IQRA NATIONAL UNIVESITY, PESHAWAR

I.D 12430

BATCH 2015

B.TECH CIVIL

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FINAL TERM

PAPER REINFORCED CONCRETE STRUCTURE

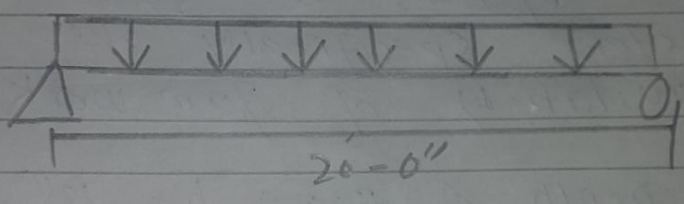
DATE: 28/9/2020

Question No 1

Design the Beam of shear and flexural stress show below as per ACI 318 . 4

$W_D = 0.75$ /Kip/ft

$W_L = 0.75$ kips



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

* Flexural an shear Desgin of Beam as per Acl

Solution

. step No 01 Size .

* For 20' length, $h_{min} = l/16 = 20 * 12/16 =$ 15"

* For grade 40, we have $= h_{min} =$

$15 \times (0.4 + 40,000/100,000) = 12''$

* This is the minumum requirment of the code for depth beam .

* Howor we select 18" deep beam

~~* Generally the maximum requirement of the Code for depth beam.~~

* Generlly the minimum beam width is 12" therefore width of the beam is taken as 12"

* the final selection of beam size depends on several factor specifically the availability of form work.

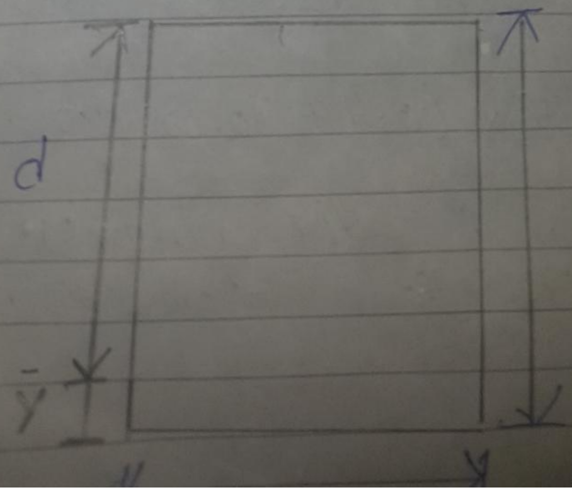
* Depth of Beam $h = 18"$

* $h = d + \bar{y}$; $\bar{y}$ is usally taken from 2.5 to 3.0 inch.

* For $\bar{y} = 2.5$ in; $d = 18 - 2.5 = 15.5$

* width of beam cross section (b_w) = 12"

* In RCD width of beam is usally denoted by b_w instead of b.



Step No2 Loads

* Self weight of beam $= \gamma_c b_w h = 0.15 \times (12 \times 18/144) = 0.225$ kips/ft

