k_a \gamma h_s = \frac{1}{3} \times 120 \times 2.5 = 100 \text{ psf}$$

here, $h_s = \frac{300}{120} = 2.5$

$$\therefore P_s = k_a \gamma h_s \times H = 100 \times 14 = 1400 \text{ lb.}$$

Passive earth pressure,

$$P_p = \frac{1}{2} k_p \gamma h^2 = \frac{1}{2} \times 3 \times 120 \times 3^2 = 1620 \text{ lb}$$

Over turning moment,

$$M_0 = P_a \times \frac{H}{3} + P_s \times \frac{H}{2}$$

$$= 3920 \times \frac{14}{3} + 1400 \times \frac{14}{2}$$

$$= 18293.33 + 9800$$

$$= 28093.33 \text{ lb-ft.}$$

Section	Weight, lb	Moment arm, ft.	Moment, lb-ft.
(I)	$9.5 \times 1.5 \times 150 = 2137.5$	$\frac{9.5}{2} = 4.75$	10153.125
(II)	$1.5 \times 12.5 \times 150 = 2812.5$	$1 + \frac{1.5}{2} = 1.75$	4921.875
(III)	$\frac{1}{2} \times 6 \times 12.5 \times 150 = 5625$	$1 + 1.5 + \frac{6}{3} = 4.5$	25312.5
(IV)	$\frac{1}{2} \times 6 \times 12.5 \times 120 = 4500$	$1 + 1.5 + \frac{2 \times 6}{3} = 6.5$	29250
(V)	$1 \times 12.5 \times 120 = 1500$	$1 + 7.5 + \frac{1}{2} = 9$	13500
Total	$W = 16575$		$M_R = 83137.5$

Stability test:

Cheque against sliding:-

Total sliding force, $P_a + P_s = 3920 + 1400 = 5320$ lb.

(without considering passive pressure)

$$FS = \frac{W \times f}{P_a + P_s} = \frac{16575 \times 0.5}{5320} = 1.56 > 1.5 \text{ (ok)}$$

(consider passive pressure)

$$FS = \frac{Wf + P_p}{P_a + P_s} = \frac{(16575 \times 0.5) + 1620}{5320} = 1.86 > 1.5 \text{ (ok)}$$

Cheque against overturnelling :-

Resisting moment, $M_R = 83137.5$ lb-ft.

over turning moment, $M_o = 28093.33$ lb-ft.

$$\therefore FS = \frac{M_R}{M_o} = \frac{83137.5}{28093.33} = 2.96 > 1.5$$

(OK)

Cheque against soil pressure :-

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

$B = 9.5'$, $c = \frac{B}{2} = 4.75'$, $A = 1 \times 9.5 = 9.5\ ft^2$

$$I = \frac{1 \times B^3}{12} = \frac{1 \times 9.5^3}{12} = 71.45$$

$$a = \frac{M_R - M_o}{W} = \frac{83137.5 - 28093.33}{16575} = 3.32$$

$$e = \frac{B}{2} - a = 4.75 - 3.32 = 1.43$$

$M = W * e = 16575 \times 1.43 = 23702.25$ lb-ft.

$$\sigma_1 = \frac{16575}{9.5} + \frac{23702.25 \times 4.75}{71.45}$$

$= 1744.74 + 1575.73$

$= 3320.47$ psf.

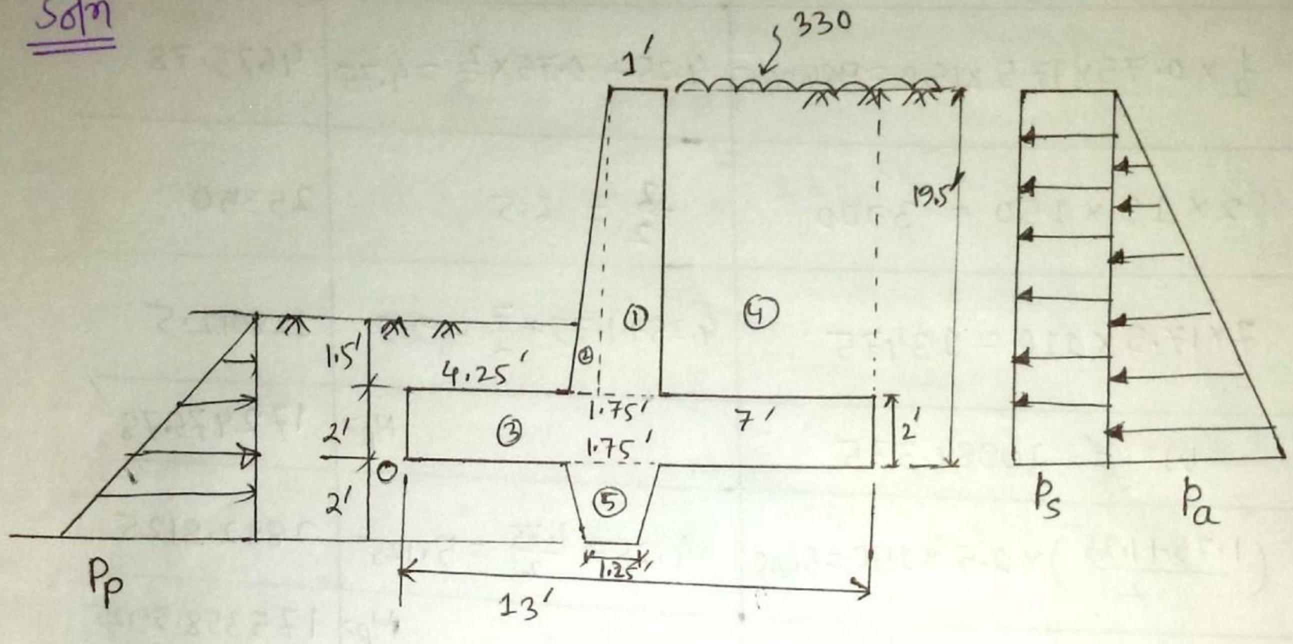
$$\sigma_2 = \frac{16575}{9.5} - \frac{23702.25 \times 4.75}{71.45}$$

$= 1744.74 - 1575.73$

$= 169$ psf.

Problem 02: Design a cantilever retaining wall to retain a backfill 16′ high which carries a surcharge 330 psf. Assume unit weight of soil 110 pcf, $\phi = 30^\circ$, $f = 0.5$, bearing capacity 5000 psf, $f_c' = 3000$ psf, $f_s = 24000$ psi.

Soln



Overall height, $H = 16 + 3.5 = 19.5'$

Width of breadth, $B = \frac{2}{3} H = \frac{2}{3} \times 19.5' = 13'$

$$k_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30^\circ}{1+\sin 30^\circ} = \frac{1}{3}.$$

$$\therefore k_p = \frac{1}{k_a} = 3.$$

$$h_s = \frac{w_s}{\gamma_s} = \frac{330}{110} = 3$$

$$\therefore p_a = k_a \gamma H = \frac{1}{3} \times 110 \times 19.5 = 715$$

$$\therefore P_a = \frac{1}{2} k_a \gamma H^2 = \frac{1}{2} \times 715 \times 19.5 = 6971.25$$

$$p_s = k_a \gamma h_s = \frac{1}{3} \times 110 \times 3 = 110$$

$$\therefore P_s = k_a \gamma h_s \times H = 110 \times 19.5 = 2145$$

$P_p = \frac{1}{2} k_p \gamma H^2 = \frac{1}{2} \times 3 \times 110 \times 3.5^2 = 2021.25$ [key consider নাকরে]

Section	Weight, lb	Moment Arm ft	Moment lbft
(1)	$1 \times 17.5 \times 150 = 2625$	$4.25 + 0.75 + \frac{1}{2} = 5.5$	14437.5
(II)	$\frac{1}{2} \times 0.75 \times 17.5 \times 150 = 984.375$	$4.25 + 0.75 \times \frac{2}{3} = 4.75$	4675.78
(III)	$2 \times 13 \times 150 = 3900$	$\frac{13}{2} = 6.5$	25350
(IV)	$7 \times 17.5 \times 110 = 13475$	$4.25 + 1.75 + \frac{7}{2} = 9.5$	128012.5
Total	$W = 20984.375$	$M_R =$	172475.78
(5)	$\left(\frac{1.75+1.25}{2}\right) \times 2.5 \times 150 = 562.5$	$4.25 + \frac{1.75}{2} = 5.125$	2882.8125
Total	$W = 21546.875$	$M_R =$	175358.5925

$$M_o = P_a \times \frac{h}{3} + P_s \times \frac{h}{2} = 6971.25 \times \frac{19.5}{3} + 2145 \times \frac{19.5}{2} = \cancel{59434.375}$$
$$= 66226.875$$

Stability calculation:

cheque for sliding,

safety of factor, $FS = \frac{W \times f}{P_a + P_s}$ [without considering Passive earth pressure]

$$= \frac{20984.375 \times 0.5}{6971.25 + 2145}$$

$$= 1.15 < 1.5 \quad \text{(Not ok)}$$

Now consider passive earth pressure,

$$FS = \frac{W \times f + P_p}{P_a + P_s} = \frac{(20984.375 \times 0.5) + 2021.25}{6971.25 + 2145}$$

$$\therefore FS = 1.37 < 1.5 \quad \text{(Not ok)}$$

Comments: A key should be provided at the base to increase the sliding resistance. The key will also serve the function of housing the reinforcement of stem.

Revised passive earth pressure,

$$P_p = \frac{1}{2} k_p \gamma h^2$$

$$= \frac{1}{2} \times 3 \times 110 \times 6^2$$

$$= 5940$$

After providing key,

$$FS = \frac{W \times f + P_p}{P_a + P_s} = \frac{(21546.875 \times 0.5) + 5940}{6971.25 + 2145}$$

$$\therefore FS = 1.83 > 1.5 \quad (OK)$$

এটা দরকার নাই $\Rightarrow$ $\boxed{FS = \frac{W \times f}{P_a + P_s} = \frac{(21546.875 \times 0.5)}{6971.25 + 2145} = 1.18 \quad (\text{Not ok})}$

Cheque Against overturning:

Befor providing key, $FS = \frac{M_R}{M_O} = \frac{172475.78}{\cancel{59434.375}\ 66226.875} = 2.60 > 1.5$ (ok)

After providing key, $FS = \frac{M_R}{M_O} = \frac{175358.5925}{\cancel{59434.375}\ 66226.875} = 2.65 > 1.5$ (ok)

Cheque against soil pressure:

$$a = \frac{M_R - M_O}{W} = \frac{175358.5925 - 66226.875}{21546.875} = 5.06$$

$$e = \frac{B}{2} - a = 6.5 - 5.06 = 1.44$$

$$A = 1 \times 13 = 13 \text{ ft}^2, \quad c = \frac{B}{2} = 6.5 \text{ ft}.$$

$$I = \frac{1 \times B^3}{12} = \frac{1 \times 13^3}{12} = 183.08 \ ft^4$$

$$M = W \times e = 21546.875 \times 1.44 = 31027.5$$

$$\sigma_1 = \frac{W}{A} + \frac{MC}{I}$$

$$= \frac{21546.875}{13} + \frac{31027.5 \times 6.5}{183.08}$$

$$= 1657.45 + 1101.59$$

$$= 2759.04 \sim 2759.$$

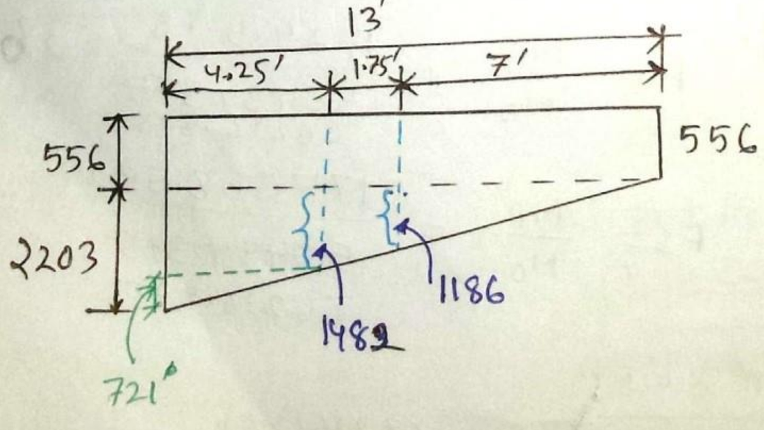
$$\sigma_2 = \frac{W}{A} - \frac{MC}{I}$$

$$= \frac{21546.875}{13} - \frac{31027.5 \times 6.5}{183.08}$$

$$= 1657.45 - 1101.59$$

$$= 555.89. \sim 556.$$

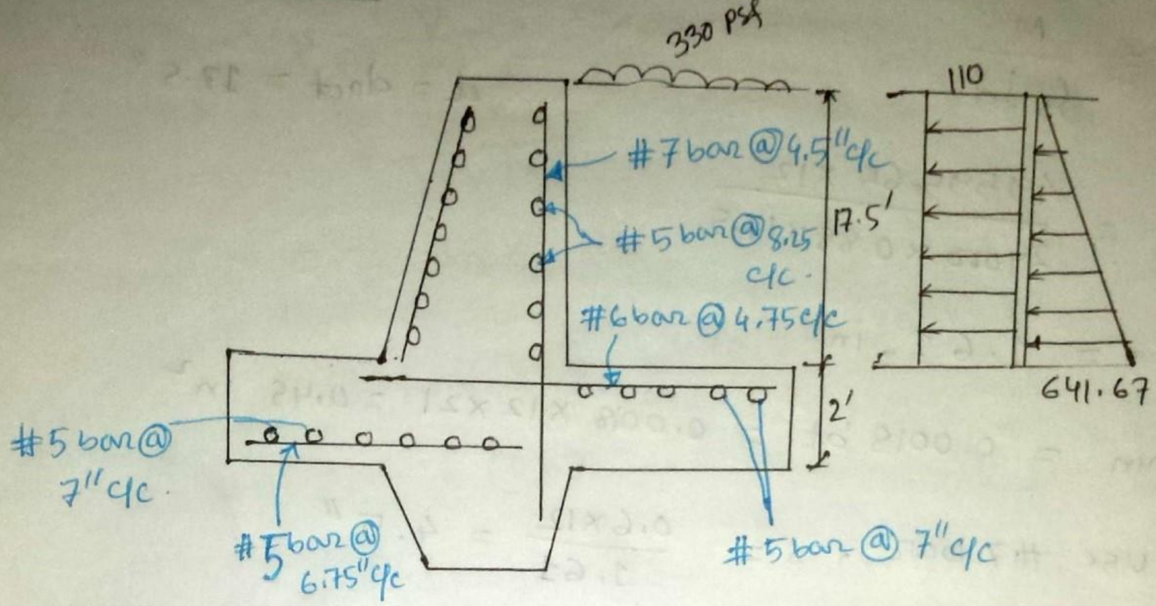
stress diagram:



$$\frac{2203 \times 7}{13} = 1186$$

$$\frac{2203 \times 8.75}{13} = 1482$$

Design of Stem :



Shear

$$V = 110 \times 17.5 + \frac{1}{2} \times 641.67 \times 17.5$$

$$= 7539.6 \text{ lb}$$

Moment,

$$M = 110 \times 17.5 \times \frac{17.5}{2} + \frac{1}{2} \times 641.67 \times 17.5 \times \frac{17.5}{3}$$

$$= 49595.65 \text{ lb-ft}.$$

Assume, $k = 0.36$, $j = 0.88$, $f_c = 0.45 f_c' = 0.45 \times 3000 = 1350$ psi.

$$\therefore R = \frac{1}{2} f_c j k = \frac{1}{2} \times 1350 \times 0.88 \times 0.36 = 213.84$$

depth check :

$$d_{req} = \sqrt{\frac{M}{Rb}},$$

$$= \sqrt{\frac{49595.65 \times 12}{213.84 \times 12}}$$

$$= 15.25''$$

Here, $M = 49595.65$. lbft

$R = 213.84$

$b = 12''$

$$d_{act} = t - cc - \frac{\phi}{2}$$

$$= 21 - 3 - \frac{8}{8 \times 2}$$

$$= 17.5''$$

$\therefore d_{act} > d_{req}$: (OK)

Reinforcement calculation :

$$A_s = \frac{M}{f_s jd} \qquad d = d_{act} = 17.5''$$

$$= \frac{49595.65 \times 12}{24000 \times 0.88 \times 17.5}$$

$$= 1.61 \text{ in}^2$$

$$A_{s\,min} = 0.0018\, bt = 0.0018 \times 12 \times 21 = 0.45 \text{ in}^2$$

So, use #7 bar , $s = \frac{0.6 \times 12}{1.61} = 4.5''$

$\therefore$ use #7 bar @ 4.5'' c/c .

Distribution reinforcement:

$$A_{st} = 0.45 \text{ in}^2$$

Use #5 bar, $s = \frac{0.31 \times 12}{0.45} = 8.25''$

USE #5 bar @ 8.25'' c/c

Development length :-

$$l_d = \frac{f_s D}{4u}$$

$$= \frac{24000 \times 7/8}{4 \times 212.83}$$

$$= 24.66''$$

$$u = \frac{3.4\sqrt{f_c'}}{D}$$

$$= \frac{3.4\sqrt{3000}}{\frac{7}{8}} = 212.8$$

$$D = \frac{7}{8}''$$

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Shear check :-

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

$V = 7539.6$

$b = 12''$

$d = 17.5''$

$$= \frac{7539.6}{12 \times 17.5}$$

$$= 35.9 \text{ lb.}$$

$$v_{all} = 1.1\sqrt{3000} = 60.25 \text{ lb}$$

$v_{all} > v$. (ok)

Design of Heel :-

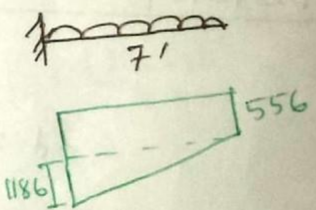
Load calculation :-

(i) self weight $= 2 \times 150 = 300$ psf.

(ii) weight of soil $= 17.5 \times 110 = 1925$ psf

(iii) surcharge $= 330$ psf.

$\therefore$ Total $= 2555$ psf



Net moment,

$$M = 2555 \times \frac{7^2}{2} - \left(556 \times \frac{7^2}{2} + \frac{1}{2} \times 1186 \times \frac{7^2}{3}\right)$$

$$= 39289.83 \text{ lb-ft.}$$

Depth check :-

$$d = \sqrt{\frac{39289.83 \times 12}{\frac{1}{2} \times 1350 \times 0.8}}$$

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{39289.83 \times 12}{213.84 \times 12}} = 13.5$$

$d_{act} = t - 3.5$

$= 24 - 3.5$

$= 20.5$

$\therefore d_{act} > d_{req}$ (ok)

Reinforcement calculation:-

$$A_s = \frac{M}{f_s j d} = \frac{39289.83 \times 12}{24000 \times 0.88 \times 20.5}$$

$$A_s = 1.1 \text{ in}^2$$

$$A_{s\,min} = 0.0018\, bt = 0.0018 \times 12 \times 24 = 0.52 \text{ in}^2$$

Use #6 bar, $s = \frac{0.44 \times 12}{1.1} = 4.75'' \text{ c/c}$

Distribution reinforcement:

$$A_{st} = 0.52 \text{ in}^2$$

Use #5 bar, $s = \frac{0.31 \times 12}{0.52} = 7'' \text{ c/c.}$

Development length:-

$$l_d = \frac{f_s D}{4u}$$

$$= \frac{24000 \times 6/8}{4 \times 139.67}$$

$$= 32.2''$$

$$u = \frac{3.4\sqrt{f_c'}}{D}$$

$$= \frac{3.4\sqrt{3000}}{6/8}$$

$$= 139.67$$

$$D = \frac{6''}{8}$$

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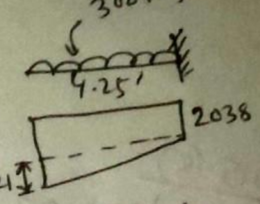
Design of Toe:

Load calculation:

weight of soil may be neglected.

(1) self weight of slab $= 2 \times 150 = 300$ psf.



Net moment,

$$M = \left(2038 \times 4.25 \times \frac{4.25}{2} + \frac{1}{2} \times 721 \times 4.25 \times \frac{2}{3} \times 4.25\right) - 300 \times 4.25 \times \frac{4.25}{2}$$

$$= 20037.33 \text{ lb ft}$$

Depth check:

Depth check is not required.

Reinforcement calculation:

$d = 20.5$

$$A_s = \frac{M}{f_s j d} = \frac{20037.33 \times 12}{24000 \times 0.88 \times 20.5}$$

$\therefore A_s = 0.55 \text{ in}^2$

$A_{smin} = 0.0018 bt = 0.0018 \times 12 \times 24 = 0.52 \text{ in}^2$

Use #5 bar, spacing, $s = \frac{0.31 \times 12}{0.55} = 6.75''$

$\therefore$ USE #5 bar @ 6.75'' c/c.

Distribution reinforcement:-

$A_{st} = 0.52 \text{ in}^2$

Use #5 bar, $s = \frac{0.31 \times 12}{0.52} = 7.15'' \sim 7''$

$\therefore$ USE #5 bar @ 7'' c/c.

Development length:

$$l_d = \frac{f_s D}{4u}$$

$$= \frac{24000 \times 5/8}{4 \times 297.96}$$

$$= 12.58''$$

$D = 5/8''$

$$u = \frac{3.4\sqrt{f_c'}}{D}$$

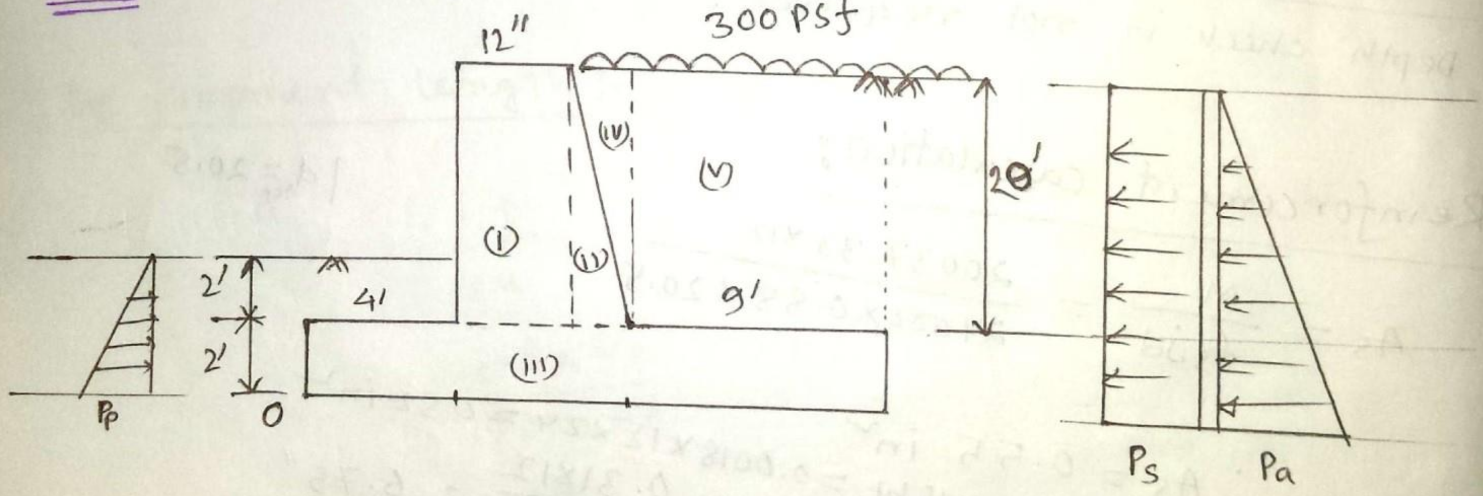
$$= \frac{3.4\sqrt{3000}}{5/8}$$

$$= 297.96$$

2015 (8)

Problem 03: A section of a cantilever retaining wall is shown in figure below. Check the external stability and design the stem of the wall. Assume (I) $\phi = 32°$ (II) unit weight of soil = 110 pcf (III) co-efficient of friction between concrete and soil = 0.5 (IV) $f_c' = 3$ KSI and (V) $f_s = 20$ KSI.

Soln



Overall height, $H = 20+2 = 22'$

width of Bredth, $B = 9+4+1.5 = 14.5'$

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 32^\circ}{1+\sin 32^\circ} = 0.307$$

$$\therefore K_p = \frac{1}{K_a} = \frac{1}{0.307} = 3.25$$

$$h_s = \frac{W_s}{\gamma_s} = \frac{300}{110} = 2.73'$$

$$\therefore P_a = K_a \gamma H = 0.307 \times 110 \times 22 = 742.94$$

$$P_a = \frac{1}{2} \times K_a \gamma H \times H = \frac{1}{2} \times 742.94 \times 22 = 8172.34$$

$$\therefore P_s = K_a \gamma h_s = 0.307 \times 110 \times 2.73 = 92.19$$

$$P_s = K_a \gamma h_s \times H = 92.19 \times 22 = 2028.22$$

$$P_p = \frac{1}{2} K_p \gamma h^2 = \frac{1}{2} \times 3.25 \times 110 \times 4^2 = 2860$$ [without considering key]

Over turning moment :-

$$M_o = P_a \times \frac{H}{3} + P_s \times \frac{H}{2}$$

$$= 8172.34 \times \frac{22}{3} + 2028.22 \times \frac{22}{2}$$

$$= 82240.91 \text{ lb ft.}$$

Section	Weight (lb)	Moment arm, ft	Moment lb-ft
(I)	$1 \times 20 \times 150 = 3000$	$4 + \frac{1}{2} = 4.5$	13500
(II)	$\frac{1}{2} \times \frac{1}{2} \times 20 \times 150 = 750$	$4 + 1 + \frac{1}{3} \times \frac{1}{2} = 5.17$	3875
(III)	$2 \times 14.5 \times 150 = 4350$	$\frac{14.5}{2} = 7.25$	31537.5
(IV)	$\frac{1}{2} \times \frac{1}{2} \times 20 \times 110 = 550$	$4 + 1 + \frac{2}{3} \times \frac{1}{2} = 5.33$	2933.33
(V)	$9 \times 20 \times 110 = 19800$	$4 + 1.5 + \frac{9}{2} = 10$	198000
Total	$W = 28450$	M=	249845.83

Stability calculation :-

Check for sliding,

Safety of factor, $FS = \dfrac{W \times f}{P_a + P_s}$ [without considering passive earth pressure]

$$= \frac{28450 \times 0.5}{8172.34 + 2028.22}$$

$$= 1.39 < 1.5 \quad \text{(Not OK)}$$

$$FS = \frac{W \times f + P_p}{P_a + P_s} = \frac{(28450 \times 0.5) + 2860}{8172.34 + 2028.22}$$ [with considering passive earth pressure]

$$\therefore FS = 1.67 > 1.5 \quad \text{(OK)}$$

Check against overturning:-

$$FS = \frac{M_R}{M_0} = \frac{249845.83}{82240.91} = 3.04 > 1.5 \quad (\text{ok})$$

Check against soil pressure:

$$a = \frac{M_R - M_0}{W} = \frac{249845.83 - 82240.91}{28450} = 5.89$$

$$e = \frac{B}{2} - a = \frac{14.5}{2} - 5.89 = 1.36$$

$$A = 1 \times 14.5 = 14.5 \ ft^2$$

$$c = \frac{B}{2} = \frac{14.5}{2} = 7.25 \ ft.$$

$$I = \frac{1 \times B^3}{12} = \frac{1 \times 14.5^3}{12} = 254.05 \ ft^4$$

$$M = W \times e = 28450 \times 1.36 = 38692$$

$$\sigma_1 = \frac{W}{A} + \frac{MC}{I} = \frac{28450}{14.5} + \frac{38692 \times 7.25}{254.05}$$

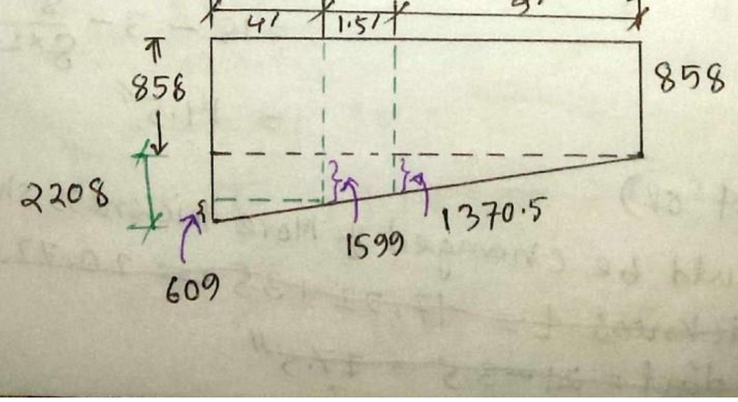
$$\sigma_1 = 1962.06 + 1104.18$$

$$\therefore \sigma_1 = 3066.24 \sim 3066$$

$$\sigma_2 = \frac{W}{A} - \frac{MC}{I} = \frac{28450}{14.5} - \frac{38692 \times 7.25}{254.05}$$

$$\therefore \sigma_2 = 857.88 \sim 858$$

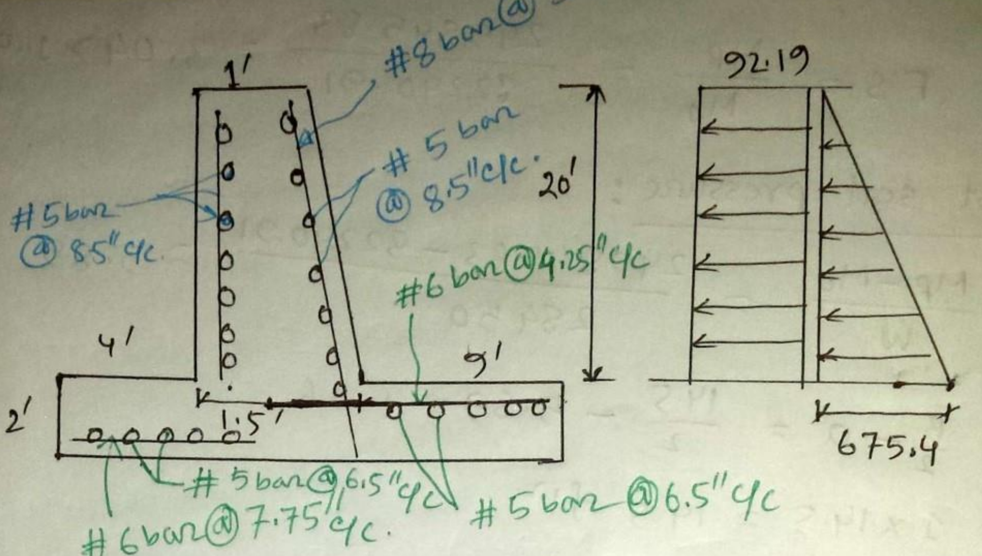
Stress diagram:



$$\frac{2208 \times 9}{14.5} = 1370.5$$

$$\frac{2208 \times 10.5}{14.5} = 1599$$

Design of stem:



Shear:

$$V = 92.19 \times 20 + \frac{1}{2} \times 675.4 \times 20 = 8597.8 \text{ lb}$$

Moment:

$$M = 92.19 \times 20 \times \frac{20}{2} + \frac{1}{2} \times 675.4 \times 20 \times \frac{20}{3}$$

$$= 63464.67 \text{ lb-ft.}$$

Assume,

$k = 0.36$, $j = 0.88$, $f_c = 0.45 f_c' = 0.45 \times 3000 = 1350$ PSI

$\therefore R = \frac{1}{2} f_c j k = \frac{1}{2} \times 1350 \times 0.88 \times 0.36 = 213.84$

depth check:

$$d_{req} = \sqrt{\frac{M}{Rb}},$$

$$= \sqrt{\frac{63464.67 \times 12}{213.84 \times 12}}$$

$$= 17.22''$$

Here,

$M = 63464.67$ lb ft.

