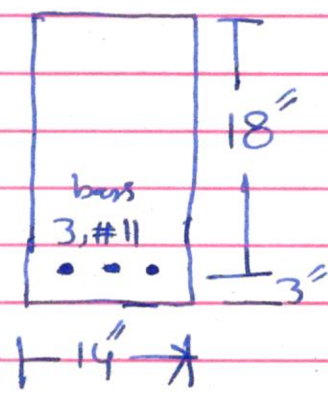
Date: ID: 15404

Q.No (01):

(A).

Section #(i)

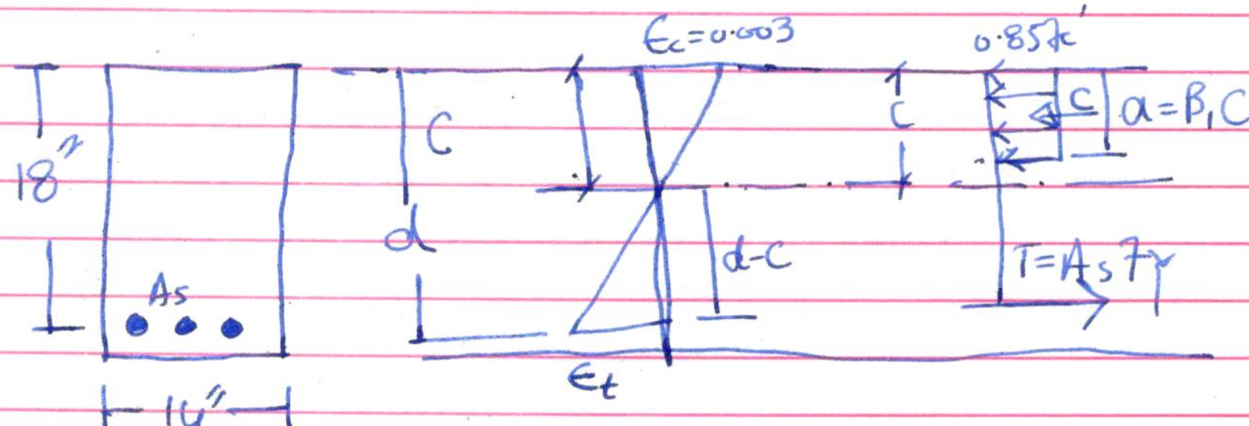


ϵ_t, ϕ & ϕM_n = ?

$f_y = 75000$ psi , $f'_c = 5000$ psi

From Appendix A , Tabele A.4

A_s = 3, #11 bars = 4.68 in^2.



$$C = T$$

$$0.85 f'_c a b = A_s f_y$$

$$a = \frac{4.68 * 75}{0.85 \times 5 \times 14} = 5.9 \text{ in}$$

Now $\beta_1 = 0.85 - \left(\frac{f'_c - 4000}{1000}\right) 0.05 = 0.80$

Date: 15404-2

So
$$C = \frac{a}{\beta_1} = \frac{5.9}{0.8} = 7.4\ in$$

Now, strain in tensile steel $= \epsilon_t$

from figure, $\epsilon_t = \frac{0.003(d-c)}{c}$

$$\epsilon_t = \frac{0.003(18-7.4)}{7.4} = 0.0043$$

$$\epsilon_t = 0.0043 < 0.005 \text{ (Transition Zone)}$$

hence, (Tension controlled) (Ductile failure)
$$\phi = 0.65 + (\epsilon_t - 0.002)\,250/3$$

$$\phi = 0.84$$

$$M_u = A_s f_y (d - a/2) = 4.68 * 75 (18 - 5.9/2)$$

$$= 5282.55\ k\text{-}in$$

$$M_n = 440.2\ k\text{-}ft$$

Applying strength reduction factor, ϕ

$\phi M_n = 0.84 * 440.2 = 369.8$ k-ft.