* $W_u = 1.2W_D + 1.6W_L$

$= 1.2 \times (0.225 + 0.75) + 1.6 \times 0.75 = 2.037$ Kips/ft

Step 3 Analysis

* Flexural Analysis

$M_u = W_u L^2/8 = 2.037 \times (20)^2 \times 12/8 = 1422$ in-kips

* Analysis for shear in beam

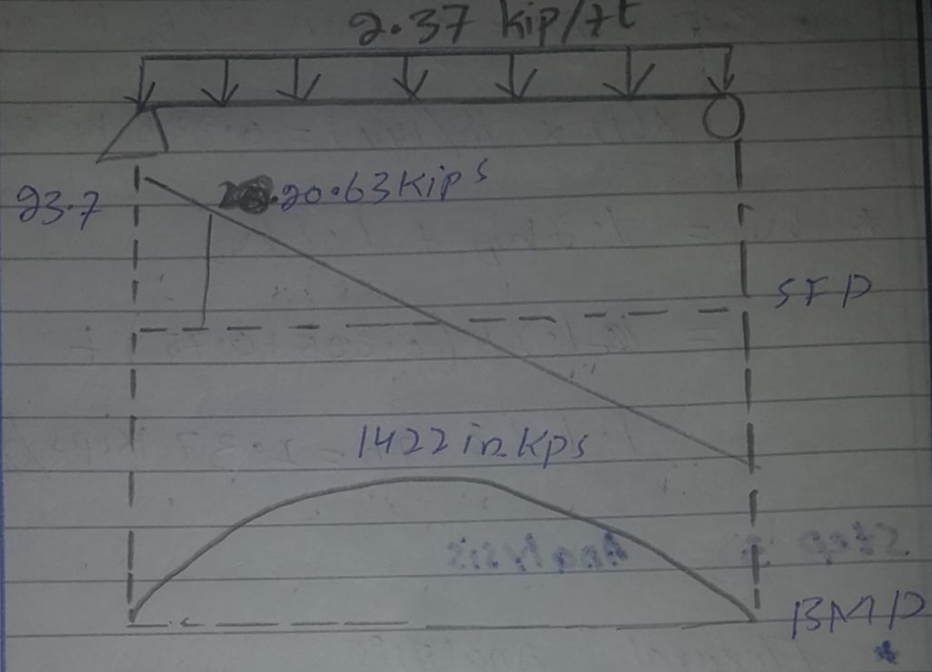
$V = 23.7$ Kips

To find V_u at a distance 'd' from face support $d = 15.5'' = 1.29'$

using similarity of triangles:

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

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



~~St~~ Setp 4 Design

* Design for flexure
    - $\phi M_n \geq M_u$ (ϕM_n is M_{desgn} or $M_{capacity}$)
    - For $\phi M_n = M_u$

* ~~As~~ $\phi A_s f_y (d - a/2)$ M_u
    - A_s $M_u / \{\phi f_y (d - a/2)\}$

- Calculate 'As' by trile and success method.

- Fist trial
- Assume $a = 4''$
- $A_s = 1422 / \{0.9 \times 40 \times \{15.5 - (4/2)\}\} = 2.92 \ in^2$
- $a = A_s f_y / (0.85 f_c' b_w)$
- $2.92 \times 40 / (0.85 \times 3 \times 12) = 3.81$ inches

Design for flexure:

2nd trial:-

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

$= 2.90 \ in^2$

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

$= 3.79$ inches

3rd trial:-

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

$= 2.90 \ in^2$

- $a = 4.49 \times 40 / (0.85 \times 3 \times 12) = 3.79$ inches

- close enough to the previous value of "a" so that $A_s = 2.90 \ inch^2$ OK

* Check for maximum and minimum reinforcement allowed by ACI

* $3(\sqrt{f'_c}/f_y)\, b_w d = 3 \times (\sqrt{3000}/\sqrt{40000})\, b_w d = 0.004 \times 12 \times 15.5 =$

$0.744\ in^2$

* $(200/f_y)\, b_w d = (200/40000) \times 12 \times 15.5 =$

$0.93\ in^2$

* $A_{s\,min} = 0.93\ in^2$

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

* $A_{s\,min}\ (0.93) < A_s\ (2.90) < A_{s\,max}\ (3.76)$ 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 explored for example.

- 7 # 6 bar ($3.08 in^2$)
- 4 # 8 bars ($3.16 in^2$)
- or Combination of two different size bar

Design of Shear

- $V_u = 20.63$ 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 stirrup
- Spacing required (S_d) = $\phi A_v f_y d / (V_u - \phi V_c)$

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

19.12"

- Maximum spacing and minimum reinforcement requirement as permitted by ACI of minimum of
- $S_{max} = A_v f_y / (50 b_w) = 0.22 \times 40000 / (50 \times 12) = 14.66"$

$S_{max} = d/2 = 15.5/2 = 7.75''$

- $S_{max} = 24''$

- $A_v f_y / 0.75\sqrt{(f_c')}\, b_w = 0.22 \times 40000 /$

 $\{0.75 \times \sqrt{(3000)} \times 12\} = 17.85''$

 there fore $S_{max} = 7.75''$

other Check.

Check for depth of beam.

$\phi V_c < \phi 8 \sqrt{f_c'}\, b_w d$

$\phi 8 \sqrt{f_c'}\, b_w d = 0.75 \times 8 \times \sqrt{3000} \times 12 \times$

 $15.5/1000 = 61.12\,K$

$\phi V_s = V_u - \phi V_c = 20.63 - 15.28 =$

 $15.35\,K < 61.12\,K$. OK.

there fore depth is OK

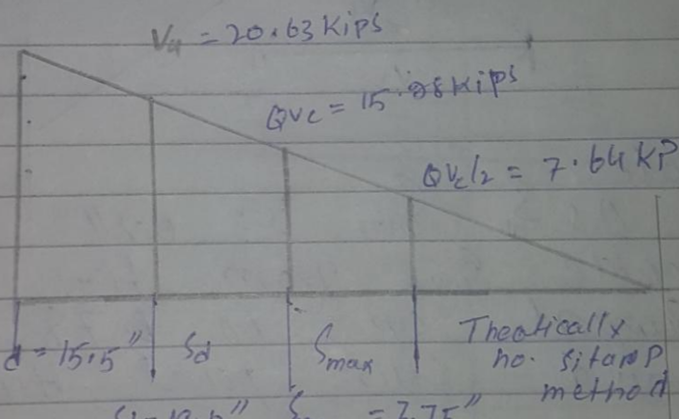
If not incres depth of beam.