$R = 213.84$

$b = 12''$

$d_{act} = t'' - cc - \frac{\phi}{2}$

$= 18'' - 3 - \frac{8}{8 \times 2}$

$= 14.5''$

Since, $d_{req} > d_{act}$ (Not ok)

Hence ~~d ch~~ section should be changed & More thickness should be provid[ed]

$\therefore$ ~~required thickness $t = 17.22 + 3.5 = 20.72 \sim 21''$~~

~~Now, $d_{act}' = 21 - 3.5 = 17.5''$~~

Reinforcement calculation:

$d = d'_{act} = \cancel{14.5}\ 14.5''$

$M = 63464.67$ lb ft

$f_s = 20$ KSI

$$A_s = \frac{M}{f_s j d}$$

$$= \frac{63464.67 \times 12}{20 \times 10^3 \times 0.88 \times 14.5}$$

$$= 2.98 \text{ in}^2$$

$A_{s\,min} = 0.0020\,bt = 0.0020 \times 12 \times 18 = 0.432 \text{ in}^2$

Use #8 bar, $s = \frac{0.79 \times 12}{2.98} = 3.18'' \sim 3.25''$

∴ Use #8 bar @ 3.25'' c/c.

Distribution reinforcement:

$A_{st} = 0.432 \text{ in}^2$

Use #5 bar, $s = \frac{0.31 \times 12}{0.432} = 8.61 \sim 8.5''$.

USE #5 bar @ 8.5'' c/c.

Development length:

$u = \frac{3.4\sqrt{f_c'}}{D}$, $D = \frac{8}{8} = 1''$

$= \frac{3.4\sqrt{3000}}{8/8}$

$= 186.23$

$$l_d = \frac{f_s D}{4u} = \frac{20000 \times 8/8}{4 \times 186.23}$$

$l_d = 26.85'' \sim 27''$

Shear check:

$V = 8597.8$ lb

$b = 12''$

$d = 14.5''$

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

$$= \frac{8597.8}{12 \times 14.5}$$

$$= 49.41 \text{ lb.}$$

$v_{all} = 1.1\sqrt{f_c'} = 1.1\sqrt{3000} = 60.25$ lb.

$v_{all} > v$ (ok)

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Design of Heel:-

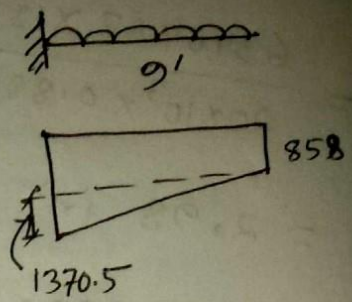
Load calculation:-

(I) Self weight $= 2 \times 150 = 300$ psf.

(II) weight of soil $= 20 \times 110 = 2200$ psf.

(III) Surcharge $= 300$ psf.

Total $= 2800$ psf.



Net moment,

$$M = 2800 \times \frac{7^2}{2} - \left(858 \times \frac{7^2}{2} + \frac{1}{2} \times 1370.5 \times 7 \times \frac{7}{3}\right)$$

$$= 68600 - (21021 + 11192.42)$$

$$= 36386.58 \text{ lb ft.}$$

আগের calculation এর সাথে মিলিয়ে 9' হবে 7' বসিয়েছি।

এইখানে change হবে 9' টা বসবে।

Depth check:

$$d_{req.} = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{36386.58 \times 12}{213.84 \times 12}} = 13.04$$

$R = 213.84$

$b = 12''$

$d_{act} = t - 3.5 = 24 - 3.5 = 20.5$

$d_{act} > d_{req}$ (ok).

Reinforcement calculation:-

$$A_s = \frac{M}{f_s jd} = \frac{36386.58 \times 12}{20000 \times 0.88 \times 20.5} = 1.21 \text{ in}^2$$

$$A_{s\,min} = 0.0020\,bt = 0.0020 \times 12 \times 24 = 0.576 \text{ in}^2$$

USE #6' bar, $s = \frac{0.44 \times 12}{1.21} = 4.36 \sim 4.25''$

USE #6 bar @ $4.25''$ c/c.

Distribution reinforcement:

$A_{st} = 0.576\ in^2$

Use #5 bar, $s = \frac{0.31 \times 12}{0.576} = 6.46'' \sim 6.5''$

Use #5 bar @ 6.5'' c/c.

Development length:

$l_d = \frac{f_s D}{4u}$

$= \frac{20000 \times 6/8}{4 \times 248.3}$

$= 15.1''$

$u = \frac{3.4\sqrt{f_c'}}{D}$, $D = \frac{6}{8}$

$= \frac{3.4\sqrt{3000}}{\frac{6}{8}}$

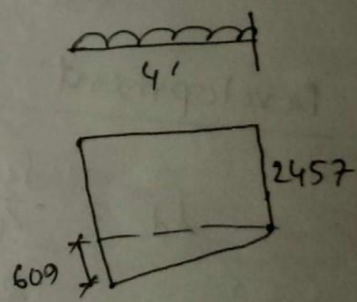
$= 248.3$

Design of Toe:

(1) Load calculation:

weight of soil may be neglected.

(1) self weight of slab $= 2 \times 150 = 300$ psf.



Net moment,

$M = (2457 \times 4 \times \frac{4}{2} + \frac{1}{2} \times 609 \times 4 \times \frac{2}{3} \times 4)$
$- 300 \times 4 \times \frac{4}{2}$

$= 20504$ lb ft

Depth check:

Depth check is not required.

Reinforcement calculation:

$$A_s = \frac{M}{f_s j d}$$ $d = d_{act} = 24 - 3.5 = 20.5$

$$= \frac{20504 \times 12}{20000 \times 0.88 \times 20.5}$$

$$= 0.68 \text{ in}^2$$

$A_{s\,min} = 0.0020\, bt = 0.0020 \times 12 \times 24 = 0.576 \text{ in}^2$

Use, #5 bar, $S = \frac{0.44 \times 12}{0.68} = 7.75''$

Use #6 bar @ 7.75''.

Distribution reinforcement:

$A_{st} = 0.576 \text{ in}^2$

USE #5 bar, $S = \frac{0.31 \times 12}{0.576} = 6.46'' \sim 6.5''$

$\therefore$ USE #5 bar @ 6.5'' c/c.

Development length:

$$l_d = \frac{f_s D}{4u}$$

$$= \frac{20000 \times 6/8}{4 \times 248.3}$$

$$= 15.1''$$

$$u = \frac{3.4\sqrt{f_c'}}{D}, \quad D = \frac{6}{8}$$

$$= \frac{3.4\sqrt{3000}}{6/8}$$

$$= 248.3$$

মোঃ রবিউল ইসলাম
রাজশাহী প্রকৌশল ও প্রযুক্তি বিশ্ববিদ্যালয়
পুরকৌশল বিভাগ
রোল নং ১৬০১১০

Column

* Define column. Classify it.

Answer: Column: A column is a vertical member supporting axial compressive load with or without moment. The cross-sectional dimension of a column generally considerably less than its height. If the unsupported length is more than 3 times, the least dimension of the cross section, then the member is considered as column.

Columns supports usually vertical load from the floor and roofs and transmits this loads to the foundation.

Types of column:

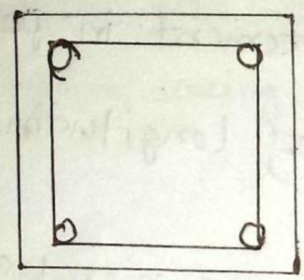
(I) Spiral column: Column's reinfored with longitudinal steel at closely spaced spiral.

(II) Tied column: Column's reinforced with longitudinal steel and lateral ties.

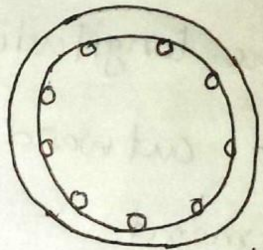
(iii) Composite column: In which a structural steel or cast iron member is thoroughly encased in a spiral column.

মোড়ানো রাখা, আবৃত রাখা

(iv) Combination columns: In which a structural steel member is wrapped with wire and encased in at least $2\frac{1}{2}''$ of concrete.



Tie column



Spiral column.

⇒ Write down minimum dimension of column.

Answer: According to ACI Code:

(i) Spiral column: সর্বনিম্ন #5 bar 6 টি

* Minimum diameter — 10"
* Minimum no of reinforcement — 6
* Minimum size of reinforcement bar = #5 bar.

(ii) Tied column: সর্বনিম্ন #5 bar 4 টি

* Minimum cross sectional area — 96 in^2
* Least dimension — 8"
* Minimum no of reinforcement — 4
* Minimum size of reinforcement — #5 bar.

** Minimum reinforcement ratio = 0.01.

** Maximum reinforcement ratio = 0.08.

* Write short note on lateral reinforcement (or tied)

Answer: Lateral reinforcement: Lateral reinforcement in the form of individual ties or continuous spiral are provided in the column.

Main function of Lateral reinforcement:

(I) To hold the longitudinal reinforcement in position.

(II) To prevent outward bulking of longitudinal reinforcement.

(III) To enhance the ductility of the member.

Specification for lateral reinforcement:

* Minimum size of the bar $\geq$ #3, if longitudinal bar is $\leq$ #10 bar.

* Size $\geq$ #4 bar, if longitudinal bar size is $\geq$ #11 bar

* Size $\geq$ #4 bar, if longitudinal bar's are bundled.

Spacing: (I) $S \leq 16d_b$ [d_b = diameter of main reinforcement]

(II) $S \leq 48d_t$ [d_t = diameter of tie]

(III) $S \leq$ least dimension of the column.

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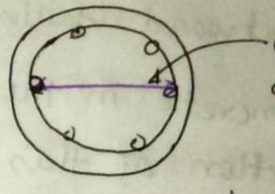
(I) Every longitudinal bar shall have lateral supports from the corner of a tied with an included angle of 135°.

(II) No longitudinal bar shall be more than 6" clear or either side from support bar.

Specification for spiral:



d_c = diameter of the core of the column.

(I) Minimum size # 3 bar.

(II) Maximum spacing, s = 3"

$$s = \frac{d_c}{6}$$

(III) Minimum spacing, s = 1"

Draw the load Vs strain diagram for column. 2015

Answer:

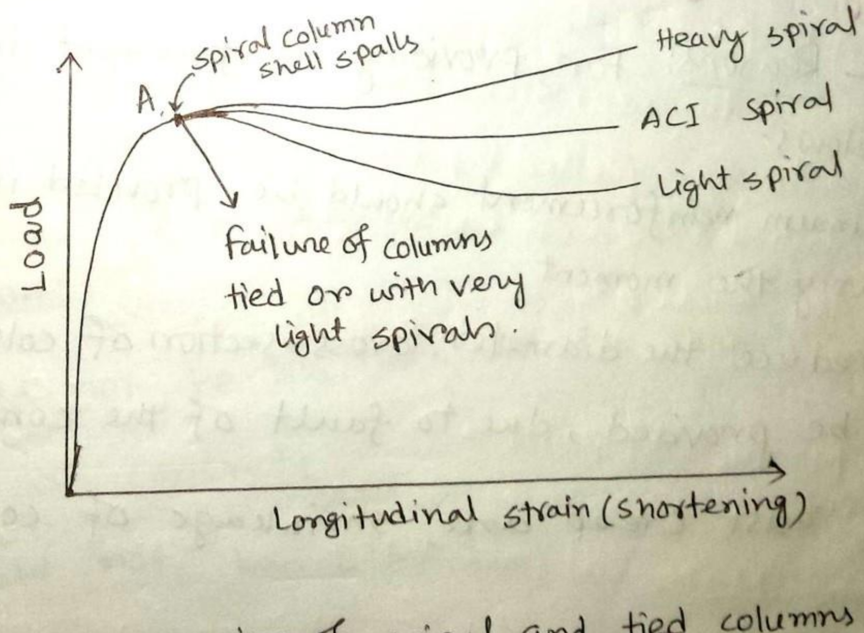


Fig. Behavior of spiral and tied columns.

At point A,

$$P = 0.85\ f_c'\ A_c + f_y\ A_s$$

At point A,

concrete fails by crushing and shearing outward along inclined plane and by bulking of steel between ties.

In a spiral column at the same load the longitudinal steel and concrete within the ~~code~~ core are prevented from outward failing by the spiral.

Concrete in the outer shell spalls of due to the confining action of this spiral the column undergoes large deformation and finally fails by yielding of the spiral or brusting of the ~~code~~ core.

A second maximum load may be achived depending on the amount of spiral.

⇒ Why reinforcement is required for column?

Answer: Reasons: For providing reinforcement in column are given below:-

(I) Minimum reinforcement should be provided in a column to carry the moment.

(II) To reduce the diameter, cross-section of column, the steel should be provided, due to fault of the construction.

(III) To resist creep and shrinkage of concrete.

Q. Flat slab / Write down the function of the column capital and drop panel.

Column capital:

(I) To provide the support to the floor slab from beneath.

(II) Reducing bending moment.

(III) Decreasing shearing stress around the column.

drop panel:

(I) Reducing shearing stress.

(II) Reducing the amount of steel requirement.

(III) Increasing effective thickness.

Q. What is reinforcement ratio? Write down the ACI specification of it.

Answer: Reinforcement ratio: The ratio of the cross-section of main reinforcement and the gross-cross-sectional area of the concrete is called the reinforcement ratio.

It is expressed by ρ_g.

$$\therefore \rho_g = \frac{A_{st}}{A_g}$$

ACI specification:

(I) ρ_g should not be less than 1% to resist the creep and shrinkage of concrete.

(II) ρ_g should not be greater than 8% because of uneconomical.

Q. What may be the reason for low value of φ of column from beam

Answer:

The following reasons are responsible for low value of φ of column from beam :-

(I) Material properties are different.

(II) Consiquence of failure are different.

(III) Crushing failure is occur on column.

Q. What is an interaction diagram? Draw and describe the different parts of I.D with equation of WSD & USD method. 2015

Answer: Interaction diagram: The diagrams that defines the failure load and failure moment for a given column for the full range of ecentricities from 0 to ∞ is knowns as interaction diagram.

Description WSD method:

The following diagram shows the interaction diagram plotted by assuming various - neutral axis and evaluating P and M for each axis.

When moment, M=0 then load, $P=P_0$ at point a.

For $P=0$, $M=m_0$. at point c.

For the balanced failure condition

$P=P_b$ and $M=M_b$. at point b.

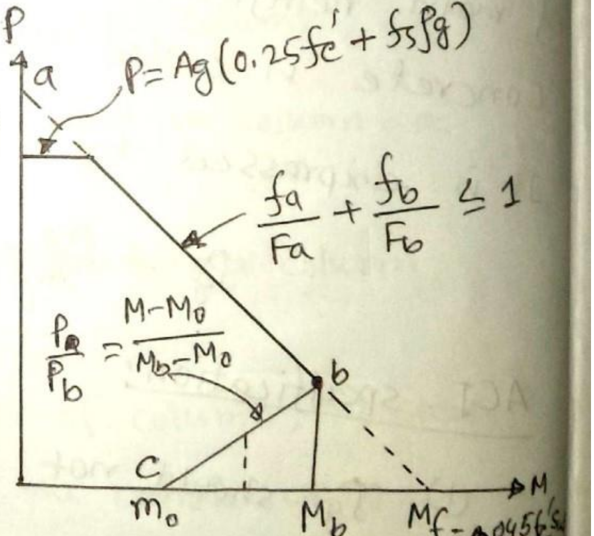
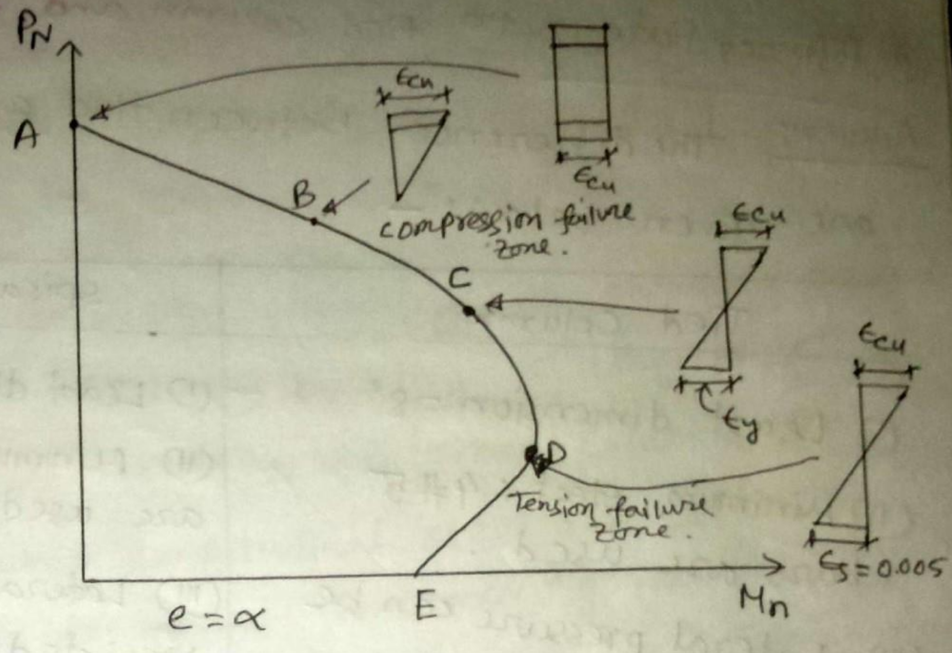


Fig. Interaction diagram at WSD.

The upper portion of point b is called compression failure zone and the lower portion of point b is called tension failure zone.

USD method:



P_N = Nominal load,
M_n = nominal moment.

Interaction diagram,

Points A corresponds to pure axial load. This is the largest axial load that a column can support.

Points B corresponds to the axial load and moment at the onset of crushing of concrete and the strain in the concrete on the opposite face is zero.

Region AC is compression control failure zone.

Points C represents balanced failure point. Points C corresponds to a strain distribution with a maximum compressive strain of 0.003 on one face of the section and a tensile strength equal to the yield strength ϵ_y in the layer of reinforcement furthest from the compression face of the column.

Points D is tensile control limit. It corresponds to strain distribution with 0.003 compressive strain on one face, and tensile strain of 0.005 on the furthest layer of steel from the compression face. CD represents the transition zone.

Points E represents the maximum moment that the column can sustain without any axial load.

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* Differentiate betn tied column and spiral column.

Answer: The differences between tied and spiral column are given below:—

Tied Column	Spiral Column
(I) Least dimension = 8"	(I) Least dimension ≥ 10"
(II) Minimum steel: 4#5 bars are used.	(II) Minimum steel: 6#5 bars are used.
(III) Lateral pressure can be resisted.	(III) Lateral pressure can't be resisted.
(IV) Outward failure can be prevented.	(IV) Outward failure can not be prevented.

Q. Why tied columns are lighter than spiral column?

Answer: Spiral column are heavier than tied column:—

In a spiral column load carrying capacity is increased by using spiral reinforcement which is continuous. Due to this steel, ultimate load is increased and hence a spiral column is heavier than a tied column.

Q. Why spiral column is stronger than tied column? Explain it.

Explanation: The equation of failure of spiral column are,

$$\phi P_{n(max)} = 0.85\phi[0.85 f'_c (A_g - A_{st}) + A_{st} f_y]$$

where, $\phi = 0.7$.

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And for tied column are,

$$\phi P_{n(max)} = 0.80 \phi [0.85 f_c' (A_g - A_{st}) + A_{st} f_y]$$

where, $\phi = 0.65$

The tied column fails by crushing and shearing outward along inclined planes and by bukling of longitudinal steel outward. But in spirally reinforced columns, when same load is reached longitudinal steel and concrete within the core are prevented from moving outward by the spiral. Thus spirally reinforced column possesses more strength than that of a tied column.

Q. What are the factors that ϕ depend on? Write down ACI specification for ϕ.

<u>Answer:</u> <u>ϕ depends on:-</u>

(I) water cement ratio.

(II) Aggregate size.

(III) Hydration.

<u>ACI specification, for ϕ:</u>

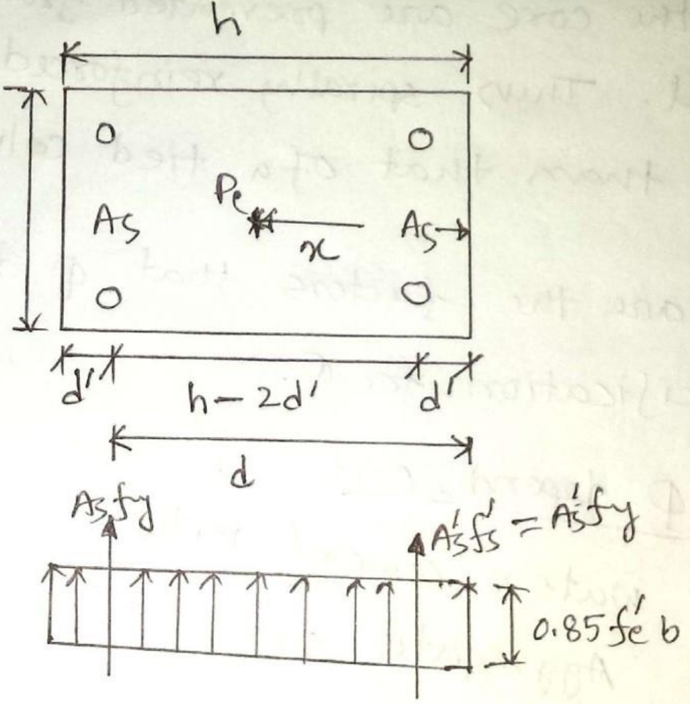
(I) For spiral column, $\phi = 0.7$.

(II) For tied column, $\phi = 0.65$.

(III) For bending, $\phi = 0.9$

(IV) For shear, $\phi = 0.75$

Plastic centroid: Plastic centroid is defined as the point of application of the resultant force for the column's cross setion (including concrete at steel forces). If the column is compressed uniformly to the failure strain $\epsilon_u = 0.003$ over its intire cross section. Eccentricity of the applied load must be measured with respect to the plastic centroid for symmetrically reinforced section plastic centroid and geometric centroid coincide.



$$x = \frac{\left(0.85 f'_c bh * \frac{h}{2}\right) + \left(A'_s f_y * d'\right) + \left(A_s f_y * d\right)}{0.85 f'_c bh + A'_s f_y + A_s f_y}.$$

$$M_b = \left\{0.85 f'_c ab * \left(\frac{h}{2} - \frac{a}{2}\right)\right\} + A'_s f'_s\left(\frac{h}{2} - d'\right) + A_s f_s\left(d - \frac{h}{2}\right)$$

Design of Axially loaded Column (USD)

$$P = A_c f_c + A_s f_s$$

$$= f_c(A_c + nA_s) \qquad [\because f_s = nf_c]$$

$$P = 0.85\, f_c' A_c + f_y A_s$$

Nominal load carring capacity,

$$P_n = (0.85\, f_c' A_c + f_y A_s) * K$$

Ultimate load carring capacity.

$$P_u = \phi P_n = \phi K(0.85\, f_c' A_c + f_y A_s)$$

$$P_u = \phi K(0.85 f_c'(A_g - A_s) + f_y A_s)$$

For tied column,

$K = 0.8$, $\phi = 0.65$.

$$\therefore \boxed{P_u = 0.65 \times 0.8[0.85 f_c'(A_g - A_s) + f_y A_s]}$$

For spiral column,

$K = 0.85$, $\phi = 0.70$

$$\therefore \boxed{P_u = 0.70 \times 0.85\,[0.85\, f_c'(A_g - A_s) + f_y A_s]}$$

Problem: Determine the ultimate axial load of a 12" square tied column reinforced with 4 #9 bars. Ties are # 3 bars placed 12" on centers. Assume $f'_c = 4KSI$ and $f_y = 60000 PSI$. Also check wheater the section satisfy ACI code requirements.

Sol^n Here, $A_g = 12 \times 12 = 144 \ in^2 > 96 \ in^2$ (ok).

Minimum reinforcement.

$A_s = 4$ no. (ok).

$\therefore A_s = 4 \times 1 = 4 \ in^2$

$$\rho_g = \frac{A_s}{A_g} = \frac{4}{144} = 0.028$$

$\therefore \ 0.01 < \rho_g < 0.08$ (ok).

Tied: Spacing,

$s = 16 d_b$ | d_b = dia of main bar.

$= 16 \times \frac{9}{8} = 18$ inch.

d_t = dia of tie

$s = 48 d_t$

$= 48 \times \frac{3}{8} = 18$ inch

$s =$ least dimension of column = 12 inch.

$\therefore$ USE # 3 bar @ 12" c/c.

So, we can conclude that column section satisfy the ACI code requirement.

$\therefore$ Ultimate load bearing capacity,

$P_u = 0.65 \times 0.8 [0.85 f'_c (A_g - A_s) + f_y A_s)]$

$= 0.65 \times 0.8 [0.85 \times 4000 (144 - 4) + 60000 \times 4]$

$= 372.32$ kips (Ans)

Q: Design a spiral column to support an axial dead load of 400 kip and LL = 150 kip, Assume $f_c' = 4$ ksi, $f_y = 60$ ksi and $\rho_g = 5\%$.

Soln

$P_u = 1.2DL + 1.6LL = 1.2 \times 400 + 1.6 \times 150 = 720$ kip.

$A_s/A_g = \rho_g \Rightarrow A_s = \frac{5}{100} \times A_g = 0.05 A_g$

Again, $P_u = \phi k\,[0.85 f_c' (A_g - A_s) + A_s f_y]$

$\Rightarrow 720 = 0.70 \times 0.85\,[0.85 * 4\,(A_g - 0.05A_g) + 0.05A_g * 60]$

$\Rightarrow A_g = 194.23\ in^2$ $\therefore A_s = 9.71\ in^2$

USE 8 #10 bar

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

$\therefore D = \sqrt{\frac{4 \times 194.23}{\pi}} = 15.73 \sim 16'' > 10''$ (ok)

$\therefore A_g = \frac{\pi}{4} \times 16^2 = 201.06\ in^2$

Assume, # 3 bar as spiral,

According to ACI code,

minimum spiral ratio.

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

$= 0.45 * \frac{4}{60} \left\{ \left(\frac{16}{13}\right)^2 - 1 \right\}$

$= 0.015$

Here,

$D = 16''$

$D_c = D - (1.5 \times 2)$

$= 13''$

$\rho_{sp} = \frac{4\,a_{sp}}{g\,D_c} \Rightarrow g = \frac{4\,a_{sp}}{D_c\,\rho_{sp}}$

$= \frac{4 \times 0.11}{13 \times 0.015}$

$= 2.25$ in

a_{sp} #3 bar

$= 0.11\ in^2$

$D_c = 13''$

Maximum spacing of spiral.

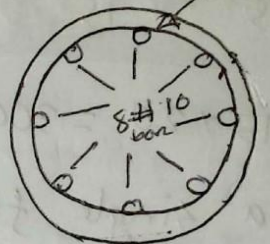
$$g = 3''.$$

$$g = \frac{D_c}{6} = \frac{13}{6} = 2.16$$

$\therefore$ g should be 2 inch.

Minimum spacing of spiral, $g = 1''$.

$\therefore$ USE # 3 bar @ 2'' c/c.



[Working diagram]

Q. Plan of a 5 storied building shown in figure. Design column A. Using following informations.

(I) Floor thickness / slab thickness = 6''

(II) LL on the floor 100 psf.

(III) Size of the beam (12'' x 24'') on both direction.

(IV) $f_c' = 4000$ PSI, $f_y = 60000$ PSI.

Column may be assumes as square section. Column may also be designed as axially loaded column.

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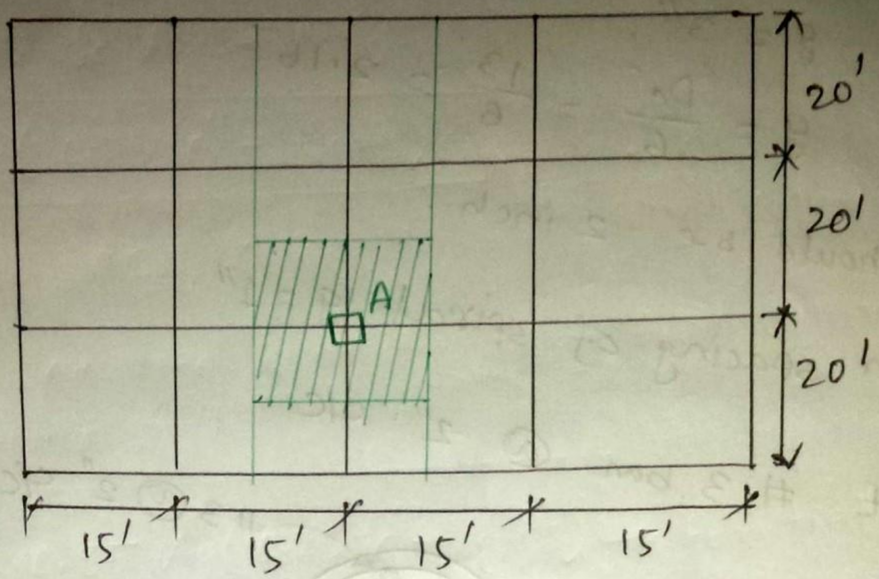


Soln

Loaded area $= 15 \times 20 = 300\ ft^2$

Total live load on a single floor.

$LL = 100 \times 300 = 30000\ lb = 30\ kip$.

$\therefore$ total live load, $LL = 5 \times 30 = 150\ kip$.

Dead load calculation:

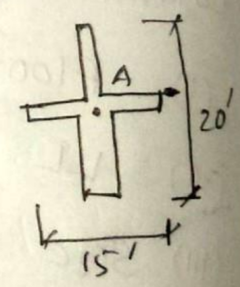
(I) Wt. of slab $= \frac{6}{12} \times \frac{150}{1000} \times 300 = 22.5\ k$

slab thickness

beam length.

(II) wt of beam $= \frac{12}{12} * \frac{24-6}{12} * \frac{150}{1000} \times (15+20)$

$= 7.875\ k$.



floor to floor height

(III) Assume (15"×15") column,

$\therefore$ self weight of column $= \frac{15}{12} \times \frac{15}{12} \times 10 \times \frac{150}{1000} = 2.34\ kip$.

$\therefore$ Total dead load $= 32.72\ kip$. [for single floor]

Total dead load on 5 floor $= 5 \times 32.72 = 163.6k$.

Ultimate load,

$\therefore$ $P_u = 1.2DL + 1.6LL$

$= 1.2 \times 163.6 + 1.6 \times 150$

$\therefore P_u = 436.32\ k$.

Again, $P_u = \phi K A_g [0.85 f_c'(1-\rho_g) + f_y \rho_g]$ $[\because \frac{A_s}{A_g} = \rho_g]$

$\therefore 436.32 = 0.65 \times 0.8 * A_g[0.85 \times 4(1-0.01) + 60 \times 0.01]$ [let $\rho_g = 1\%$]

$\therefore A_g = 211.55\ in^2 > 96\ in^2$ (ok).

$\therefore A_s = \rho_g * A_g = 0.01 \times 211.55 = 2.12\ in^2$.

$\therefore$ USE 4 #7 bar, $A_s = 2.4\ in^2$.

