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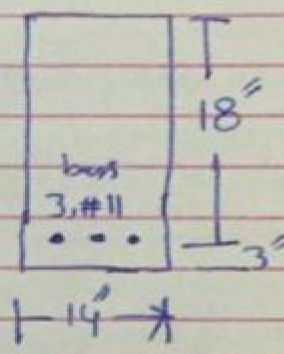
SUBJECT	Advance Design of reinforced concrete structures
PROGRAMME	M.S (STRUCURE ENGINNERING)

ANSWERS

Q.No (01):
(A).

Section #(i)

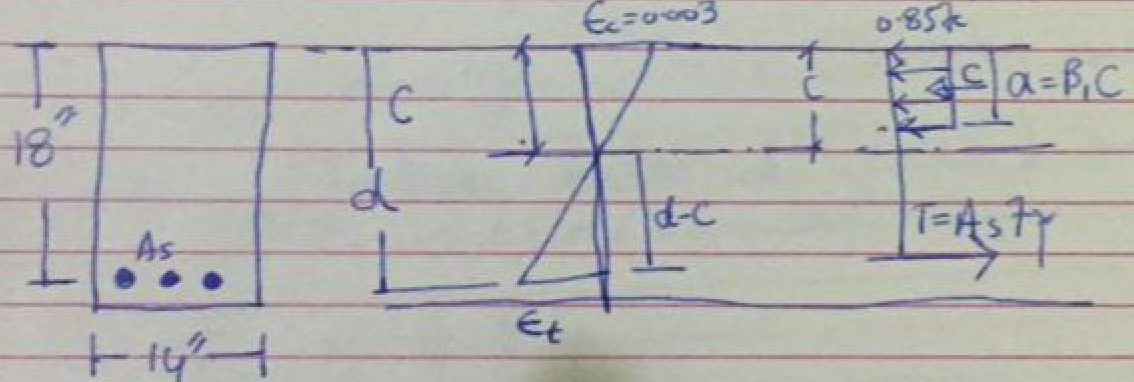


ϵ_t, ϕ & $\phi M_n = ?$

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

From Appendix A , Tabele A.4

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



$$C = T$$

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

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

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

So

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

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)

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

$$\phi = 0.84$$

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

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

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

Applying strength reduction factor, ϕ

DARSI NOTES

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

Now

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

From Appendix A, Table A-7;

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

$$\rho_{min} = 0.0028$$

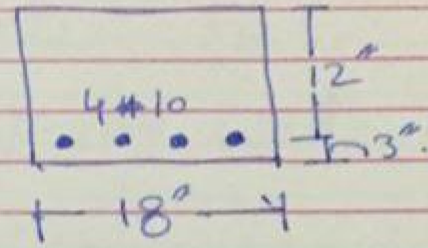
So,

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

$$\rho > \rho_{min}.$$

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.

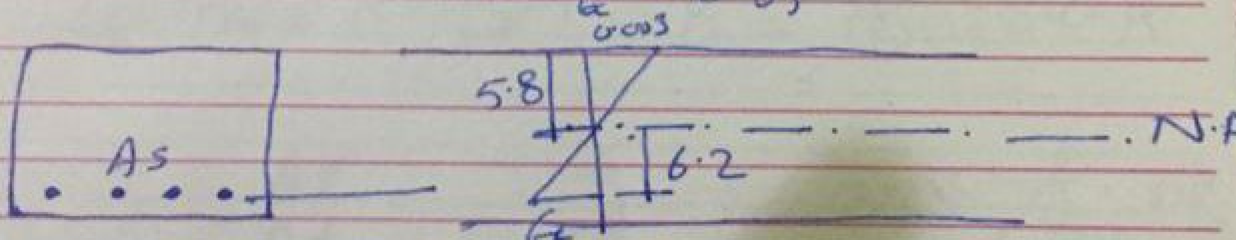
ϵ_t, ϕ and $\phi M_n = ?$

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

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

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

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



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

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

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

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

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

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

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

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:

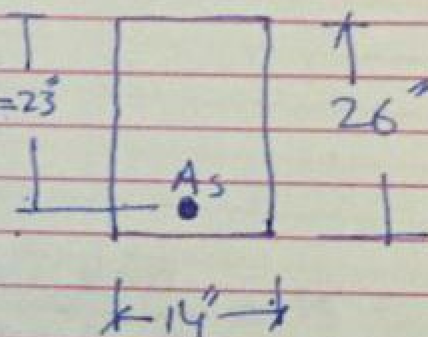
Assuming a 14" x 26" Beam size.

$F_y = 60{,}000$ Psi

$f_c' = 4000$ Psi

$M_D = 154$ k-ft

$M_L = 410$ k-ft.



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

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

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

Assuming $f_s = f_y = 60{,}000$ Psi

$$\rho_{max} = 0.0181$$

$$\Rightarrow 0.0181 = \frac{A_s}{14 \times 26} \Rightarrow A_s = 6.6 \text{ in}^2.$$

Try 3, #14 bars : $A_s = 6.75 \text{ in}^2$.

DARSI NOTES

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

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

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

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

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

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

$d = 23$

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

$$= 0.0039 > \epsilon_y$$

~~So $\phi = 0.90$~~

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

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

$\phi M_n \geq M_u$

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

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

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.

Date:

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

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

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

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

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

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

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

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

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

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

$\phi = 0.75$

Earlier From Equation (A),

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

Balanced Factored Moment is

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

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

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

Now

$\Phi M_n \geqslant M_u$

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

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

ϵ_{sc} = Strain in Compression steel

f_{sc} = Stress in Compression steel

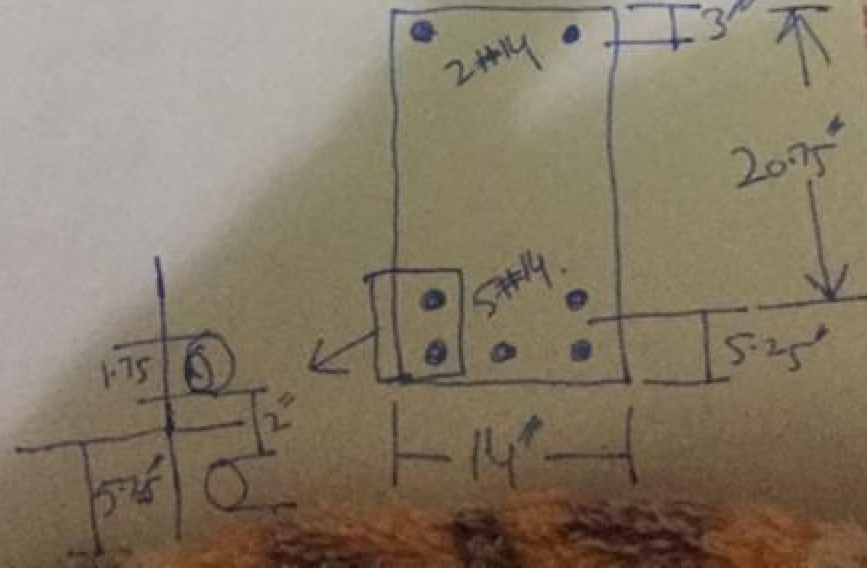
Now

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

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

Try 2, #14 bars $A_{sc} = 4.5$ in².

Now designed Section is



From Appendix A; Table A 12;

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

So, Considering $\rho = 0.0033$

$A_s = 0.0033 \times 18 \times 12 \times 18.5 = 13.9 \text{ in}^2$

From Table A.4:

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

$A_s = 14.06 \text{ in}^2$.

Development Length:

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

If $\frac{c_b}{d_b} > 2.5$, then use $\frac{c_b}{d_b} = 2$

c_b = Side Cover $= 3.5''$

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

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

$\Sigma V = 0 \Rightarrow$

$P_n = 246.7 + 49.9 - 24.46$

$= 272.14\ K$

$\phi P_n = 0.65 * P_n = 176.9\,K > P_u$

~~$M_n = P_n$~~

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

$M_n = +(246.7 * 8.42) + 49.9(6.9)$

$- 272.14 * 2.3$

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

ok.

Date

Q.NO (02):

Design a square Column for given Conditions:

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

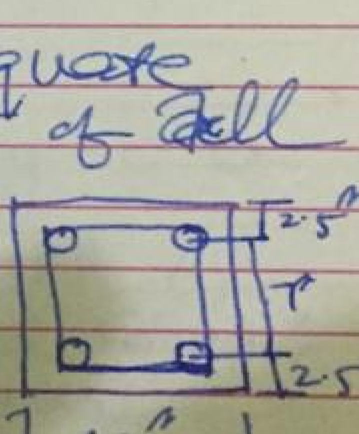
$f_c' = 4000\ Psi$, $f_y = 60{,}000\ Psi.$

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 < ϵ_y)

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

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

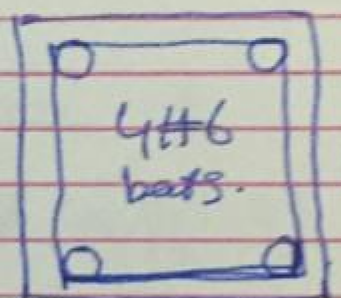
Date

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

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, from strain diagram.

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

⇒ $f_s' = (0.00196)(29 \times 10^3)(0.88)$

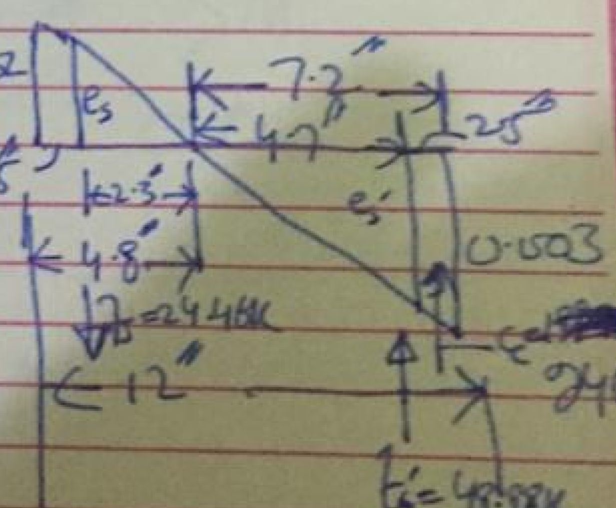
$f_s' = 49.98 K$

⇒ $f = 56.8 \, ksi$

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

⇒ $f_s' = (0.00096)(29 \times 10^3)(0.88)$

$f_s' = 24.46 K$



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

$C_c = 0.85 f_c [(6.12)(12) - (8)(0.88)] =$ [illegible] K

Q.No (03)

A:

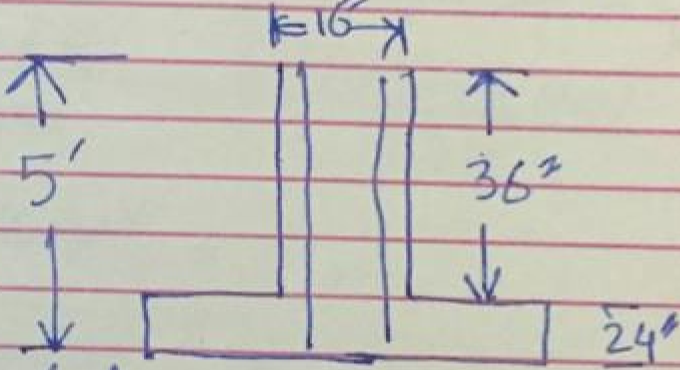
Design of Square Footing for given data:

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

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

$f'_c = 3000\ Psi$

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:

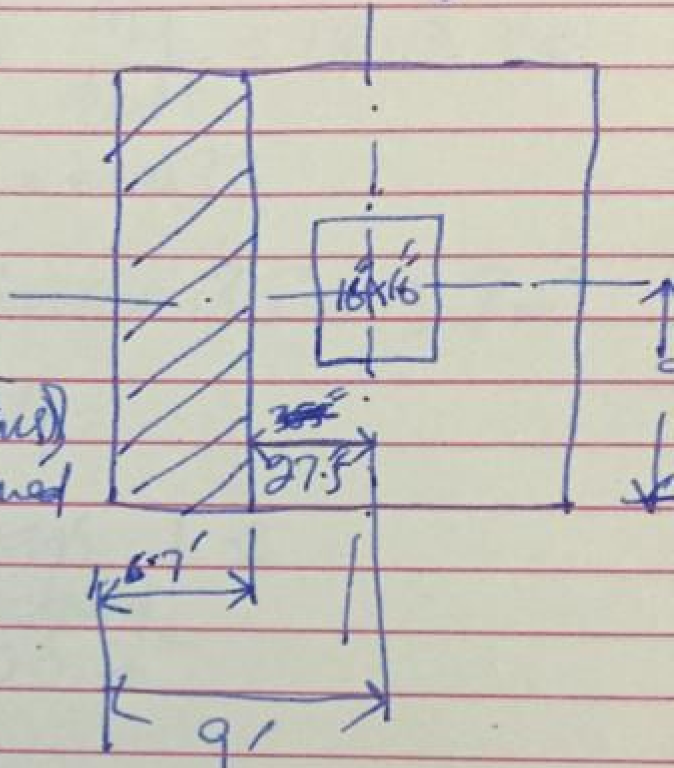
The 'd' required for two-way shear is the largest value obtained from the following expressions.

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

&

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

Step 5: Depth required for one-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 $= d = \dfrac{V_u}{\Phi 2 \sqrt{f'c}\, b_w}$

$d = \dfrac{164 * 1000}{0.75 \sqrt{3000} * 18 * 12} = 18.5'' < 19.5$

Ultimate moment @ Face of the Column i.e

$M_u = (9 \times 8.33) * 1.36 * \dfrac{8.33}{2} = 425\ k$

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