Now

$$\rho = \frac{A_s}{bd} = \frac{4.68}{14 \times 18} = 0.0186$$

From Appendix A, Table A-7;

ρ_{min} for $f_y = 75$ ksi & $f_c' = 5$ ksi

$\rho_{min} = 0.0028$

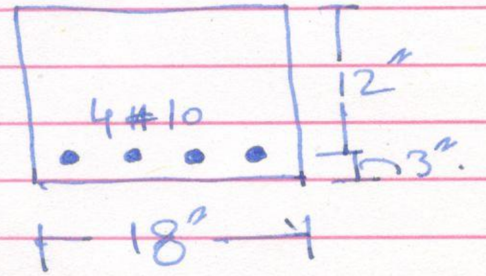
So,

~~$A_{s(min)} = 0.0028 \times 14 \times 18 =$~~

$\rho > \rho_{min}$.

So, Flexural design strength of the section is 369 k-ft and Tension controlled.

Section (ii)



$f_y = 60{,}000$ Psi

$f_c' = 4000$ Psi.

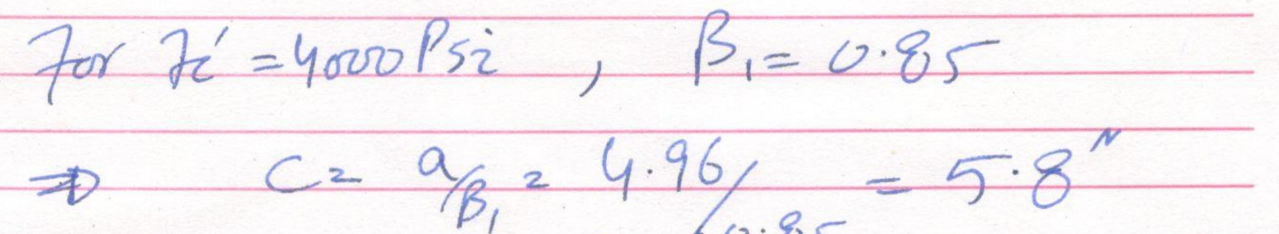
ϵ_t , ϕ and $\phi M_n = ?$

From Table A-4, $A_s = 5.06\ in^2$.

$$a = \frac{5.06 * 60}{0.85 * 4 * 18} = 4.96\ in$$

For $f_c' = 4000$ Psi , $\beta_1 = 0.85$

$$\Rightarrow \quad c = a/\beta_1 = 4.96/0.85 = 5.8''$$



$$\epsilon_t = \frac{0.003(6.2)}{5.8} = 0.0032 > \epsilon_y$$

Tension Controlled.

$\epsilon_t = 0.0032 < 0.005$, So

Strength Reduction factor will be

$$\phi = 0.65 + (0.0032 - 0.002)\,250/3$$

$$\phi = 0.75.$$

Now

$$M_n = A_s f_y (d - a/2)$$

$$= \frac{5.06 \times 60}{12}(12 - 4.96/2)$$

$$M_n = 240.86 \text{ k-ft}.$$

$$\phi M_n = 0.75 \times 240.8 = 180.6 \text{ k-ft}.$$

$$\rho = \frac{A_s}{bd} = \frac{5.06}{18 \times 12} = 0.023 > \rho_{min} \text{ ok}$$

Q.No (01)

B. Design a doubly reinforced beam for $M_D = 154$ k-ft and $M_L = 410$ k-ft. If $f'_c = 4000$ Psi and $f_y = 60,000$ Psi

Soln:

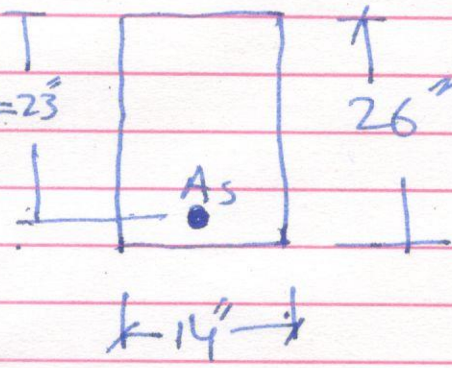
Assuming a 14" x 26" Beam size.