Design of tie:

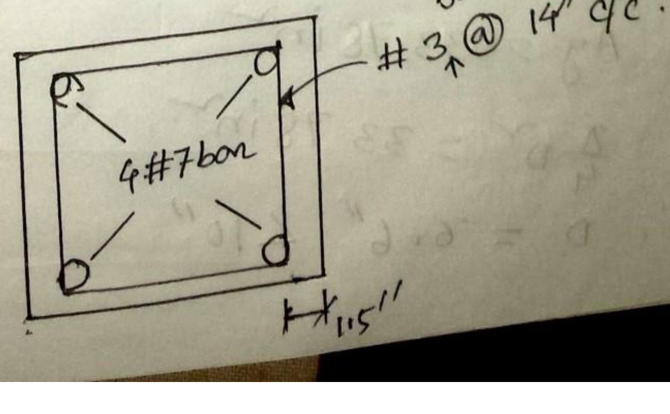
USE # 3 bar,

$\therefore S = 16 D_b = 16 \times \frac{7}{8} = 14''$

$S = 48 D_t = 48 \times \frac{3}{8} = 18''$

$S =$ least dimension $= 15''$

$\therefore$ USE # 3 bar @ $14''$ c/c.



Design of Axially loaded Column by WSD method :-

For spiral column,

$$P = 0.25 f_c' A_g + A_s f_s$$

$$\boxed{P = A_g(0.25 f_c' + \rho_g f_s)}$$ [$\because A_s = A_g \rho_g$] For spiral column.

According to ACI Code,

Axial load carried capacity of a tied column is 85% of spiral column.

$$\boxed{\therefore P = 0.85 A_g(0.25 f_c' + \rho_g f_s)} \rightarrow \text{For tied column}$$

Problem: An axially loaded column is to be designed to carry a working load of 50 kips. Design the column following WSD method Assume $f_c' = 4000$ PSI and $f_s = 24000$ PSI, as (i) The column is spiral column: (ii) The column is a tied column

Soln

Design as a spiral column :-

We know, for spiral column,

$$P = A_g(0.25 f_c' + \rho_g f_s)$$

Assume, $\rho_g = 0.02$

Given, $P = 50^k$

$$\therefore 50 = A_g(0.25 \times 4 + 0.02 \times 24)$$

$$\therefore A_g = 33.78 \text{ in}^2$$

$$\therefore \frac{\pi}{4} D^2 = 33.78 \text{ in}^2$$

$$\therefore D = 6.6'' < 10''$$

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∴ Assume, $D = 10$ inch.

∴ $A_g = \frac{\pi}{4} D^2 = \frac{\pi}{4} \times 10^2 = 78.54 \text{ in}^2$.

∴ $50 = 78.54 \, [0.25 \times 4 + \rho_g \times 24]$

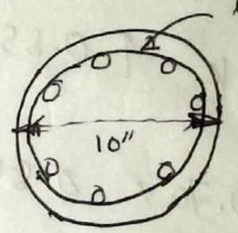
∴ $\rho_g = -0.015$.

∴ $\rho_g = 0.01$.

∴ $A_s = \rho_g A_g = 0.01 \times \frac{\pi}{4} \times 10^2 = 0.79 \text{ in}^2 \sim 3\#5$ bar.

∴ we cannot use this no of bar because Code give us a specific no.

So we use $6\#5$ bar.



Design as a tied colum:

$P = 0.85 A_g (0.25 f_c' + \rho_g f_s)$

$50 = 0.85 \times A_g (0.25 \times 4 + 0.01 \times 24)$

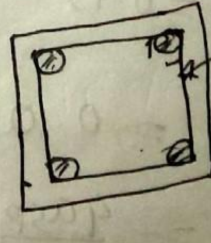
∴ $A_g = 39.74 \text{ in}^2 < 96''$

∴ $A_g = 96 \text{ in}^2$.

Assume column section, $10'' \times 10''$

∴ $A_s = \rho_g A_g = 0.01 \times 10 \times 10 = 1 \text{ in}^2$

∴ USE $4\#5$ bar.



USD

Problem: 2015 3(b) A circular column having a diameter of 25 inch is reinforced with 8 #10 bars. Calculate the ultimate load carrying capacity of the column. Also design the spiral assuming a clear cover of 2 inch. Consider $f_c' = 4000$ PSI and $f_y = 60000$ PSI.

Soln

Given,

$$A_g = \frac{\pi}{4} D^2 = \frac{\pi}{4} \times 25^2 = 490.87 \text{ in}^2$$

$$A_s = 8 * 1.27 = 10.16 \text{ in}^2$$

we know that,

$$P_u = \phi K \left[0.85 f_c' (A_g - A_s) + A_s f_y\right]$$

$$= 0.70 \times 0.85 \left[0.85 \times 4000 * (490.87 - 10.16) + 10.16 \times 60000\right]$$

$$= 1335.2 \text{ kip.}$$

Assume, #3 bar as spiral,

According to ACI Code,

minimum spiral ratio,

Here,
$D = 25''$
$D_c = 25 - (2\times2)$
$= 21''$

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

$$= 0.45 \times \frac{4}{60} \left\{ \left(\frac{25}{21}\right)^2 - 1 \right\}$$

$$= 0.0125$$

$$\therefore \rho_{sp} = \frac{4 a_{sp}}{g D_c} \Rightarrow g = \frac{4 a_{sp}}{\rho_{sp} * D_c}$$

$$\therefore g = \frac{4 \times 0.11}{0.0125 \times 21} = 1.67 \text{ in} \sim 2''$$

Maximum spacing of spiral,

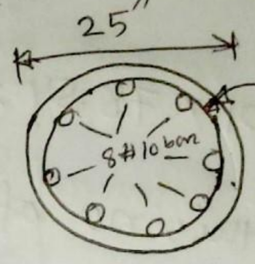
$g = 3''$

$g = \frac{Dc}{6} = \frac{21}{6} = 3.5''$

$\therefore$ g should be $2''$

$\therefore$ Minimum spacing of spiral, $g = 1''$

$\therefore$ USE # 3 bar @ 2" c/c.



#3 bar @ 2" c/c.

working diagram

WSD

2014 7(b) Design completely by WSD method a circular column to carry an axial load of 75 kips. Assume $f_c' = 4000$ PSI and $f_s = 24000$ PSI.

Soln we know that,

$P = A_g(0.25 f_c' + \rho_g f_s)$

$\therefore 75\times10^3 = A_g(0.25\times4000 + 0.02\times24000)$

Assume, $\rho_g = 0.02$

Given, $P = 75$ kips.

$\therefore A_g = 50.68 \text{ in}^2$

$\therefore \frac{\pi}{4}D^2 = 50.68$

$\therefore D = 8.03 \angle 10''$

$\therefore D = 10'' \Rightarrow A_g = \frac{\pi}{4}\times D^2 = \frac{\pi}{4}\times10^2 = 78.54 \text{ in}^2$

$P_c = 0.25 f_c' A_g = 0.25\times4000\times78.54 = 78540$ lb.

$\therefore P_s = P \sim P_c = 75\times10^3 \sim 78540 = 3540 = 3.544.$

$\therefore A_s = \frac{P_s}{f_s} = \frac{3540}{24000} = 0.1475 \text{ in}^2 \sim 2\#3$ bar.

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$75 = 78.54\,[0.25 \times 4 + \rho_g \times 24]$

$\rho_g = -0.0018$.

$\therefore \rho_g = 0.01$

$\therefore A_s = A_g \rho_g = 0.01 \times 78.54 = 0.79\ in^2 \sim 3\#5$ bar.

$\therefore$ we can not use this no of bar because code gives us a specific no.

So we use $6\#5$ bar.

Spacing: Assume Cover $= 1.5''$

$D_c = D - 2 \times 1.5 = 10 - 2 \times 1.5 = 7''$

$\therefore S_{max} = \frac{D_c}{6} = \frac{7}{6} = 1.16'' \sim 1.5''$

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

$\therefore$ Use spacing $= 1.5''$.

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

$= 0.45 \times \frac{4}{60}\left\{\left(\frac{10}{7}\right)^2 - 1\right\}$

$= 0.031$

$\rho_{s2} = \frac{4A_{sp}}{SD_c} = \frac{4 \times 0.11}{1.5 \times 7} = 0.04$

$\rho_{s2} > \rho_{s1}$, (ok)

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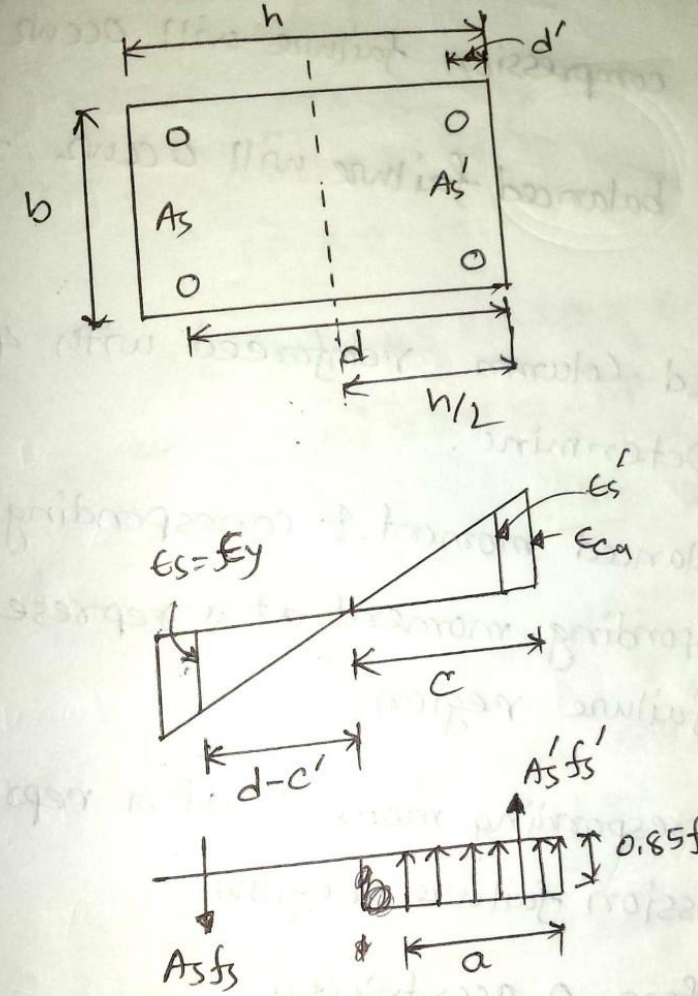
Strain compatability analysis:

For balanced condition,

$\epsilon_{cu} = 0.003$,

$\epsilon_s = \epsilon_y = \dfrac{f_s}{E_s} = \dfrac{60}{29000} = 0.0021$



$$\frac{\epsilon_u}{c} = \frac{\epsilon_y}{d-c} \Rightarrow c\,\epsilon_y = \epsilon_u d - \epsilon_u c;$$

$$a = 0.85c = \beta_1 c;$$

$$\therefore c = \left(\frac{\epsilon_u}{\epsilon_u + \epsilon_y}\right) d \,,$$

$\because \epsilon_s' = \epsilon_u \dfrac{c-d'}{c}$

$f_s' = f_y$,

$f_s' = \epsilon_s' E_s = E_s \epsilon_u \dfrac{c-d'}{c} \leq f_y$

For balanced failure $f_s' = f_y$

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

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

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

* If $e > e_b$, then tension failure will occur, $f_s = f_y$.
* If $e < e_b$ then compression failure will occur.
* If $e = e_b$ then balanced failure will occur, $f_s' = f_s = f_y$.

Problem: A reinforced column reinforeed with 4 #9 bar as shown in figure. Determine.

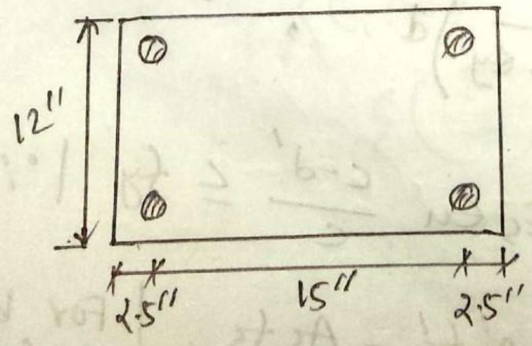
(a) Balanced load, balanced moment, & corresponding eccentricity.

(b) The load and corresponding moment at a representative point in the tension failure region.

(c) The load and corresponding moment at a representative point in the compression failure region.

(d) The axial load for 0 ecentricity.

Assume $f_c' = 4000 PSI$ and $f_y = 60000 PSI$. &



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Soln

(a) For balanced condition, $\epsilon_{cu} = 0.003$,

$$\epsilon_s = \epsilon_y = \frac{f_y}{E_s} = \frac{60}{29000} = 0.0021$$

$$c = c_b = \frac{\epsilon_u}{(\epsilon_u + \epsilon_y)} d$$

$$= \left(\frac{0.003}{0.003 + 0.0021}\right) * 17.5$$

$$= 10.29 \text{ inch.}$$

$$\therefore a = \beta_1 c = 0.85 \times 10.29 = 8.75 \text{ inch.}$$

$$A_s' = 2 \times 1 = 2 \text{ in}^2$$

$$A_s = 2 \times 1 = 2 \text{ in}^2$$

$$\because \epsilon_s' = \epsilon_u \frac{c - d'}{c}$$

Now,

$$f_s' = \epsilon_s' E_s = E_s * \epsilon_u \frac{c - d'}{c} \leq f_y$$

$$= 29000 * 0.003 * \frac{10.29 - 2.5}{10.29}$$

$$\therefore f_s' = 65.86 \text{ KSI} \leq f_y.$$

$$\therefore f_s' = 60 \text{ KSI.}$$

$f_s = f_y$ [for balanced condition]

$\therefore$ compressive force,

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

$$\therefore P_b = 0.85 f_c' ab + A_s' f_y - A_s f_y$$

$$= 0.85 \times 4 \times 8.75 * 12 + 2 \times 60 - 2 \times 60$$

$$= 357 \text{ K}$$ [balanced load]

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∴ Compressive force exerted by concrete.

$$C = 0.85 f_c' ab = 0.85 \times 4 \times 8.75 \times 12 = 357\ k.$$

balanced Moment:

we know that,

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

$$= \left\{0.85 * 4 * 8.75 \times 12 * \left(\frac{20}{2} - \frac{8.75}{2}\right)\right\} + 2 \times 60 * \left(\frac{20}{2} - 2.5\right) + 2 * 60 \left(17.5 - \frac{20}{2}\right)$$

$$= 2008.125 + 900 + 900$$

$$= 3808.125\ k\ \text{inch}.$$

$$\therefore \text{ecentricity, } e_b = \frac{M_b}{P_b} = \frac{3808.125 \text{ kip-inch}}{357 \text{ kip}}$$

$$\therefore e_b = 10.66 \text{ inch. Ans}$$

(b) For tension failure,

If we choose of a higher value than C_b, then $C < C_b$

represents a point in the tension control region. Let,

assume, $c = 6''$ ∴ $a = 0.85 \times 6 = 5.1$ inch.

At this condition, $f_s = f_y$

$$f_s' = \epsilon_s' E_s = E_s \epsilon_u \frac{c - d'}{c} = 29000 * 0.003 * \frac{6 - 2.5}{6}$$

$$\therefore f_s' = 50.75 \text{ KSI}$$

compressive force exerted by concrete.

$C = 0.85 f_c' ab = 0.85 \times 4 \times 5.1 \times 12 = 208.08$ kso.

Load,

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

$= 0.85 f_c' ab + A_s' f_s' - A_s f_y$ $[\because f_s > f_y]$

$= (0.85 \times 4 \times 5.1 \times 12) + (2 \times 50.75) - (2 \times 60)$

$= 208.08 + 101.5 - 120$

$= 189.58$ k.

Moment,

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

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

$= 0.85 \times 4 \times 5.1 \times 12\left(\frac{20}{2} - \frac{5.1}{2}\right) + 2 \times 50.75\left(\frac{20}{2} - 2.5\right)$
$+ 2 \times 60\left(17.5 - \frac{20}{2}\right)$

$= 1550.196 + 761.25 + 900$

$= 3211.45$ kip inch.

$= 267.6$ kip-ft.

$\therefore$ Eccentricity, $e = \frac{M}{P} = \frac{267.6 \text{ k-ft}}{189.58 \text{ k-ft}}$

$\therefore e = 1.41$ ft Ans

(c) Any value of c larger than $c_b = 10.66$ inch will represents a point in the compression controlled region.

Let, Assume, $c = 18''$.

$$\varepsilon_s = \varepsilon_u \left(\frac{d-c}{c}\right) = 0.003 * \frac{17.5-18}{18}$$

$$\therefore \varepsilon_s = -0.00008.$$

$$\therefore f_s = E_s \varepsilon_s = 29000 \times (-0.00008)$$

$$\therefore f_s = -2.42 \text{ KSI}.$$

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

Compressive force exerted by concrete,

$$C = 0.85 f_c' ab = 0.85 \times 4 \times 15.3 \times 12 = 624.24 \text{ K}.$$

$$f_s' = E_s \varepsilon_u \frac{c-d'}{c} = 29\times10^6 \times 0.003 \cdot \frac{18-2.5}{18} \leq f_y$$

$$\therefore f_s' = 74.92 \leq f_y.$$

$$\therefore f_s' = f_y.$$

$\therefore$ Load,

$$P_n = 0.85 f_c' ab + A_s' f_s' - A_s f_s$$
$$= 0.85 f_c' ab + A_s' f_y - \cancel{A_s f_y} . A_s f_s$$
$$= 0.85 \times 4 \times 15.3 \times 12 + 2 \times 60 - 2 \times (-2.42)$$
$$= 624.24 + 120 + 4.84$$
$$= 749 \text{ K}.$$

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Moment,

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

$= 0.85 \times 4 \times 15.3 \times 12\left(\frac{20}{2}-\frac{15.3}{2}\right) + 2 \times 60\left(\frac{20}{2}-2.5\right) + 2\times(-2.42)\left(17.5-\frac{20}{2}\right)$

$= 1466.964 + 900 - 36.3$

$= 2330.664$ kip-inch.

$\therefore$ Eccentricity, $e = \frac{M}{P} = \frac{2330.664}{749} = 3.11''$ (inch)

(d) For 0 eccentricity, $c = \alpha$, $e = 0$, $a = h$.

$P_n = 0.85 f_c' A_g + A_{st} f_y$

$= 0.85 \times 4 (A_g - A_{st}) + A_{st} f_y$

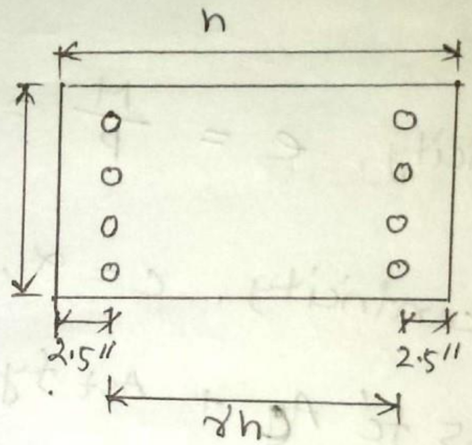
$= 0.85 \times 4 (20\times12 - 4) + 4\times60$

$= 1042.4^{K}$.

Design of Column USD Method:

Problem: Design a column using the following data.
DL = 220 kips, LL = 335 kips, Dead load moment = 165 kip-ft.
Live load moment = 165 kip-ft. Also, $f_c' = 4$ KSI, $f_y = 60$ KSI.
Column section in limited in dimension of $20'' \times 25''$.
$\boxed{b \times h}$

Solution:



$$A_g = 20 \times 25$$
$$= 500 \text{ in}^2$$

$$\gamma = \frac{25-(2\times2.5)}{25}$$

$\therefore \gamma = 0.8$.

For tied column, $\phi = 0.65$,
Factored load,
$$P_u = 1.2DL + 1.6LL = (1.2 \times 220) + (1.6 \times 335) = 800 \text{ kip}.$$

Factored Moment,
$$M_u = 1.2M_{DL} + 1.6M_{LL} = 1.2\times165 + 1.6\times165 = 462 \text{ kip-ft}.$$

$$K_n = \frac{P_u}{\phi f_c' A_g} = \frac{800}{0.65\times4\times500} = 0.62$$

$$R_n = \frac{M_u}{\phi f_c' A_g h} = \frac{462\times12}{0.65\times4\times500\times25} = 0.18$$

From table, K_n Vs R_n.

$\rho_g = 0.02 \sim 0.03$ [Eye estimation]

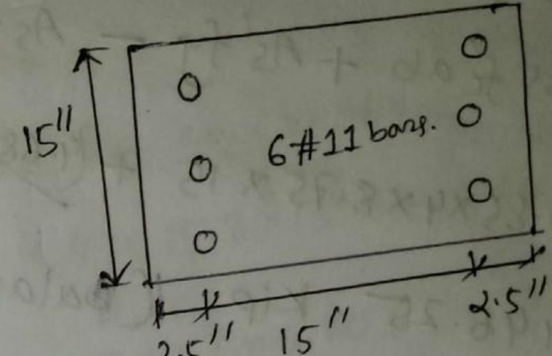
$\rho_g = 0.028$

$\therefore A_s = A_g \rho_g = 500 \times 0.028 = 14\ in^2$

USE 14 # 9 bar.

Ans

2015 6(b) A 15"x20" column is reinforced with 6#11 bars as shown in figure below. Find the ultimate load and corresponding moment for an eccentricity of e = 8". Assume $f_c' = 4$ KSI, $f_y = 60$ KSI



Soln For balanced failure condition, $\epsilon_{cu} = 0.003$.

$$\epsilon_s = \epsilon_y = \frac{f_y}{E_s} = \frac{60}{29000} = 0.0021$$

$$c = c_b = \left(\frac{\epsilon_u}{\epsilon_u + \epsilon_y}\right) d$$

$$= \left(\frac{0.003}{0.003 + 0.0021}\right) \times 17.5$$

$$= 10.29 \text{ inch}$$

$$\therefore a = \beta_1 c = 0.85 \times 10.29 = 8.75 \text{ inch}$$

$A_s = 3\#11$ bar $= 3 \times 1.56 = 4.68\ in^2$

$A_s' = 3\#11$ bar $= 3 \times 1.56 = 4.68\ in^2$

Now, $f_s' = \epsilon_s' E_s = E_s\ \epsilon_u \frac{c - d'}{c} \leq f_y$

$$= 29000 \times 0.003 * \frac{10.29 - 2.5}{10.29} \leq f_y$$

$$\therefore f_s' = 65.86 \text{ KSI} \leq f_y$$

$\therefore f_s' = 60$ KSI.

$f_s = f_y$ [for balanced condition]

compression zone,

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

$\therefore P_b = 0.85 f_c' ab + A_s' f_y - A_s f_y$

$= 0.85 \times 4 \times 8.75 \times 15 + \cancel{(4.68 \times 60)} - \cancel{(4.68 \times 60)}$

$= 446.25$ Kip (balanced load)

$\therefore$ Compressive force exerted by concrete.

$C = 0.85 f_c' ab = 446.25$ Kip.

Balanced Moment:

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

$= \{0.85 \times 4 \times 8.75 \times 15 (\frac{20}{2} - \frac{8.75}{2})\} + 4.86 \times 60 (\frac{20}{2} - 2.5)$

$+ 4.86 \times 60 (17.5 - \frac{20}{2})$

$= 2610.16 + 2187 + 2187$

$= 6884.16$ kip-inch.

$\therefore$ eccentricity, $e_b = \frac{M_b}{P_b} = \frac{6884.16}{446.25} = 15.43''$

Scanned by CamScanner

Since $e < e_b$ then compression failure will occur.

Assume, $c = 18''$,

$\therefore \epsilon_s = \epsilon_u \left(\frac{d-c}{c}\right) = 0.003 * \frac{17.5-18}{18}$

$\therefore \epsilon_s = -0.00008$.

$\therefore f_s = E_s \epsilon_s = 29000(-0.00008) = -2.42\,ksi$.

$a = \beta_1 c = 0.85 \times 18 = 15.3''$.

compression force exerted by concrete.

$C = 0.85 f_c' a b = 0.85 \times 4 \times 15.3 \times 15 = 780.3$ Kip.

$f_s' = E_s \epsilon_u \frac{c-d'}{c} = 29 \times 10^6 \times 0.003 \; \frac{18-2.5}{18} \le f_y$

$\therefore f_s' = 74.92 \le f_y$

$\therefore f_s' = f_y$.

$\therefore$ Load,

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

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

$= (0.85 \times 4 \times 15.3 \times 15) + (4.86 \times 60) - 4.86(-2.42)$

$= 780.3 + 291.6 + 11.76$

$= 1083.66$ Kip. Ans

Moment,

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

$= 0.85 \times 4 \times 15.3 \times 15\left(\frac{20}{2} - \frac{15.3}{2}\right) + 4.86 \times 60\left(\frac{20}{2} - 2.5\right) + 4.86 \times (-2.42) \times \left(17.5 - \frac{20}{2}\right)$

$= 1833.705 + 2187 - 88.209$

$= 3932.5$ Kip-inch. Ans

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Compression + Bending in WSD Method:

Problem: A 16" x 25" column is reinforced with 8 # 11 bars.

Calculate (a) Allowable axial load.

(b) Allowable moment if axial load is 360 kips.

(c) The allowable moment if the simultaneous axial load is 100 kips.

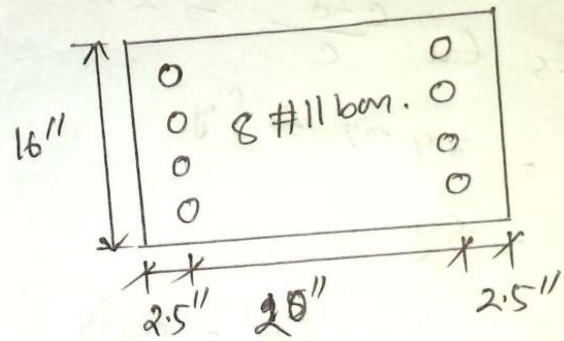
(d) The balanced axial load and Moment, P_b, M_b.

when $f_c' = 4$ KSI, $f_y = 50$ KSI.

Soln

$A_s = 8$ # 11 bar

$\therefore A_s = 8 \times 1.56 = 12.48 \text{ in}^2$



$A_g = 16 \times 25 = 400 \text{ in}^2$

$\rho_g = \frac{A_s}{A_g} = \frac{12.48}{400}$

$\therefore \rho_g = 0.0312$

$$P_{all} = 0.85 A_g (0.25 f_c' + \rho_g f_s)$$

$$= 0.85 \times 400 (0.25 \times 4 + 0.0312 \times 20)$$

$$= 552.16 \text{ k}$$ Ans (a)

(b) We know that for analysis.

$$\frac{f_a}{F_A} + \frac{f_b}{F_b} = 1$$

$$f_a = \frac{\text{Axial load}}{\text{Gross Area}} = \frac{P}{A_g} = \frac{360}{400} = 0.9$$

$$F_a = 0.34\, f_c' \,(1 + \rho_g m)$$
$$= 0.34 \times 4 (1 + 0.0312 \times 14.71)$$
$$= 1.98 \text{ KSI}$$

$$m = \frac{f_y}{0.85 f_c'} = \frac{50}{0.85 \times 4} = 14.71$$

F_b = Allowable bending stress for bending alone.

$$F_b = 0.45\, f_c' = 0.45 \times 4 = 1.8 \text{ KSI}$$

$$f_b = \frac{\text{Bending Moment}}{\text{section modulus of uncracked transform section}}$$

$$\therefore f_b = \frac{M}{S_{ut}}$$

$$S_{ut} = \frac{I_{ut}}{c} = \frac{1}{c} * I_{ut}$$

$$I_{ut} = \frac{bh^3}{12} + 2 * \frac{A_s}{2}(2n-1)\left(\frac{h}{2} - d'\right)^2$$
$$= \frac{16 \times 25^3}{12} + 12.48(2 \times 8 - 1)\left(\frac{25}{2} - 2.5\right)^2$$
$$= 39553.33 \text{ in}^4$$

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

$$c = \frac{h}{2} = \frac{25}{2}$$
$$\therefore c = 12.5$$

$$\therefore S_{ut} = \frac{I_{ut}}{c} = \frac{39553.33}{12.5} = 3164.3 \text{ in}^3$$

$$\therefore \frac{0.9}{1.98} + \frac{\frac{M}{3164.3}}{1.8} = 1.$$

$$\therefore M = 3106.76 \text{ kip-inch}$$
$$\therefore M = 259 \text{ k-ft.}$$

$$\therefore e = \frac{M}{P} = \frac{259}{360} = 0.72 \text{ ft} = 8.6 \text{ inch}$$ Ans

(d) Formula calculate balanced Eccentricity:

$$e_b = (0.67\ \rho_g m + 0.17)d$$

$$= (0.67 \times 0.0312 \times 14.71 + 0.17) \times 22.5$$

$$= 10.74 \text{ inch.}$$

$$e < e_b.$$

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = 1.$$

$$f_b = \frac{M_b}{S_{ut}} = \frac{P_b e_b}{S_{ut}}.$$

$$f_a = \frac{P}{A_g} = \frac{P_b}{A_g}.$$

$$\therefore \frac{\frac{P_b}{A_g}}{F_a} + \frac{\frac{P_b e_b}{S_{ut}}}{F_b} = 1$$

$$\Rightarrow \frac{P_b}{A_g F_a} + \frac{P_b e_b}{S_{ut} F_b} = 1$$

$$\Rightarrow \frac{P_b}{400 \times 1.98} + \frac{P_b \times 10.74}{3164.3 \times 1.8} = 1$$

$$\Rightarrow P_b = 317.57^{k}$$

$$\therefore M_b = P_b e_b = 317.57 \times 10.74 = 3410.72 \text{ k-inch.}$$

(c) Let us consider [P = 100 kips]

Tension will govern for this load condition.

Now, $\frac{P}{P_b} = \frac{M - M_0}{M_b - M_0}$

$M_0 = 0.4 A_s f_y (d - d')$
$= 0.4 \times 4 \times 1.56 \times 50 (22.5 - 2.5)$
$= 2496$ kip-inch

$\Rightarrow \frac{100}{317.57} = \frac{M - 2496}{3410.72 - 2496}$

$\Rightarrow M = 2784$ kip-inch.

$\therefore e = \frac{M}{P} = \frac{2784}{100} = 27.95$ inch.

$e > e_b$. (ok)

<u>Note:</u> $e < e_b$, $c > c_b \rightarrow$ compression govern করবে।
$e > e_b$, $c < c_b \rightarrow$ tension govern করবে।

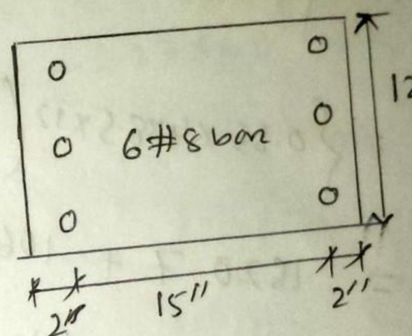
Question Solve

Column

মো: রবিউল ইসলাম CE'130110.

Tied USD

Question 2014 2(b) The column section shown below in the figure is reinforced with 6#8 bars. Estimate by USD method, the ultimate load of the column for an ecentricity of 12 inch. Data given, $f_c' = 4$ KSI and, $f_y = 60$ KSI.



soln

Given,

$b = 12''$

$h = 19''$

$d = 17''$

$d' = 2''$

$A_s = 3 \times 0.79 = 2.37 \text{ in}^2$

$A_s' = 3 \times 0.79 = 2.37 \text{ in}^2$

$\epsilon_u = 0.003$

$\epsilon_y = \frac{f_y}{E_s} = \frac{60000}{29 \times 10^6}$

$\therefore \epsilon_y = 0.0021$

$$c = c_b = \frac{\epsilon_u}{\epsilon_u + \epsilon_y} d$$

$$= \frac{0.003}{0.003 + 0.0021} * 17$$

$$= 10 \text{ inch.}$$

$\therefore a = \beta_1 c = 0.85 \times 10 = 8.5$ inch

Now, $f_s' = E_s \epsilon_s' = E_s * \epsilon_u \frac{c - d'}{c} \le f_y$

$$= 29000 * 0.003 * \frac{8.5 - 2}{8.5} \le f_y$$

$$= 66.53 \text{ KSI} \le f_y$$

$\therefore f_s' = 60$ KSI.