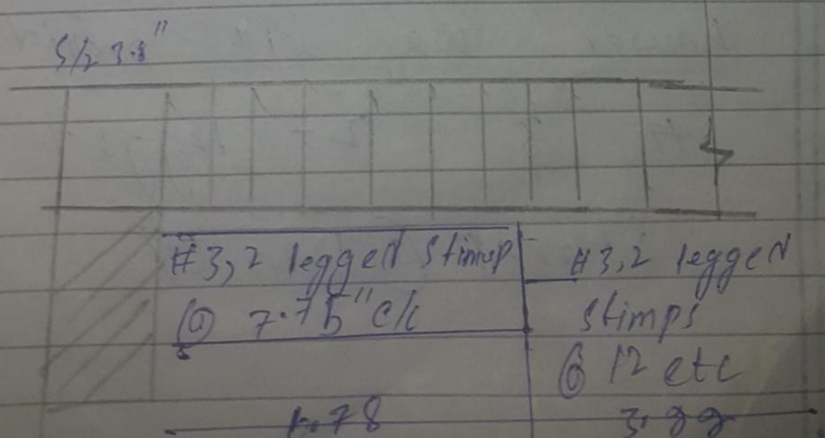
other Check

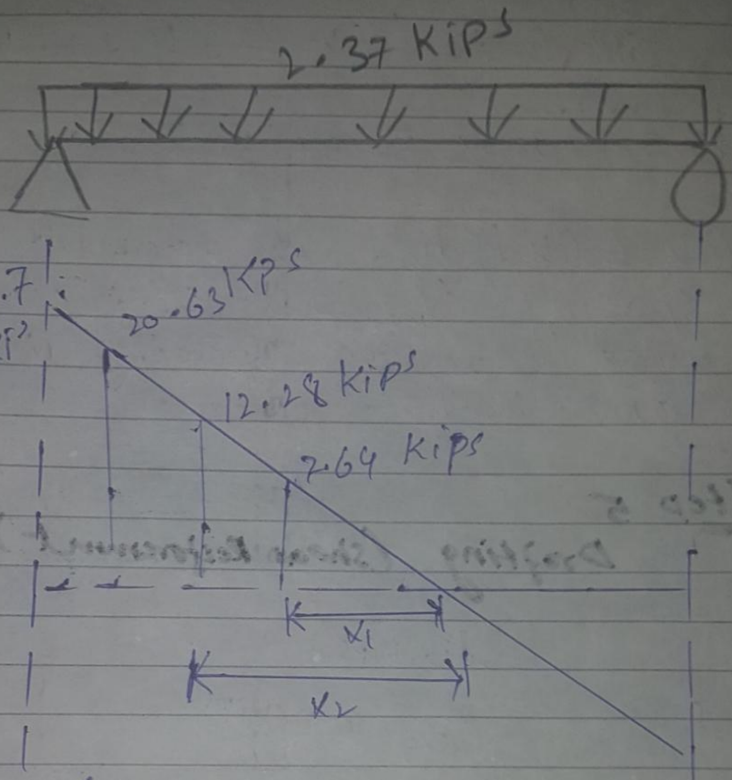
- Check if $\phi V_s \le \phi 4\sqrt{f'_c}\, b_w d$
- 5.35 Kips < 30.56 Kips ok
- $\phi V_s \le \phi 4\sqrt{f'_c}\, b_w d$ the maximum spacing (S_{max}) is OK. other wise redice spacing by one half.

Step 5

Drafting (shear Reinforcement)





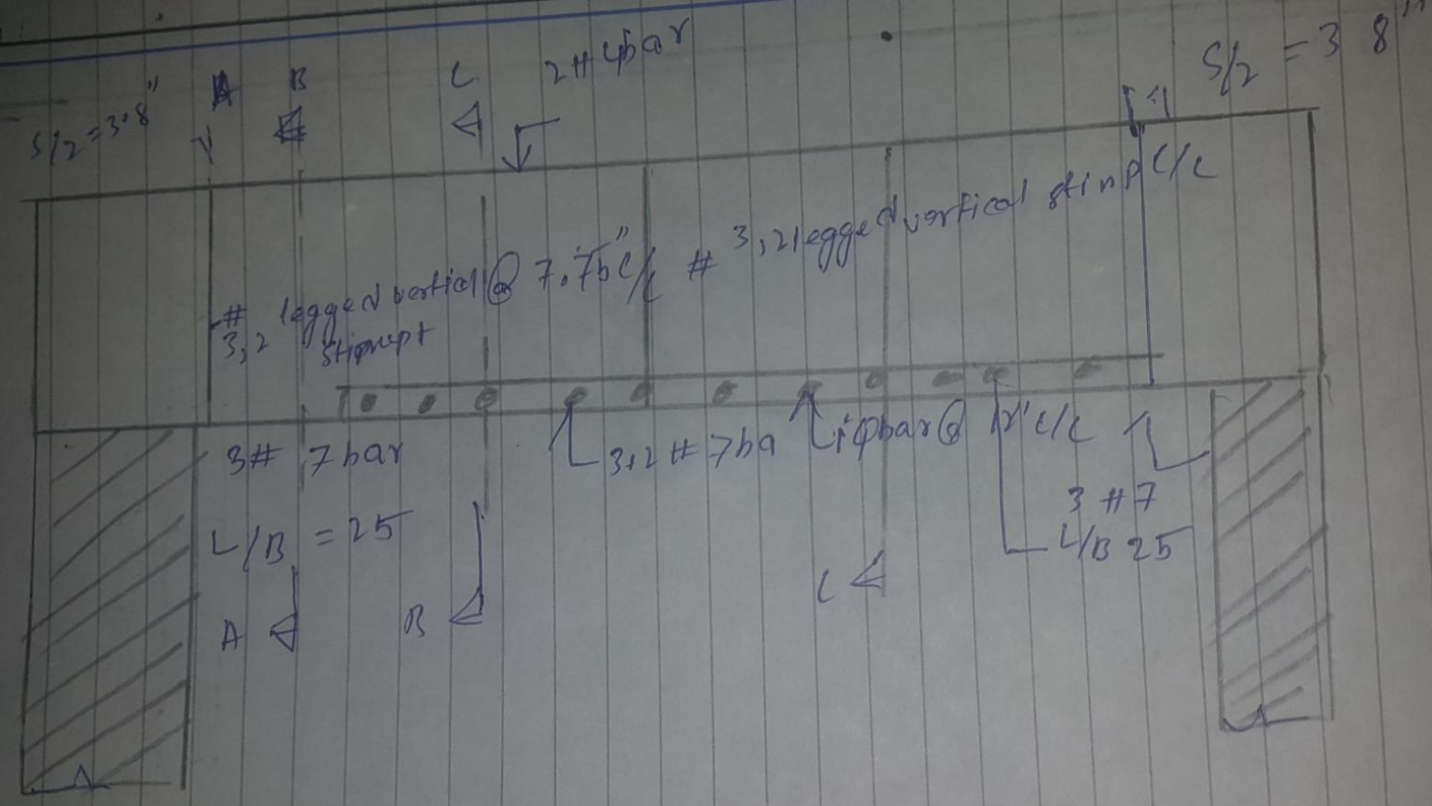


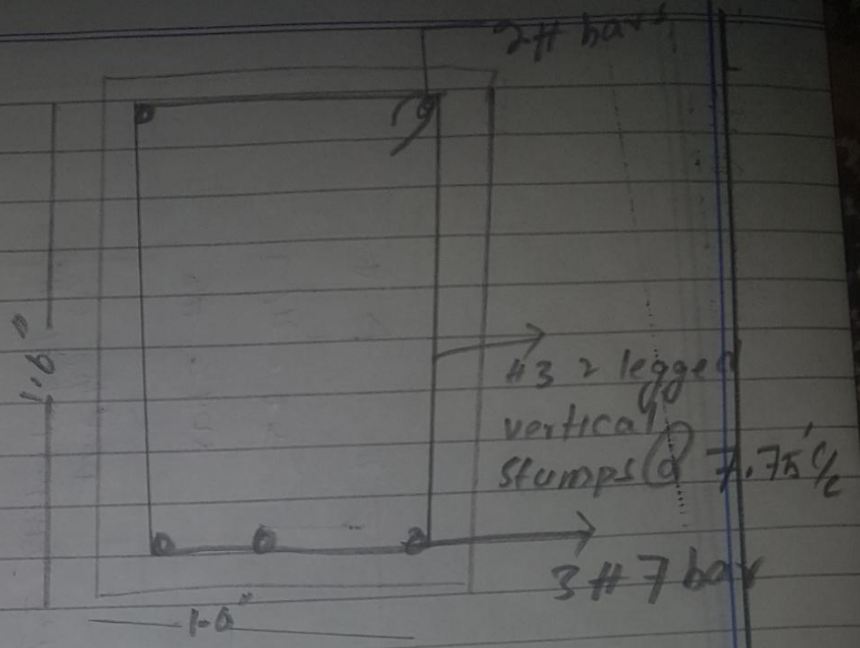
Note

As S_d S_{max} we will provide S_{max} from the support up to 6.78 ft Beyond this point theoretically no reinforcement is requisite hower we will provide