$F_y = 60,000$ Psi

$f'_c = 4000$ Psi

$M_D = 154$ k-ft

$M_L = 410$ k-ft



Factored Moment $= M_u = 1.2(M_D) + 1.6(M_L)$

$M_u = 1.2(154) + 1.6(410)$

$= 840.8$ k-ft

Assuming $f_s = f_y = 60,000$ Psi

$\rho_{max} = 0.0181$

$\Rightarrow 0.0181 = \dfrac{A_s}{14 \times 26} \Rightarrow A_s = 6.6\ in^2$

Try 3, #14 bars : $A_s = 6.75\ in^2$

Now $\rho = \dfrac{6.75}{14 \times 23} = 0.0209 > \rho_{max}$

Now, we have to check tensile steel strain which must be greater than yield strain i.e

$$\frac{\epsilon_s}{\cancel{(d-c)}} = \frac{0.003}{c}(d-c)$$

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{6.75 \times 60}{0.85 \times 4 \times 14} = 8.5 \text{ in}$$

$$c = \frac{a}{\beta_1} = \frac{8.5}{0.85} = 10 \text{ in}$$

($\beta_1 = 0.85$ for $f_c' = 4000$ psi).

$$\epsilon_s = \frac{0.003}{10}(\overset{d=23''}{23} - \overset{10}{10}) = 0.0039 > \epsilon_y$$

$$= 0.0039 > \epsilon_y$$

~~So $\phi = 0.90$~~

$$\cancel{\phi M_n = \phi \times A_s \times f_y (d - a/2)}$$

$$\cancel{= \frac{0.9 \times 6.75 \times 60 (23 - 8.5/2)}{12}} =$$

Date:

$\phi M_n \geq M_u$

$\Rightarrow \phi M_{n_c} = 334.55$ k-ft

$M_{n_c} = \frac{334.55}{0.8} = 418.2$ k-ft.

Now

$418.2 = A_{sc} * f_y * (d - d')$

Assuming $\epsilon_{sc} = \epsilon_y \Rightarrow f_{sc} = f_y$.

$418.2 = A_{sc} * 60 * (23 - 3)$

$A_{sc} = 0.35 \text{ in}^2$.

Assuming 2, #4 i.e $A_{sc} = 0.39 \text{ in}^2$.

A_{sc} = Area of steel required in compression zone of the beam.

Now checking strain in compression steel under ultimate moment.

As $\epsilon_s > \epsilon_y$. Hence, tension Controlled.
Strength reduction factor will be

$$\phi = 0.65 + (0.0039 - 0.002)\,250/3$$

$$\phi = 0.8$$

Now, Nominal moment Capacity of 3, #14 bars i.e

$$\phi M_n = 0.8 * \frac{6.75 * 60 *}{12}\left(23 - 8.5/2\right)$$

$$\phi M_n = 506.25 \text{ K-ft.} \quad — \quad A$$

Given Factored Moment is

$$= 840.8 \text{ K-ft.}$$

As $M_u = 840.8 > \phi M_n = 506.25$

Compression steel provision is mandatory to counter for Balance factored Moment i.e

$$= 840.8 - 506.25$$

$$M_{u_{balance}} = 334.55 \text{ K-ft.}$$

Now checking ~~strain~~ in tensile & Compressive steel at ultimate load of the designed section.

i.e $c = 10$, $a = 8.5$, $A_{sc} = 4.5 \text{ in}^2$

$A_{st} = 6.75 \text{ in}^2$.

$d = 20.75$ & $d' = 3$.

$$\epsilon_{st} = \frac{0.003}{c}(d-c) = \frac{0.003}{10}(20.75-10)$$

$\epsilon_{st} = 0.003225 > \epsilon_y$

Now $\phi = 0.65 + (\epsilon_{st} - 0.002) 250/3$

$\phi = 0.75$

Earlier From Equation Ⓐ,

$\phi M_n = 0.75 * 632.8125 = 474.6 \text{ k-ft}$.

Balanced Factored Moment is

$840.8 - 474.6 = 366.2 \text{ k-ft}$.

$\Rightarrow \phi M_{nc} \geq 366.2 \text{ k-ft}$

$\Rightarrow M_{nc} = 488.3 \text{ k-ft}$

$\phi M_n \geq M_u$

$\Rightarrow M_n = \dfrac{M_u}{\phi} = \dfrac{334.55}{0.8} = 418.2\ k\text{-}ft.$

Now Assuming $\epsilon_{sc} > \epsilon_y$ & $f_{sc} = f_y$

ϵ_{sc} = Strain in Compression steel

f_{sc} = Stress in Compression steel

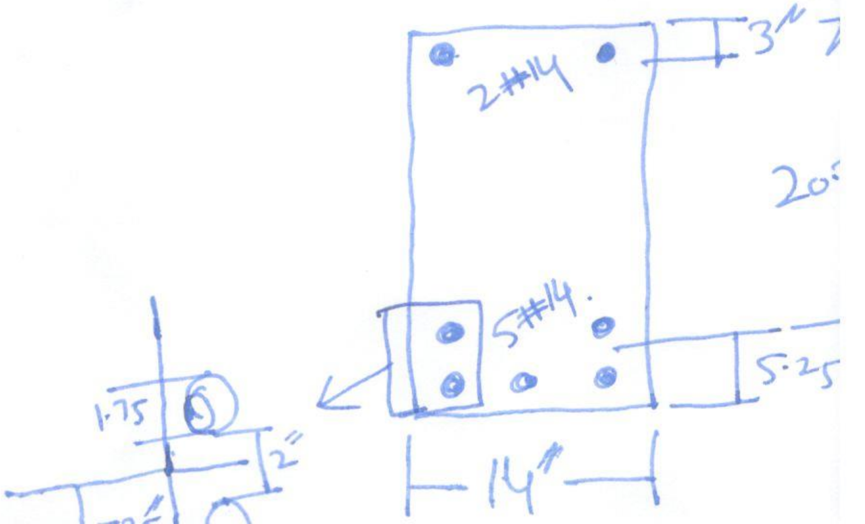
Now

$418.2 \times 12 = A_{sc} * 60 * (23-3)$

$\Rightarrow A_{sc} = 4.182\ in^2.$

Try 2, #14 bars $A_{sc} = 4.5\ in^2.$

Now designed Section is

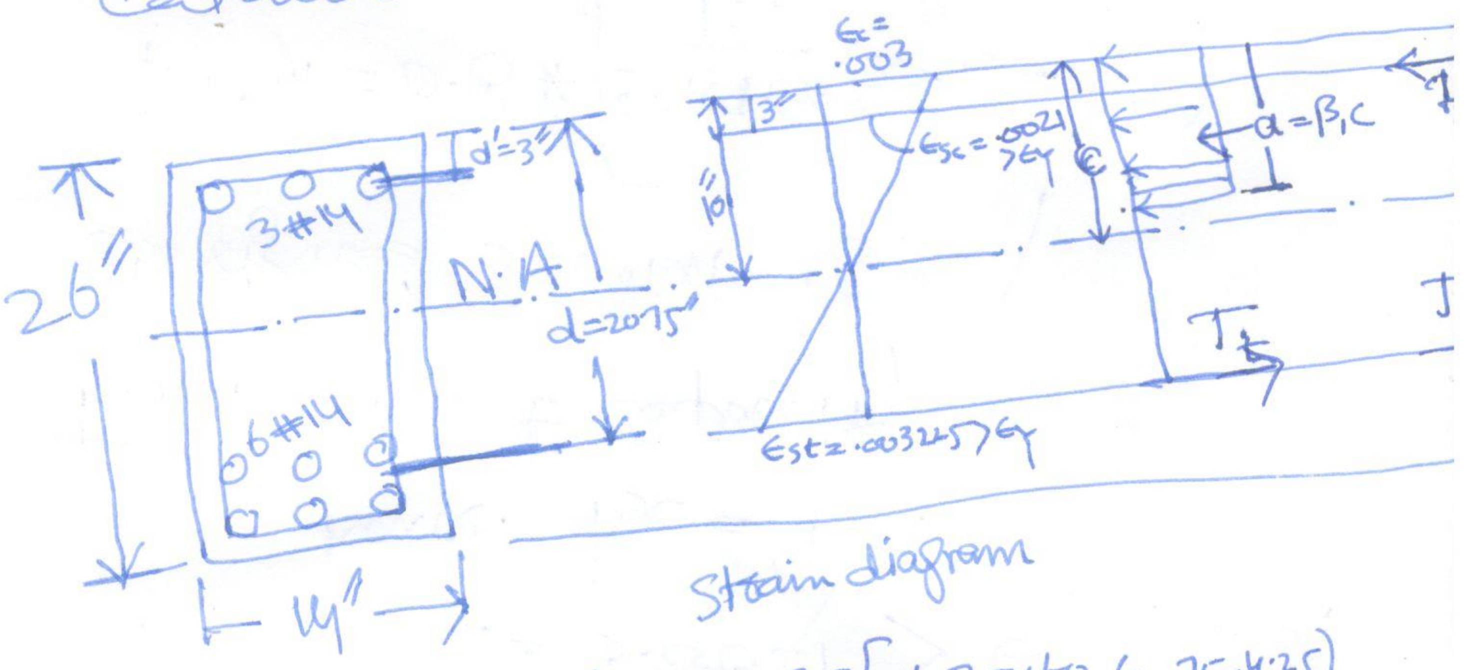


Now

$$488.3 * 12 = A_{sc} * f_y (d - d')$$

$$\Rightarrow A_{sc} = \cancel{1 in^2} \; 5.5 in^2.$$

Try 3 #14 above as well to counter for extra cracked moment. So the Final design section is alongwith calculated values.