$f_s = f_y$.

Balanced load

$$P_b = 0.85 f_c' a b + A_s' f_y - A_s f_y$$

$$= 0.85 \times 4 \times 8.5 \times 12 + 2.37 \times 60 - 2.37 \times 60$$

$$= 346.8 \text{ kip.}$$

Balanced Moment:

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

$$= \left\{ 0.85 \times 4 \times 8.5 \times 12 \left(\frac{19}{2} - \frac{8.5}{2} \right) \right\} + 2.37 \times 60 \left(\frac{19}{2} - 2 \right) + 2.37 \times 60 \left(17 - \frac{19}{2} \right)$$

$$= 1820.7 + 1066.5 + 1066.5$$

$$= 3953.7 \text{ kip-inch}$$

$$\therefore e_b = \frac{M_b}{P_b} = \frac{3953.7}{346.8} = 11.4 \text{ inch} < e = 12''$$

$e_b < e$, tension controlled failure will occur.

Let, $C = 9''$, $a = \beta_1 C = 0.85 \times 9 = 7.65''$

$$f_s' = E_s \epsilon_s' = E_s \epsilon_u \frac{c - d'}{c} \le f_y.$$

$$= 29000 \times 0.003 \times \frac{9-2}{9} \le f_y$$

$$= 67.67 \text{ KSI} \le f_y$$

$$\therefore f_s' = 60 \text{ KSI}$$

$$f_s = E_s \epsilon_u \frac{d - c}{c} = 29000 \times 0.003 \times \frac{17-9}{9} \le f_y$$

$$f_s = 77.33 > 60 \text{ KSI}.$$

$$\therefore f_s = 60 \text{ KSI}$$

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

$= 0.85 \times 4 \times 7.65 \times 12\left(\frac{19}{2} - \frac{7.65}{2}\right) + 2.37 \times 60\left(\frac{19}{2} - 2\right) + 2.37 \times 60 \left(17 - \frac{19}{2}\right)$

$= 1771.281 + 1066.5 + 1066.5$

$= 3904.3$ k - inch.

$\therefore P_n = \frac{M_n}{e} = \frac{3904.3}{12} = 325.4$ k.

$\therefore P_n = 0.85 f_c' ab + A_s' f_s' - A_s f_y$

$325.4 = 0.85 \times 4 \times a \times 12 + 2.37 \times 60 - 2.37 \times 60$

$\therefore a = 7.97$ inch.

$\Rightarrow a = \beta_1 c$

$\therefore c = \frac{7.97}{0.85} = 9.3'' \approx 9''$

so, $P_n = 325.4$ k.

$M_n = 3904.3$ kip-inch.

$P_u = \alpha \phi P_n = 0.8 \times 0.65 \times 325.4 = 169.2$ k

$M_u = \alpha \phi M_n = 0.8 \times 0.65 \times 3904.3 = 2030.24$ k-inch.

Spiral WSD:-

2011 7(b) Design completely a circular reinforcement concrete column to support a concentrated load of 420 kips using WSD method. Data, $f_c' = 3000$ PSI, $f_y = 60000$ PSI, $f_s = 24000$ PSI.

Soln

We know that,

Assume,
$\rho_g = 2\% = 0.02$
$f_s = 24000$ PSI.
$f_c' = 3000$ PSI

$$P = A_g(0.25 f_c' + \rho_g f_s)$$

$$420000 = A_g(0.25 \times 3000 + 0.02 \times 24000)$$

$$A_g = 341.46 \text{ in}^2$$

$$\frac{\pi}{4} D^2 = 341.46$$

$$\therefore D = 20.85 \sim 20.5''$$

$$\therefore A_g = \frac{\pi}{4} D^2 = 330 \text{ in}^2$$

$$P_c = 0.25 f_c' A_g = 0.25 \times 3000 \times 330 = 247500$$

$$P_s = P - P_c = 420000 - 247500 = 172.5 \text{ K}$$

$$P_s = A_s f_s$$

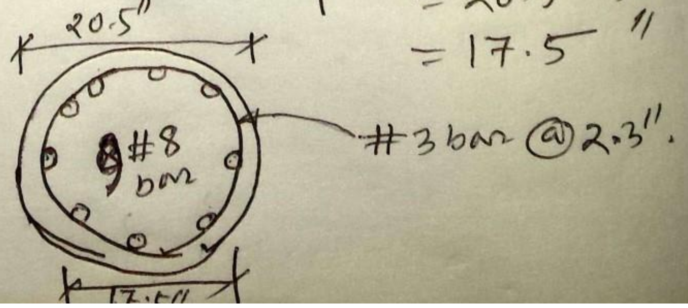
$$\therefore A_s = \frac{P_s}{f_s} = \frac{172.5}{24} = 7.1875 \text{ in}^2$$

Using, ~~8#9~~ ~~8#9 bar~~. 9 #8 bar.

$$S_{max} = \frac{D_c}{6} = \frac{17.5}{6} = 2.92'' \sim 3''$$

$D_c = D - 2 \times 1.5$
$= 20.5 - 3$
$= 17.5''$

$\therefore S_{max} = 3''$



Check:

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

$= 0.45 \times \frac{4}{60} \left\{ \left(\frac{20.5}{17.5}\right)^2 - 1 \right\}$

$= 0.011$

$\rho_{sp} = \frac{4 a_{sp}}{g D_c}$

$g = \frac{4 a_{sp}}{\rho_{sp} \times D_c} = \frac{4 \times 0.11}{0.011 \times 17.5} = 2.3''$

Maximum spacing of spiral,

$g = 3''$

$g = \frac{D_c}{6} = 3''$

$g_{min} = 1''$

$\therefore$ g should be $2.3''$

$\therefore$ Use # 3 bar @ $2.3''$ c/c.

মো: রবিউল ইসলাম
রাজশাহী প্রকৌশল ও প্রযুক্তি বিশ্ববিদ্যালয়
পুরকৌশল বিভাগ
রোল নং: ১৬০১১০

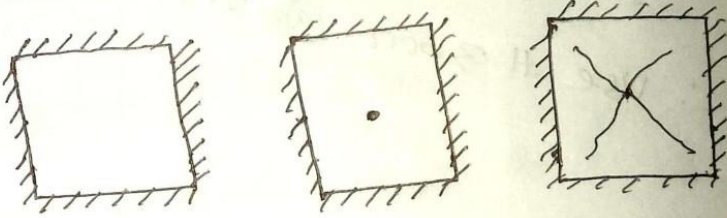
YIELD LINE THEORY

2015, 14, 12, 08, 01:

Q: Explain yield line theory. state its assumptions. what are the limitations of yield line theory.

Answer: Yield Line theory: The method of analyzing the reinforced concrete slab which utilize the plastic deformation is called the yield line theory.

Explanation:



(a) (b) (c)

(a) Let us consider an isotropically reinforced square slab {fig(a)} subjected to gradually increased load under simply supported condition.

(b) If load increases, the centre of slab will crack on the tension side {fig.(b)}.

(c) Any further increase in load will cause distribution of moment, to the uncracked position which will result in increase of crack about centre (fig-c)

(d) At this condition slab carries maximum load any further increase of load will cause the slab to collapse.

Guide lines for establishing axes of rotation and yield lines are summarized as follows:—

Assumption:

(I) Yield lines are straight lines.

(II) Yield lines represents axes of rotation.

(III) The supported edges of the slab will also establish axes of rotation.

(IV) An axes of rotation will pass over any column support.

~~Its orienta~~

(V) Yield lines form under concentrated loads, radiating outward from the point of application

(VI) A yield line between two slab segments must pass through the point of intersection of the axes of rotation of the adjacent slab segments.

Limitation:

(I) It doesn't give any information about stresses, deflection and load condition.

(II) It is to be ensured that earliest failure will not be occurred due to shear, bond or other causes.

Q. Define positive yield line, negative yield line, isotropically reinforced slab, orthrotopically reinforced slab.

Answer: Positive yield line: Yield line associated with tension at the bottom of a slab is called the positive yield line.

Negative yield line: Yield line associated with tension at the top of a slab is called the negative yield line.

Isotropically reinforced slab: If a slab is reinforced in orthogonal direction ultimate resisting moment is the same in these two direction such slab is called isotropically reinforced slab.

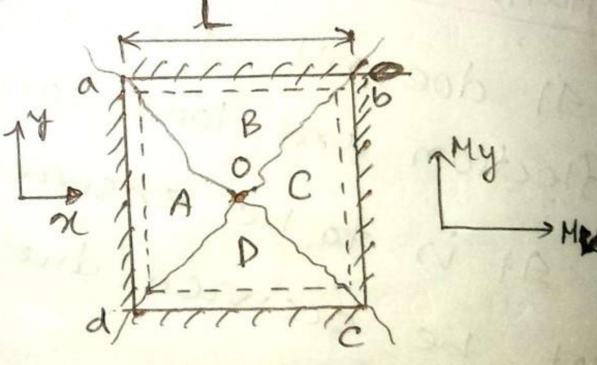
Orthotropically reinforced slab: If the ultimate strengths are different in two perpendicular direction, the slab is called orthotropically reinforced slab.

2014,10

Q. Find out the ultimate load for a square slab. Simply supported in all sides and uniformly loaded.

Derivation:

Let us consider the square slab with a load intensity of w as shown in figure.



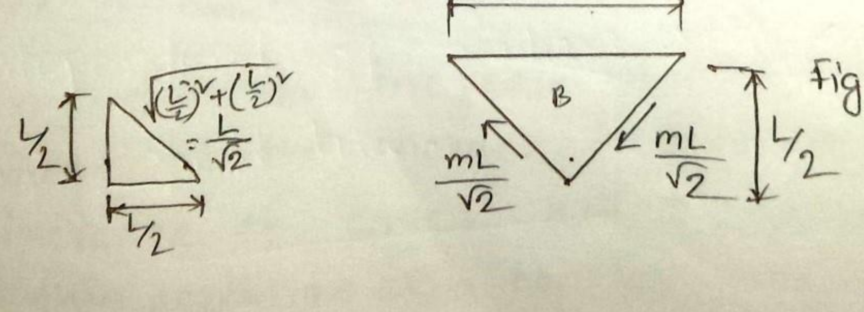


Fig. Analysis of a square two-way slab by segment equilibrium equation.

~~considering slab element, B.~~

~~External moment = MuL~~

considering the moment equilibrium of any one of the identical slab segments about its support, one obtains.

$$\frac{1}{2}\times L\times\frac{L}{2}\times w\times\frac{1}{3}\times\frac{L}{2} - 2\,\frac{mL}{\sqrt{2}}\cdot\frac{1}{\sqrt{2}} = 0$$

$$\text{or,}\quad \frac{wL^2}{4}\cdot\frac{L}{6} - mL = 0$$

$$\therefore\quad m = \frac{wL^2}{24}$$

Q. Define plastic hinge?

Answer: Plastic hinge: If a reinforced concrete member as structural form is subjected to progressive loading moment at certain section will increase as the magnitude of the load increases. After yielding, moment in that section will not increase with further increase of load that is the section behaves like a hinge. This concept is known as plastic hinge theory.

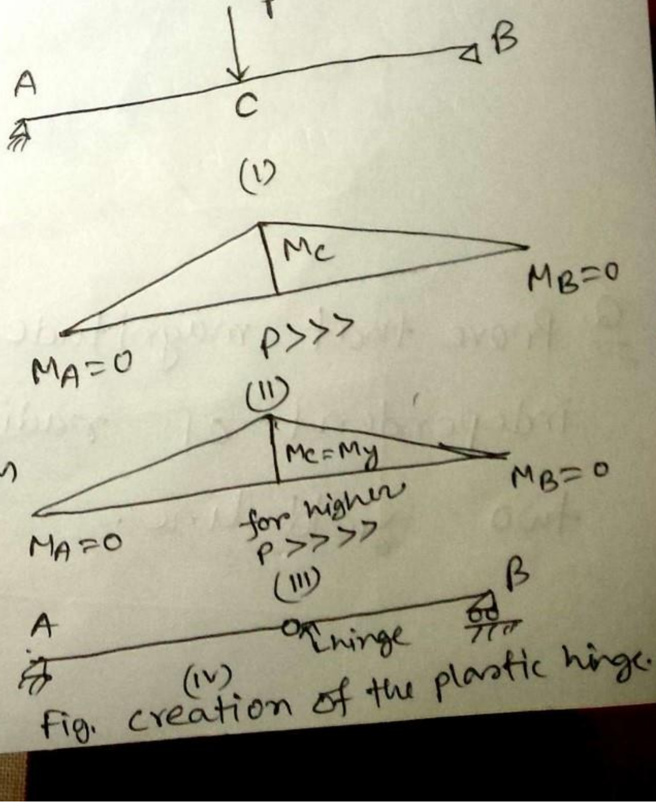


Fig. creation of the plastic hinge.

Explanation:

Here, considering the following fig. Showing the formation of plastic hinge. where for increasing load the moment increases at (II), (III) diagram. But after reaching at yield position moment will not increase and a hinge is formed as the plastic hinge at figure (IV)
So, from the above fig. with the explanation formation of plastic hinge is clearly understood.

2013

Q. Define. Collapse Mechanism.

Answer: Collapse mechanism: If a structure is subjected to progressive loading, moment at some section increases as if magnitude of load increases. The section with maximum moment is frusted and plastic hinge is formed at that section. Plastic hinge is formed also at another section depending on the boundary conditions and loading pattern for a certain number of plastic hinge at the structure and it reaches a mechanism for which it collapse.

This mechanism is known as collapse mechanism.

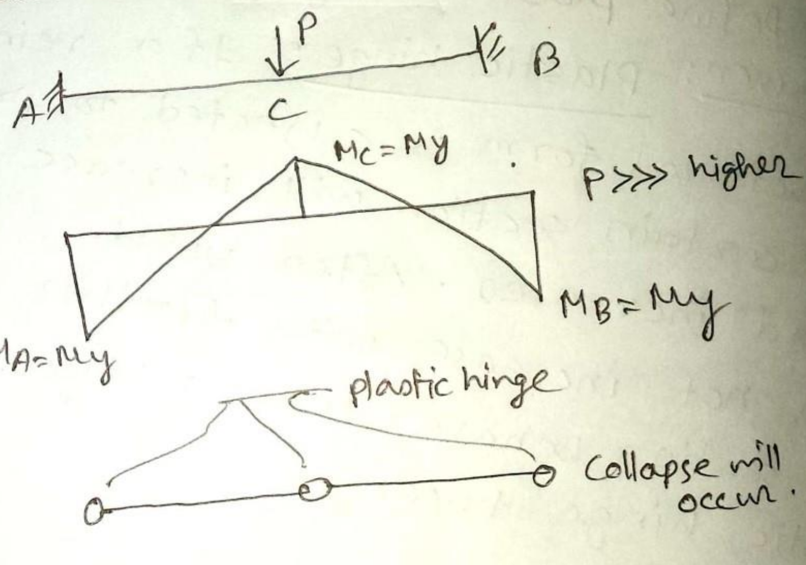


Fig. Collapse mechanism.

Q. Prove that magnitude of concentrated load is independent of radius and angle between two yield line.

proof: If a concentrated load acts on a reinforced concrete slab at an interior location, away from any edge or corner a negative yield line will form with positive yield lines radiating outward from the load point.

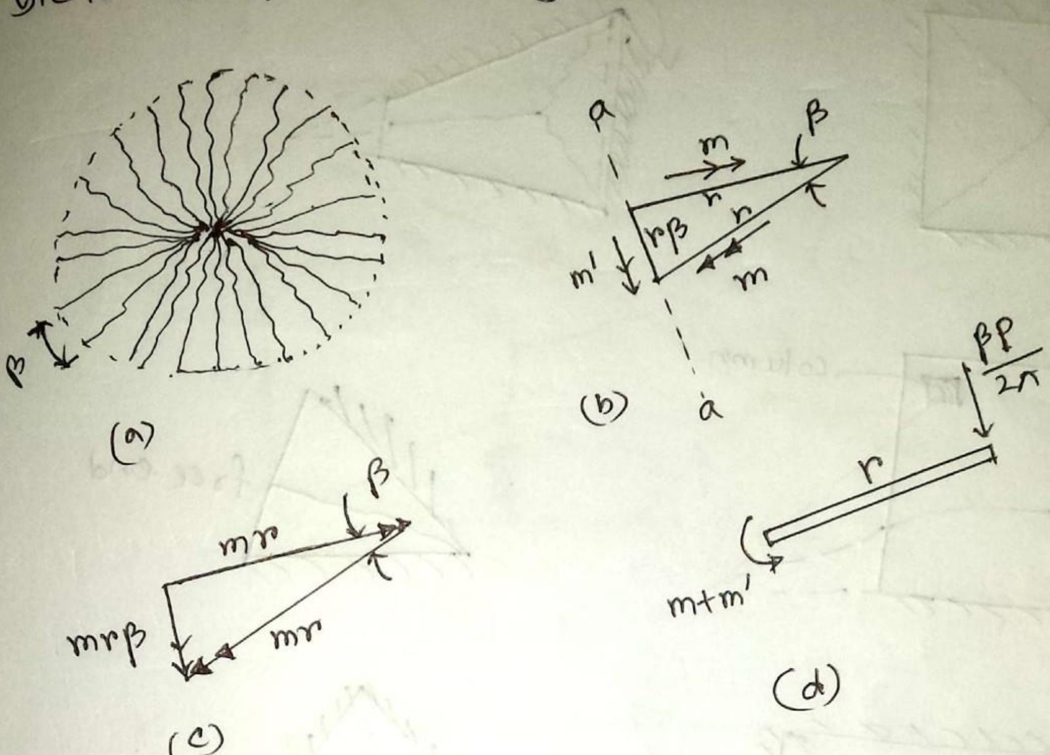


Fig. Fan patterns at concentrated load of yield geometry.

If positive and negative resisting moment per unit length are m & m' and r is radius, β is single elements central angle and

Fig. (c) shows vector addition of resultant ~~for~~ moment.

Fig. (d) shows the fractional part of the total load.

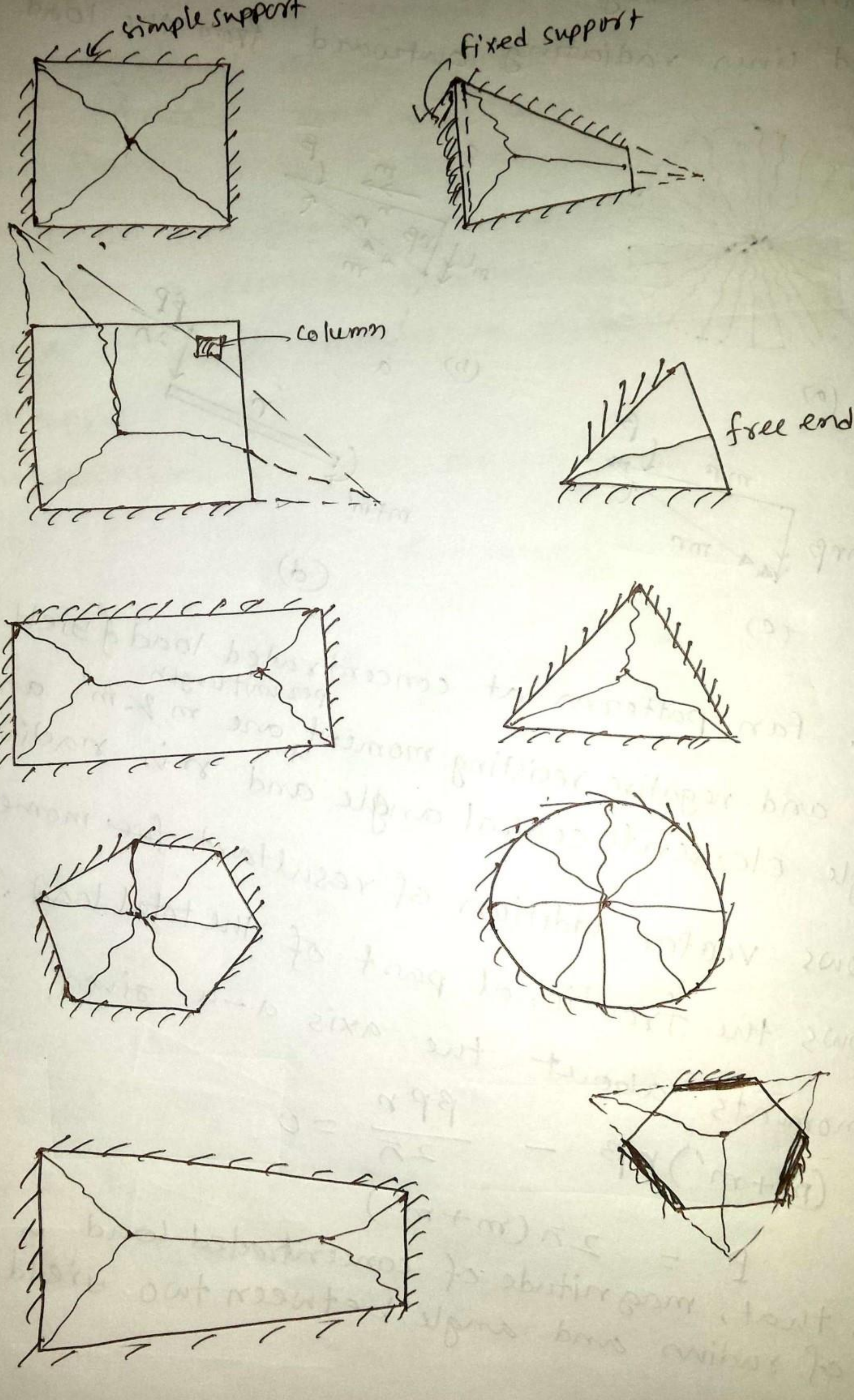
Now Taking moments about the axis a–a gives

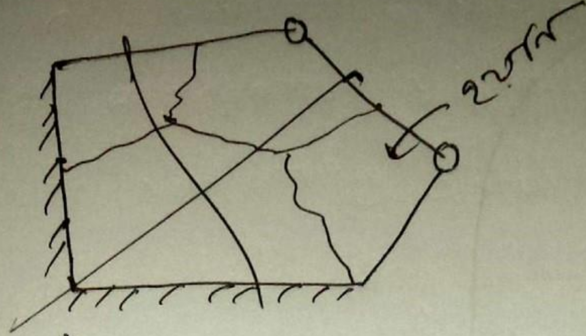
$$(m+m')r\beta - \frac{\beta P r}{2\pi} = 0$$

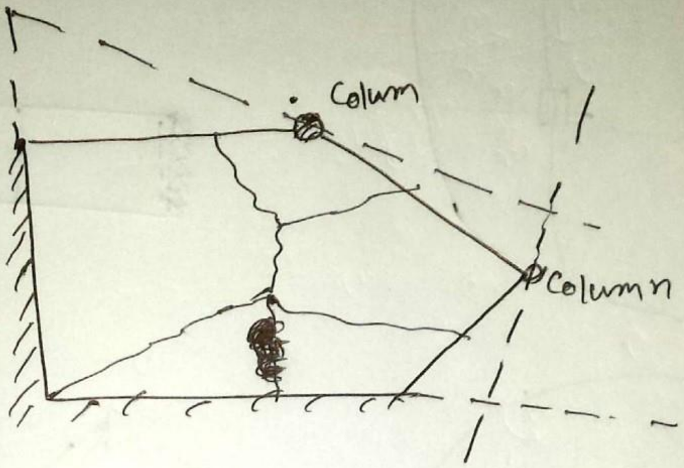
$$\therefore \quad P = 2\pi(m+m')$$

which shows that, magnitude of concentrated load is independent of radius and angle between two yield line.

Draw Yield Line Diagram



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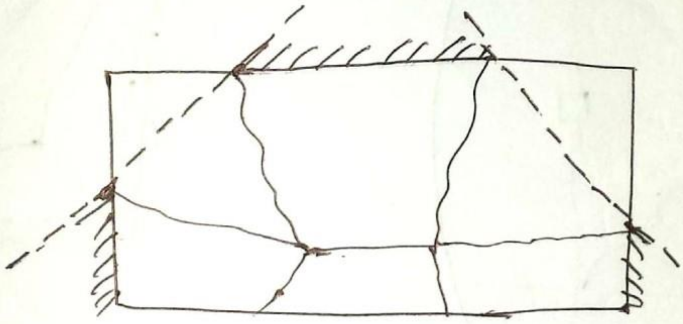

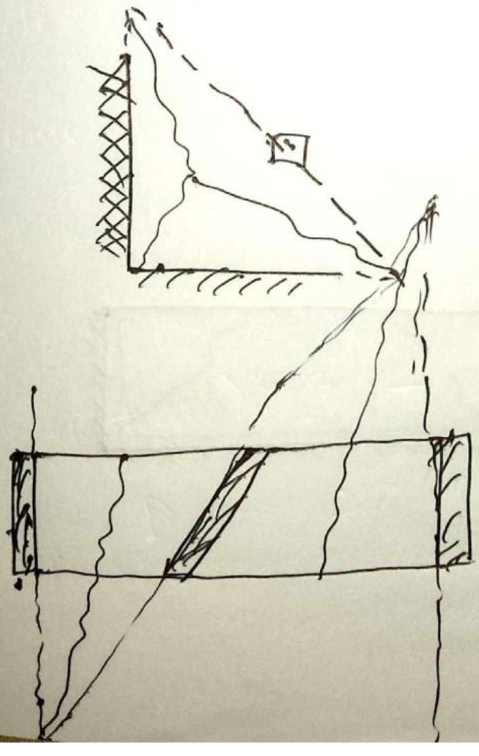

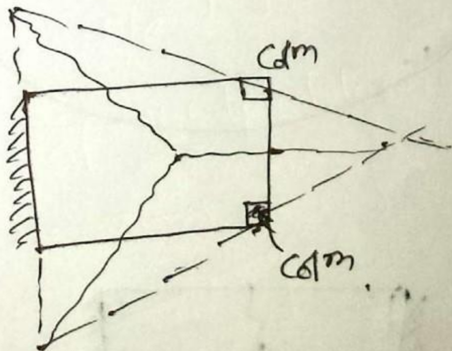
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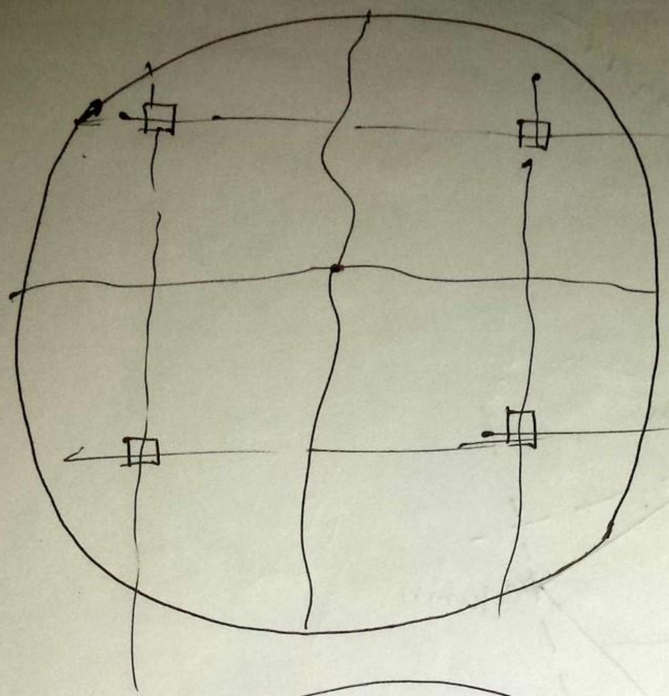
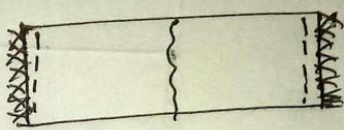
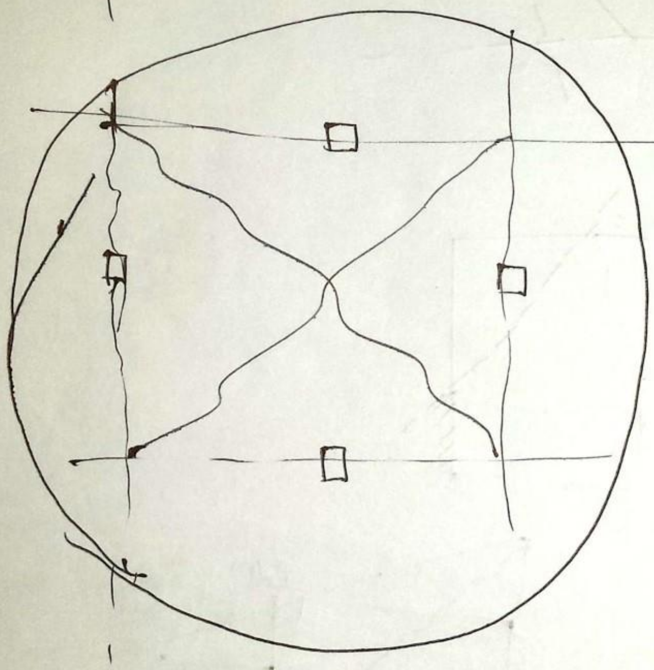
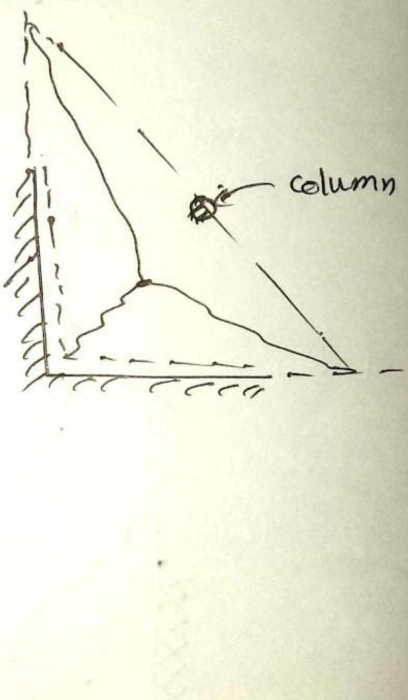
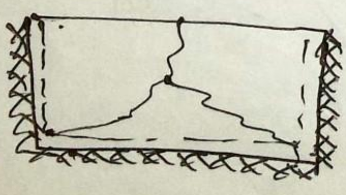
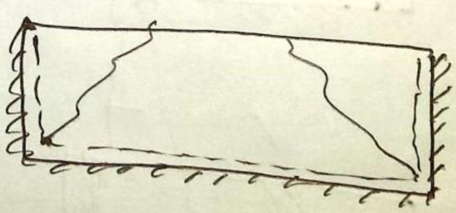







column





মো: রবিউল ইসলাম
রাজশাহী প্রকৌশল ও প্রযুক্তি বিশ্ববিদ্যালয়
পুরকৌশল বিভাগ
রোল নং: ১৬০০১০

Reinforced Concrete – II (CE-3217)

FOOTING

Q. Define Foundation. What are the essential requirements in foundation design? What are the preventing measures for transmitting settlement? 2015.

Answer: **Foundation:** The part of a structure that is usually placed below the surface of the ground and that transmits the load to the underlying soil or rock is known as Foundation/ footings/ Substructure.

Requirements:

(I) The amount of settlement of the structure shall be limited to an acceptable small value.

(II) Differential settlement of various parts of the structure shall be eliminated as far as possible.

Prevention of settlement:

(I) Transmitting the load coming from the structure to the soil stratum having sufficient strength.

(II) Distributing the load over a sufficiently large area.

(III) Application of proper compaction to the substructure before construction.