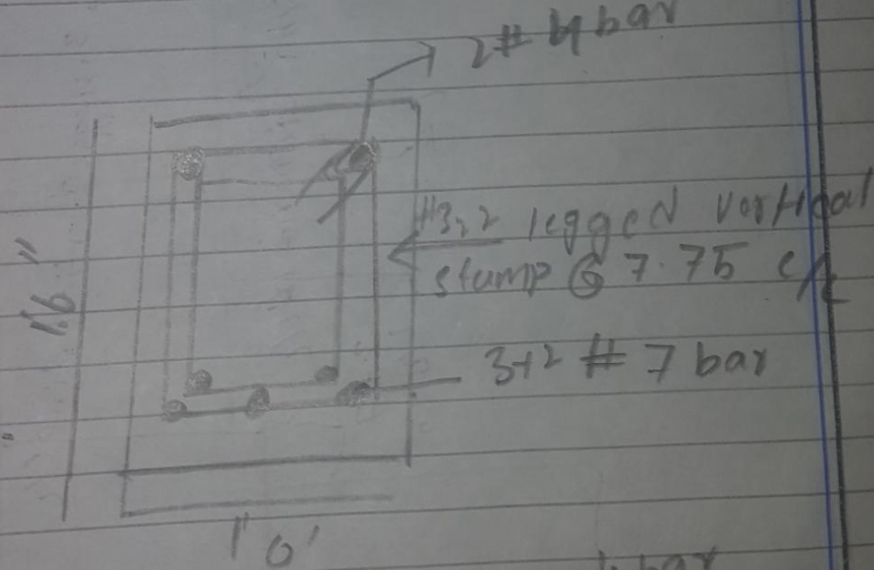
3 2- legged

Stirrup @ 12in c/c .

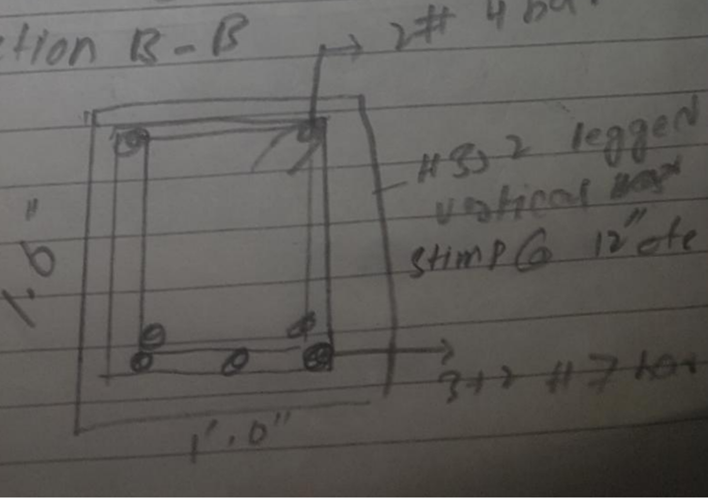

S/2 = 3'8"
A
B
C
2# 4 bar
S/2 = 3 8"
3,2 legged vertical @ 7.75" c/c
stirrup
3,2 legged vertical stirrup c/c
3# 7 bar
3+2 # 7 ba
4 bar @ 12" c/c
3 #7
L/B 25
L/B = 25
A
B
C



Section A-A



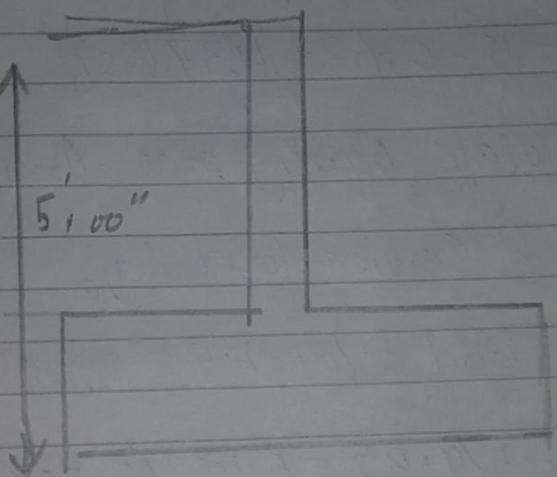
Section B-B



Question No 2

12 in thick Connert wall Carried

12 in
dead Load 10 kip/ft
L load 12.5 kips



Step #01 Estimate the thickness of footting, h.

→ Assuming a laile thickness h = 12 in
- Effective depth d = 12 - 3 in Cover - 1/2 (bar diameter) = 0.85 in

Step 2
Calculate weigh of fill and wight of Connerete W

- $W = W_{conc} + W_{fill} = 1 \times 0.15 + 4 \times 0.12$
  $= 0.63\ ksf$

Step 3 Calculate effective bearing Capacity q_e

- $q_e = q_a - w$

$q_e = 5 - 0.63 = 4.37\ ksf$

Step 4 Calculate bearing area A_{req}

- A_{req} = Service load / q_e

Service load = 10 + 12.5 = 22.5 kips

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

- Trying a footing 5 ft 2 in wide.

Step 5 Calculate design pressure on base of footing due to factored load q_u

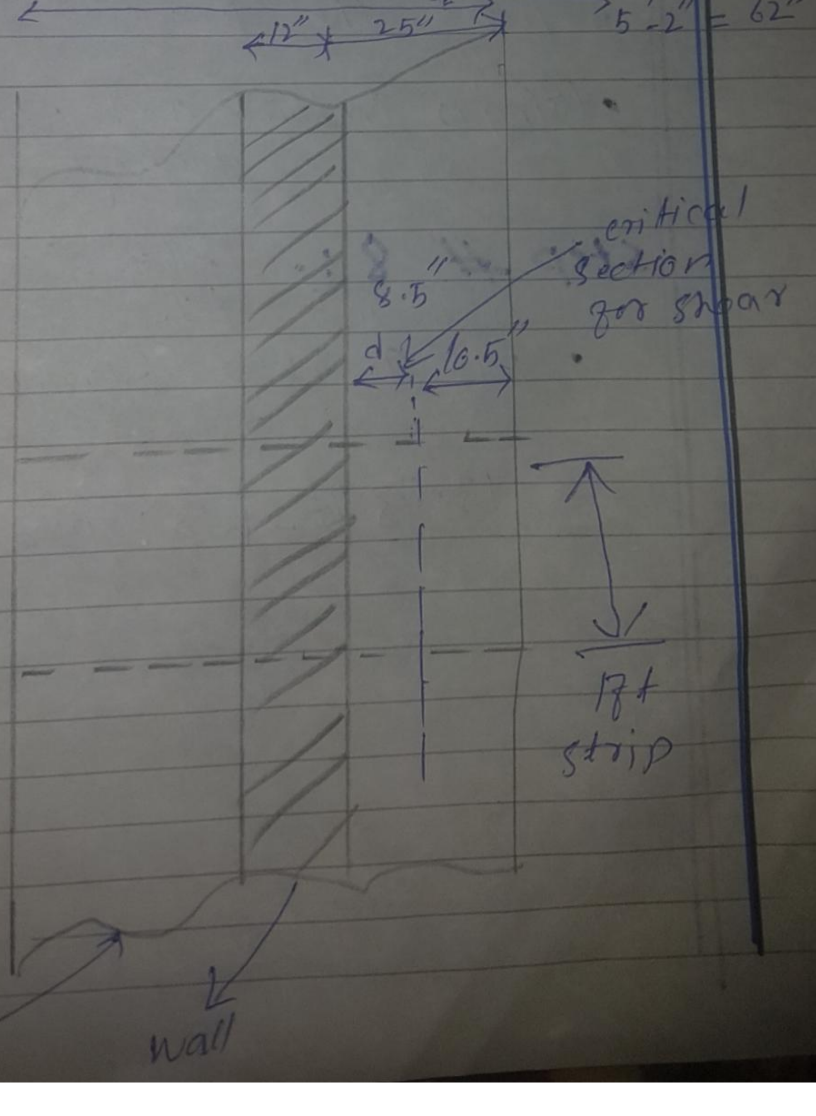
q_u = Factored load / Bearing area

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

- $q_u = 32/5.17 = 6.19\ ksf$

step #06 Calculate the critical Shear V_u

- 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 deformation) $= 8.5$ in.
- $V_u = q_u b(k-d)$
- $V_u = 6.19 \times 1\{(25-8.5)/12\} = 8.51$ kips/ft



Step #07 Check the Shear Capacity ϕV_c

- Check the thickness for Shear.
- Shear Capacity $\phi V_c = \phi 2 \sqrt{f'_c}\, bd$

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

$\phi V_c = 9.05$ kips

- Since $\phi V_c > V_u$, the footing depth is OK otherwise chose a new thickness and repeat the previous steps

- Using 12 in thick and 5ft 2in wide footing

step # 8 :- Calculate maxium moment M_u

- $M = q_u b k^2 / 2 = 6.19 \times 1 \times ((25/12)^2 / 2)$

$= 13.43$ ft - kips per ft.