$$\text{Strength Capacity} = 0.75\left[\frac{6.75 \times 60}{12}(20.75 - 4.25) + \frac{6.75 \times 60}{12}(20.75 - 3\right.$$

$$\phi M_n = 866.9 k\text{-}ft$$

$$\phi M_n > M_u.$$

So, strength OK and behaviour ... is tension

Date——

Q.NO (02):

Design a square column for given conditions:

$P_u = 154\,K$, $M_u = 15\,K\text{-}ft.$

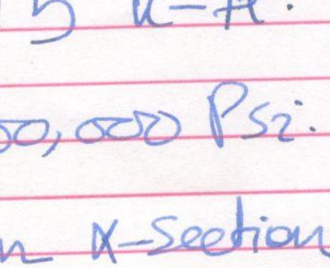
$f'_c = 4000\,Psi$, $f_y = 60{,}000\,Psi.$

For assumption of column X-section,

$$154 = (0.6)(4)(A_g)$$

$$\Rightarrow A_g = 64.2\ in^2$$

Assume 12" x 12" Square column. with reinforcement of all four faces i.e



Now, Assuming $\phi = 0.65$

(Strain in tensile steel $< \epsilon_y$)

$$P_n = \frac{P_u}{\phi} = \frac{154}{0.65} = 237\,K$$

$$M_n = \frac{M_u}{\phi} = \frac{15}{0.65} = 23.1\,K\text{-}ft.$$

For assumption of Reinforcement, using interaction diagram i.e

$$K_n = \frac{P_n}{f_c' A_g} = \frac{237}{4 \times 144} = 0.41$$

$$R_n = \frac{P_n}{f_c' A_g} \left(\frac{e}{h}\right) \Rightarrow$$

$$e = \frac{M_u}{P_u} = \frac{15 \times 12}{154} = 1.169 \text{ in}$$

$$\Rightarrow R_n = 0.41 \left(\frac{1.169}{12}\right) = 0.04$$

Now $\gamma = \frac{7}{12} = 0.58$ say 0.6

From Appendix A, Graph 6, interaction diagram for tied columns with bars on all four faces,

Point on graph is below 1% of steel area, so assuming

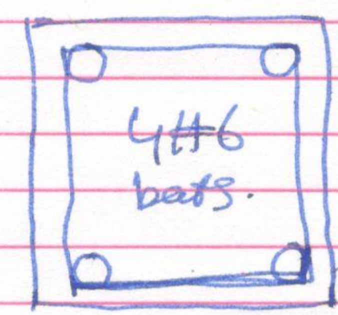
$$\rho = 0.01$$

⇒ $A_s = 0.01 * 12 \times 12 = 1.44\ in^2$.

Try 4, #6 bars,

$A_s = 1.77\ in^2$.

So, the designed section is



Checking design strength of Column:

~~A~~ Assuming 0.003 concrete strain at ultimate loading, then from strain diagram.

$\epsilon_s' = \frac{0.003}{7.2} * 4.7 = 0.00196$

⇒ $f_s' = (0.00196)(29\times10^3)(0.88)$

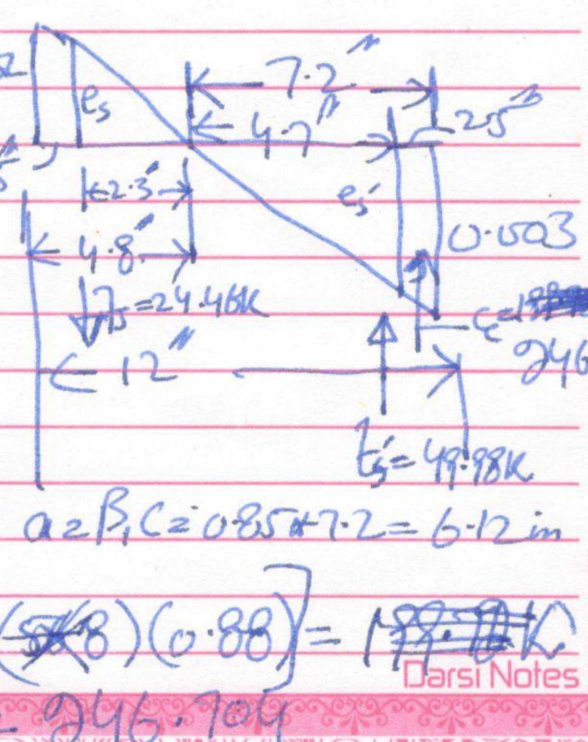
$f_s' = 49.98K$

⇒ $f = 56.8\ ksi$

$\epsilon_s = \frac{.002}{4.8} * (2.3) = 0.00096$

⇒ $f_s' = (0.00096)(29\times10^3)(0.88)$

$f_s' = 24.46K$



$a = \beta_1 c = 0.85 * 7.2 = 6.12\ in$

$C_c = 0.85 f_c'[(6.12)(12) - (8)(0.88)] = \cancel{177.6}K$

$= 246.704$

$\Sigma V = 0 \Rightarrow$

$P_n = 246.7 + 49.9 - 24.46$

$= 272.14\ K$

$\phi P_n = 0.65 * P_n = 176.9\ K > P_u$ OK.

$\cancel{M_n = P_n}$

$\Sigma M = 0$ at tensile steel $\Rightarrow$

$M_n = +(246.7 * \overset{8.42}{\cancel{6.9}}) + 49.9(6.9)$

$\cancel{+} - 272.14 * 2.3$

$= 149.6\ k\text{-}ft > M_u$

OK.

Q.No (03)

A:

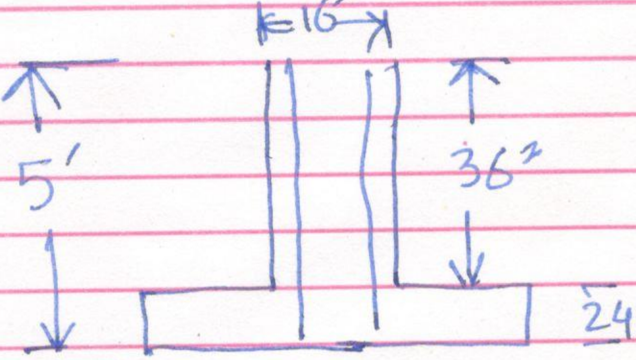
Design of Square Footing for given data:

$P_D = 154K$, $P_L = 160k$

$\gamma_{soil} = 100\ lb/ft^3$, $f_y = 60{,}000$ Psi

$f_c' = 3000$ Psi

Allowable Earth pressure $= q_a = 1540\ lb/ft^2$



Assuming following data

Normal weight concrete unit weight $= \gamma_c = 150\ lb/ft^3$

Footing depth = 2ft = 24"

Effective soil pressure $= q_e$

$$\Rightarrow q_e = 1540 - \left(\frac{24}{12} \times 150\right) - \left(\frac{36}{12} \times 100\right)$$

$$q_e = 940\ lb/ft^2 = 0.94\ k/ft^2.$$

Date______

Step 2: Area of Footing $= \dfrac{P_D + P_L}{q_e}$

$$= \frac{154+160}{0.94} = 334.04\ ft^2$$

Try (use) 18' × 18' Footing Area

$$= 324\ ft^2.$$

Step 3: Ultimate Bearing capacity $= q_u$

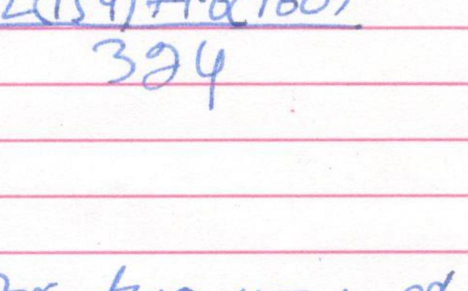
$$q_u = \frac{1.2\,P_D + 1.6\,P_L}{\text{Footing Area}} = \frac{1.2(154)+1.6(160)}{324}$$

$$q_u = 1.36\ k/ft^2.$$

Step 4: Depth Required for two way or Punching Shear:

The 'd' required for two-way Shear is the largest value obtained from the following expressions:

(i) $$d = \frac{V_{u2}}{\phi\, 4\sqrt{f'_c}\, b_o}$$



Eq

(ii) $$d = \frac{V_{u2}}{\phi\left(\frac{\alpha_s d}{b_o}+2\right)\sqrt{f'_c}\cdot b_o}$$

$\Rightarrow$ b_0 = Perimeter around Punching Area

i.e $b_0 = 4 * 35.5 = 142$ in

$\alpha_s = 40$ for interior column.
(Four sided perimeter).

V_{u_2} = Hatched Area in figure * q_u

$$= \left(324 - \frac{35.5 \times 35.5}{12 \times 12}\right) * 1.36$$

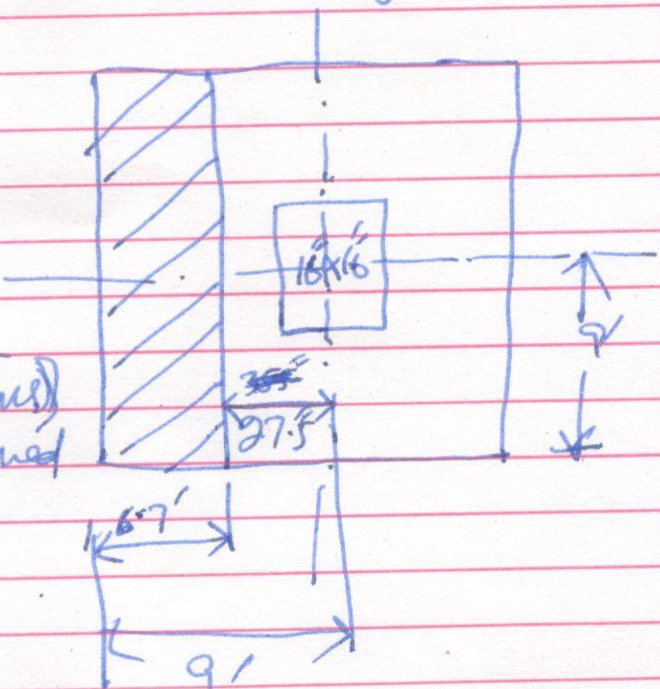
$V_{u_2} = 428.74$ K $= 428740$ lbs

(i)
$$d = \frac{428740}{0.75 * 4\sqrt{3000}\, 142} = 18.4'' < 19.5''$$
O.K.

(ii)
$$d = \frac{428740}{0.75\left(\frac{40 \times 19.5}{142} + 2\right)\sqrt{3000} * 142} = 9.8'' < 19.5'' \text{ OK.}$$

So, for Punching Shear, Footing depth is OK.

Step 5: Depth required for on-way Shear



$V_{u_1} = (18 \times 6.7) * 1.36$

$= 164\ K$

(Because of overlapping of Main Bars (in both directions) clear cover has been assumed as 4.5").

Required Depth $= d = \dfrac{V_u}{\phi 2 \sqrt{f'c}\, b_w}$

$$d = \frac{164 * 1000}{0.75 \sqrt{3000} * 18 * 12} = 18.5" < 19.5" \text{ ok}$$

Ultimate moment @ Face of the Column:
i.e

$$M_u = (9 \times 8.33) * 1.36 * \frac{8.33}{2} = 425\ k\text{-}ft$$

$$\frac{M_u}{\phi b d^2} = \frac{425 * 1000 * 12}{0.9 * (18 \times 12)(19.5)^2} = 69\ psi$$

Date

From Appendix A; Table A 12;

$\rho_{min} = 0.0033$ and $\frac{M_u}{\phi b d^2} = 190.3$ Psi

So, Considering $\rho = 0.0033$

$A_s = 0.0033 * 18 \times 12 * 18.5 = 13.9 \ in^2$.

From Table A.4:

Use 9 #11 bars i.e (in both directions).

$A_s = 14.06 \ in^2$.

Development Length:

$$\frac{l_d}{d_b} = \frac{3}{40} \cdot \frac{f_y}{\lambda\sqrt{f_c'}} \cdot \frac{\psi_t \cdot \psi_e \cdot \psi_s}{\frac{c_b}{d_b}} \longrightarrow (1)$$

if $\frac{c_b}{d_b} > 2.5$, then use $\frac{c_b}{d_b} = 2.5$

C_b = Side Cover = 3.5"

$\Rightarrow \frac{3.5}{1\frac{1}{8}} = 2.54$

so use $\frac{c_b}{d_b} = 2.5$

Using Equation (1) i.e

$$\frac{l_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{1\sqrt{3000}} \times \frac{1 \times 1 \times 1}{2.5}$$

$$= 32.86$$

$$\frac{l_d}{d_b} * \left(\frac{13.9}{14.06}\right) \Rightarrow$$

$$32.86 * 0.988 \Rightarrow$$

$$\frac{l_d}{d_b} = 32.48$$

⇒ Development length $= l_d = 44.6''$

Say $\boxed{45''}$.

Available $= (9 \times 12) - (8) - (3.5)$

$= 84.5'' > l_d$ o.k