Q. Discuss the pressure development under the footing for different types of soil. 2015

Answer: **Pressure developed under footing:-**

From observation and elastic analysis, it is known that pressure distribution under footing is not always same. It depends on (I) rigidity of footing.

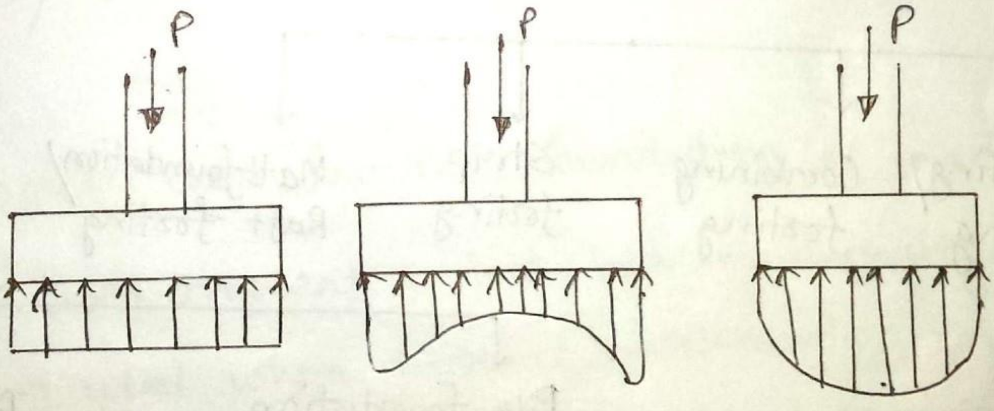
(II) Soil type.

(III) condition of soil.

The usually assumed uniform pressure distribution is shown in fig (1).

For cohesive soil pressure under edges are largest and decreased toward centre, which is shown in fig.(2).

For cohesionless soil i.e granular soil (sand) pressure under cetitre is maximum and decreased towards both edge as shown in fig.(3).



Fig(1). Fig.2 fig.3

Q. Write down the objectives of footings. 2015,

Answer: **Objectives of footings:**

(I) To distribute structural load over large area.

(II) To provide a level surface for building operation.

(III) To take the substructure deep into ground to prevent overturning.

(IV) To prevent unequal settlement.

Q. Discuss different types of footings.

Answer: **Types of footings:**

Foundation / Footings

- Shallow foundation
  - Spread footing / pad footing
  - Combining footing
  - Strip footing
  - Mat foundation / Raft footing
- Deep foundation
  - Pile foundation
  - Drilled shaft.

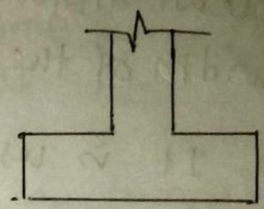
Shallow foundation :-

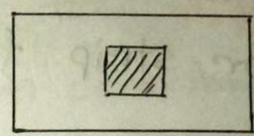
(I) Spread footing / Isolated footing :

(a) Simple type of footing.

(b) Construction cost is cheap.

(c) It is used when soil is relatively cheap or when the column load from superstructure is relatively low.

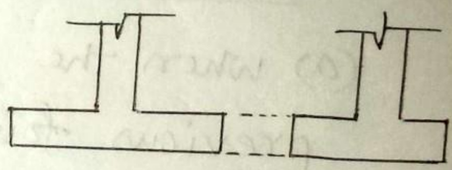




(II) Combined footing :

(a) Construction cost is higher than spread footing.

(b) It is used when the bearing capacity is relatively low or when the foundation can not be extended due to the end of property line.



(III) Strip footing :

(a) Commonly used for the foundation of load bearing wall.

(b) It is used on weak or low bearing capacity soil.

(c) It is used when distance between two consiqutive spread footing is equal to the length of the size of pad - or spread footing.

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(IV) Mat foundation/ raft footing:

(a) When the strip foundation become so wide that the clear distance between them is about the same as the width of the strip, mat foundation is used.

(b) It is useful in restricting the differential settlement.

(c) When the depth to suitable bearing capacity strata for strip footing loading becomes ~~too~~ deep.

(B) Deep foundation:

(I) Pile Foundation:

(a) When the bearing capacity with respect to other previous foundation types is low.

(b) Where soils are particularly affected by seasonal change (expansive soil)

(II) Drilled Shaft:-

Drilled shaft refers to cast in situ or cast in place pile generally having a diameter of about 2.5 ft or more without reinforcement. A single drilled shaft may be used instead of group pile.

Q. Explain different types of spread footing with neat sketch.

Answer: Explanation :-

Spread footing is mainly of two types as wall footing and coloumn footing. A wall footing is simply a structure of reinforced concrete wider than the wall.

The single column footings may be square, rectangular or circular. If in case of single column footings, if passes beyond the exterior wall, sometimes combined footing are used. Different type of footing as spread as shown in figure.

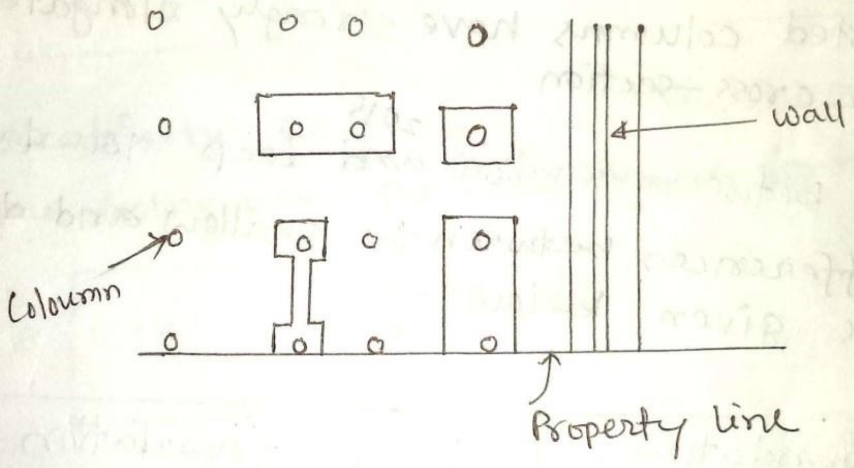


Fig. Different types of spread footing

Q. State different between square footing and rectangular footing. When you will recomment rectangular footings?

Answer: The differences between the square and rectangular footing are given next page:-

Square footing	Rectangular footing
(I) Spacing of bar is constant.	(I) Spacing of bar is variable.
(II) Less steel area is required	(II) More steel area is required
(III) Reinforcement is distributed equally and uniformly in the both direction.	(III) Reinforcement is distributed uniformly over width in longer direction.

Recommendation of Rectangular footing:-

(I) If space restriction governs the choice.

(II) If supported columns have strongly elongated, rectangular cross-section.

Q. Distinguish between shallow and deep foundation. 2015

Answer: The differences between the shallow and deep foundation are given below:-

Shallow foundation	Deep foundation
(1) The foundation is laid on soil in a shallow depth.	(1) The foundation is laid on soil in a deeper depth.
(2) Less strong foundation than deeper foundation.	(2) More strong foundation than shallow foundation.
(3) Ex, square rectangular footing.	(3) Ex:- Coloumn footing in deeper depth.

Loads, Bearing Pressures, And Footing size:

For concentrically loaded footings, the required area is determined from,

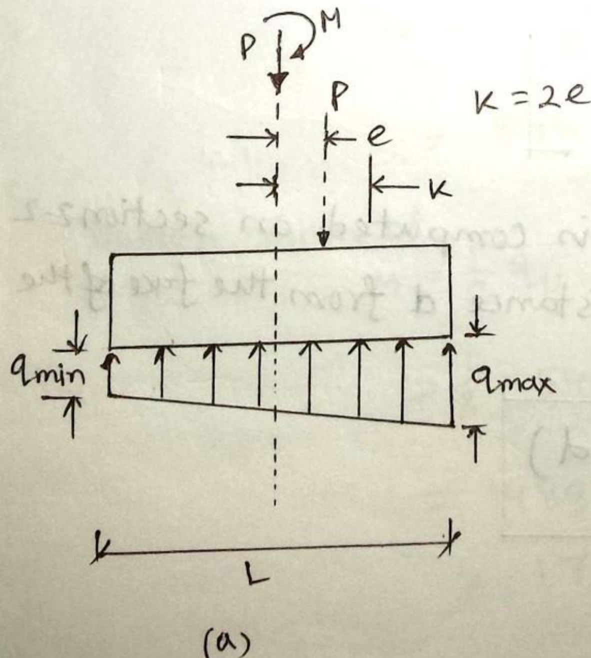
$$\boxed{A_{req} = \frac{D+L}{q_a}}$$

In addition, most codes permit a 33 percent increase in allowable pressure when the effect of wind W or earthquake E are included, in which case,

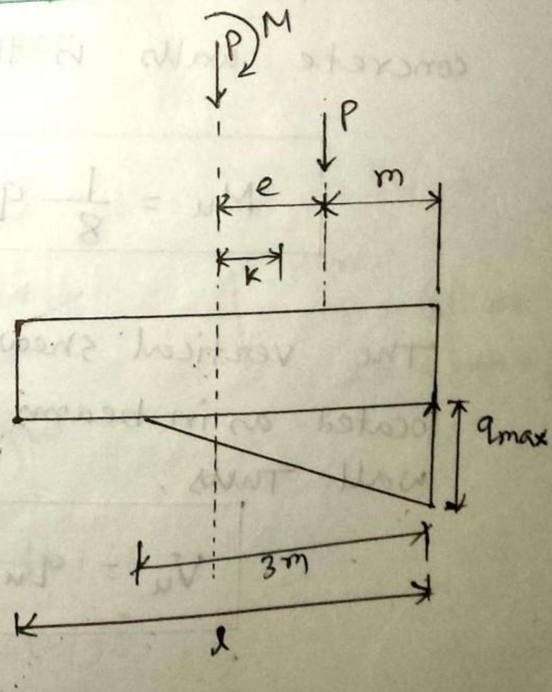
$$\boxed{A_{req} = \frac{D+L+W}{1.33\, q_a} \quad \text{or} \quad \frac{D+L+E}{1.33\, q_a}}$$

If eccentricity $e = \frac{M}{P}$ does not exceed the kern distance k of the footing area, the usual flexure formula.

$$\boxed{q_{\substack{max \\ min}} = \frac{P}{A} \pm \frac{Mc}{I}}$$



(a)

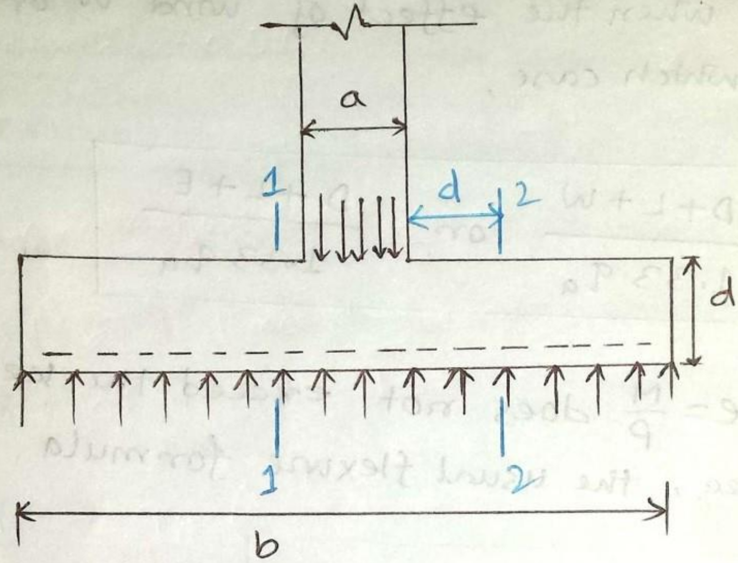


(b)

For rectangular footings of size $L \times b$, the maximum pressure can be found from.

$$\boxed{q_{max} = \frac{2P}{3bm}}$$

b = width of the foundation.



The maximum bending moment in footings under concrete walls is therefore given by,

$$\boxed{M_u = \frac{1}{8} q_u (b-a)^2}$$

The vertical shear force is computed on section 2-2, located as in beams, at a distance d from the face of the wall. Thus,

$$\boxed{V_u = q_u \left(\frac{b-a}{2} - d \right)}$$

* Design of wall footing by WSD and USD Method:-

Example 16.1:- Design of wall footing:- A 16 inch concrete wall supports a dead Load D = 14 kips/ft, and a Live Load L = 10 kips/ft. The allowable bearing pressure is $q_a = 4.5$ kips/ft^2 at the level of the bottom of the footing. which is 4 ft below grade. Design a footing for this wall using 4000 PSI concrete and Grade 60 steel.

Solution: Assume that, thickness of footing = 12 inch.

and, assume 3 ft fill on the top of the footing.

$$\therefore q_e = 4.5 \times 1000 - \left(\frac{12}{12} \times 150 + 3 \times 100\right)$$

[Assume, $\gamma_{soil} = 100$ pcf]

$$= 4050 \text{ } \cancel{\text{kips}} \text{ lb}$$

The required width of the footing,

$$B_{req} = \frac{DL + LL}{q_e} = \frac{(14 + 10) \times 1000}{4050} = 5.92 \text{ ft} \sim 6 \text{ ft}.$$

$\therefore$ A 6 ft wide footing will be assumed.

$\therefore$ Bearing pressure developed.

$$q_u = \frac{1.2DL + 1.6LL}{B_{req}}$$

$$= \frac{(1.2 \times 14 + 1.6 \times 10)\,1000}{6}$$

$$= 5467 \text{ psf.}$$

Design moment,

$$M = \frac{1}{8} q_u (B - a)^2$$

Here,
$B = 6$ ft
$a = 16$ inch $= \frac{4}{3}$ ft.

$$= \frac{1}{8} \times 5467 \left(6 - \frac{4}{3}\right)^2$$

$$= 14882.39 \text{ lb-ft/ft}$$

$$= 178590 \text{ in-lb/ft.}$$

Depth check for moment:

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

$$= \sqrt{\frac{178590}{295.3125 \times 12}}$$

$$= 7.09 < 12-3 = 9''$$

(So, depth check OK)

for,

$f_c' = 4000$ PSI

$f_y = 60000$ PSI

$\therefore f_s = 24000$ PSI

~~n = 9~~,

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

$$r = \frac{f_s}{f_c} = \frac{24000}{0.45 * 4000} = 13.33$$

$$k = \frac{n}{n+r} = \frac{8}{8+13.33} = 0.375$$

$j = 0.875$

$R = \frac{1}{2} f_c j k = 0.5 \times 0.45 \times 4000 \times 0.875 \times 0.375$

$\therefore R = 295.3125$ PSI

*** Depth check for shear:**

Shear at a distance 'd' from the face of the wall,

$$V_u = q_u \left(\frac{b-a}{2} - d \right)$$

$$= 5467 \left(\frac{16 - 1.33}{2} - \frac{9}{12} \right)$$

$$= 8665.195 \text{ lb/ft}$$

Again, $V_c = 1.1 \sqrt{f_c'}\, bd$

$V_c = 1.1 \sqrt{4000} \times 12 \times d = 834.84\, d$

Now $V_u = V_c \Rightarrow V_c = V_u$

$\therefore 834.84\, d = 8665.195$

$\therefore d = 10.38$ (Not ok)

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Steel calculation :

$$A_s = \frac{M}{f_s j d} = \frac{178590}{24000 \times 0.875 \times 9} = 0.95 \text{ in}^2/\text{ft}.$$

Using # 6 bar , $S = \frac{0.44 \times 12}{0.95} = 5.6$ inch.

6 bar @ 5.5 in (provided)

Shrinkage reinforcement, $A_{sb} = 0.0018\, bt$

$= 0.0018 \times 12 \times 12$

$= 0.2592$

Using # 3 bar @ $\frac{0.11 \times 12}{0.2592} = 5.09''$ c/c.

Bond check :

$$U_{all} = \frac{3.4\sqrt{f_c'}}{D} = \frac{3.4\sqrt{4000}}{\frac{6}{8}} = \cancel{268.88} \text{ PSI}.$$

$= 286.71$ PSI

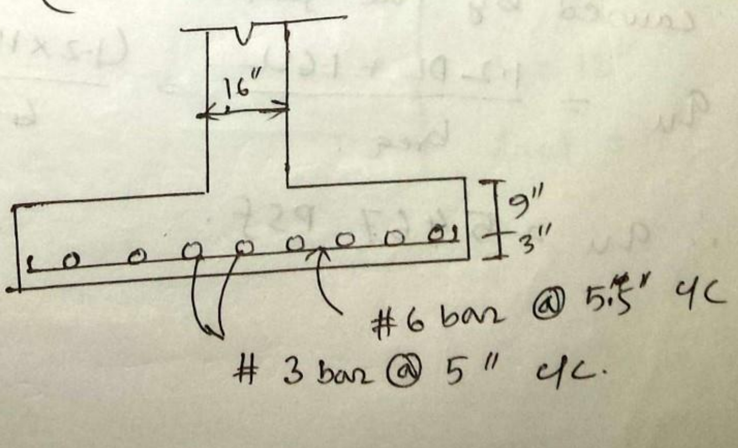
$\therefore \cancel{L_d = \frac{f_y D}{4 U_{all}} = \frac{60000 \times 6/8}{4 \times 202.38} = 55.58 \text{ inch}}$.

$$l_d = \frac{f_s D}{4u} = \frac{24000 \times 6/8}{4 \times 286.71} = 15.7''.$$

Length available :

$L = \frac{1}{2}(b - a - 2cc) = \frac{1}{2}(6 \times 12 - 16 - 2 \times 3) = 25''$

$L > l_d$ (ok)



Example 16.1: A 16" concrete wall supports a dead load D = 14 kips/ft and a live load L = 10 kips/ft. The allowable bearing pressure is $q_a = 4.5$ kips/ft^2 at the level of the bottom of the footing, which is 4 ft below grade. Design a footing for this wall using 4000 psi concrete and Grade 60 steel.

Solution: Given that,

$D.L = 14$ kips/ft, | $f_c' = 4000$ psi

$LL = 10$ kips/ft | $f_y = 60000$ psi.

$q_a = 4.5$ kips/ft^2

$\therefore$ Total load, $W_T = D.L + L.L = 24$ klf

Let, thickness of footing = 12 inch, clear cover = 3 inch.

$d_{eff} = 12 - 3 = 9$ inch.

Assume, 3 ft fill on the top of the footing.

$\therefore q_e = 4.5 \times 10^3 - (3 \times 100 + \frac{12}{12} \times 150)$ | Let, $\gamma_{soil} = 100$ lb/ft

$= 4050$ psf.

$\therefore$ Therefore require width of footing,

$$b = \frac{24000 (W_T)}{4050} = 5.93 \text{ ft} \approx 6 \text{ ft}.$$

Now the bearing pressure for strength design of the footing caused by the factored loads,

$$q_u = \frac{1.2 DL + 1.6 LL}{b_{req}} = \frac{(1.2 \times 14 + 1.6 \times 10) \times 10^3}{6}$$

$\therefore q_u = 5467$ psf.

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Design moment, $M_u = \frac{1}{8} q_u (b-a)^2$

$= \frac{1}{8} \times 5467 \left(6 - \frac{16}{12}\right)^2$ lbft/ft.

$= 14882.38 \times 12$ in lb/ft.

$= 178590$ in lb/ft.

Shear at a distance $d = 9''$, from the face of the wall.

$$V_u = q_u \left(\frac{b-a}{2} - d \right)$$

$$= 5467 \left(\frac{6 - \frac{16}{12}}{2} - \frac{9}{12} \right)$$

$= 8656$ lb/ft.

Depth check:

$$d = \sqrt{\frac{M_u}{\phi R b}}$$

$$= \sqrt{\frac{178590}{0.9 \times 1041 \times 12}}$$

$= 3.98''$

Since,

$d_{act} > d_{rea}$

(ok)

Here,

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

$$\rho = 0.75 \times 0.85 \frac{f_c'}{f_y} \beta_1 \times \frac{87000}{87000 + f_y}$$

$$= 0.75 \times 0.85 \times \frac{4000}{60000} \times 0.85 \times \frac{87000}{87000 + 60000}$$

$= 0.0214$

$$\therefore R = 0.0214 \times 60000 \left(1 - 0.59 \times \frac{0.0214 \times 60000}{4000}\right)$$

$\therefore R = 1041$ psi.

$b = 12''$

$d_{act} = 12'' - 3'' = 9''$

* **Depth check for shear:**

shear at a distance $d = 9''$ from the face of the wall,

$V_u = 8565$ lb/ft.

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

$V_u = 2 \times 0.75\sqrt{4000} \times 12\, d$

$\Rightarrow 8565 = 2 \times 0.75 \sqrt{4000} \times 12\, d$

$\therefore d = 7.52'' < 9''$

$\therefore$ depth check is (ok).

Reinforcement calculation:

$$A_s = \frac{M_u}{\phi f_y (d - \frac{a}{2})}$$

$$A_s = \frac{178590}{0.9 \times 60000(9 - 0.74 A_s)}$$

$\therefore 0.22 A_s^2 - 2.718 A_s + 1 = 0$

$\therefore A_s = 0.4 \text{ in}^2$

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

$$a = \frac{A_s \times 60000}{0.85 \times 4000 \times 12}$$

$\therefore a = 1.47 A_s$

$\therefore \frac{a}{2} = 0.74 A_s$

According to ACI code,

$$A_{s\,min} = \frac{3\sqrt{f_c'}\, bd}{f_y} \geq \frac{200 bd}{f_y}$$

$$= \frac{3\sqrt{4000} \times 12 \times 9}{60000} \geq \frac{200 \times 12 \times 9}{60000}$$

$= 0.34 < 0.36$

$\therefore A_s = 0.4 \text{ in}^2/\text{ft}$ (taken) use # 6 bar @ $\frac{0.44 \times 12}{0.4} = 13.25''$ c/c.

Distribution reinforcement:

$A_{st} = 0.0018\,bt = 0.0018 \times 12 \times 12$

$\therefore A_{st} = 0.2592\ in^2$

use # 3 bar, @ $\frac{0.11 \times 12}{0.2592} = 5$ in c/c.

Development length:

$V_{all} = \frac{4.8\sqrt{f_c'}}{D} = \frac{4.8 \times \sqrt{4000}}{6/8} = \cancel{227.68\ psi}\ 404.77$

$\therefore L_d = \frac{f_y D}{4 V_{all}} = \frac{60000 \times 6/8}{4 \times 404.77} = 27.8''$

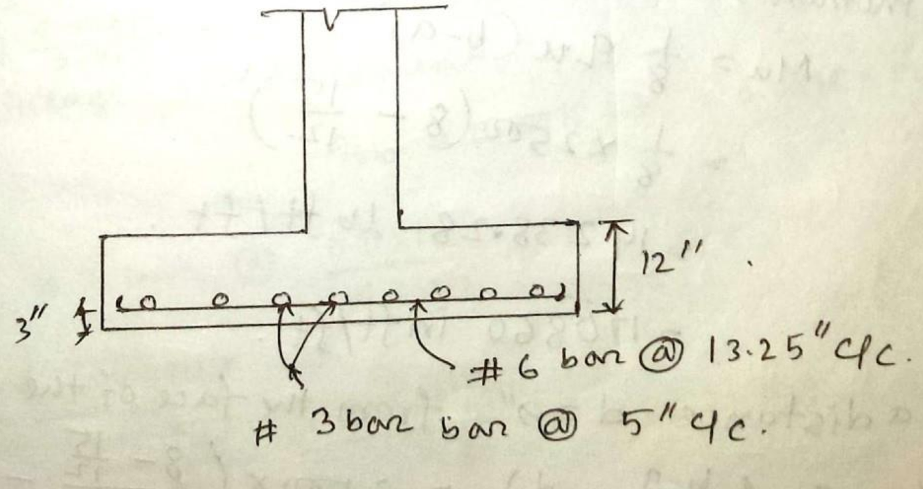
$L_d = \frac{0.04 f_y D}{\sqrt{f_c'}} = \frac{0.04 \times 60000 \times 6/8}{\sqrt{3000}} = 32.86''$

Available length,

$L = \frac{1}{2}(B - a - 2 \times c)$

$= \frac{1}{2}(6 \times 12 - 16 - 2 \times 3)$

$= 25.$



2015 4(c): Design a reinforced concrete footing for a masonry wall 15" thick carrying a total load of 20 k/ft. The safe bearing capacity of soil is 3000 psf. Assume, $f_c' = 4000$ psi and $f_s = 24000$ psi.

Soln

Given,

$W_T = 20$ k/ft, $f_c' = 4000$ psi, $f_s = 24000$ psi.

Let, thickness of footing = 12 inch, clear cover = 3"

$\therefore d_{eff} = 12 - 3 = 9"$

Assume, 3 ft fill on the top of the footing.

$q_e = 3000 - (3 \times 100 + \frac{12}{12} \times 150)$ | Let, $\gamma_{soil} = 100$ lb/ft^3

$= 2550$ psf

Therefore required width of footing,

$b = \frac{W_T}{q_e} = \frac{20 \times 1000}{2550} = 7.84 \sim 8$ ft.

Now the bearing pressure developed,

$q_u = \frac{W_T}{b_{req}} = \frac{20 \times 10^3}{8} = 2500$ psf.

Design moment,

$M_u = \frac{1}{8} q_u (b-a)^2$

$= \frac{1}{8} \times 2500 \left(8 - \frac{15}{12}\right)^2$

$= 14238.28$ lb ft/ft.

$= 170860$ in ft/ft.

Shear at a distance d = 9" from the face of the wall,

$V_u = q_u \left(\frac{b-a}{2} - d\right) = 2500 \times \left(\frac{8 - \frac{15}{12}}{2} - \frac{9}{12}\right)$

$\therefore V_u = 6562.5$ lb/ft.

Depth check for moment:

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

$$= \sqrt{\frac{170860}{295.3125 \times 12}}$$

$$= 6.94''$$

dact > d. (ok)

for,

$f_c' = 4000$ PSI.

$f_c = 0.45 f_c' = 1800$ PSI

$f_s = 24000$ PSI.

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

$$r = \frac{f_s}{f_c} = \frac{24000}{1800} = 13.33$$

$$k = \frac{n}{n+r} = \frac{8}{8+13.33} = 0.375$$

$j = 0.875$

$R = \frac{1}{2} f_c j k = 0.5 \times 0.45 \times 4000 \times 0.875 \times 0.375$

$\therefore R = 295.3125$ PSI

Depth check for shear:

$V_u = 6562.5$ lb/ft.

$V_c = 1.1\sqrt{f_c'}\, bd = 1.1\sqrt{4000} \times 12 \times d$

$\therefore 6562.5 = 834.84\, d$

$\therefore d = 7.86'' <$ dact.

(ok)

Reinforcement calculation:

$$A_s = \frac{M}{f_s j d} = \frac{170860}{24000 \times 0.875 \times 9} = 0.91\ in^2$$

Use #6 bar @ $\frac{0.44 \times 12}{0.91} = 5.8 \sim 5.75''$ c/c.

Shrinkage reinforcement,

$A_{st} = 0.0018\, bt = 0.0018 \times 12 \times 12 = 0.2592\ in^2.$

using #3 bar @ $\frac{0.11 \times 12}{0.2592} = 5''$ c/c.

Development length:

$$u = \frac{3.4\sqrt{f_c'}}{D} = \frac{3.4\sqrt{4000}}{6/8} = 286.71 \text{ PSI}.$$

$$l_d = \frac{f_s D}{4u} = \frac{24000 \times 6/8}{4 \times 286.71} = 15.7'' \sim 16''$$

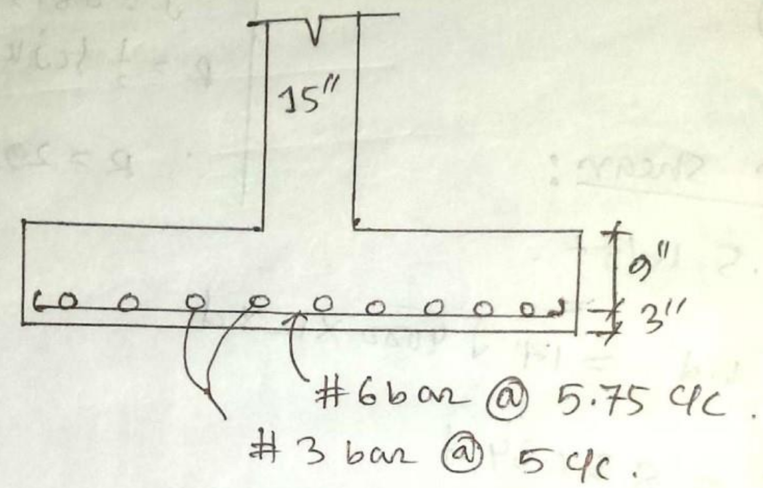
Length available,

$$L = \frac{1}{2}(b - a - 2c)$$

$$= \frac{1}{2}(8\times12 - 15 - 2\times3)$$

$$= 37.5$$

$L > l_d$ (ok)



Q. 2013 7(b) Design a reinforced concrete footing for a masonry wall 10 inch thick carrying a load of 12 kips/foot. The safe bearing capacity of soil is 3000 PSI. Assume. $f_c' = 4000$ PSI, $f_y = 50000$ PSI, Use WSD method.

Soln Given that,

$W_T = 12$ klf. $f_c' = 4000$ PSI $\therefore f_c = 0.45 f_c' = 1800$ PSI

$f_y = 50000$ PSI. $\therefore f_s = 0.4 f_y = 20000$ PSI

Let, the thickness of footing $= 12''$, clear cover $= 3''$.

$$\therefore d_{eff} = 12'' - 3'' = 9''$$

Assume 3 ft fill on the top of the footing.

$$\therefore q_e = 3000 - \left(3\times100 + \frac{12}{12}\times150\right)$$ | let, $\gamma_{soil} = 100$ lb/ft^3

$$= 2575 \text{ psf.}$$

Therefore, required width of footing,

$$b = \frac{W_T}{q_e} = \frac{12 \times 1000}{2575} = 4.66 \sim 5 \text{ ft}.$$

Now, the bearing pressure developed,

$$q_u = \frac{W_T}{Area} = \frac{12 \times 10^3}{5} = 2400 \text{ psf}.$$

Design moment,

$$M_u = \frac{1}{8} q_u (b-a)^2 = \frac{1}{8} \times 2400 \left(5 - \frac{10}{12}\right)^2$$

$\therefore M_u = 5208.33$ lbft/ft.

$\therefore M_u = 62500$ inch lb/ft.

Shear at a distance $d = 9''$ from the face of the wall.

$$V_u = q_u\left(\frac{b-a}{2} - d\right) = 2400\left(\frac{5 - \frac{10}{12}}{2} - \frac{9}{12}\right)$$

$V_u = 3200$ Plf.

Depth check for moment:

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

$$= \sqrt{\frac{62500}{317.34 \times 12}}$$

$= 4.05''$

$d_{act} = 9'' > d = 4.05''$

(ok)

$$r = \frac{f_s}{f_c} = \frac{20000}{1800} = 11.11$$

$$n = \frac{E_s}{E_c} = \frac{29 \times 10^6}{57000\sqrt{4000}}$$

$\therefore n = 8.$

$$\therefore k = \frac{n}{n+r} = \frac{8}{8+11.11} =$$

$k = 0.41$

$\therefore j = 0.86$

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

$= 0.5 \times 1800 \times 0.86 \times 0.41$

$= 317.34$

$b = 12''$

Depth check for shear:

$V_u = 3200$ lb/ft.

$V_c = 1.1\sqrt{f_c'}\, bd$

$3200 = 1.1\sqrt{4000} \times 12 \times d$

$\therefore d = 3.83''$

$d_{act} > d$ (ok).