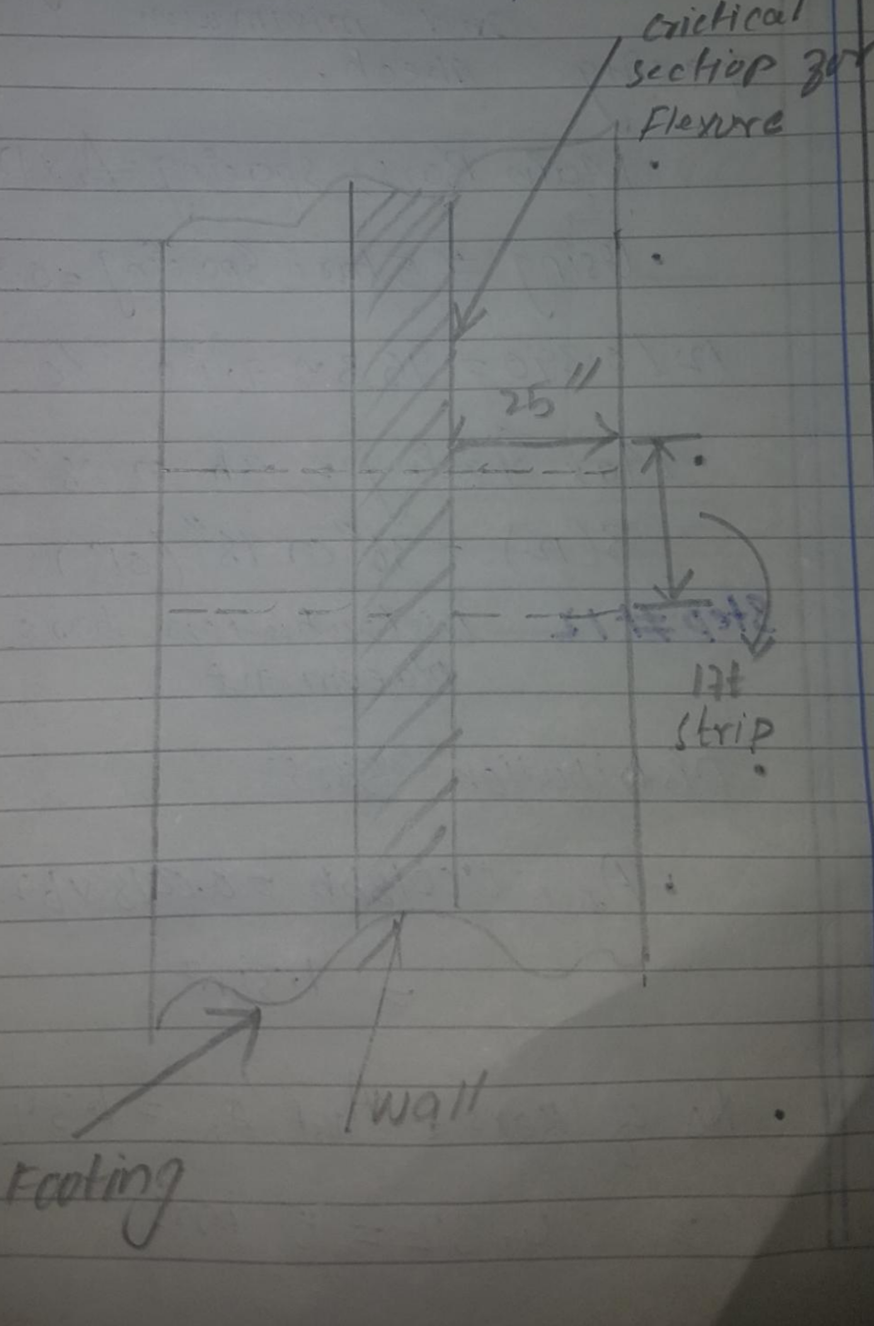
Step #9 Calculate steel area, A_s

- Now using trial and success method for determining A_s,

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

$$a = \quad a = 0.9h$$

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



Step #10: Minimum renifoxment Check

- 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

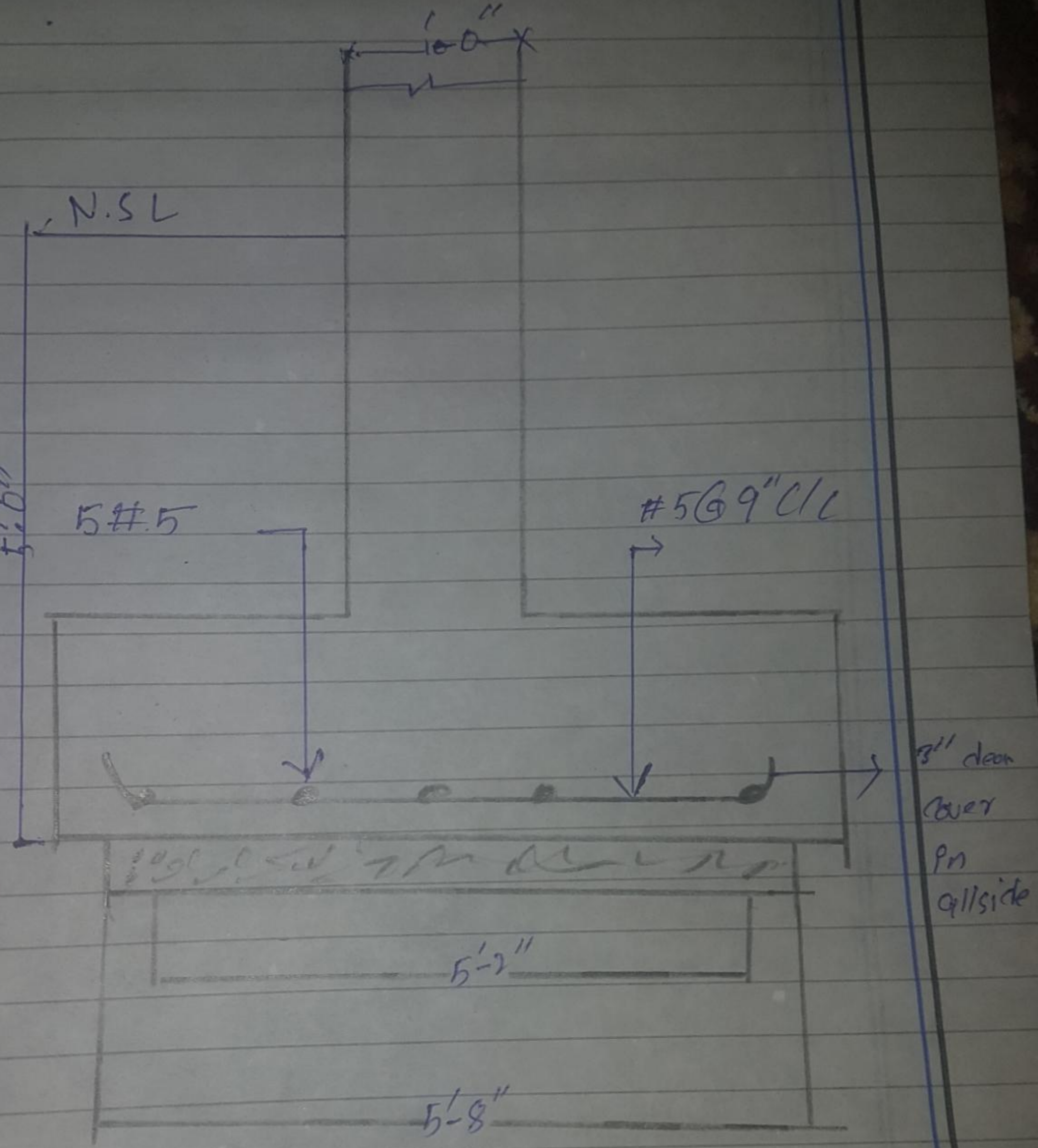
Step #11 Main Bars Spacing and mirimum spacing check.

- Main Bars: spacing $= A_b \times 12 / A_s$
- Using #5 bars Spacing $= 0.31 \times 12 / 0.390 = 9.53 \approx 9$ in. c/c
- Max Spacing $= 3h$ or $18'' =$ $3(12) = 36''$ or $18''$ (OK)

Step #12 Distrubution baos placement

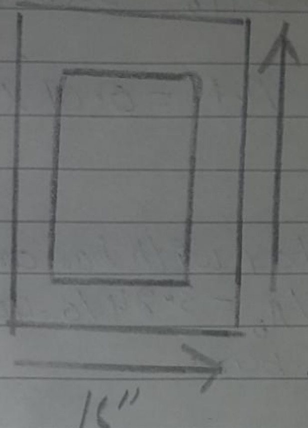
- Distribution Bars:
  - $A_{dist} = 0.0018bh = 0.0018 \times 62 \times 12$ $= 1.3 in^2$
- No of Bars $= A_{dist} / A_b = 1.34 / 0.31$ $= 4.32 = 5$ bars

Step # 13



Question No 3

Design a 18"x18" Colum for a factored axil Compressive load of 300 kip the material strength ar $f_c' = 3$ksi an $f_y = 40$ksi



Solution:-

Norimal strength (ϕP_n) of axially loaded Colamn is

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

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

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

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

$\Rightarrow$ $492 \text{ kips} > (P_u = 300 \text{ kips})$ O.K

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

Main Bars:

- Using #6 bar with bar area $A_b = 0.44 \text{ in}^2$
- No of bar $= A_s/A_b = 3.24/0.44 = 7.36 = 8$ bar
- use 8 #6 bars

Tie bar

- Using #3 bar with bar area $A_b = 0.11 \text{ in}^2$

Center-to-center spacing shall not exceed the least of:

(1) $16 d_b$ of longitudinal bar $= 16 \times 0.75 = 12''$

(11) $48 d_b$ of tie bar $= 48 \times 3/8 = 18''$

(3) Smallest dimension member $= 18''$

therefore use #3 ties @ $12''$ c/c