Name	Nauman Khan
ID	6829
Department	Btech (civil)
Subject	RCD
Date	28/09/2020
Submitted to	Engr. Humair Arshad

QNo1

Given data:

⇒ $W_D = 0.75$ Kip/ft

⇒ $W_L = 0.75$ Kip/ft

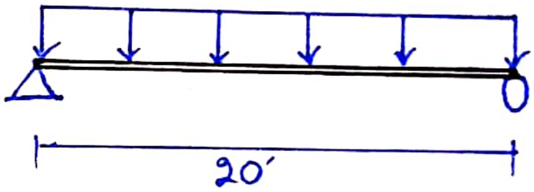
⇒ $f'_c = 3$ Ksi

⇒ $f_y = 40$ Ksi

Required data:

⇒ Design Beam

Solution:



Step No1: Sizes

⇒ for 20' length, $h_{min} = \cancel{15"} \dfrac{l}{16} = \dfrac{20\times12}{16} = 15"$

⇒ For grade 40, we have = $h_{min} = 15" \times \left(0.4 + \dfrac{40{,}000}{100{,}000}\right)$

$= 12"$

However we select 18" deep beam

Generally the minimum beam width is 12",

Therefore, width of the beam is taken as '12")

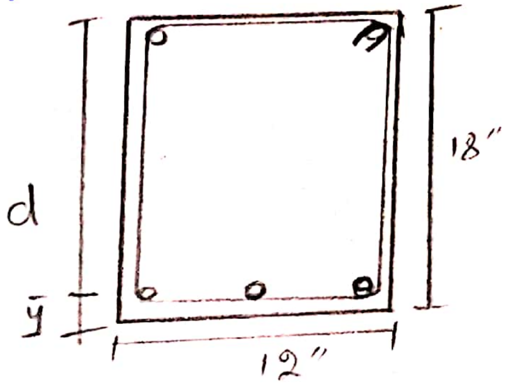
⇒ The final selection of beam size depend on several factors specially the availability of form work.

⇒ Depth of beam $= h = 18''$

⇒ $h = d + \bar{y}$; $\bar{y}$ is usually taken from 2.5 to 3"

⇒ For $\bar{y} = 2.5$ inch; $d = 18 - 2.5 = 15.5''$

⇒ Width of beam Cross Section $(b_w) = 12''$



Step No 2 : Loads

⇒ Self weight of beam $= \gamma_c b_w h$

$$= 0.15 \times \left(12 \times \frac{18}{144}\right) = 0.225 \text{ Kips/ft}$$

⇒ $W_u = 1.2 W_D + 1.6 W_L$

$$= 1.2 \times (0.225 + 0.75) + 1.6(0.75)$$

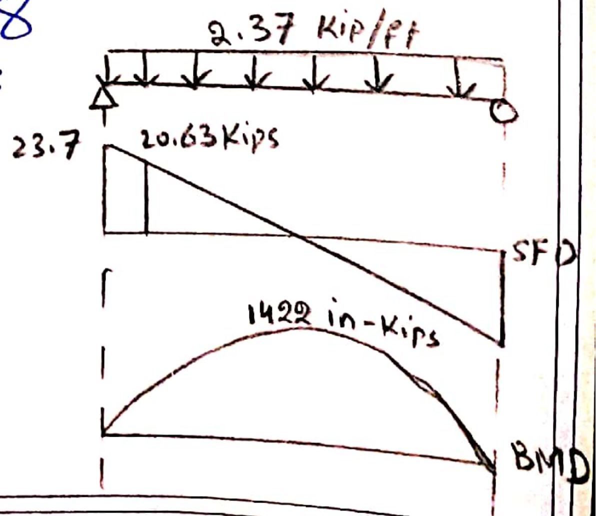
$$= 2.37 \text{ Kips/ft}$$

Step No 3 Analysis;

⇒ Flexural Analysis:

$$M_u = \frac{W_u l^2}{8} = \frac{2.37 \times (20)^2 \times 12}{8} = 1422 \text{ in-Kips}$$

⇒ Analysis for Shear in Beam:



$V = 23.7$ Kips

To find V_u at a distance "d" from face of Support, $d = 15.5'' = 1.29'$

⇒ $V_u/(10 - 1.29) = 23.7/10$

$$\Rightarrow V_u = \frac{23.7 \times (10 - 1.29)}{10}$$

$$\Rightarrow \boxed{V_u = 20.63 \text{ Kips}}$$

Step No 4: Design

$\Rightarrow$ Design for ~~flexural~~ flexure:

$\Rightarrow \phi M_n \geq M_u$ (ϕM_n is M_{design} or $M_{capacity}$)

$\Rightarrow$ For $\phi M_n = M_u$

$$\Rightarrow \phi A_s f_y (d - a/2) = M_u$$

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

Now As find by trial and Success Method.

1st trial:

$\Rightarrow$ Assum $a = 4''$

$$\Rightarrow A_s = \frac{1422}{0.9 \times 40 \times [15.5 - 4/2]}$$

$$= 2.92 \text{ in}^2$$

$$\Rightarrow a = A_s f_y / (0.85 f_c' b_w)$$

$$= \frac{2.92 \times 40}{0.85 \times 3 \times 12} = 3.81 \text{ inches}$$

2nd trial:

$$A_s = \frac{1422}{0.9 \times 40 (15.5 - \frac{3.81}{2})} = 2.90 \text{ in}^2$$

$$a = \frac{2.9 \times 40}{0.85 \times 3 \times 12} = 3.79 \text{ inch}$$

3rd trial:

$$\Rightarrow A_s = \frac{1422}{0.9\times 40(15.5 - \frac{3.79}{2})} = 2.90\ in^2$$

$$\Rightarrow a = \frac{2.90 \times 40}{0.85\times 3\times 12} = 3.79\ in$$

Close enough to previous value of "a"

So that $A_s = 2.90\ in^2$ is OK.

Now; Check for maximum and minimum reinforcement allowed by ACI:

$$\Rightarrow A_{s_{min}} = 3\left(\sqrt{\frac{f_c'}{f_y}}\right) b_w d \geq \left(\frac{200}{f_y}\right) b_w . d$$

$$\Rightarrow 3\left(\sqrt{\frac{f_c'}{f_y}}\right) b_w d = 3\times\sqrt{\frac{3000}{40000}} \times 12\times 15.5 = 0.744\ in^2$$

$$\Rightarrow \left(\frac{200}{f_y}\right) b_w d = \left(\frac{200}{40000}\right)\times 12\times 15.5 = 0.93\ in^2$$

$$\Rightarrow \boxed{A_{s_{min}} = 0.93\ in^2}$$

Now $A_{s_{max}} = 0.27\left(\frac{f_c'}{f_y}\right) b_w d = 0.27\times\left(\frac{3}{40}\right)\times 12\times 15.5$

$$= 3.76\ in^2$$

Now $A_{s_{min}}(0.93) < A_s(2.90) < A_{s_{max}}(3.76) \rightarrow OK.$

Now; Bar placement: 5 #7 bars will provide 3.0 in^2 of steel area which is slightly greater than required

Other option can be explored. For example

- ★ 7 #6 bar (3.08 in^2)
- ★ 4 #8 bar (3.16 in^2)

or Combination of two different size bars.

Design for Shear:

$\Rightarrow V_u = 20.63$ Kips

$\Rightarrow \phi V_c =$ (capacity of concrete in Shear) $= \phi 2\sqrt{f_c'}\, b_w d$

$$= \frac{0.75 \times 2 \times \sqrt{3000} \times 12 \times 15.5}{1000} = 15.28\text{ K}$$

As $\phi V_c < V_u$, Shear Reinforcement is required.

$\Rightarrow$ Assuming #3, 2 legged (0.22 in^2), vertical Stirrups.

$\Rightarrow$ Spacing required $(S_d) = \dfrac{\phi A_v f_y d}{V_u - \phi V_c}$

$$= \frac{0.75 \times 0.22 \times 40 \times 15.5}{20.63 - 15.28}$$

$$\simeq 19.12''$$

=> Maximum Spacing and minimum reinforcement requirement as permitted by ACI is minimum of:

$$\Rightarrow S_{max} = \frac{A_v f_y}{50 b_w} = \frac{0.22 \times 40000}{50 \times 12}$$

$$\Rightarrow S_{max} = 14.66''$$

and

$$S_{max} = \frac{d}{2} = \frac{15.5}{2} = 7.75''$$

Also

$$\Rightarrow S_{max} = 24''$$

$$\Rightarrow \frac{A_v f_y}{0.75\sqrt{f_c'}\, b_w} = \frac{0.22 \times 40000}{0.75 \times \sqrt{3000} \times 12} = 17.85''$$

=> Therefore $S_{max} = 7.75''$

=> <u>Other checks:</u>

$$\Rightarrow \phi V_s \leq \phi 8\sqrt{f_c'}\, b_w d$$

$$\Rightarrow \phi 8\sqrt{f_c'}\, b_w d = 0.75 \times 8\sqrt{3000} \times 12 \times \frac{15.5}{1000} = 61.12K$$

$$\Rightarrow \phi V_s = V_u - \phi V_c = 20.63 - 15.28 = 5.35 < 61.12$$

$\longrightarrow$ OK

Therefore depth is ok, If not increase the depth of Beam.

⇒ Check if " $\phi V_s \leq \phi 4 \sqrt{f_c'}\, b_w d$

⇒ $\phi 4 \sqrt{f_c'}\, b_w d = 0.75 \times 4 \times \sqrt{3000} \times 12 \times \frac{15.5}{1000}$

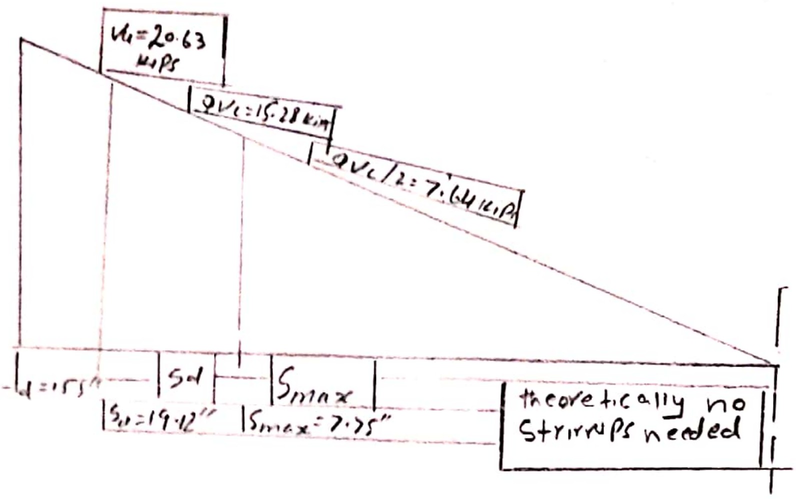
$= 30.56\ K$.

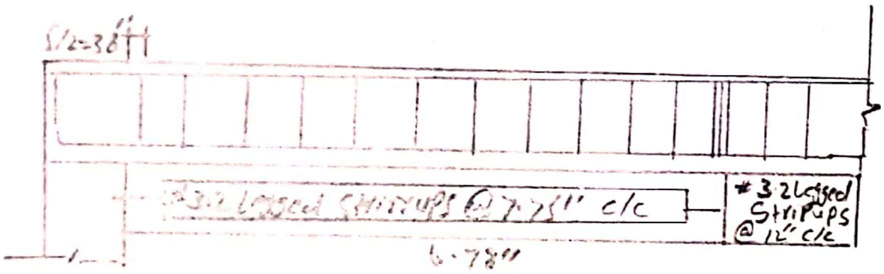
⇒ $5.35K < 30.56K$ ⟶ OK

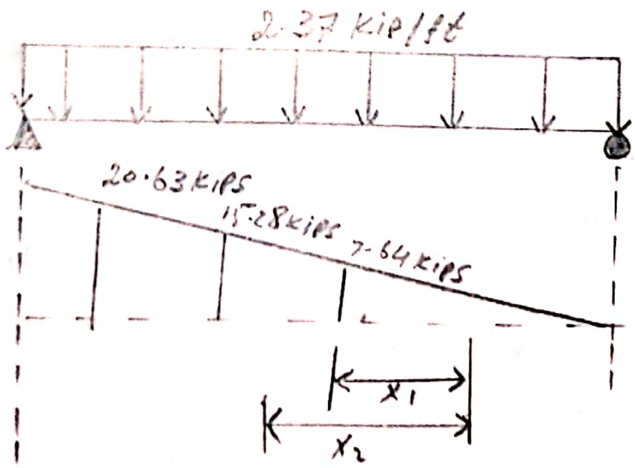
⇒ $\phi V_s \leq \phi 4 \sqrt{f_c'}\, b_w d$, the maximum spacing ($S_{max}$) is OK.

otherwise Reduce spacing by half.

⇒ Step 05: Drafting (Shear Reinforcement)



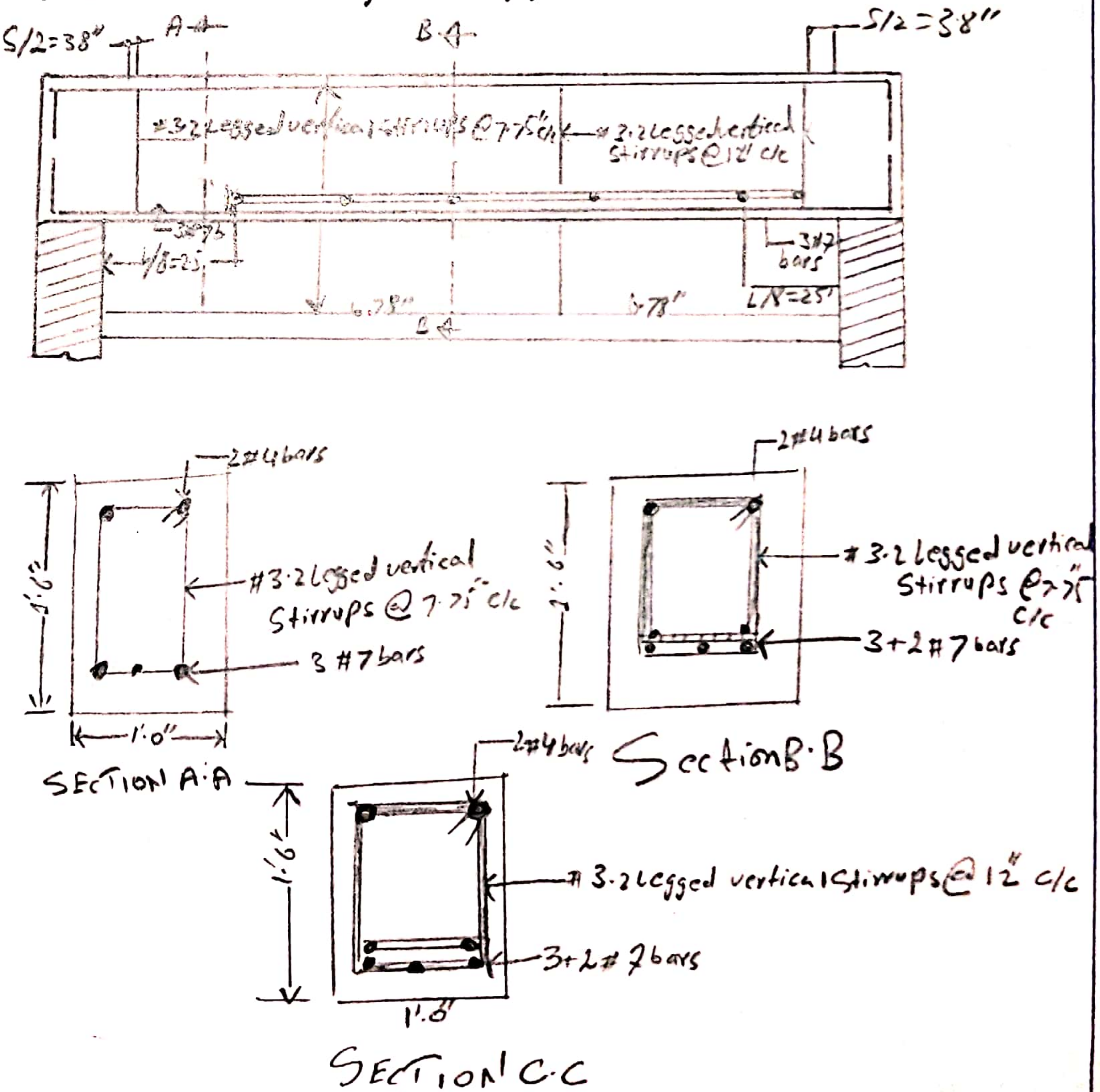




Note:

As S_d < S_{max1} we will provide S_{max} from the support up to 6-78 ft Beyond this point theoretically no reinforcement is required however, we will provide # 3 2-legged Stirrups @ 12 in c/c.

=> Drafting (Flexural Reinforcement):



Q.No2

Given data:

=> Thickness of Concrete wall = 12 inch

=> D. Load = 10 Kip/ft

=> L. Load = 12.5 Kip/ft

=> allowable bearing Capacity = q_a = 5000 psf

=> f_c' = 3500 psi

=> f_y = 60,000 psi

Required: => density of soil = 120 lb/ft³

=> Design Wall footing

Solution:

Step#1: Estimate thickness of footing; h

=> Assuming a trial thickness, h = 12 in

=> Effective depth; d = 12 − 3 in. cover − ½ (bar dia)

≈ 8.5 inch

Step#2: Calculate weight of fill and weight of Concrete, W:

$$\Rightarrow W = w_{conc} + W_{fill} = (1 \times 0.15) + (4 \times 0.12) = 0.63 \text{ Ksf}$$

Step#3 Calculate effective bearing capacity, q_e

$$\Rightarrow q_e = q_a - W$$

$$\Rightarrow q_e = 5 - 0.63$$

$$\Rightarrow \boxed{q_e = 4.37 \text{ Ksf}}$$

Step No 4: Calculate bearing Area; A_{req}

$\Rightarrow A_{req} = \dfrac{\text{Service load}}{q_e}$

$\Rightarrow$ Service load $= 10 + 12.5 = 22.5$ Kip/ft

$\Rightarrow A_{req} = \dfrac{22.5}{4.37} = 5.15\ ft^2$ per foot of length

$\Rightarrow$ Trying a footing 5 ft 2 in wide.

Step 5: Calculate design pressure on base of footing due to factored loads, q_u:

$\Rightarrow q_u = \dfrac{\text{factored load}}{\text{Bearing area}}$

$\Rightarrow$ factored load $= 1.2(10) + 1.6(12.5)$

$= 32$ Kips

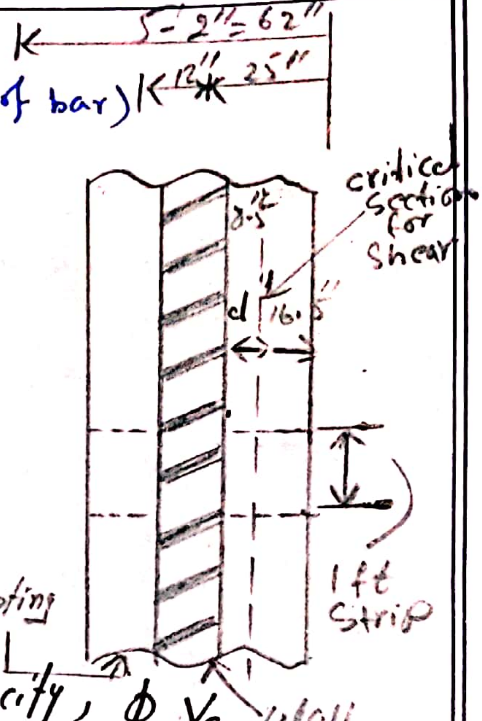
$\Rightarrow q_u = \dfrac{32}{5.17} = 6.19$ ksf

Step No 6: Calculate Critical Shear, V_u:

$\Rightarrow$ only one way shear is significant in wall footing, hence determining critical shear at a distance "d" from the face of support

$\Rightarrow d = 12 - 3\text{in cover} - \frac{1}{2}(\text{dia of bar})$

$\simeq 8.5$ in



$\Rightarrow V_u = q_{u} b(K-d)$

$\Rightarrow V_u = 6.19 \times 1 \frac{(25-8.5)}{12}$

$= 8.51$ Kips/ft.

Step#7: check the Shear Capacity, ϕV_c

$\Rightarrow$ check the thickness for Shear.

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

$= \frac{\{0.75 \times 2 \times \sqrt{3000} \times 12 \times 8.5\}}{1000}$

$\Rightarrow \phi V_c = 9.05$ Kips

$\Rightarrow$ Since $\phi V_c > V_u$, The footing depth is ok.

$\Rightarrow$ Using 12 in thick and 5 ft 2in wide footing

Step #8 Calculate Maximum moment, M_u

$\Rightarrow M_u = \frac{q_u b K^2}{2} = \frac{6.19 \times 1 \times (\frac{25}{12})^2}{2}$

$\Rightarrow M_u = 13.43$ ft-Kips per ft.

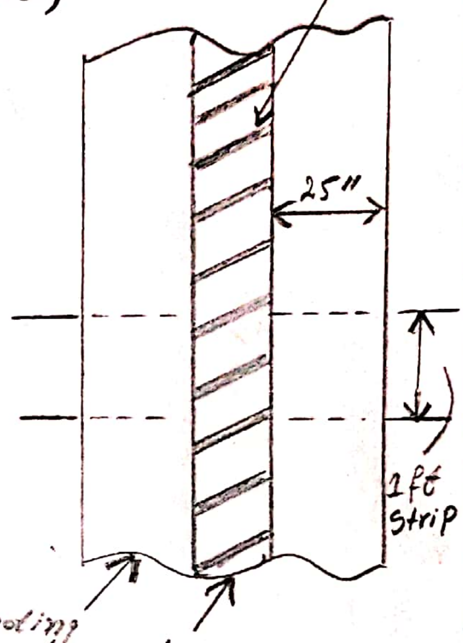
$\Rightarrow$ **Step # 09:** Calculate Steel Area;

$\Rightarrow$ Now using trial and Success Method for determining A_s;

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

$\Rightarrow A_s = 0.390\ in^2$ per foot

$\therefore a = 0.2h$



$\Rightarrow$ **Step # 10:** Minimum Reinforcement check.

$$\Rightarrow A_{s_{min}} = 0.0018\,bh = 0.0018 \times 12 \times 12 = 0.26\ in^2/ft$$

$\Rightarrow A_s(0.390\ in^2) > A_{s_{min}}(0.26\ in^2) \longrightarrow$ OK

$\Rightarrow$ **Step # 11:** Main Bars Spacing and Maximum Spacing check:

$\Rightarrow$ Main Bars: Spacing $= A_b \times \frac{12}{A_s}$

using #5 bars, Spacing $= 0.31 \times \frac{12}{0.390} = 9.53 \approx 9''$ c/c

=> Max Spacing = $3h$ or $18'' = 3(12) = 36''$ or $18''$

→ ok.

Step#12: Distribution Bar placement.

=> Distribution

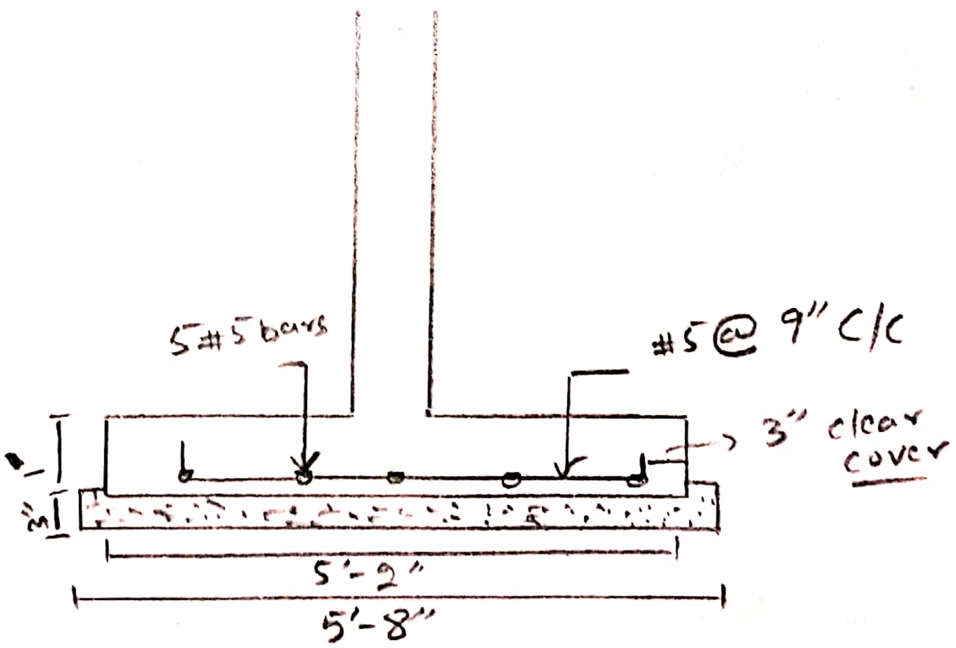
=> $A_{distt} = 0.0018\,Bh = 0.0018 \times 62 \times 12$

$= 1.34\ in^2$

=> No. of Bars $= \dfrac{A_{dist}}{A_{bar}} = \dfrac{1.34}{0.31} = 5$ bars.

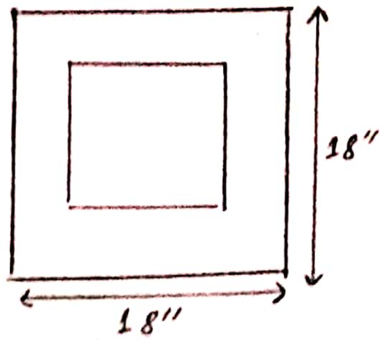
Step# 13

Drafting:



QNO3

Givendata



- ⇒ Compressive load = 300 Kips
- ⇒ $f_c' = 3$ Ksi
- ⇒ $f_y = 40$ Ksi
- ⇒ Cross section of tied Column = 18" × 18"

Solution:

⇒ Nomial Strength ($\alpha\phi P_n$) of axially Loaded column is:

$\Rightarrow \alpha\phi P_n = 0.80\phi\{0.85 f_c' (A_g - A_{st}) + A_{st} f_y\}$

$\Rightarrow A_g = 18 \times 18 = 324\ in^2$

$\Rightarrow$ Let $A_{st} = 1\%$ of $A_g = 0.01 \times 324 = 3.24$

$\Rightarrow \alpha\phi P_n = 0.80 \times 0.65 \times \{0.85 \times 3 \times (324 - 3.24) + 3.24 \times 40\}$

$= 492$ kips $> (P_u = 300$ kips$)$, ok.

therefore, $A_{st} = 0.01 \times 324 = 3.24\ in^2$

⇒ Main Bars:

⇒ using #6 bar, with bar area $A_b = 0.44 in^2$

⇒ No. of bars $= A_s / A_b = 3.24 / 0.44 = 7.36 \approx 8$ bars

⇒ Use 8 #6 bars

⇒ Tie Bars:

⇒ using #3 bar, with bar area $A_b = 0.11 in^2$

∴ Center-to-center Spacing shall not exceed The Least of:

(i) 16 d_b of Longitudinal bar = 16×0.75 = 12"

(ii) 48d_b of tie bar = 48 × 3/8 = 18"

(iii) Smallest dimension of member = 18"

therefore use #3 tie @ 12" c/c

End