Reinforcement calculation:

$A_s = \dfrac{M}{f_s j d} = \dfrac{62500}{20000 \times 0.86 \times 9} = 0.4\ in^2$

use # 6 bar @ $\dfrac{0.44 \times 12}{0.4} = 13''$ c/c.

$A_{smin} = 0.002\, bt = 0.002 \times 12 \times 12 = 0.288\ in^2$.

use # 3 bar @ $\dfrac{0.11 \times 12}{0.288} = 4.5''$ c/c.

Development length:

$\therefore L_d = \dfrac{f_s D}{4u} = \dfrac{20000 \times 6/8}{4 \times 404.77}$

$u = \dfrac{4.8\sqrt{f_c'}}{D} = \dfrac{4.8\sqrt{4000}}{6/8}$

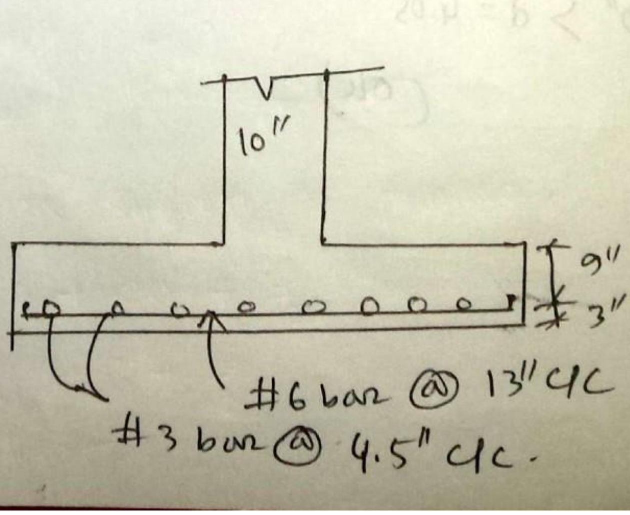
$\therefore u = 404.77$ psi.

$L_d = 9.26 \sim 9.50''$

length available:

$L = \frac{1}{2}(b - a - 2cc) = \frac{1}{2}(5 \times 12 - 10 - 2 \times 3) = 22''$

$L > L_d$ (ok)



2012 3(c) Design a reinforced concrete wall footing to support a 10" thick brick wall carrying a 15 kip/ft dead load and a 10 kip/ft LL. The base of the footing is 4ft below the grade. Use $f_c' = 3000$ PSI and $f_y = 60000$ PSI.

Soln Given that,

$DL = 15$ KLf
$LL = 10$ KLf.

$f_c' = 3000$ PSI
$f_c = 0.45 \times f_c' = \cancel{1800}$ 1350 PSI
$f_y = 60000$ PSI.

Total load, $W_T = 15 + 10 = 25$ pLf.

Let, thickness of footing = 12 inch, clear cover = 3 inch.

$d_{eff} = 12 - 3 = 9''$.

Since 4ft below the base hence 3ft fill on the top of the footing. Assume [$q_a = 5$ kips/ft^2.]

Let, $\gamma_{soil} = 100$ lb/ft^3.

$\therefore q_e = 5 \times 10^3 - \left(3 \times 100 + \frac{12}{12} \times 150\right)$

$\therefore q_e = 4550$ PSf.

Therefore required width of footing,

$b = \frac{W_T}{q_e} = \frac{25 \times 10^3}{4550} = \cancel{6.17}$ $5.5' \sim 6'$

$\therefore q_u = \frac{1.2\ DL + 1.6\ LL}{b_{req}} = \frac{(1.2 \times 15 + 1.6 \times 10) \times 10^3}{6 \times 1}$

$\therefore q_u = 5667$ PSf.

Design moment,

$M_u = \frac{1}{8} q_u (b-a)^2 = \frac{1}{8} \times 5667 \left(5 - \frac{10}{12}\right)^2$

$\therefore M_u = \cancel{36599.4 \text{ ftlb/ft}}$ 18909.67 ft lb/ft

$\therefore M_u = \cancel{439192.5 \text{ in lb/ft}} = 226916.125$ inch lb/ft

Shear at distance $d = 9''$ from the face of the wall

$$V_u = q_u\left(\frac{b-9}{2} - d\right)$$

$$= 5667\left(\frac{6-\frac{10}{12}}{2} - \frac{9}{12}\right)$$

$$= 10389.5 \text{ lb/ft}.$$

depth check for moment:

$$d = \sqrt{\frac{M}{\phi R b}}$$

$$= \sqrt{\frac{226916.125}{0.9 \times 778.35 \times 12}}$$

$$= 5.19''$$

$d_{act} > d$ (ok)

shear check

$$\rho = 0.75 \times 0.85 \beta_1 \frac{f_c'}{f_y} \times \frac{\epsilon_u}{\epsilon_u + \epsilon_y}$$

$$= 0.75 \times 0.85 \times 0.85 \times \frac{3}{60} \times \frac{0.003}{0.003 + \frac{60}{29000}}$$

$$= 0.016$$

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

$$= 0.016 \times 60000 \left(1 - 0.59 \times \frac{0.016 \times 60}{3}\right)$$

$$= 778.35 \text{ PSI}$$

$b = 12''$

Depth check for shear:

$$V_u = \phi V_c = 2\phi \sqrt{f_c'}\, b d$$

$$10389.5 = 2 \times 0.75 \times \sqrt{3000} \times 12 \times d$$

$$d = 10.5''$$

$d_{act} = 9'' < d = 10.5''$ (not ok)

$V_u = 10389.5$ lb

$b = 12''$

~~$f_c' = 4000$~~

$f_c' = 3000$ psi

$\therefore$ Actual thickness should be provided,

$$t = d_{req} + cc + \frac{\phi}{2}$$

$$= 10.5 + 3 + \frac{6}{8}$$

$$= 14.25''$$

Reinforcement calculation:

$$A_s = \frac{M}{\phi f_y (d - \frac{a}{2})}, \qquad a = \frac{A_s f_y}{0.85 f'_c b}$$

$$= \frac{A_s \times 60}{0.85 \times 3 \times 12}$$

$$a = 1.96 A_s$$

$$\therefore \frac{a}{2} = 0.98 A_s.$$

$$A_s = \frac{43919.25}{0.9 \times 60000 (10.5 - 0.98 A_s)}$$

$$\cancel{1.204 A_s^2 - 12.9 A_s + 1 = 0}$$

$$\therefore \cancel{A_s} =$$

Shear check:

$$V_{ud} = q_{ud} \times \left(\frac{b-a}{2} - \frac{d}{12} \right)$$

$$= 5667 \times \left(\frac{6 - \frac{10}{12}}{2} - \frac{d}{12} \right)$$

$$= 14639.75 - 472.25\, d$$

again, $v_c = 2\phi \sqrt{f'_c}\, bd$

$$v_c = 2 \times 0.75 \sqrt{3000} \times 12 \times d$$

$$v_c = 985.9\, d.$$

$v_c = v_u$

$$\therefore 9859\, d = 14639.75 - 472.25 d.$$

$$\therefore d = 10.03'' \sim 10''$$

ওপরের Moment এর জন্য depth check : আবার Reinforced calculation

Reinforcement calculation:

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

$$A_s = \frac{226916.125}{0.9 \times 60000 \left(10 - \frac{1.96 A_s}{2}\right)}$$

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

$$= \frac{A_s \times 60}{0.85 \times 3 \times 12}$$

$$= 1.96 A_s$$

$\therefore 0.23 A_s^2 - 2.38 A_s + 1 = 0$

$\therefore A_s = 0.44 \text{ in}^2$

Use # 6 bar @ $\frac{0.44 \times 12}{0.44} = 12''$ c/c.

Use # 6 bar @ 12" c/c.

$A_{smin} = 0.0020\, bt = 0.0020 \times 12 \times 13.75$

$= 0.33 \text{ in}^2$

$t = d + cc + \frac{\phi}{2}$

$= 10 + 3 + \frac{6}{8}$

$= 13.75$

Using # 5 bar @ $\frac{0.31 \times 12}{0.33} = 11.25''$ c/c.

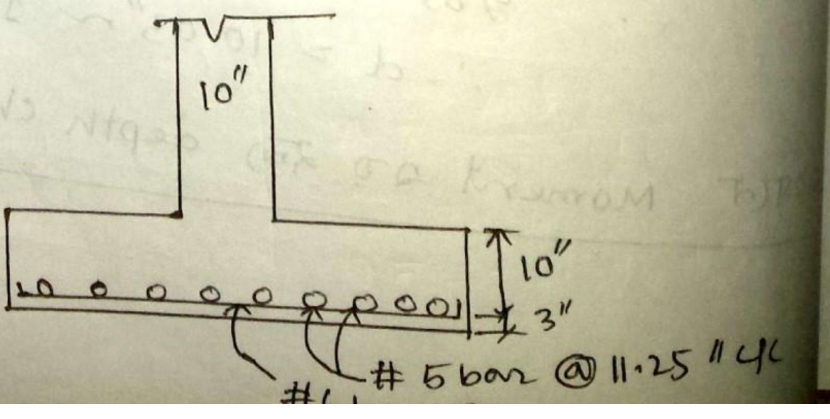
Development length:

$L_d = 0.04 \frac{A_s f_y}{\sqrt{f'_c}} = 0.04 \times \frac{0.42 \times 60000}{\sqrt{3000}} = 18.4''$

Length available, $L = \frac{1}{2}\{b - a - 2cc) = \frac{1}{2}(6 \times 12 - 10 - 2 \times 3)$

$\therefore L = 28''$

$L > L_d$ (OK)



মো: রবিউল ইসলাম CE'130110 RUET.

Square Footing:

(I) Area $= \dfrac{DL + LL}{q_a - \gamma D}$, γ = unit weight of soil $= 125\ lb/ft^3$

D = depth of footing [না দেওয়া থাকলে 5' ধরতে হবে]

q_a = Allowable bearing pressure.

(II) $q_{ud} = \dfrac{1.2DL + 1.6 \times LL}{Area}$

(III) **Punching shear check:**

V_{ud} = Area of footing $\times q_{ud}$ − Area of column perimeter

$V_c = 4\phi\sqrt{f_c'}\, b_o d$ [$\phi = 0.75$].

$V_{ud} = V_c \Rightarrow d = ?$

(IV) **Beam shear check:**

$$V_{ud} = \left(\frac{L}{2} - \frac{a}{2\times 12} - \frac{d}{12}\right) \times L \times q_{ud}$$

$V_c = 2\phi\sqrt{f_c'}\, bd$ [$\phi = 0.75$]

$V_c > V_u$ (ok)

(V) **Moment check:**

$M_u = \dfrac{WL^2}{2}$,

$A_s = \dfrac{M_u}{\phi f_y (d - \frac{a}{2})}$

$A_{s\,min} = \dfrac{200}{f_y} bd$

$\phi = 0.9$

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

$h = d + 1.5 d_b + 3''$

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Example 16.2: A column 18 inch square with $f_c' = 4$ KSI reinforced with 8#8 bar of $f_y = 60$ KSI. supports a dead load of 225 kips and a live load of 175 kips. The allowable soil pressure q_a is 5 kips/ft^2. Design a square footing with base 5 ft below grade, using $f_c' = 4$ KSI, and $f_y = 60$ KSI.

Solution: Given that,

$DL = 225$ kips, $LL = 175$ kips.

$q_a = 5$ k/ft^2, $D = 5'$

footing size $= 18'' \times 18''$, $f_c' = 4$ KSI, $f_y = 60$ KSI

$$\text{Area required for footing} = \frac{DL + LL}{q_a - \gamma D}$$

$$= \frac{225 + 175}{5 - \frac{125}{1000} \times 5}$$

$$= 91.43 \text{ ft}^2$$

Now, Area $= 91.43$ ft^2

$L \times L = 91.43$.

$\therefore L = 9.56' \sim 9.5'$

Area $= 9.5^2 = 90.25$ ft^2.

Ultimate pressure developed,

$$q_{ud} = \frac{1.2DL + 1.6LL}{\text{Area}} = \frac{1.2 \times 225 + 1.6 \times 175}{90.25}$$

$\therefore q_{ud} = 6.1$ ksf.

Punching shear check:

V_{ud} = Area of footing $\times q_{ud}$ − Area of column $\times q_{ud}$

$$= 90.25 \times 6.10 - \left(\frac{18+d}{12}\right)^2 \times 6.10$$

$$= 550.5 - (18+d)^2 \times 0.0424$$

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Again, $V_c = 4\phi\sqrt{f_c'}\ b_o d$ | b_o = perimeter

$= \frac{4\times 0.75\sqrt{4000}\times(18+d)4\times d}{1000}$

$= 0.759\,(18+d)\,d$

At equilibrium,

$V_{ud} = V_c$

$\Rightarrow 550.5 - (324 + 36d + d^2)\times 0.0424 = 0.759(18+d)d$

$\Rightarrow 550.5 - 13.74 - 1.5264d - 0.0424d^2 = 13.66d + 0.759d^2$

$\Rightarrow 0.8014d^2 + 15.1864d - 536.76 = 0$

$\therefore d = 18.08 \sim 19''$

Beam shear check:

$V_{ud} = \left(\frac{L}{2} - \frac{a}{2\times12} - \frac{d}{12}\right)\times L\times q_{ud}$

$= \left(\frac{9.5}{2} - \frac{18}{2\times12} - \frac{19}{12}\right)\times 9.5\times 6.1$

$= 140.05\ k$

$\therefore V_c = 2\phi\sqrt{f_c'}\ bd$ | $b = 9.5' = 9.5\times12$

$= \frac{2\times0.75\times\sqrt{4000}\times9.5\times12\times19}{1000}$

$= 205.48^k > 140.05^k$

$\therefore V_c > V_{ud}$ (ok).

(3) Moment check:

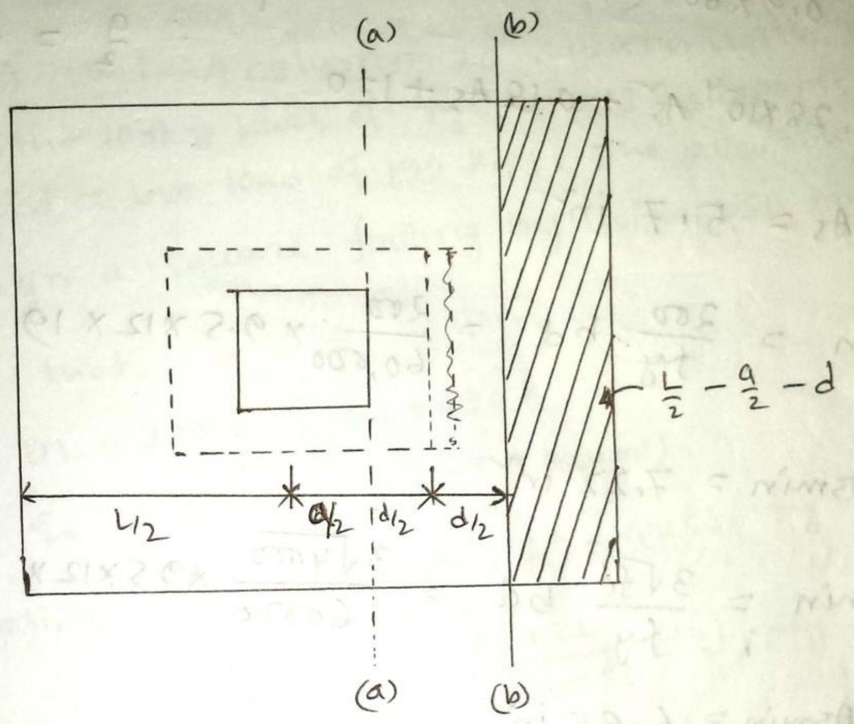
Moment along line a-a by the side of column.

$$M = \frac{1 \times 9.5 \times 6.10 \left(\frac{9.5}{2} - \frac{18}{2 \times 12} \right)^2}{2}$$

$\frac{wl^2}{2}$, $w = bq_{ud}$

$L/2$ $a/2$

$M = 463.6$ kip ft.



$$\therefore d = \sqrt{\frac{M}{\phi R b}}$$

$$= \sqrt{\frac{463.6 \times 1000 \times 12}{0.9 \times 0.908 \times 12 \times 9.5 \times 1000}}$$

$= 7.73'' < 19''$ (OK)

$$\rho = 0.85 \beta_1 \frac{f_c'}{f_y} \frac{\epsilon_u}{\epsilon_u + \epsilon_t}$$

$$= 0.85 \times 0.85 \times \frac{4}{60} \times \frac{0.003}{0.003 + 0.005}$$

$= 0.018$

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

$$= 0.018 \times 60 \left(1 - 0.59 \times \frac{0.018 \times 60}{4}\right)$$

$= 0.908$

$b = 9.5' = 9.5 \times 12''$

Reinforcement calculation :

$$A_s = \frac{M_u}{\phi f_y (d - \frac{a}{2})}$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{A_s \times 60}{0.85 \times 4 \times 9.5 \times 12}$$

$$a = 0.15 A_s.$$

$$\therefore \frac{a}{2} = 0.075 A_s$$

$$A_s = \frac{463.6 \times 12}{0.9 \times 60 \times (19 - 0.075 A_s)}$$

or, $7.28 \times 10^{-4} A_s^2 - 0.18 A_s + 1 = 0$

$\therefore A_s = 5.7 \text{ in}^2$

$$A_{s\,min} = \frac{200}{f_y} bd = \frac{200}{60,000} \times 9.5 \times 12 \times 19$$

$\therefore A_{s\,min} = 7.22 \text{ in}^2$

$$A_{s\,min} = \frac{3\sqrt{f'_c}}{f_y} bd = \frac{3\sqrt{4000}}{60000} \times 9.5 \times 12 \times 19$$

$\therefore A_{s\,min} = 6.85 \text{ in}^2$

$\therefore A_s = A_{s\,min} = 7.22 \text{ in}^2$

USE, 12 #7 bar

Bond check :

$$L_d = 0.04 \frac{A_s f_y}{\sqrt{f'_c}} = 0.04 \times \frac{0.6 \times 60000}{\sqrt{4000}} = 22.77''$$

Allowable, $L_d = \frac{L}{2} - \frac{a}{2} - cc = \frac{9.5 \times 12}{2} - \frac{18}{2} - 3 = 45'' > 27.$ (ok)

$$h = d + 1.5 d_b + 3'' = 19 + 1.5 \times \frac{7}{8} + 3 = 23.31'' \sim 24''$$

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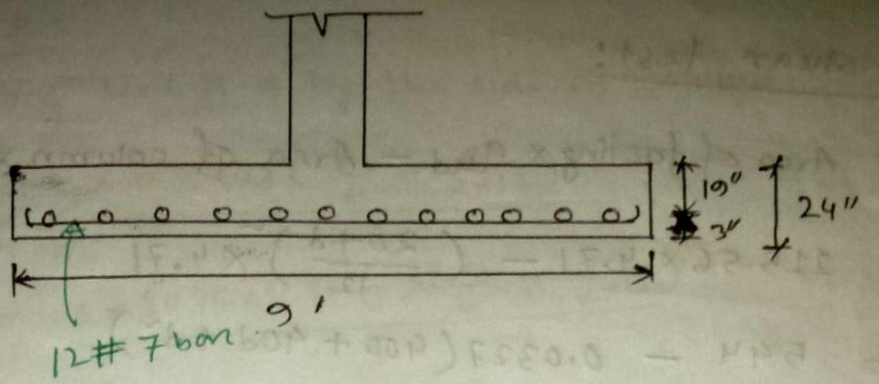

Problem : (2015) 7(b) : A column 20" square, with $f_c' = 4$ KSI reinforced with 10 # 8 bars of $f_y = 50$ KSI, supports a dead load of 200 kips and a live load of 190 kips. The allowable soil pressure is 4 ksf. Design a square footing by using USD method.

Soln Given that,

$DL = 200$ K , $LL = 190$ K

$q_a = 4$ ksf, $D = 5'$ [Assume]

footing size $= 20'' \times 20''$, $f_c' = 4$ KSI, $f_y = 50$ KSI.

Let, $\gamma_{soil} = \gamma = 125$ lb/ft³

$$\text{Area required for footing} = \frac{DL+LL}{q_a - \gamma D}$$

$$= \frac{200+190}{4 - \frac{125}{1000} \times 5}$$

$$= 115.55 \text{ ft}^2$$

Now, Area $= 115.55$

$L \times L = 115.55$

$\therefore L = 10.75'$

$\therefore$ Area $= 10.75^2 = 115.56$ ft²

Ultimate pressure developed.

$$q_{ud} = \frac{1.2DL + 1.6LL}{\text{Area}} = \frac{1.2 \times 200 + 1.6 \times 190}{115.56}$$

$\therefore q_{ud} = 4.71$ ksf

Punching shear test:

V_{ud} = Area of footing × q_{ud} − Area of column × q_{ud}.

$$= 115.56 \times 4.71 - \left(\frac{20+d}{12}\right)^2 \times 4.71$$

$$= 544 - 0.0327(400 + 40d + d^2)$$

$$= 544 - 13.08 - 1.308d - 0.0327d^2$$

$$= 530.92 - 1.308d - 0.0327d^2$$

Again, $V_c = 4\phi\sqrt{f'_c}\, b_o d$ | b_o = perimeter.

$$= \frac{4 \times 0.75\sqrt{4000} \times 4(20+d) \times d}{1000}$$

$$= 0.76(20+d)d$$

At equilibrium,

$$0.76(20+d)d = 530.92 - 1.308d - 0.0327d^2$$

$$\Rightarrow 15.18 + 0.76d^2 = 530.92 - 1.308d - 0.0327d^2$$

$$\therefore 0.7927d^2 + 1.308d - 515.74 = 0$$

$$\therefore d = 24.69 \sim 25''$$

Beam shear check:

$$V_{ud} = \left(\frac{L}{2} - \frac{a}{2\times12} - \frac{d}{12}\right) \times L \times q_{ud}$$

$$= \left(\frac{10.75}{2} - \frac{20}{2\times12} - \frac{25}{12}\right) \times 10.75 \times 4.71$$

$$= 124.47\ k$$

$$\therefore V_c = 2\phi\sqrt{f'_c}\, bd = \frac{2\times0.75\sqrt{4000} \times 10.75 \times 12 \times 25}{1000}$$ | $b = 10.75' = 10.75\times12$

$\therefore V_c = 305.95^k$, $V_c > V_{ud}$ (Ok)

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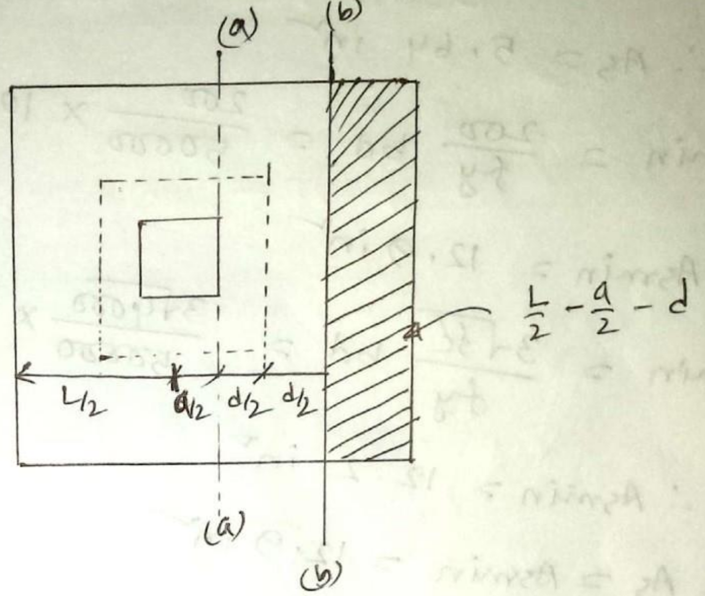
Moment check:

Moment along line a-a by the side of column.

$$M = \frac{1 \times b \times q_{ud} \left(\frac{b}{2} - \frac{a}{2 \times 12} \right)^2}{2}$$

$$= \frac{1 \times 10.75 \times 4.71 \left(\frac{10.75}{2} - \frac{20}{2 \times 12} \right)^2}{2}$$

$= 522.19$ k-ft.



$$d = \sqrt{\frac{M}{\phi R b}}$$

$$= \sqrt{\frac{522.19 \times 12}{0.9 \times 0.9215 \times 10.75 \times 12}}$$

$= 7.26'' < 25''$. (ok)

$$\rho = 0.85\, \beta_1 \frac{f_c'}{f_y} \frac{\epsilon_u}{\epsilon_u + \epsilon_t}$$

$$= 0.85 \times 0.85 \times \frac{4}{50} \frac{0.003}{0.003 + 0.005}$$

$= 0.021675$

$= 0.022$

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

$$= 0.022 \times 50 \left(1 - 0.59 \times \frac{0.022 \times 50}{4}\right)$$

$= 0.9215$

$b = 10.75' = 10.75 \times 12$ inch.

Reinforcement calculation:

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

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{A_s \times 50}{0.85 \times 4 \times 10.75 \times 12} = 0.11 A_s$$

$$A_s = \frac{522.19 \times 12}{0.9 \times 50 \left(25 - \frac{0.11 A_s}{2}\right)}$$

$$\Rightarrow 3.95 \times 10^{-4} A_s^2 - 0.1795 A_s + 1 = 0$$

$$\therefore A_s = 5.64 \text{ in}^2$$

$$A_{s\,min} = \frac{200}{f_y} bd = \frac{200}{50000} \times 10.75 \times 12 \times 25$$

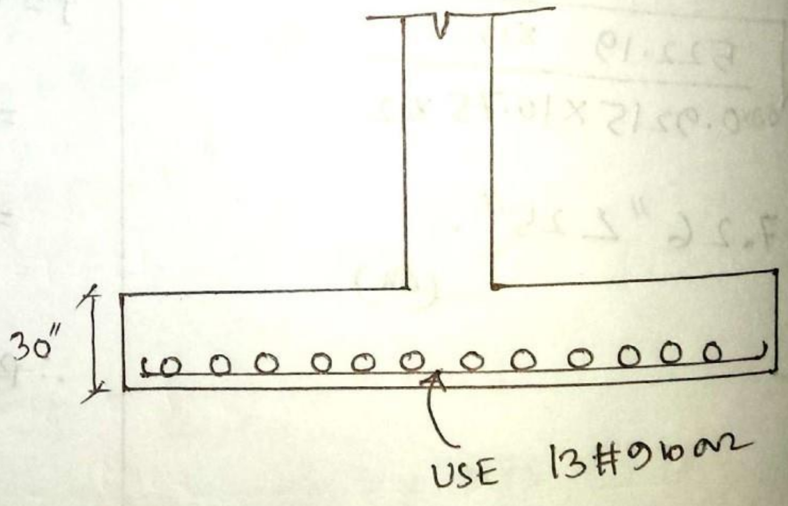
$$\therefore A_{s\,min} = 12.9 \text{ in}^2$$

$$A_{s\,min} = \frac{3\sqrt{f'_c}}{f_y} bd = \frac{3\sqrt{4000}}{50000} \times 10.75 \times 12 \times 25$$

$$\therefore A_{s\,min} = 12.2 \text{ in}^2$$

$$\therefore A_s = A_{s\,min} = 12.9 \text{ in}^2$$

USE 13 # 9 bar.



Bond check:

$$L_d = 0.04 \frac{A_b f_y}{\sqrt{f'_c}}$$

$$= 0.04 \times \frac{1 \times 50000}{\sqrt{4000}}$$

$$= 31.62''$$

$$\sim 32''.$$

Allowable,

$$L_d = \frac{L}{2} - \frac{a}{2} - cc$$

$$= \frac{10.75 \times 12}{2} - \frac{20}{2} - 3$$

$$= 51.5'' > 32'' \text{ (OK)}$$

$$h = d + 1.5 d_b + 3'' = 25 + 1.5 \times \frac{9}{8} + 3''$$

$$\therefore h = 29.68 \sim 29.75'' \sim 30''$$

মো: রবিউল ইসলাম
রাজশাহী প্রকৌশল ও প্রযুক্তি বিশ্ববিদ্যালয়
পুরকৌশল বিভাগ
রোল নং: ১৩০০১১০

Combined Footing

⇒ $\text{Area} = \dfrac{D.L + L.L}{q_a - \gamma D - \text{Surcharge}}$

⇒ Beam shear check.

$$V_{ud} = V_{max} - \frac{d}{12} W_u \quad [q_{ud} \times B = W_u]$$

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

$\boxed{V_{ud} = V_c}$ → d = ? পাওয়া যাবে।

* Punching shear test.

Exterior column

$$V_{ud} = (1.2DL + 1.6LL) - \text{Upward soil pressure}$$

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

⇒* Steel ⇒ $A_s = \dfrac{M_u}{\phi f_y (d - \frac{a}{2})}$, $a = \dfrac{A_s f_y}{0.85 f_c' b}$

$$A_s = \frac{3\sqrt{f_c'}}{f_y} bd \;, \quad A_s = \frac{200}{f_y} bd.$$

* Design of transverse beam under interior column:-

$d' = d - 1 = ?$

$b = d +$ interior column এর $b = ?$

Problem 16.3 Design a combined footing supporting one exterior and one interior column :-

An exterior 24 X 18 in column with D = 170 Kips, L = 130 Kips, and an interior 24 X 24 in column with D = 250 Kips, L = 200 Kips are to be supported on a combined rectangular footing whose outer end cannot protrude beyond the outer face of the exterior column. The distance centre to center of column is 18 ft, 0 inch and the allowable bearing pressure of the soil is 6000 psf. The bottom of the footing is 6 ft below grade, and a surcharge of 100 psf is specified on the surface. Design the footing for $f_c' = 3000$ psi, $f_y = 60000$ psi.

