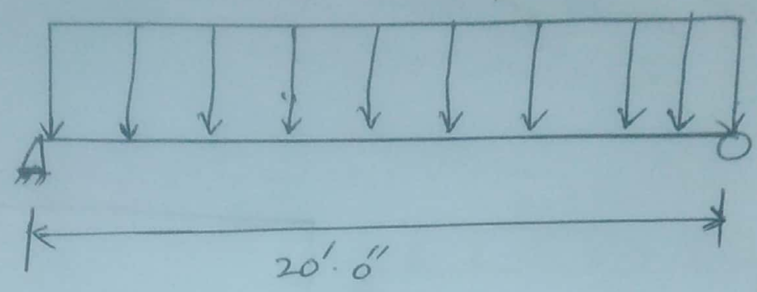
Question No 1.

Design the beam for shear and flexural stresses shown below as as ACI 318-14.

$W_D = 0.75$ kip/ft
$W_L = 0.75$ kips/ft



Take $f'_c = 3$ksi and $f_y = 40$ksi

SOLUTION.

Step No 1: For 20' length, $h_{min} = \frac{L}{16} = \frac{20 \times 12}{16} = 15''$

For grade 40, we have = $h_{min} = 15'' \times (0.4 + 40000/100000)$
$= 12''$.

This is the minimum requirement of the code for depth of beam. How ever we select 18" depth beam. Generally the minimum beam width is 12". therefore the width of the beam is taken 12".
The final selection of beam size depends on several factors specifically the availability of frame work.

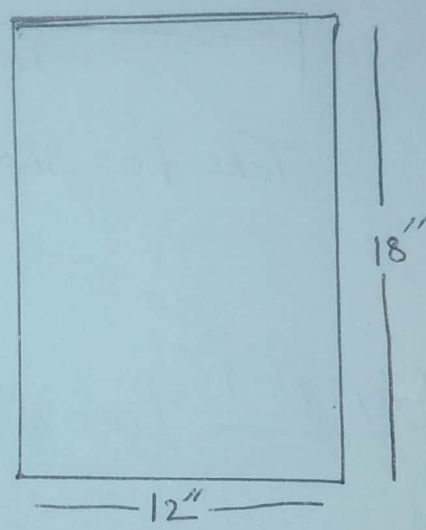
Depth of beam = h = 18"

h = d + y', y' is usually taken from 2.5 to 3.0 Inches

For y' = 2.5 in. d = 18 - 2.5 = 15.5".

width of beam Cross section = (bw) = 12".

In RCD, width of beam is usually denoted by bw.



Step No 2: Sizes

Self weight of beam =

$\gamma_c b_w h = 0.15 \times (12 \times 18/144) = 0.225$ kips/ft.

$W_u = 1.2 w_d + 1.6 w_L$.

$= 1.2 \times (0.225 + 0.75) + 1.6 \times 0.75 = 2.37$ kips/ft

Step No 3: Analysis:

Flexural Analysis.

$M_u = W_u l^2/8 = 2.37 \times (20)^2 \times 12/8 = 1422$ in-kips.

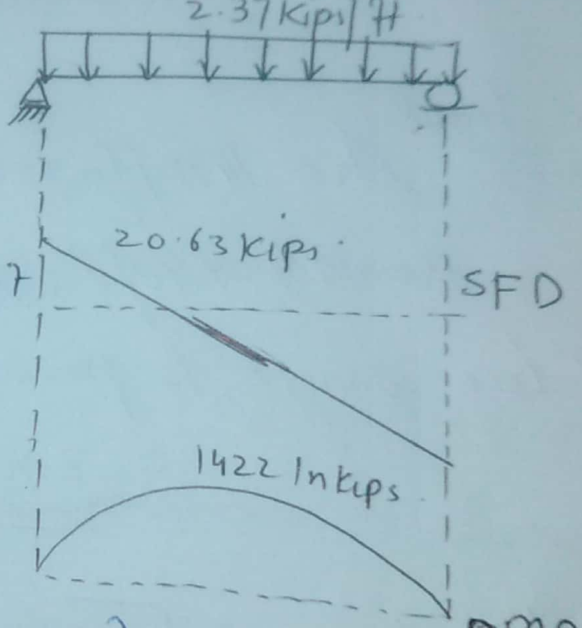
Analysis for shear in beam:

$V = 23.7$ kips

To find V_u at a distance 'd' from face of support,
$d = 15.5'' = 1.29'$
Using Similarity of triangles

$V_u / (10 - 1.29) = 23.7/10$

$V_u = 23.7 \times (10 - 1.29)/10 = 20.63K$



Step No 4: Design.

Design for flexure.

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

→ For $\phi M_n = M_u$

→ $\phi A_s f_y (d - a/2) = M_u$

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

→ Calculate, A_s, by trail and sucess method.

First trail:

Assume $a = 4''$

$A_s = 1422/[0.9 \times 40 \times \{15.5 - (4/2)\}] = 2.92 in^2$

$a = A_s f_y/(0.85 f'_c b_w)$

$= 2.92 \times 40/0.8 \times 3 \times 12 = 3.81$ inch

Second Trail,

$A_s = 1422/[0.9 \times 40 \times \{15.5 - (3.82/2)\}] = 2.90 inch^2$

$a = 2.90 \times 40/(0.85 \times 3 \times 12) = 3.79 in$

Third trail,

$A_s = 1422/[0.9 \times 40 \times \{15.5 - (3.79/2)\}] = 2.90 in^2$

$a = 4.49 \times 40/(0.85 \times 3 \times 12) = 3.78$ Inch

Close enough to previous values of (a) so that $A_s = 2.90 in^2$ ok

Check,

$A_{s min} = 3\sqrt{f'_c}/f_y) b_w d \geq (200/f_y) b_w d$

$3\sqrt{f'_c}/f_y) b_w d = (3 \times \sqrt{3000}/40000) b_w d =$

$= 0.004 \times 12 \times 15.5 = 0.744 in^2$

$= (200/f_y) b_w d = (200/40000) \times 12 \times 15.5 = 0.93 in^2$

$A_{s min} = 0.93 in^2$

$A_{s min} = 0.27 (f'_c/f_y) b_w d = 0.27 \times (3/40) \times 12 \times 15.5 = 3.76 in^2$

$A_{s min} (0.93) \leq A_s (2.90) < A_{s min} (3.76)$ ok

Bar placement: 5 #7 bars will provide 3.0 in^2 of steel area which is slightly greater than required. other option can be explore. For example

→ 7 #6 bars (3.08) in^2.

→ 4 #8 bars (3.16) in^2.

or Combination of two different size bars.

$V_u = 20.63$ kips.

ϕV_c = (Capacity of Concrete in Shear) $= \phi 2\sqrt{f_c}\, b_w d$.

$= 0.75 \times 2 \times \sqrt{3000} \times 12 \times 15.5/1000 = 15.28$ kips.

As $\phi V_c < V_u$. Shear reinforcement is Required.

Assuming #3, 2 legged (0.22 in^2), Vertical stirrups.

Spacing Required (S_d) $= \phi A_v f_y d/(V_u - \phi V_c)$.

$= 0.75 \times 0.22 \times 40 \times 15.5/(20.63 - 15.28)$.

$= \underline{19.12''}$.

Maximum Spacing & minimum reinforcement requirement as permitted by ACI is minimum of.

$S_{max} = A_v f_y/(50 b_w) = 0.22 \times 40000/(50 \times 12) = 14.66''$

$S_{max} = d/2 = 15.5/2 = 7.5''$

$S_{max} = 24''$

$A_v f_y / 0.75\sqrt{(f'_c)}\, b_w = 0.22 \times 40000 / \{(0.75 \times \sqrt{(3000)}) \times 12 = 17.85''$

Therefore $S_{max} = 7.75''$

<u>Checks</u>:

$= \phi V_s \le \phi 8 \sqrt{f'_c}\, b_w d.$

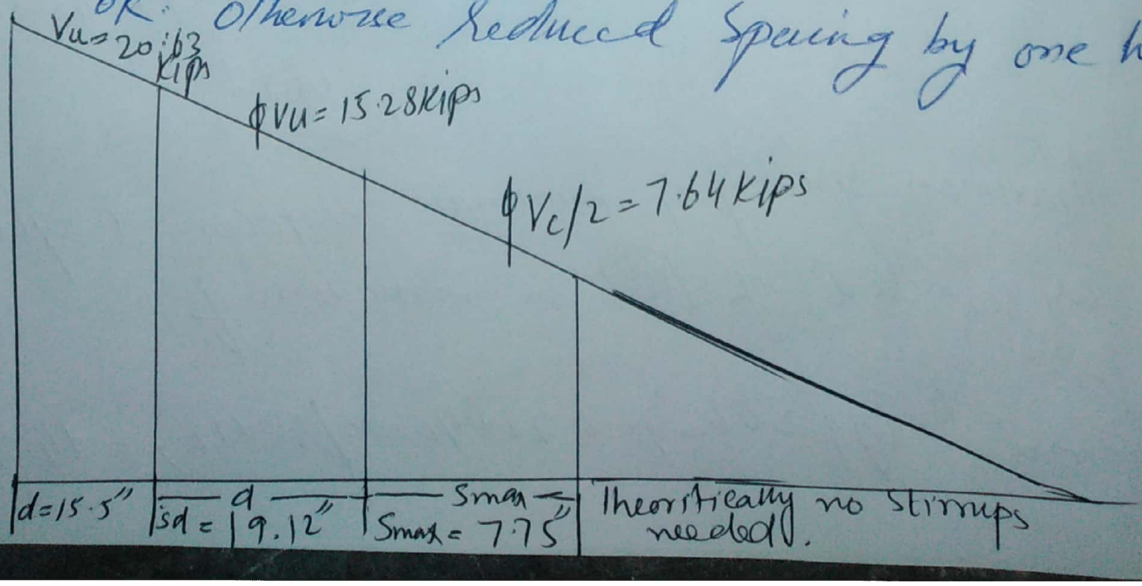
$= \phi 8 \sqrt{f'_c}\, b_w d = 0.75 \times 8 \times 3000 \times 12 \times 15.5 / 1000 = 61.12k$

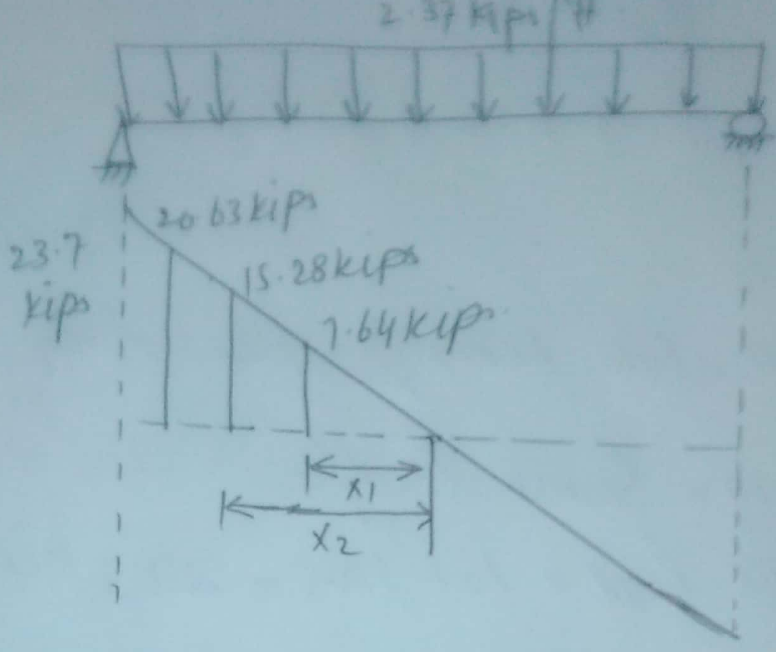
$\phi V_s = V_u - \phi V_c = 20.63 - 15.28 = 5.35k \le 61.12k.$ ok

Check if $\phi V_s \le \phi 4 \sqrt{f'_c}\, b_w d$

$= 5.35$ kips < 30.56 kips ok.

$\phi V_s \le \phi 4 \sqrt{f'_c}\, b_w d$, the maximum spacing ($S_{max}$) is ok. otherwise reduced spacing by one half.





Question No2:

Solution. Step #1.

Estimate the thickness of footing h.

→ Assuming a trail thickness, $h = 12\,in$.

Effective depth, $d = 12 - 3\,in.\ \text{cover} - \frac{1}{2}(\text{bar diameter})$
$\approx 8.5\,in$

Step #2 Calculate weight of fill and weight of concrete, W.

$W = W_{Concrete} + W_{Fill} = 1 \times 0.15 + 4 \times 0.12 = 0.63\ ksf.$

Step # 3.
Calculate effective bearing capacity

$q_e = q_a - w$

$q_e = 5 - 0.63 = 4.37$ ksf

Step # 4
Calculate bearing Area, Areq

A_{req} = Service load / q_e

Service load = 10 + 12.5 = 22.5 Kips/ft

A_{req} = 22.5/4.37 = 5.15 ft^2 per foot of length

Trying a footing 5 ft 2 in wide.

Step # 5.
Calculate design pressure on base of footing due to factored loads, qu

qu = Factored loads / Bearing area

Factored load = 1.2(10) + 1.6(12.5) = 32 Kips

qu = 32/5.17 = 6.19 ksf

Step # 6,
Calculate the critical shear. Vu.

→ only one way shear is significant in wall footing.

Hence determining critical shear at distance d from the face of support

$d = 12 - 3$ in. cover $- \frac{1}{2}$ (bar diameter) ≈ 8.5 in.

$V_u = q_u b(k-d)$

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

$= 8.51$ kips/ft

Step #7.

Check shear capacity.

Shear capacity, $\phi V_u = \phi\, 2\sqrt{f'c}\, bd$.

$= \{0.75 \times 2 \times \sqrt{3500} \times 12 \times 8.5\} / 1000$.

$\phi V_c = 9.05$ kips.

Since, $\phi V_c > V_u$, the footing depth is ok. otherwise choose a new thickness and repeat the previous steps. Using 12 in thick and 5'. 2inch wide footing.

Step # 8.

Calculate maximum moment, M_u.

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

$= 13.43$ ft-kips per ft.

Step # 10.
Calculate steel Area As
Now using trail and success method for determining As

$A_s = M_u / \phi f_y (d - a/2)$ a = 0.2h

$A_s = 0.390\ in^2$ per foot

Min Reinforcement.

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

$A_s (0.390\ in^2) > A_{s\,min} (0.26\ in^2)$ ok.

Step # 11.
Main bars spacing and Maximum spacing check

Main bars. spacing = $A_b \times 12 / A_s$.

using #5 bars, spacing = $0.31 \times 12 / 0.390 = 9.53 \approx$ 9in c/c

Max spacing = 3h or 18" = (3)(12) = 36" or 18"

ok.

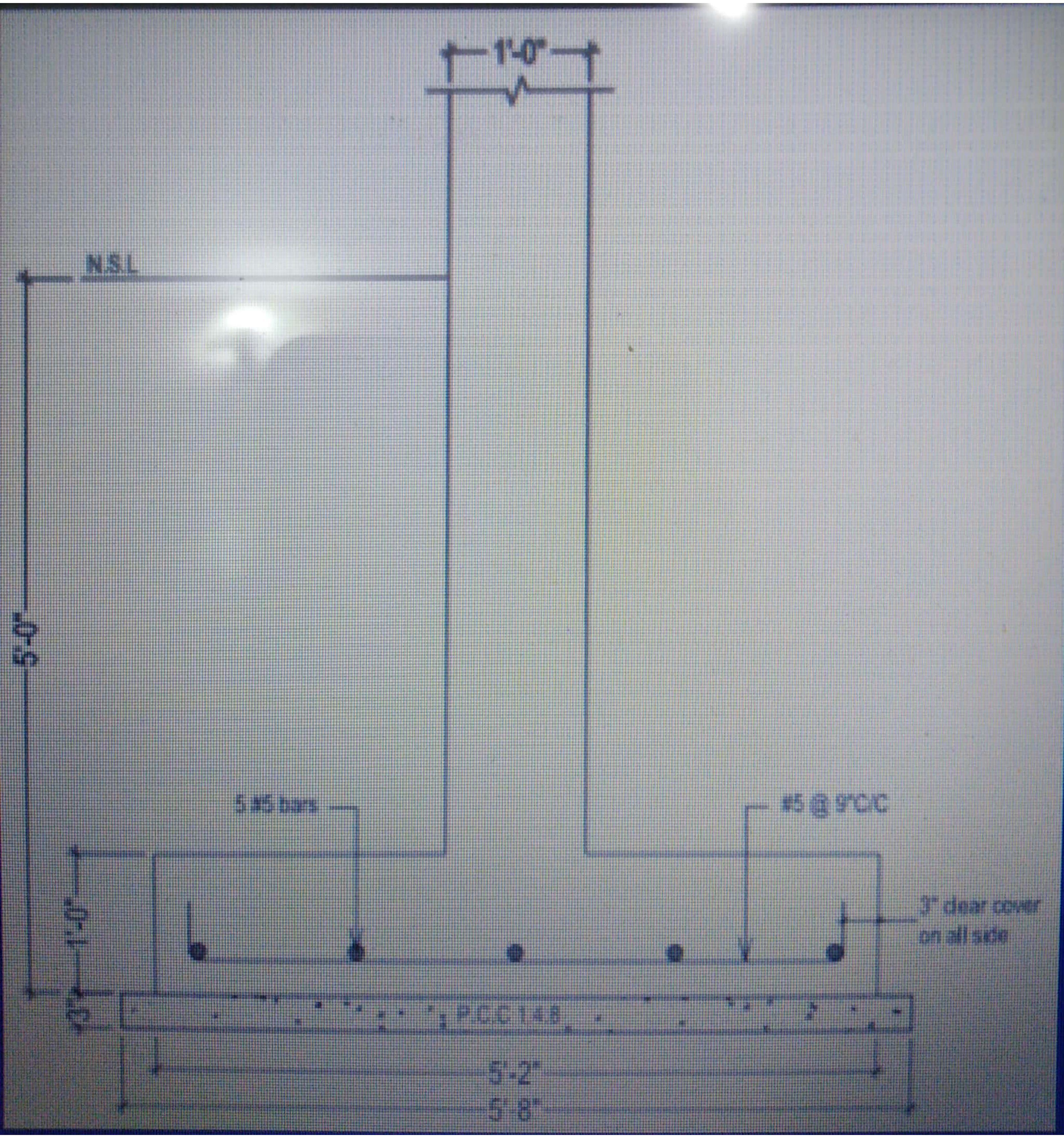
Step # 12.

Distribution bars placing

Distribution bars.

$A_{dist} = 0.0018Bh = 0.0018 \times 62 \times 12 = 1.34 in^2$.

No. of bars $= A_{dist} = 1.34/0.31 = 4.32 \simeq 5$ bars



1'-0"
N.S.L
5'-0"
5 #5 bars
#5 @ 9"C/C
3" clear cover
on all side
1'-0"
3"
P.C.C 1:4:8
5'-2"
5'-8"

Question No 3:

Solution:

Nominal Strength ($Q\phi Pn$) of axially loaded Column is

$$Q\phi Pn = 0.800\,(0.85 f'c)(Ag - Ast) + As\,Fy.$$

$$Ag = 18 \times 18 = 324\, in^2$$

Let $Ast = 1\%$ of $Ag = 0.01 \times 324 = 3.24$.

$$Q\phi Pn = 0.80 \times 0.65 \{0.85 \times 3\,(324 - 3.24)\,3.24 +$$

$= 492$ Kips $>$ $Pn = 300$ kips ok.

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

Main bar.

Using #6 bar with bar Area, $A_b = 0.44 \, in^2$.

$$\text{No. of bars} = \frac{A_s}{A_b} = \frac{3.24}{0.44} \Rightarrow 7.36 \approx 8 \text{ bars}.$$

Tie bars:

Using # bar with bar Area $A_b = 0.11 \, in^2$.

C/c Spacing shall axial the least of.

① 16 db of longitudnal bars = ~~18″~~
$16 \times 0.75 = 12''$.

② 48 db of tie bars = $48 \times 3/8 = 18''$.

③ Smallest diameter of number = 18″.

Therefore use #3 bars ties @ 12″ C/c.