Soln Given that,

$q_a = 6000$ psf, $D = 6'$, Surcharge $= 100$ psf.

$$\text{Area required for footing} = \frac{D.L + L.L}{q_a - \gamma D - \text{Surcharge}}$$

$$= \frac{(170 + 130 + 250 + 200)1000}{6000 - 125 \times 6 - 100}$$

$$= 145.63 \text{ ft}^2$$

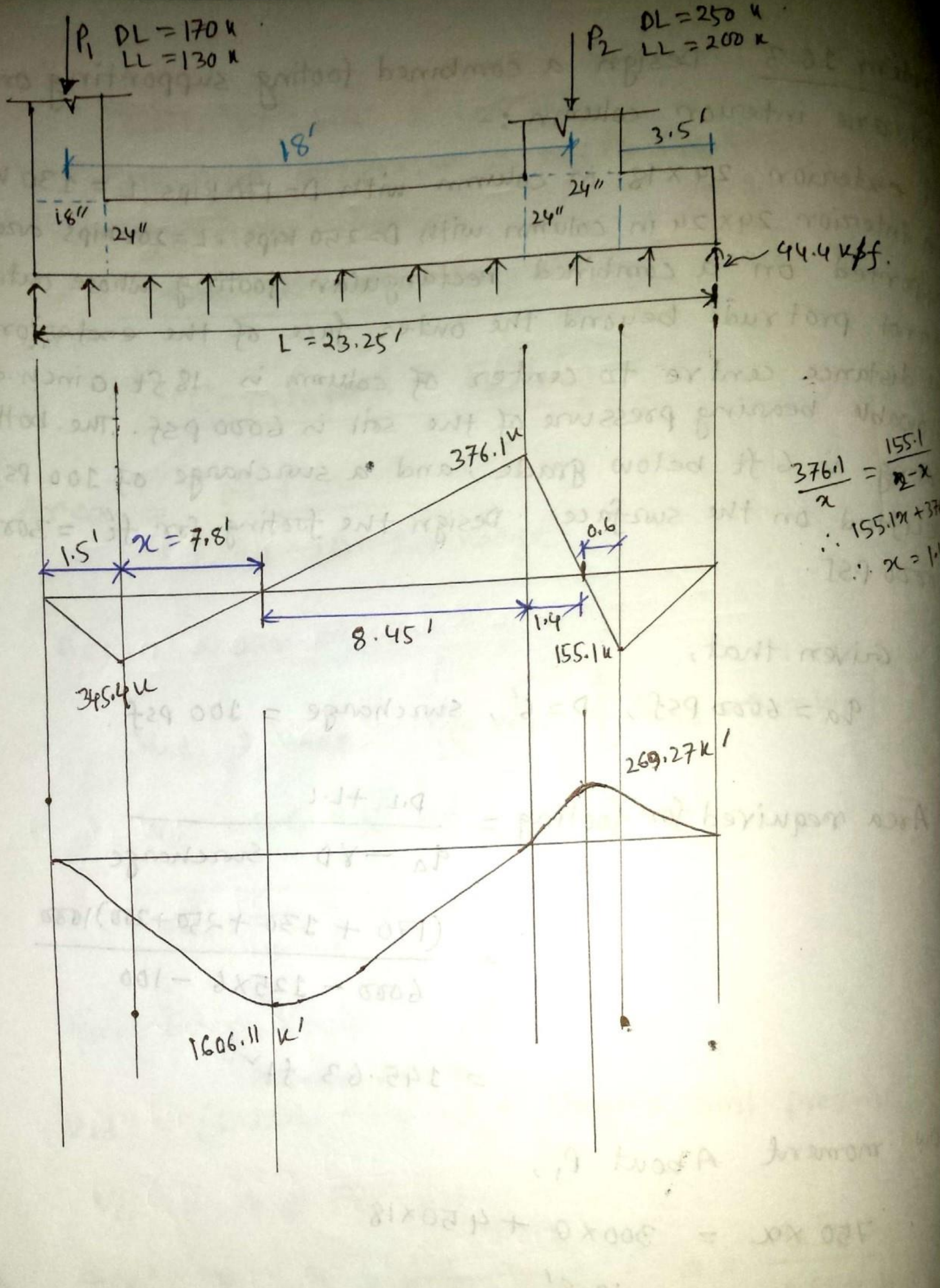
Now moment About P_1,

$$750 \times x = 300 \times 0 + 450 \times 18$$

$$\therefore x = 10.8'$$

Now, $\frac{L}{2} = x + \frac{18}{2 \times 12}$ $\Rightarrow L = (10.8 + 0.75) \times 2 = 23.1' \sim 23.25'$

$$\therefore B = \frac{145.63}{23.25} = 6.26 \sim 6.5'$$



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Ultimate pressure developed,

$$q_{ud} = \frac{1.2DL + 1.6LL}{Area}$$

$$= \frac{1.2(170+250) + 1.6(130+200)}{23.25 \times 6.5}$$

$$= 6.83 \text{ ksf}.$$

$\therefore$ Intensity of pressure per linear feet along length

$$= 6.83 \times 6.5$$

$$= 44.4 \text{ k/ft}.$$

The length of cantilever end from right end,

$$= 23.25 - \left(18 + \frac{9}{12} + \frac{12}{12}\right)$$

$$= 3.5'$$

→ Beam shear test.

$$V_{ud} = V_{max} - \frac{wd}{12}$$

$$= 376.1 - \frac{44.4 \times d}{12}$$

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

$$= \frac{2 \times 0.75 \times \sqrt{3000} \times 6.5 \times 12 \times d}{1000}$$

$$= 6.41d$$

Now,

$$V_c = V_{ud}$$

$$\therefore 6.41d = 376.1 - \frac{44.4d}{12}$$

$$\therefore d = 37.2'' \sim 37.5''$$

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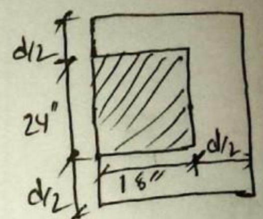
Punching shear test:

$V_{ud} = (1.2DL + 1.6LL)$ − upward soil pressure.

$= (1.2 \times \overset{170}{\cancel{200}} + 1.6 \times \overset{130}{\cancel{200}}) - (18 + \frac{d}{2})(\frac{24+d}{144}) \times 6.83$

$= 412 - (18 + \frac{37.5}{2})(\frac{24+37.5}{144}) \times 6.83$



$= 304.8$ K

$V_c = 4\phi\sqrt{f'_c}\, b_o d$

$= \frac{4 * 0.75\sqrt{3000} * \{2(18 + \frac{37.5}{2}) + (24 + 37.5)\} \times 37.5}{1000}$

$= 831.85$ K > 304.8 K (OK)

Moment check:

$\rho = 0.85\, \beta_1 \frac{f'_c}{f_y} \times \frac{\epsilon_u}{\epsilon_u + \epsilon_y}$

$= 0.85 \times 0.85 \times \frac{3}{60} \times \frac{0.003}{0.003 + 0.005}$

$= 0.014$

$R = \rho f_y (1 - 0.59 \frac{\rho f_y}{f'_c})$

$= 0.014 \times 60 (1 - 0.59 \times \frac{60 \times 0.014}{3})$

$= 0.705$ k/in²

$d = \sqrt{\frac{M}{\phi R b}} = \sqrt{\frac{1606 \times 12}{0.9 \times 0.705 \times 6.5 \times 12}} = 19.72'' < 37.5''$ (OK)

Reinforcement calculation :

$$A_s = \frac{M}{\phi f_y (d - \frac{a}{2})}$$

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

$$= \frac{A_s \times 60}{0.85 \times 3 \times 6.5 \times 12}$$

$$= 0.302 A_s.$$

$$A_s. = \frac{1606 \times 12}{0.9 \times 60 (37.5 - \frac{0.302 A_s}{2})}$$

$$\therefore \; 0.151 A_s^2 - 37.5 A_s + 356.88 = 0$$

$$\therefore A_s = 9.91 \text{ in}^2.$$

Use 10 # 9 bar.

Reinforcement for cantilever part:-

But not less than,

(1) $A_s = \frac{200}{f_y} bd = \frac{200}{60000} \times 6.5 \times 12 \times 37.5$

$$\therefore A_s = 9.75 \text{ in}^2$$ use 10 # 9 bar.

Minimum required,

$$A_s = \frac{269.27 \times 12}{0.9 \times 60 (37.5 - 0.151 A_s)}$$

$$\Rightarrow 0.151 A_s^2 - 37.5 A_s + 59.84 = 0$$

$$\therefore A_s = 1.61 \text{ in}^2$$

$$A_{s\,min} = \frac{3\sqrt{f'_c}}{f_y} bd = \frac{3 * \sqrt{3000}}{60000} \times 6.5 \times 12 \times 37.5$$

$$= 8.01 \text{ in}^2$$

Design of transverse beam under interior column.

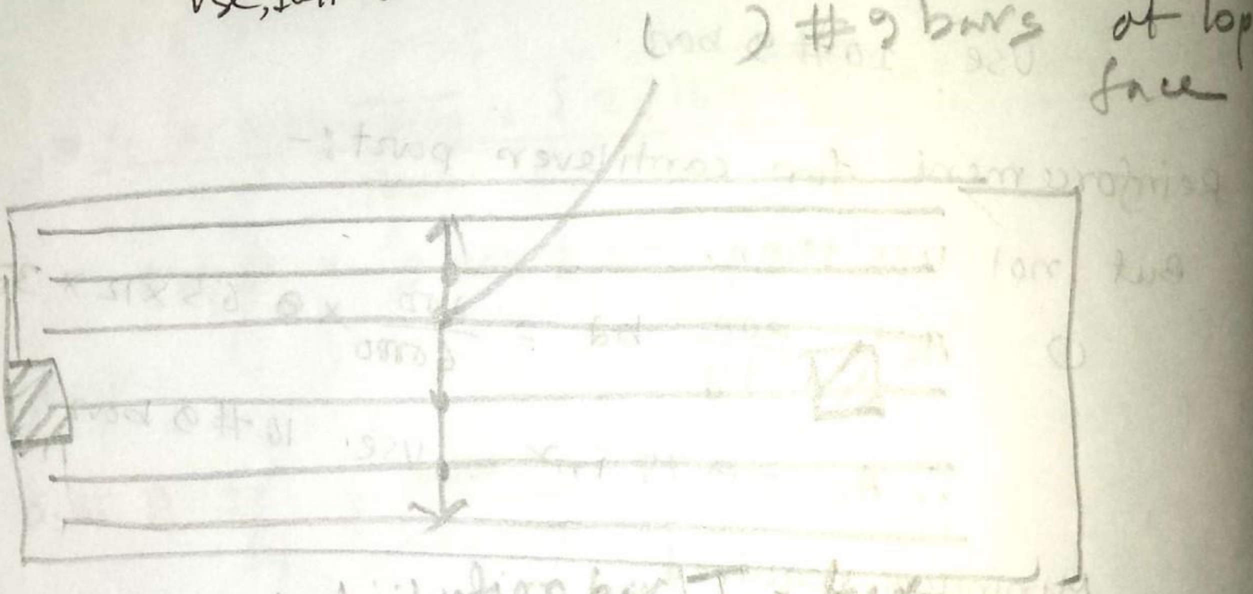
$$d' = 37.5 - 1 = 36.5''$$

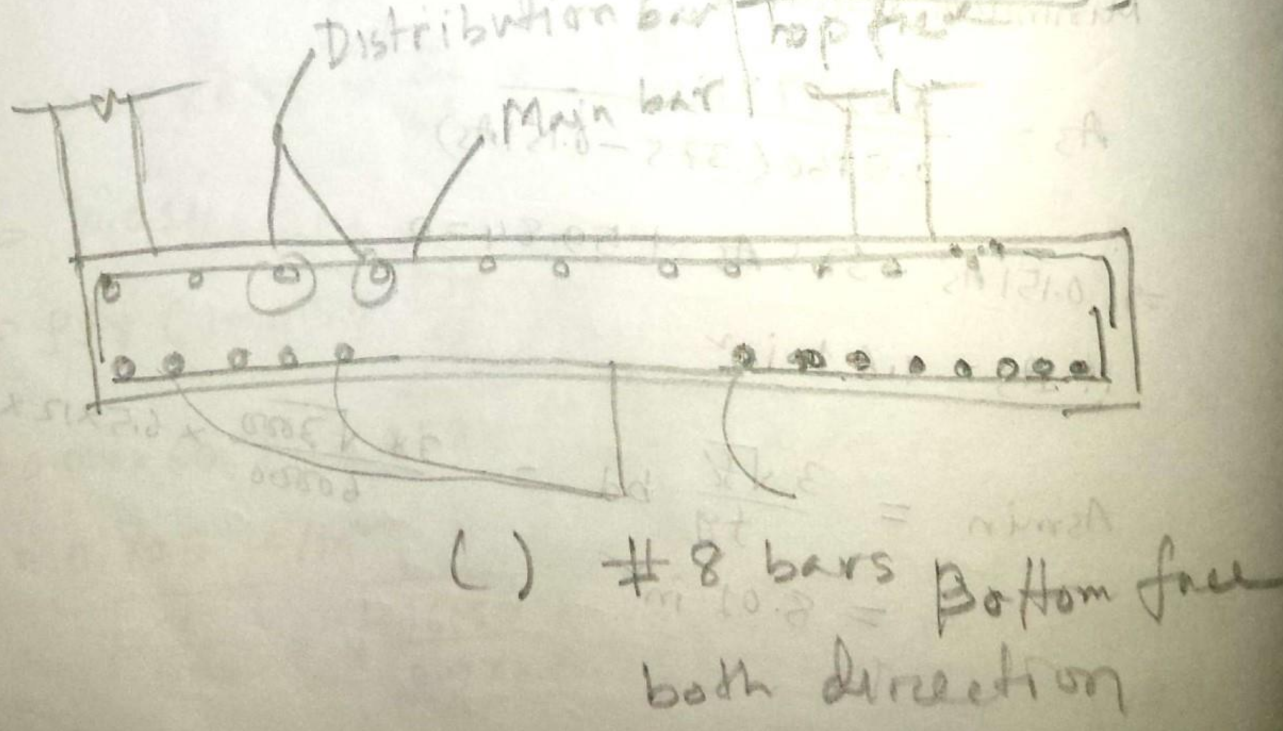
$$b = d + 24 = 37.5 + 24 = 61.5''$$

$$\therefore A_s = \frac{200}{f_y} b d' = \frac{200}{60000} \times 61.5 \times 36.5$$

$$\therefore A_s = 7.5 \text{ in}^2$$

Use, 10# 8 bar.





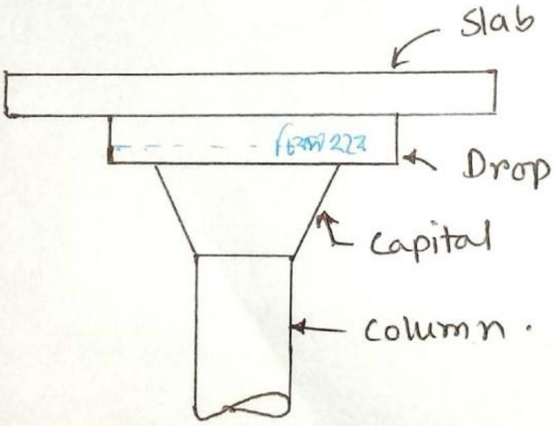
মো: রবিউল ইসলাম
রাজশাহী প্রকৌশল ও প্রযুক্তি বিশ্ববিদ্যালয়
পুরকৌশল বিভাগ
রোল নং: ১৬০১১০.

Flat slab

Q. Define flat slab and flat plate.

Answer: **Flat slab:** A flat slab is a slab which is constructed in such a way monolithically cast directly slab with column without the aid of beams and girders.

Flat plate: Special types of flat slab construction in which neither drop panel nor column capital are used is called flat plate.



Drop/panel, Dropped panel: The slab has uniform thickness throughout the entire floor area, symmetrical about the column, may be made somewhat thicker than the rest of the slab, the thickened portion of the slab thus formed constituting what is known as a dropped panel or drop.

Function of dropped panel: 2015

(I) To reduce the shearing stress.

(II) To increase the effective slab thickness.

(III) To decrease the compression stresses.

(IV) To reduce the amount of steel.

Reduce → SS, CS, AS
Increase → ET.

N.B → Dropped panel is not economical for live load less than 150 psf.

Column capital: 2015 The column in practically al cases flare out ~~for~~ toward the top, forming a capital of a shape somewhat similar to an inverted truncated cone, which is known as column capital.

Function of column capital:

(I) To gives the support for the floor slab from beneath.

(II) To decrease bending moment.

(III) Decreases shearing stress around the column.

N.B: The effective diameter of column capital should be equal at the point at which 45° line from the base of the capital intersects the bottom of the slab or dropped panel.

Advantages of flat slab:

(I) It is economical when superimposed load is large.

(II) For multistoried building less height is required.

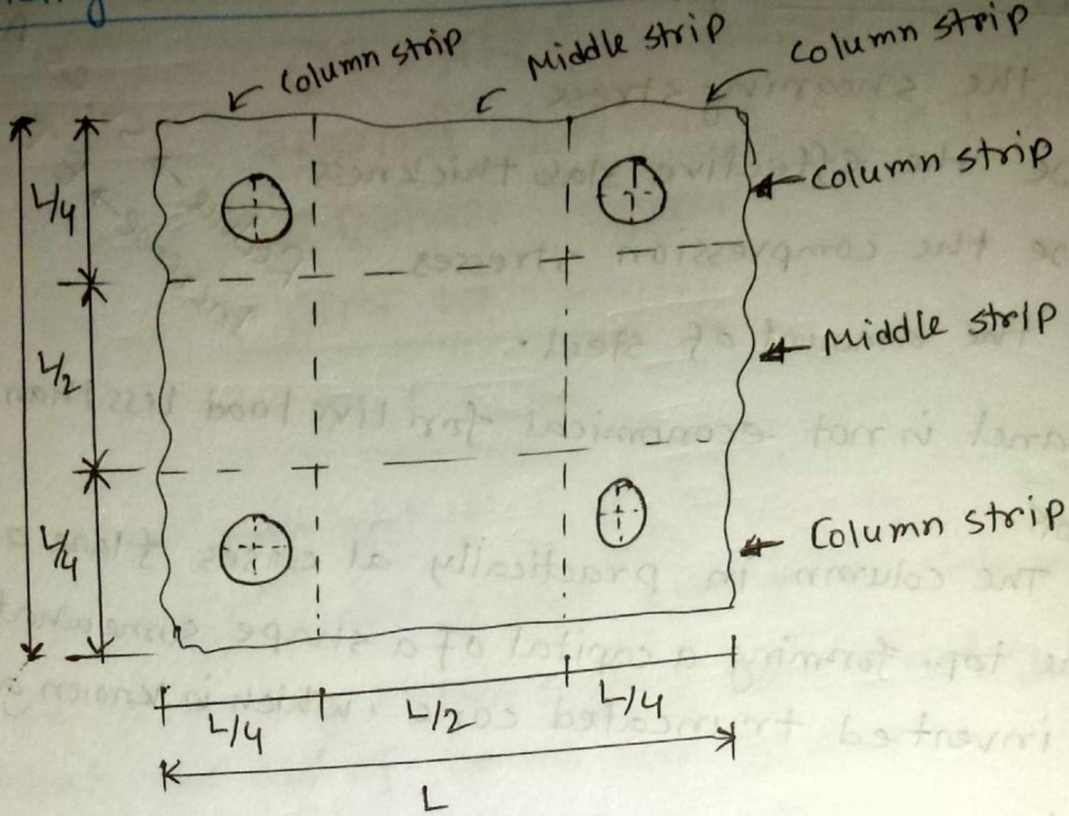
(III) The slab formwork is much simpler.

(IV) Light can passes through uniformly.

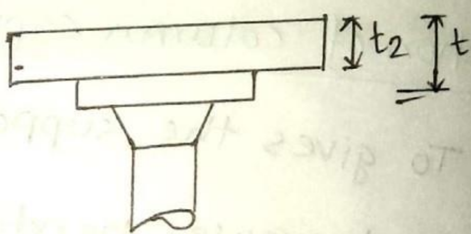
(V) It resist the fire.

(VI) Reinforcement can be easily done.

Bending Moment in Flat slab Floors:



Thickness of Slab:



$t_2 = \frac{L}{40}$ or 4" for without drop panel.

$t_2 = 5"$ for with drop panel.

$$t_2 = 0.024L\left(1 - \frac{2c}{3L}\right)\sqrt{\frac{w'}{\frac{f_c'}{2000}}} + 1 \text{, with drop panel}$$

$$t_2 = 0.028L\left(1 - \frac{2c}{3L}\right)\sqrt{\frac{w'}{\frac{f_c'}{2000}}} + 1\tfrac{1}{2} \text{, without drop panel.}$$

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

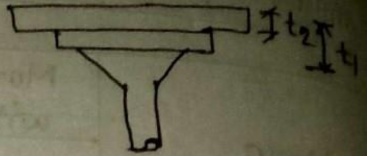
$$F = 1.15 - \frac{c}{L} \geq 1$$

where,
W = total load.
L = span length c/c distance.
w' = uniformly distributed load.
c = column capital's dia.
M_0 = design moment.

Moment distribution: in interior panels ACI Code.

Strip	Moment in slabs without drop panel		Moment in slabs with dropped panel.	
	Negative	Positive	Negative	Positive
Column strip	$0.46 M_0$	$0.22 M_0$	$0.50 M_0$	$0.20 M_0$
Middle strip	$0.16 M_0$	$0.16 M_0$	$0.15 M_0$	$0.15 M_0$

Procedure:



Thickness:

$t_2 \geq 4''$ (without dP), $t_2 \geq 5''$ → with dP

$$t_2 = \frac{L}{40}$$

$$t_2 = 0.024L\left(1-\frac{2c}{3L}\right)\sqrt{\frac{w'}{\frac{f_c'}{2000}}} + 1 \quad \text{, with drop panel.}$$

$$t_2 = 0.028L\left(1-\frac{2c}{3L}\right)\sqrt{\frac{w'}{\frac{f_c'}{2000}}} + 1\tfrac{1}{2} \quad \text{, without drop panel.}$$

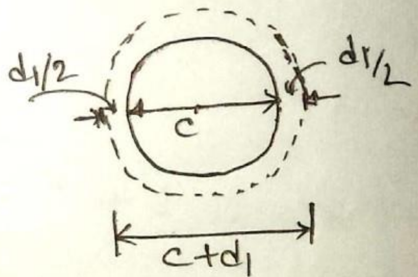
c = dia of column capital $= 0.15L$ to $0.25L$ (এই range এর ভিতর নিতে হবে)

$W' = D.L + L.L$, $DL = \frac{t}{12} \times 150$

$t_1 \leq 1.5\, t_2$

Punching shear check:

(a) Around column capital:



with drop panel,

$d_1 = t_1 - 1.5$ → clear cover

outside drop, $d_2 = t_2 - 1.5$

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

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

$v_c = 2\sqrt{f_c'}$ $\quad v_c > v$ (ok)

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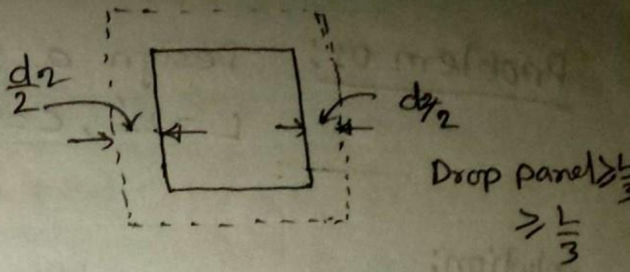
(b) Around dropped panel:

$V = SLW' - d_2^2 w'$

$v = \frac{V}{A} = \frac{V}{b_0 d_2}$

$v_c > v$ (ok)



Beam shear check:

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

$V = \frac{L}{2} \times S \times w' - \left(\frac{a}{2} + \frac{d_2}{12}\right) \times S \times w'$ | $V = \frac{wL^2}{2}$ (given by Zia sir)

$v = \frac{V}{b_0 d}$

$v_c = 1.1\sqrt{f_c'}$

$v_c > V$ (ok)

Moment calculation:

$M_0 = 0.09\, wLF\left(1 - \frac{2c}{3L}\right)^2$, w = total load.

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

Depth check:

a) where drop panel is present, $d_1 = \sqrt{\frac{M}{Rb}}$, b = 75% of drop panel width.

$d_{eff} = t_1 \Rightarrow d_1$

b) where drop panel is absent,

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

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

Reinforcement calculation:

a) Column strip, $A_s(-) / A_s(+) = \frac{M}{f_s j d}$

b) Middle strip, $A_s(+) / A_s(-) = \frac{M}{f_s j d}$

Problem 01: Design a flat slab when DL = 100 PSF, LL = 200 PSF, $L = 22'$, $c = 4'$, $f_c' = 3000$ PSI, $f_s = 20000$ PSI

Solution:

Thickness calculation:

$$t_2 = \frac{L}{40} = \frac{22 \times 12}{40} = 6.6'' \sim 8''$$

$$t_2 \geq 4''$$

$$t_2 = 0.024L\left(1 - \frac{2c}{3L}\right)\sqrt{\frac{w'}{\frac{f_c'}{2000}}} + 1$$

$$= 0.024 \times 22\left(1 - \frac{2 \times 4}{3 \times 22}\right)\sqrt{\frac{300}{\frac{3000}{2000}}} + 1$$

$$= 7.6'' \sim 8''$$

Here,
$w' = DL + LL$
$= 200 + 200$
$= 300$ PSf.
$f_c' = 3000$ PSI.
$c = 4''$
$L = 22'$

$$t_1 \leq 1.5\, t_2 \leq 1.5 \times 8'' \leq 12''$$

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

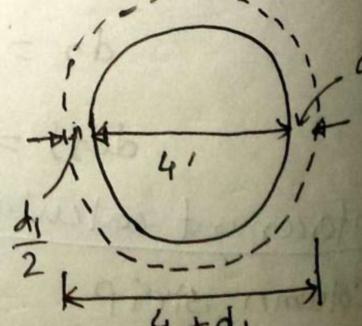
Punching shear check:

a) Around column capital:

Within drop, $d_1 = t_1 - \text{cover} = 11'' - 1.5'' = 9.5''$

Outside drop, $d_2 = t_2 - 1.5 = 8 - 1.5 = 6.5''$

$$c + d_1 = 4 + \frac{9.5}{12} = 4.79'$$



$$V_{dev} = w'\left(22 \times 22 - \frac{\pi}{4}(4.79)^2\right)$$
$$= 300\left(22 \times 22 - \frac{\pi}{4}(4.79)^2\right)$$
$$= 139.79 \text{ K}.$$

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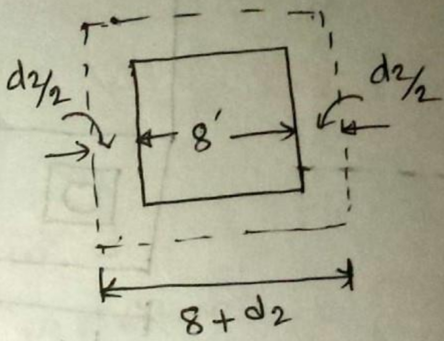
$$v = \frac{V_{dev}}{b_o d} = \frac{139.79 \times 1000}{\pi(4.79 \times 12) \times 9.5}$$

$\therefore v = 81.48$ PSI.

$v_{all} = 2\sqrt{f_c'} = 2\sqrt{3000} = 109.54$ PSI.

$v_{all} > v$ (ok)

(b) Around drop panel:



Length of drop panel $+ d_2 = 8 + \frac{6.5}{12}$

$= 8.54'$

$V_{dev} = w'(22 \times 22 - 8.54 \times 8.54)$

$= 300(22 \times 22 - 8.54 \times 8.54)$

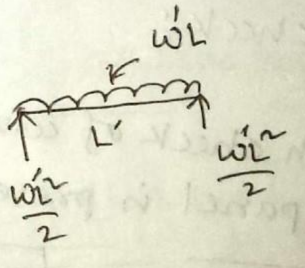
$= 123.32$ K.

$$v = \frac{V_{dev}}{b_o d} = \frac{123.32 \times 1000}{4 \times 8.54 \times 12 \times 6.5} = 46.28 \text{ PSI}.$$

$\therefore v_{all} = 2\sqrt{f_c'} = 2\sqrt{3000} = 109.54$ PSI.

$v_{all} > v$ ok.

Beam shear check:



$V_{dev} = \frac{w'L^2}{2}$

$= \frac{300 \times 22^2}{2}$

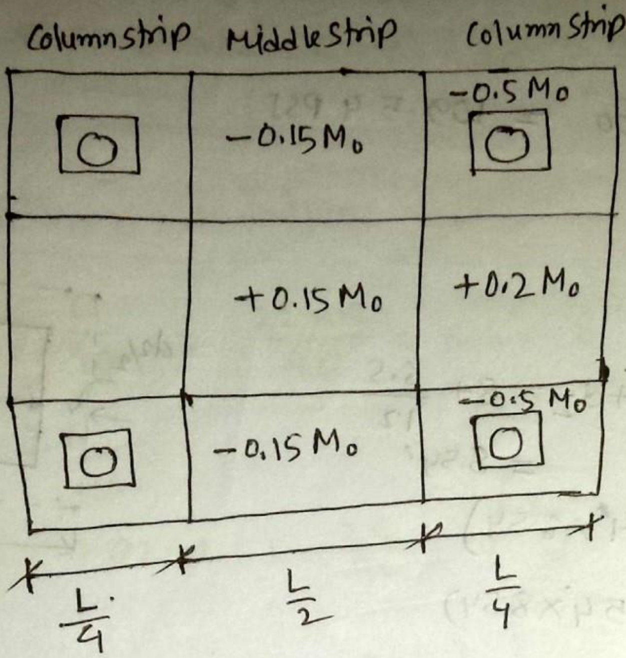
$= 72.6$ K.

$$v = \frac{V_{dev}}{bd} = \frac{72.6 \times 10^3}{22 \times 12 \times 6.5} = 42.307 \text{ PSI}.$$

$v_{all} = 1.1\sqrt{f_c'} = 1.1\sqrt{3000} = 60.25$ PSI.

$\therefore v_{all} > v$ (ok)

Moment calculation:



	33303.6 lb-ft	111012 lb-ft
	33303.6 lb-ft	44404.8 lb-ft
	33303.6 lb-ft	111012 lb-ft

$M_0 = \overset{0.09}{w} LF \left(1 - \frac{2c}{3L}\right)^2$

$= 0.09 \times 300 \times 22 \times 22 \times 22 \times 1 \left(1 - \frac{2\times 4}{3\times 22}\right)^2$

$= 222024$ lb-ft

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

$= 1.15 - \frac{4}{22}$

$= 0.97 \sim 1$

w = total load on panel

$= 22 \times 22 \times 300$

Depth check:

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

$d_1 = \sqrt{\frac{M}{Kb}} = \sqrt{\frac{111012 \times 12}{0.75 \times 223 \times 8 \times 12}}$

$\therefore d_1 = 9.1''$

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

$K = 0.75\ R$

$= 0.75 \times \frac{1}{2} f_c\ j\ k$

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

$r = \frac{f_s}{f_c} = \frac{20000}{0.45\times 3000} = 14.81$

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

$j = 1 - k/3 = 0.87$

$R = \frac{1}{2} f_c j k = 223$

$\therefore K = 0.75\ R$

$b = 8' = 8 \times 12''$

(b) Depth check of slab where drop panel is absent.

$$d_2 = \sqrt{\frac{44404.8 \times 12}{0.75 \times 223 \times \frac{22}{2} \times 12}} \qquad d_2 = \sqrt{\frac{M}{Kb}} \qquad b = \frac{L}{2} = \frac{22}{2} \times 12$$

$= 4.9''$

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

Reinforcement Calculation:

(a) At column strip:

$$A_s(-) = \frac{M}{f_s jd} = \frac{111012 \times 12}{20000 \times 0.87 \times 6.5} = 8.05 \text{ in}^2$$

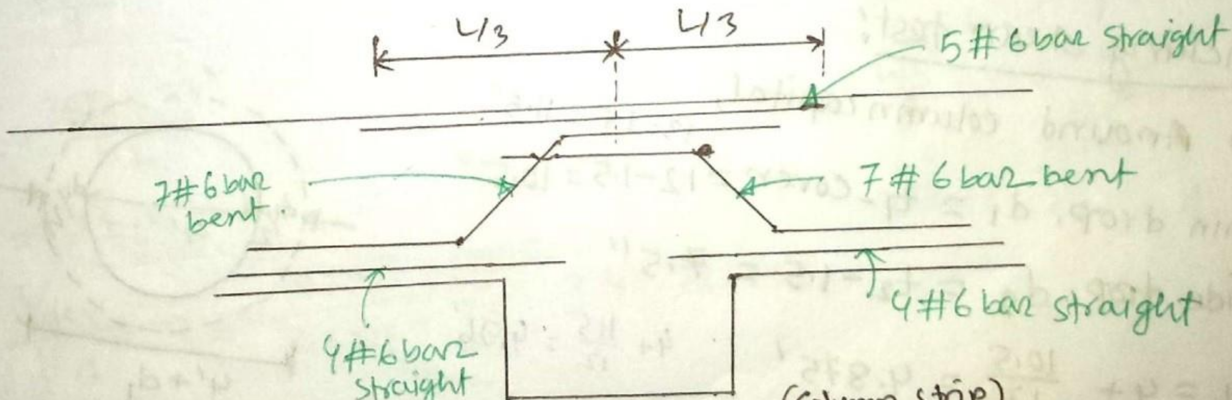
use 19 #6 bar.

$$A_s(+) = \frac{M}{f_s jd} = \frac{44404.8 \times 12}{20000 \times 0.87 \times 6.5} = 4.71 \text{ in}^2$$

use 11 #6 bar.

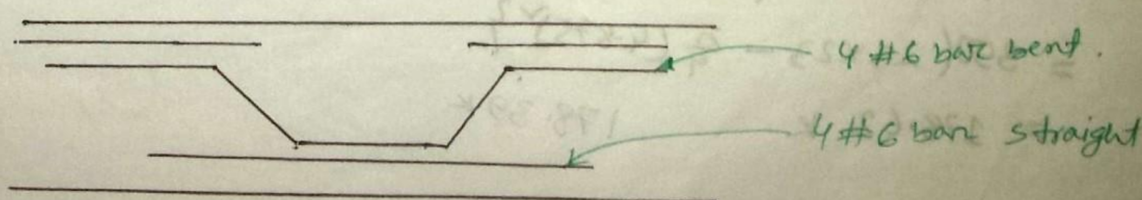
(b) At middle strip:

$$A_s(+) = A_s(-) = \frac{33303.6 \times 12}{20000 \times 0.87 \times 6.5} = 3.54 \text{ in}^2, \text{ USE } 8 \# 6 \text{ bar.}$$



(Column strip)

33% bar will straight $= 11 \times 0.33 = 3.63 \sim 4$ bar straight.



(Middle strip) 50% bar straigt, 50% bent.

2015 5(b): A flat slab floor system is to support a dead load of 125 psf including its self weight and a service live load of 225 psf having column spacing of 23ft on centers in both directions. Using 8-ft drop panel, design the slab. Assume $f_c' = 3000$ PSI and $f_y = 50000$ PSI.

Solution:

Thickness calculation:

Here,
$w' = DL + LL$
$= 125 + 225 = 350$ psf
$f_c' = 3000$ PSI
$L = 23'$
Assume,
$c = 4'$

$$t_2 = \frac{L}{40} = \frac{23}{40} = 6.9'' \sim 8''$$

$$t_2 \geq 4''$$

$$t_2 = 0.024L\left(1 - \frac{2c}{3L}\right)\sqrt{\frac{w'}{\frac{f_c'}{2000}} + 1}$$

$$= 0.024 \times 23\left(1 - \frac{2\times4}{3\times23}\right)\sqrt{\frac{350}{\frac{3000}{2000}} + 1}$$

$$= 8.45'' \sim 9''$$

$$\therefore t_2 = 9''$$

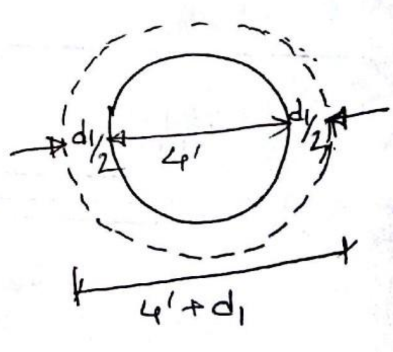
$$t_1 = 1.5\, t_2 = 1.5\times9 = 13.5 \sim 12'' \sim 13''$$

Punching shear test:

(a) Around column capital,

within drop, $d_1 = t_1 - \text{cover} = 12 - 1.5 = 10.5''$ $\quad 13 - 1.5 = 11.5''$

outside drop, $d_2 = t_2 - 1.5 = 7.5''$



$c + d_1 = 4 + \frac{10.5}{12} = 4.875'$ $\quad 4 + \frac{11.5}{12} = 4.96'$

$$V_{dev} = w'\left(23\times23 - \frac{\pi}{4}(4.875)^2\right) \quad 4.96$$

$$= 350\left(23\times23 - \frac{\pi}{4}(4.875)^2\right)$$

$$= 178.62 \text{ K.} \quad 178.39 \text{ K}$$

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$$v = \frac{V_{dev}}{b_o d_1} = \frac{178.62 \times 10^3}{\pi(4.875 \times 12) \times 10.5} = 92.56 \text{ PSI}$$

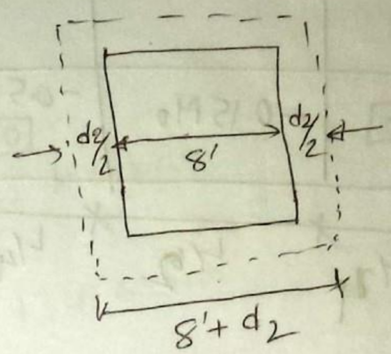
$$11.5 \quad = 82.96 \text{ PSI.}$$

$v_{all} = 2\sqrt{f_c'} = 2\sqrt{3000} = 109.54$ PSI.

$v_{all} > v$ (ok).

(b) Around drop panel:

Length of drop panel $+ d_2 = 8 + \frac{7.5}{12} = 8.625'$



$V_{dev} = w'(23 \times 23 - 8.625 \times 8.625)$

$= 350 * (23^2 - 8.625^2)$

$= 159.11$ K

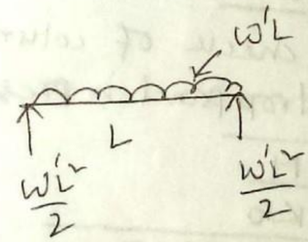
$$v = \frac{V_{dev}}{b_o d} = \frac{159.11 \times 10^3}{4 \times 8.625 \times 12 \times 7.5} = 51.24 \text{ PSI.}$$

$v_{all} = 1.1\sqrt{f_c'} = 1.1\sqrt{3000} = 60.25$ PSI.

$v_{all} > v$ (ok)

Beam shear check:

$$V_{dev} = \frac{w'L^2}{2}$$



$$= \frac{350 \times 23^2}{2}$$

$= 92.575$ K.

$$v = \frac{V_{dev}}{b d} = \frac{92.575 \times 1000}{23 \times 12 \times 7.5} = 44.72 \text{ PSI.}$$

$v_{all} = 1.1\sqrt{f_c'} = 1.1\sqrt{3000} = 60.25$ PSI.

$\therefore v_{all} > v$ (ok).

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Moment calculation:

	middle strip	column strip
$\boxed{0}$	$-0.15 M_0$	$-0.5 M_0$ $\boxed{0}$
	$+0.15 M_0$	$+0.2 M_0$
$\boxed{0}$	$-0.15 M_0$	$-0.5 M_0$ $\boxed{0}$
$L/4$	$L/2$	$L/4$

	-44931.075 lb ft	149770.25 lb-ft
	44931.075 lb-ft	59908.1 lb-ft.
	-44931.075 lb-ft	149770.25 lb-ft

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

$$= 0.09 \times 350 \times 23 \times 23 \times 23 \times 1 \left(1 - \frac{2 \times 4}{3 \times 23}\right)^2$$

$$= 299540.5 \text{ lb-ft}$$

$F = 1.15 - \frac{c}{L} = 1.15 - \frac{4}{23} = 0.98$

w = total load on the panel.

$= 350 \times 23 \times 23$

$L = 23'$

$c = 4'$

Depth check:

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

$$d_1 = \sqrt{\frac{M}{Kb}}$$

$$= \sqrt{\frac{149770.25 \times 12}{0.75 \times 223 \times 8 \times 12}}$$

$d_1 = 10.58$

$\therefore d_{eff} = 12 - 1.5 = 10.5 < 10.58$ (Not ok)

$\therefore d_{eff} = 13 - 1.5 = 11.5'' > 10.58''$ (ok)

Here, $f_y = 50000$ PSI, or $f_s = 20000$ PSI

$k = 0.75R$.

$n = \frac{E_s}{E_c} = \frac{29 \times 10^6}{57000\sqrt{f_c'}} = 9.28 \sim 9$

$r = \frac{f_s}{f_c} = \frac{20000}{0.45 \times 3000} = 14.81$

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

$j = 1 - k/3 = 0.87$

$R = \frac{1}{2} f_c j k = 223$

$b = 8' = 8 \times 12$ inch.

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(b) Depth check of slab where drop panel is not present.

$$d_2 = \sqrt{\frac{59908.1 \times 12}{0.75 \times 223 \times \frac{23}{2} \times 12}} \qquad \Big| \quad b = \frac{L}{2} = \frac{23}{2} \times 12 \text{ inch.}$$

$$= 5.58''$$

$d_{eff} = 7.5'' > 5.58''$ (ok)

Reinforcement calculation:

(a) At column strip:

$$A_s(-) = \frac{149770.25 \times 12}{20000 \times 0.87 \times 11.5} = 8.98 \text{ in}^2$$ USE 21 #6 bar.

$$A_s(+) = \frac{59908.1 \times 12}{20000 \times 0.87 \times 7.5} = 5.51 \text{ in}^2$$ USE 13 #6 bar.

(b) At middle ~~column~~ strip:

$$A_s(+) = A_s(-) = \frac{44931.075 \times 12}{20000 \times 0.87 \times 7.5} = 4.13 \text{ in}^2$$ USE 10 #6 bar.

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মো: রবিউল ইসলাম CE'130110

FLAT PLATE (USD)

Procedure:

Step-1:

thickness of slab, $t = 5''$ (minimum)

$$t = \frac{(800 + 0.005 f_y)\, l_n}{36000}$$ → ইঞ্চিতে নিতে হবে।

Step-2:

Load calculation.

$DL = \frac{t}{12} \times 150$ psf.

$LL =$ (Given) psf.

$W_u = 1.2DL + 1.6LL$

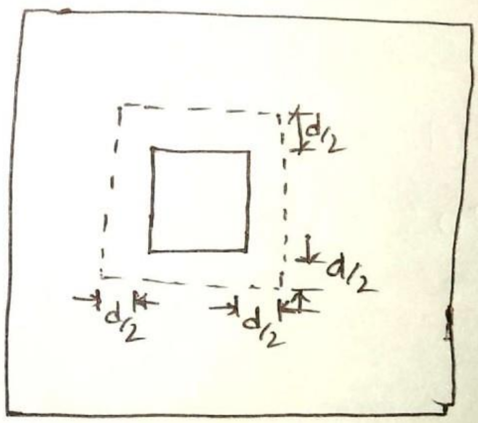
$d_{long} = d_L = t - 0.75 - \frac{4}{8\times 2}$ (bar dia.)

$d_{short} = d_s = d_l - 0.5$

Step-3:

Punching shear test:

$V_u = W_u$ {panel area − critical section area)



$v_{all} = 4\varphi\sqrt{f'_c}\, b_o d$

$v_{all} > v_u$ (ok)

[Beam shear test দরকার নাই]

Moment calculation:

M_o (long direction) $= 0.125\, W_u\, l_s\, l_l^2$ (clear span l_l, c/c span l_s)

M_o (Short direction) $= 0.125\, W_u\, l_l\, l_s^2$ (clear span l_s, c/c span of l_l)

Depth check:

$$\rho = \rho_{max} = 0.85\,\beta_1 \frac{f_c'}{f_y} \frac{\epsilon_u}{\epsilon_u + \epsilon_y}$$

$$R = \rho f_y \left(1 - \frac{\beta}{\alpha} \frac{f_y}{f_c'} \rho\right)$$ [যখন $f_c' \doteq 4\,KSI$ পর্যন্ত $\frac{\beta}{\alpha} = 0.59$]

$$d = \sqrt{\frac{M}{\phi R b}}$$, b = shorter length.

Table:

working diagram.

Problem: A 18 x 20 ft interior panel of a flat plate floor system carries 60 psf LL and dead load 100 psf including self weight. Design the panel Assuming column size 15" x 15". $f_c' = 4000\,psi$, $f_y = 60\,ksi$.

Soln

$t = 5''$

$$t = \frac{(800 + 0.005 f_y)\, l_n}{36000}$$

$l_n = 20' - \frac{15}{12} = 225''$

$$= \frac{(800 + 0.005 \times 60000) \times 225}{36000}$$

$$= 6.87'' \sim 7''$$

Load calculation:

DL = 100 psf.

LL = 60 psf.

$W_u = 1.2\,DL + 1.6\,LL = 216\ psf$.

$$d_l = t - 0.75 - \frac{4}{8 \times 2} = 6''$$

$$d_s = d_l - 0.5 = 5.5''$$

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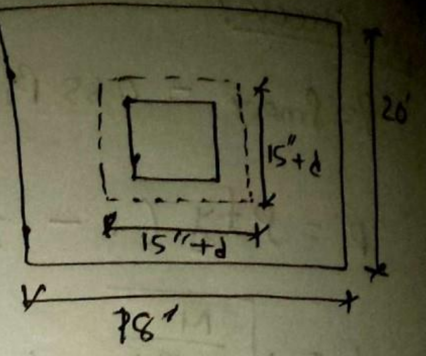
Punching shear test:



$$V_u = W_u \{\text{panel area} - \text{critical section area}\}$$

$$= 216\left(18\times20 - \frac{21}{12}\times\frac{21}{12}\right)$$

$$= 77098.5 \text{ lb}.$$

Let, $d = 6''$

$\therefore\ 15'' + 6'' = 21''$

$$V_{all} = 4\varphi\sqrt{f_c'}\ b_o\,d$$

$$= 4 * 0.75 * \sqrt{4000} * 4\times21\times6$$

$$= 95627.27 \text{ lb}.$$

$V_{all} > V_u$ (ok).

Moment calculation:

$$M_{o\text{-long}} = 0.125\, W_u\, l_s\, l_l^2$$

$$= 0.125\times216\times18\times18.75^2$$

$$= 170859.375 \text{ lb-ft}.$$

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

$l_s = 18'$

$$M_{o\text{-short}} = 0.125\, W_u\, l_l\, l_s^2$$

$$= 0.125\times216\times20\times16.75^2$$

$$= 151503.75 \text{ lb-ft}$$

$l_s = 18 - \frac{15}{12} = 16.75'$

$l_l = 20'$

depth check:

$$\rho = 0.85\ \beta_1\ \frac{f_c'}{f_y}\ \frac{\epsilon_u}{\epsilon_u+\epsilon_y}$$

$$= 0.85\times0.85\times\frac{4}{60} * \frac{0.003}{0.003+0.005}$$

$$= 0.018.$$

$$R = \rho f_y\left(1 - 0.59\frac{\rho f_y}{f_c'}\right) = 0.018\times60000\left(1 - 0.59\times\frac{60\times0.018}{4}\right)$$

$$R = 907.956$$

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$$d = \sqrt{\frac{M}{\phi R b}}$$

$$= \sqrt{\frac{170859.375 \times 12}{0.9 \times 907.956 \times 201}}$$

$$= 3.53'$$

$d_i > d$ (ok)

b = shorter length
$= 18 \times 12 - 15$
$= 201$

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

Direction	Longer, $M_n = \frac{M_o}{\phi} = \frac{170859.375}{0.9} = 189.84$ k-ft				Shorter, $M_n = \frac{M_o}{\phi} = \frac{151503.75}{0.9} = 168.34$ k-ft.			
Strip	Column	Middle	Column	Middle	Column	Middle	Column	Middle.
Sign	—	+	—	+	—	+	—	+
K	0.49	0.21	0.16	0.14	0.49	0.21	0.16	0.14
$M_r = KM_n$ (k-ft)	93.1	39.87	30.37	26.58	82.5	35.35	26.94	23.57
d (in)	6	6	6	6	5.5	5.5	5.5	5.5
b	9	9	9	9	9	9	9	9
$a = 17.65 \frac{A_s}{b}$	0.16 As	0.16 As	0.16 As	0.16 As	0.16 As	0.16 As	0.16 As	0.16 As
$A_s = \frac{M_r}{\phi f_y (d - a/2)}$	3.62	1.51	1.14	0.99	3.19	1.33	1.0	0.88
$A_{s\,min} = 0.0018bt$	1.36	1.36	1.36	1.36	1.36	1.36	1.36	1.36
Use #4 bar	19	8	7	7	16	7	7	7
$S_{av} = \frac{b}{n}$	5.68	13.5	15.43	15.43	6.75	15.43	15.43	15.43
$S_{max} = 2t$	14	14	14	14	14	14	14	14
Provide bar.								

মো: রবিউল ইসলাম

রাজশাহী প্রকৌশল ও প্রযুক্তি বিশ্ববিদ্যালয়

পুরকৌশল বিভাগ

রোল নং: ১৬০১১০

TWO WAY SLAB

2015, 2013

1(a) Prove that a slab can be designed as a one-way slab if the long to short span ratio of the slab is larger than 2.

Answer: **Proof:**

Considering the fig(a) of a panel of a slab where larger and shorter direction as B and A are shown.

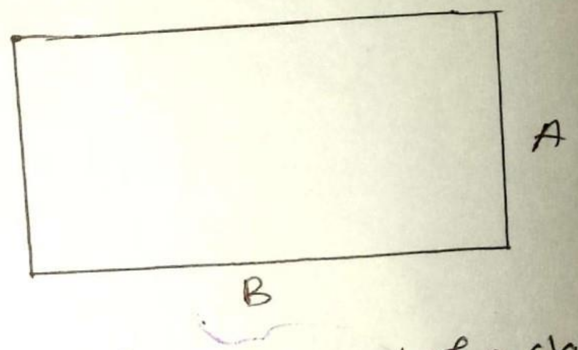

fig (a). panel of a slab.

Here, for the square type of panel the B and A are equal so that there ratio is 1. If there is taken the trial and error method and increasing and decreasing the value of moment of B and A. the ratio of B will be is greater than 2. And then the moment in shorter neglected and only then having moment direction which showes it is one way slab and its long to shorter span ratio is greater than 2.

So, $\frac{MB}{MA} > 2 \Rightarrow \frac{1}{MA} > 2 \Rightarrow \frac{B}{A} > 2$

So, For one way slab design the long to shorter span ratio of the slab is larger than 2.

2014, 03, 05

Distigush betn one way slab and two way slab.

Answer: Differences: The differences between the one way slab and two way slab are given below:-

One way Slab	Two way slab.
(I) The ratio of long span to short span is more than 2. i.e. $\frac{Ll}{Ls} > 2$	(I) The ratio of long span to short span is less than 2. i.e. $\frac{Ll}{Ls} < 2$.
(II) Total load distributed along short length.	(II) Total load is distributed along both length.
(III) Main steels are provided along short direction only	(III) Main steels are provided along both direction.
(IV) Deforms into a cylindrical surface.	(IV) Deforms into a dished surface.

2012 Why it is necessary to provide more reinforcement in shorter direction of a rectangular two way slab?

or, prove majority load in short direction.

proof:

Considering the following an isolated simple supported slab,

where,

A = span in short direction.

B = span in long direction.

W_B = load distributed along B.

W_A = load distributed along A.

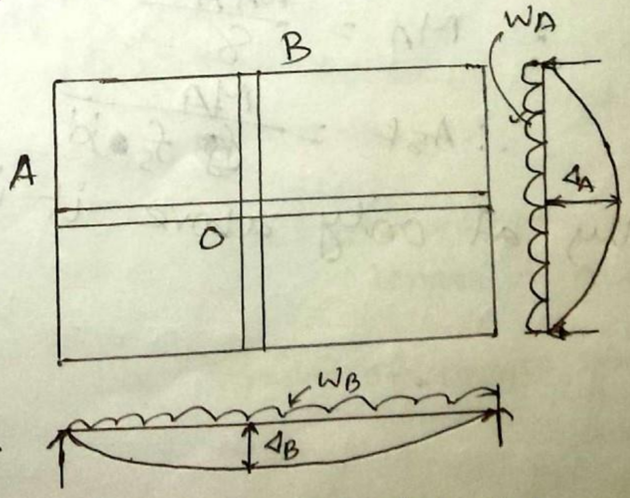

Fig. simple supported slab.

$W = W_A + W_B$ —— (I)

W = total load distributed on slab.

Considering a unit strip along short direction and another along long direction shown in figure.

Then we get,

$$\Delta_B = \frac{5}{384} \frac{W_B B^4}{EI} \quad \text{—— (II)}$$

and $$\Delta_A = \frac{5}{384} \frac{W_A A^4}{EI} \quad \text{—— (III)}$$

Equating (II) & (III) we get.

$$\Delta_B = \Delta_A$$

$$\therefore \frac{5}{384} \frac{W_B B^4}{EI} = \frac{5}{384} \frac{W_A A^4}{EI}$$

$$\therefore \frac{W_A}{W_B} = \left(\frac{B}{A}\right)^4$$

$$\therefore W_A = W_B \times \left(\frac{B}{A}\right)^4 \quad \text{—— (IV)}$$

Here, B>A , so loads are majority in short direction. If load is major then moment will be high and more reinforcement should be provided.

$$\therefore M_A = \frac{W_A A^2}{8} \quad \text{—— (V)}$$

$$\therefore A_{st} = \frac{M_A}{f_s\, jd} \quad \text{—— (VI)}$$

Finally at only above it is done.

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Q. Why special reinforcement is required at exterior corners of a two way slab? Draw a qualitative diagram of steel placement for an exterior corner.

Answer: Reasons:

Corner reinforcement: The difflection which is occur in a two way floor system can be resist by using reinforcement in the corner of floor slab, ~~is known~~. This reinforcement is known as corner reinforcement.

Reasons: For the following reasons special reinforcement is required at ~~reint~~ exterior corner of a two way slab:-

(I) To resist twisting moment developed at corner.

(II) To resist the tendency of cracking diagonally along the panel.

(III) To resist the tendency of cracking perpendicular to the diagonal.

Qualitative diagram:

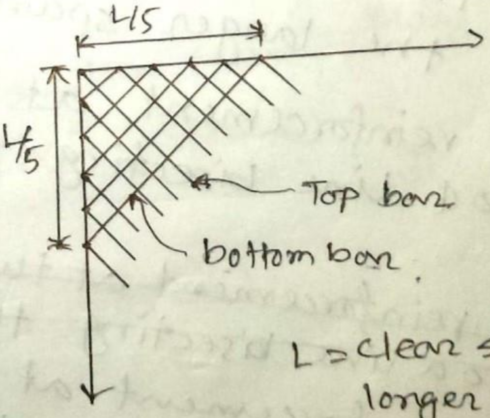

Fig. details of corner reinforcement

Q. Class

Why corner reinforcement is provided in two way slab. Write ACI code specification for such reinforcement.

Answer: Corner reinforcement: To resist twisting moment of a slab, the reinforcement developed at the corner of a slab, the reinforcement is given is called the corner reinforcement.

Corner reinforcement provides for following reasons:—

The twisting moments are occured at only the exterior corners of a two way slab system. Where they tend to crack the slab that at the bottom along the panel diagram and the top perpendicular to the panel diagram. To resist this twisting moment corner reinforcement is provided at exterior corners in both the bottom and top of the slab.

Specification for corner reinforcement:—

(I) Special reinforcement shall be provided at exterior corners in both bottom and top of the slab, for a distance in each direction from the corner equal to one fifth the larger span of the corner panel.

(II) Corner reinforcement at the top of the slab shall be parallel to a line bisecting the angle at the relevant corner.

~~(III) Corner reinforcement at the top of the slab shall be parallel to a line bisecting the angle at the relevant corner~~

(III) Corner reinforcement at the bottom of the slab shall be perpendicular to a line bisecting the angle at the relevant corner.

(IV) The top and bottom corner reinforcement shall be of size and spacing equivalent to that required for the maximum positive moment in the panel.

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prove that in a isotropically reinforced slab. the yield moment in all directions are equal and torsion is zero.

Answer: Proof:

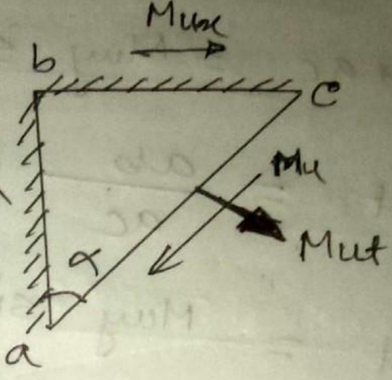


fig (a)

Let considering fig (a). where.

Mux = ultimate moment per unit length along x-axis.

Muy = ultimate moment per unit length along Y-axis.

Mu = ultimate moment along yield line

Mut = Torsional moment per unit width normal to the yield line.

Taking summession of forces along yield line.

$-Mu * ac + Muy * \cos\alpha\ ab + Mux \sin\alpha * bc = 0$

$\therefore Mu = \frac{ab}{ac} Muy \cos\alpha + \frac{bc}{ac} Mux \sin\alpha$

$= Muy \cos^2\alpha + Mux \sin^2\alpha$

for isotropically reinforced slab, $Mux = Muy$

$Mu = Mux(\cos^2\alpha + \sin^2\alpha) = Mux$

$\therefore Mu = Mux = Muy$ (proved)

Taking summession of forces normal to yield line

$$Mut * ac = Muy \sin\alpha * ab - Mux \cos\alpha \times bc$$

$$\Rightarrow Mut = \frac{ab}{ac} Muy \sin\alpha - \frac{bc}{ac} Mux \cos\alpha$$

$$\Rightarrow Mut = Muy \sin\alpha \cos\alpha - Mux \sin\alpha \cos\alpha$$

$$\Rightarrow Mut = Mux \sin\alpha \cos\alpha - Mux \sin\alpha \cos\alpha \quad [\because Mux = Muy]$$

$$\therefore Mut = 0$$ (proved)

22/04/19

4:04 PM

মো: রবিউল ইসলাম (TWO WAYSLAB)

WSD Method

Procedure:

(I) **Dimension check:**

A = Short direction.

B = Long direction.

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

6" above → 0.5 বাড়বে } round করতে হবে.
6" below → 0.25 বাড়বে

(II) **Thickness:**

$t_{min} = 3.5''$

$t = \frac{\text{perimeter}}{180} = \frac{2(A+B)\times 12}{180}$ inch.

< বড়টা >

(III) **Load calculation:**

$DL = \frac{t}{12} \times 150$ psf.

LL না দেওয়া থাকলে 40 psf.

$w = T.L. = DL + LL$

(IV) **Moment calculation:**

$m = \frac{A}{B}$, কোন case এ পড়বে সেটা দেখে $C_{A\,D.L}$, $C_{B\,D.L}$, $C_{A\,L.L}$, $C_{B\,LL}$.

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

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: $M_A(+) = (C_{A.LL} * W L_A^2) + (C_{A\,DL} * W L_A^2)$

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

(v) **Depth check:**

$d = \sqrt{\frac{M}{Rb}}$, $b = 12''$, $d_{act} = t - 1$. $d_{act} > d$ (ok)

(vii) **Reinforcement calculation:**

In short direction & Longer direction দুই ক্ষেত্রে একই হবে same process.

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

$A_{smin} = 0.0018\, bt$ ($f_y = 60$ ksi)

$= 0.0020\, bt$ ($f_y = 40$ or 50 ksi)

Spacing, $S = ?$

$S_{max} = 3t$

(−) $A_s = \frac{M}{f_s j d}$

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

Working diagram:

Problem: Design a slab panel 20'×25' to support a uniform Live load of 125 psf. The panel is a corner panel of a group. A 3000 psi concrete and steel $f_y = 60000$ psi are used in connection with ultimate strength design.

Solution: WSD তে যেমন হয়েছিল শুধু steel calculation USD তে আলাদা হবে।

(I) $A = 20'$, $B = 25'$, $\frac{B}{A} = \frac{25}{20} = 1.25 < 2$.

$\therefore$ two way slab confirm.

(II) Thickness, $t = 3.5''$

$$t = \frac{2(A+B)\times 12}{180} = \frac{2(20+25)\times 12}{180} = 6''$$

$\therefore$ Design thickness, $t = 6''$

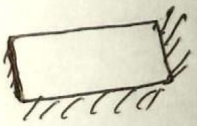
(III) Load calculation.

$$D.L = \frac{t}{12}\times 150 = \frac{6}{12}\times 150 = 75 \text{ psf}.$$

$LL = 125$ psf.

$\therefore$ Total load, $W_T = DL + LL = 75 + 125 = 200$ psf.

(IV) Moment calculation,



$$m = \frac{A}{B} = \frac{20}{25} = 0.80 \text{ (case - 4)}$$

$C_{A\,neg} = 0.071$, $C_{A\,DL} = 0.039$, $C_{A\,LL} = 0.048$

$C_{B\,neg} = 0.029$, $C_{B\,DL} = 0.016$, $C_{B\,LL} = 0.020$

At continuous end,

$(-)M_A = C_{A\,neg}\, W_T L_A^2 = 0.071 * 200 * 20^2 = \cancel{8236}$ 5680 lbft

$(-)M_B = C_{B\,neg}\, W_T L_B^2 = 0.029 \times 200 \times 25^2 = 3625$ lb-ft

At midspan,

$(+)\ M_A = C_{ADL} \cdot W_{D.L} * L_A^2 + C_{ALL} \cdot W_{LL} * L_A^2$

$= (0.039 * 75 * 20^2) + (0.048 * 125 * 20^2)$

$= 3570$ lb-ft.

$(+)\ M_B = C_{BDL} \cdot W_{DL} \cdot L_B^2 + C_{BLL} \cdot W_{LL} \cdot L_B^2$

$= (0.016 \times 75 \times 25^2) + (0.020 \times 125 \times 25^2)$

$= 2312.5$ lb-ft

(v) Depth check:

$b = 12''$ (always)

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

$d = \sqrt{\frac{5680 \times 12}{202 \times 12}}$

$= 5''$

$d_{act} = 6 - 1 = 5'' > d$ (ok)

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

$r = \frac{f_s}{f'_c} = \frac{24000}{0.45 \times 3000} = 17.77$

$k = \frac{n}{n+r} = 0.34$

$j = 1 - k/3 = 0.88$

$R = \frac{1}{2} f'_c\, j\, k = \frac{1}{2} \times 0.45 \times 3000 \times 0.88 \times 0.34$

$\therefore R = 202$

Reinforcement calculation:

short direction

$(+)\ A_s = \frac{M}{f_s j d} = \frac{3570 \times 12}{24000 \times 0.88 \times 5} = 0.41\ in^2$

$A_{s\,min} = 0.0018\, bt = 0.0018 \times 12 \times 6 = 0.1296\ in^2$

USE # 3 bar @ $\frac{0.11 \times 12}{0.41} = 3.25''$ c/c.

USE # 4 bar @ $\frac{0.20 \times 12}{0.41} = 5.75''$ c/c.

$(-)\ A_s = \frac{5680 \times 12}{24000 \times 0.88 \times 5} = 0.65\ in^2$

USE # 4 bar @ $\frac{0.20 \times 12}{0.65} = 3.75''$ c/c.

Long direction

(+) $A_s = \frac{2312.5 \times 12}{24000 \times 0.88 \times 5} = 0.26 \text{ in}^2$

USE # 3 bar @ $\frac{0.11 \times 12}{0.26} = 5'' \text{ c/c}$

(−) $A_s = \frac{3625 \times 12}{24000 \times 0.88 \times 5} = 0.42 \text{ in}^2$.

USE # 3 bar @ $\frac{0.11 \times 12}{0.42} = 3.25'' \text{ c/c}$.

development length.

$V = \frac{w_a l_A}{2} = \frac{200 \times 20}{2} = 2000 \text{ lb}$.

$u_d = \frac{V}{\Sigma_0 jd} = \frac{2000}{\pi * \frac{3}{8} * \frac{12}{3.25} \times 0.88 \times 5}$

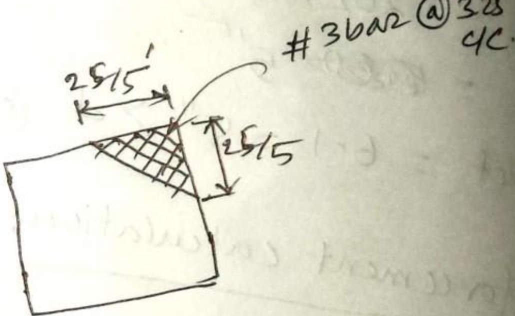
$u_d = 104.5 \text{ lb}$.

$L_d = \frac{f_s D}{4 u_d} = \frac{24000 \times 3/8}{4 \times 104.5} = 21.5''$

$L_d \text{ min} = 12D = 12 \times \frac{3}{8} = 4.5''$

$l_d = 21.5''$

Corner reinforcement:



working diagram: