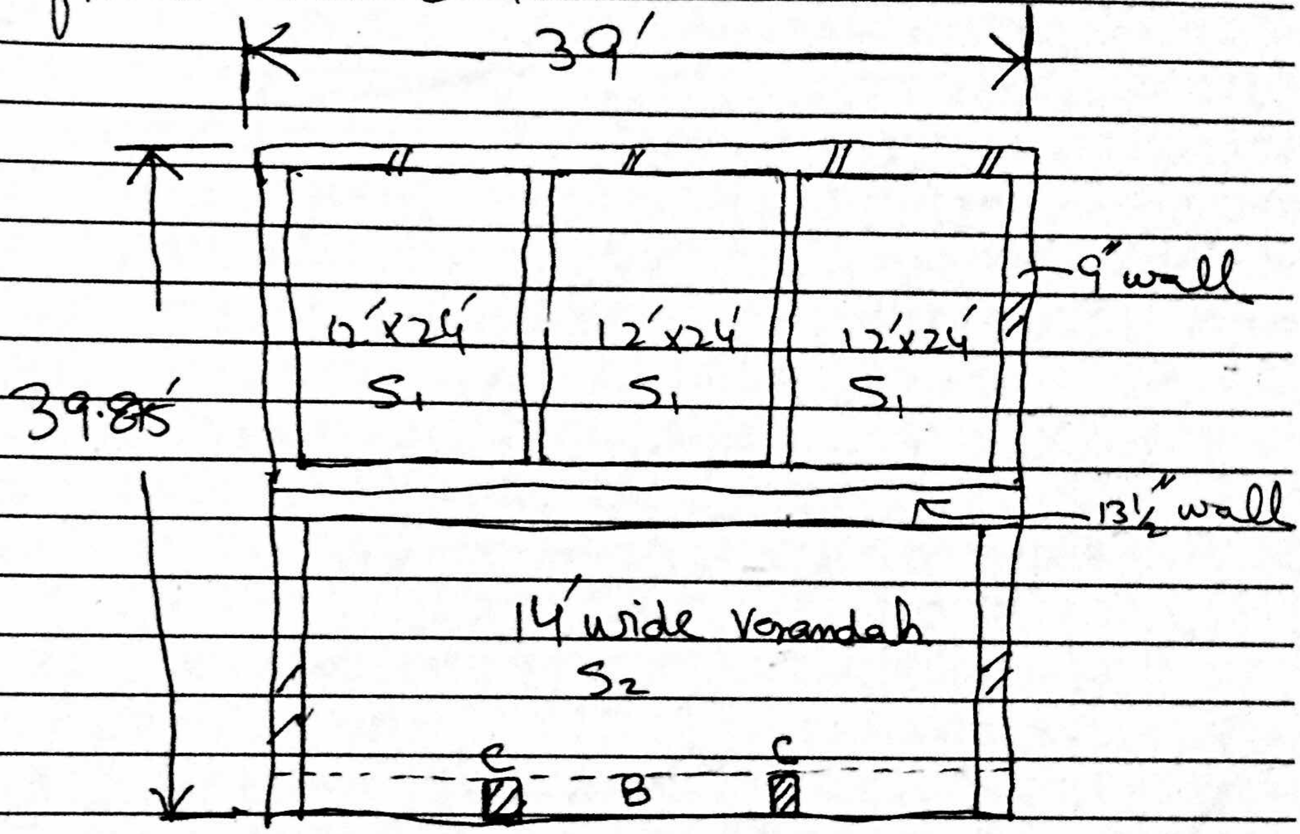
Date:

Design of Residential House given below i.e



Total Covered area = 39 x 39.875
= 1555.125 ft².

Concrete Compressive Strength = 3 ksi

Steel yield Strength = 40 ksi

Load on Slab: 4" thick mud

: 2" thick brick tile.

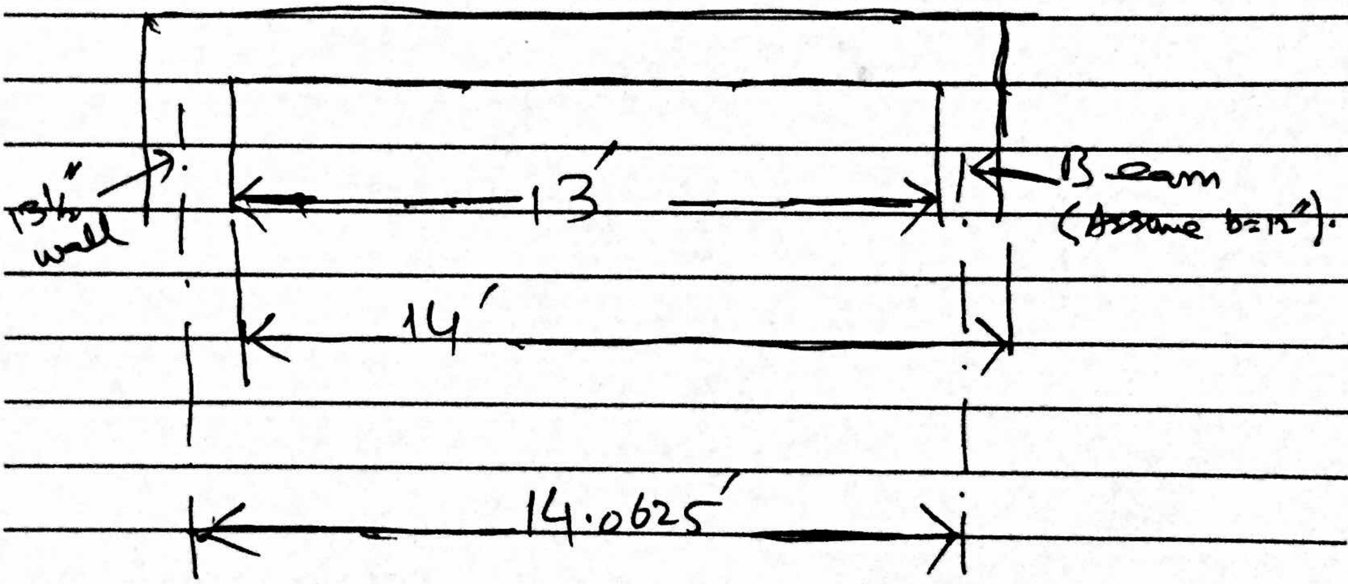
Date:

Design S_1, S_2, Column, Beam and Column Footing:

Solution:

(1) Design of Slab "S_2"

$\Rightarrow \frac{l_b}{l_a} = \frac{37.5}{14} = 2.7 > 2$ "one way slab".



Span length According ACI 8.9.1 is minimum of

(i) $l_n + h_f = 13 + 6'' = 13.5'$

(ii) C/C distance b/w Supports = 14.0625'

where h_f = Slab thickness

Assume Slab thickness = 6"

Date:

therefore Span length $= l = 13.5'$.

Minimum Slab thickness as Per ACI 9.5.2.1 is

$$= \frac{l}{20}\left(0.4 + \frac{fy}{100,000}\right)$$

$$= \frac{13.5}{20}\left(0.4 + \frac{40,000}{100,000}\right) * 12 = 6.48''$$

Assume Slab thickness $= h_f = 6.5''$

therefore l = Span length $= 13 + \frac{6.5}{12}$

$$= 13.54'$$

$$d = h_f - 0.75 - \frac{(3/8)}{2} = 5.56''$$

Step No 2: Loading:

Slab Dead loads:

Slab $= \frac{6.5}{12} * 0.15 = 0.08125\ K/ft^2$

Mud $= \frac{4}{12} * 0.12 = 0.04\ K/ft^2$

tile $= \frac{2}{12} * 0.12 = 0.02\ K/ft^2$

Date:

(4)

Service Dead load = $0.14125\ k/ft^2$

Service Live Load = $0.04\ k/ft^2$

Factored load = 1.2DL + 1.6LL

$W_u = 1.2(0.14125) + 1.6(0.04)$

$= 0.2335\ k/ft^2$

Step No 3: Analysis:

$M_u = \frac{W_u l^2}{8} = \frac{0.2335 * (13.54)^2 * 12}{8}$

$= 64.2\ k\text{-}in/ft$

Step No 4: Design:

$A_{s\,min} = 0.002 * b * h$ (ACI 10.5.4)

$= 0.002 * 12 * 6.5 = 0.156\ in^2$

$\Rightarrow a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.156 * 40}{0.85 * 3 * 12} = 0.204\ in$

$\Rightarrow \phi M_n = \phi A_s f_y (d - a/2) = 30.65\ k\text{-}in/ft < M_u$

Date:

(5)

Therefore

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

take $a = 0.2d = 0.2 * 5.56 = 1.1''$

$\Rightarrow A_s = 0.36\ in^2$

$\Rightarrow a = \frac{A_s f_y}{0.85 f_c b} = 0.465''$

take $a = 0.465''$

$\Rightarrow A_s = 0.335\ in^2$

$\Rightarrow a = 0.44'' \Rightarrow A_s = 0.334\ in^2$ O.K.

so take $A_s = 0.33\ in^2/ft$

Using #3 bar, Area $= 0.11\ in^2$

Spacing required $= \frac{0.11}{0.334} \times 12 = 3.95''$ say $3.5''$

Use #3 @ $3.5''$ c/c as Main Reinforcement. ($A_s = 0.44\ in^2/ft$)

Shrinkage Steel: $A_s = 0.15\ in^2$

Darsi Notes

Date:

Using #3, bars:

Spacing required = $\frac{0.156}{0.11} = \frac{0.11}{0.156} \times 12 = 8.5''$

Use #3 @ 8" c/c As shrinkage Reinforcement.

- Maximum spacing for Main Steel according ACI 7.6.5 is minimum of

i) $3 \times h_f = 3 \times 6.5 = 19.5''$

ii) $18''$

so given spacing O.K.

- Maximum spacing for shrinkage Steel according ACI 7.12.2 is min of

i) $5 \times h_f = 5 \times 6.5 = 32.5''$

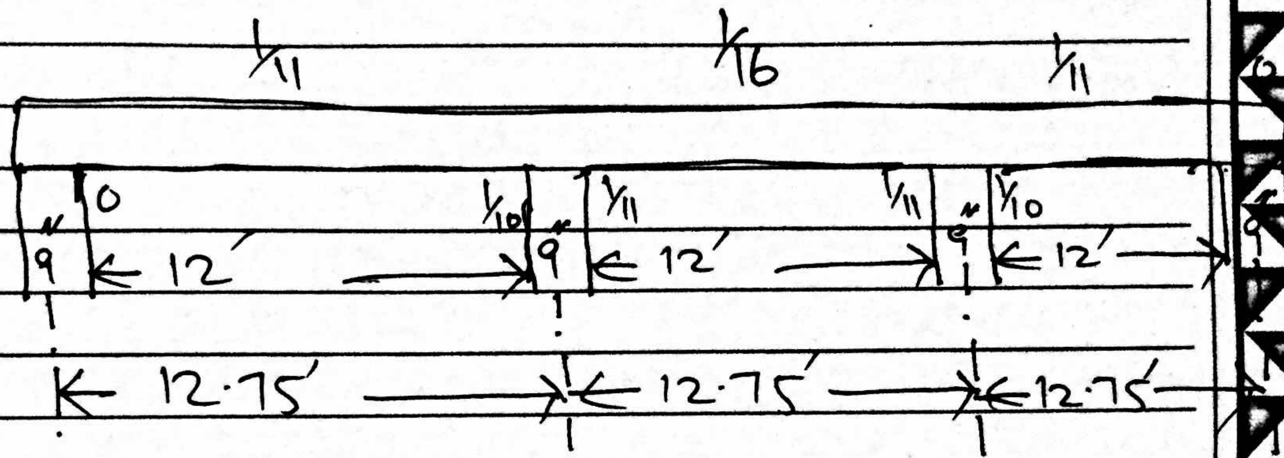
ii) $18''$

so given spacing OK.

Date:

Design of Slab 'S1':

$L_b / L_a = \frac{24}{12} = 2$ (design as one way slab).



~~take~~ $h_t = 6.5"$ ⇒

Span length = (i) $12 + 6.5/12 = 12.54'$
= (ii) $c/c = 12.75'$

So Span length = 12.54'

Minimum thickness as per ACI 9.5.2.1

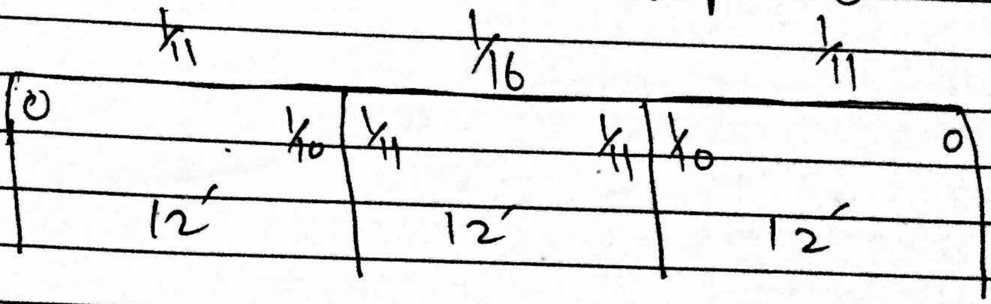
$= l/24 = \frac{12.54}{24} \times 12 = 6.3"$

So $6.5" = h_t$ O.K.

Factored load on slab = $0.2335\ k/ft^2$
(already Calculated).

Date:

ACI Moment Coefficients As per 8.3.3 are



+ve moment @ Exterior Spans i.e

$$= \frac{W_u l^2}{11} = \frac{0.2335*(12)^2}{11} = 36.7 \text{ k-in/ft}$$

$$-ve \text{ moment} = \frac{W_u l_n^2}{10} = 40.4 \text{ k-in/ft}$$

+ve moment @ interior Span

$$= \frac{0.2335*(12)^2}{16} = 25.2 \text{ k-in/ft}$$

Now $h_f = 6.5''$, $d = 5.5''$

Design:

$A_{s\,min} = 0.002 * 12 * 6.5 = 0.156 \text{ in}^2$

$\Rightarrow a =$ ~~0.04 in~~ 0.204 in

$\Rightarrow \phi M_n = \phi A_s f_y (d - a/2) = 0.9*0.156*40*(5.5 - \frac{0.204}{2})$

$\phi M_n = 30.3$ k-in

Darsi Notes

Date:

Using #3 @ 8" c/c : $A_s = 0.17\ in^2/ft$

$\Rightarrow a = 0.22\ in$

$\Rightarrow \phi M_n = 32.98\ k\text{-}in$

So, Shrinkage Reinforcement and Main reinforcement for the moment at interior span will be

#3 @ 8" c/c

Now design for the moment @ ~~Mid~~ Exterior spans i.e

$M_u = 36.7\ k\text{-}in$

$\Rightarrow$ let $a = 0.2 * d = 0.2 * 5.5 = 1.1"$

$$\Rightarrow A_s = \frac{M_u}{\phi f_y (d - a/2)} = \frac{36.7}{0.9 * 40 * (5.5 - 1.1/2)} =$$

$= 0.21\ in^2$

$\Rightarrow a = 0.274"$ $\Rightarrow A_s = 0.19\ in^2$

$\Rightarrow a = 0.25"$ $\Rightarrow A_s = 0.19\ in^2$ hence O.K

Use #3 @ 6" c/c , $A_s = 0.22\ in^2/ft$

Date:

So for +ve moment @ exterior spans use

#3 @ 6" c/c $A_s = 0.22\ in^2/ft$

Design for -ve moment @ interior supports i.e

$M_u = 40.4$ k-in

Let $a = 0.2 \times d = 1.1"$

$\Rightarrow A_s = \frac{40.4}{0.9 \times 40 \times (5.5 - a/2)} = 0.227\ in$

$\Rightarrow a = 0.296" \Rightarrow A_s = 0.21\ in^2/ft$

$\Rightarrow a = 0.274" \Rightarrow A_s = 0.21\ in^2/ft$ o.k.

Use #3 @ 6" c/c , $A_s = 0.22\ in^2/ft$

$\Rightarrow a = 0.288"$

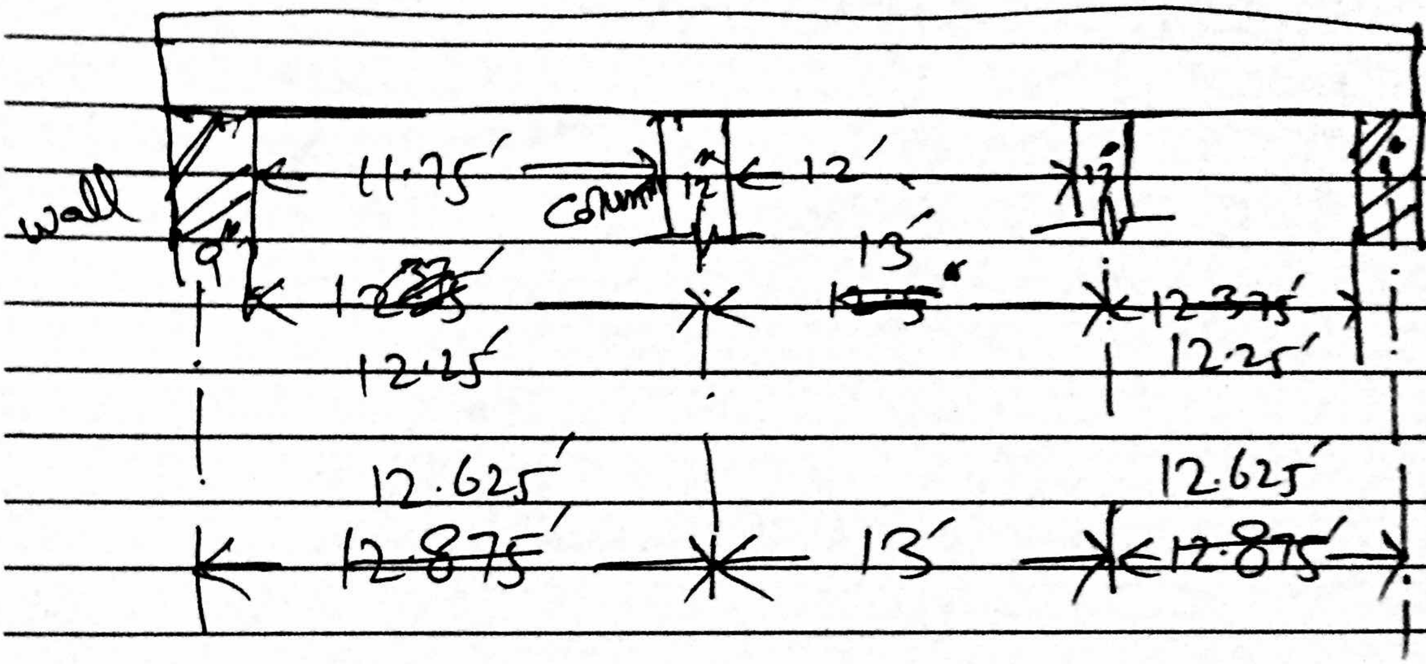
$\Rightarrow \phi M_n = 42.4$ k-in $> M_u$ hence o.k

So, for -ve moment @ supports

Use **#3 @ 6" c/c** $A_s = 0.22\ in^2$

Date:

Design of Beam:



Assuming Beam dimensions : 12" x 18"

$d = h - 3 = 18 - 3 = 15"$

Minimum thickness requirement as per ACI 9.5.2.1

Span length : (i) c/c distance : 12.625'

(ii) $11.75' + 18" = 13.25'$

So take $l = 12.625'$.

$\Rightarrow \frac{l}{18.5}\left(0.4 + \frac{f_y}{100{,}000}\right) = 6.5"$

So take $h = 18"$

Date:

Loads:

Service D.L From Slab = 0.14125 K/ft^2

" L.L " " = 0.04 K/ft^2

So,

D.L on beam from Slab = 5 * 0.14125

= 0.71 K/ft

L.L From Slab = 5 * 0.04 = 0.2 K/ft

Beam Self wt = $\left(\frac{12 \times 18}{144}\right) \times 0.15$

= 0.225 K/ft

Total Dead load = 0.71 + 0.225

= 0.935 K/ft

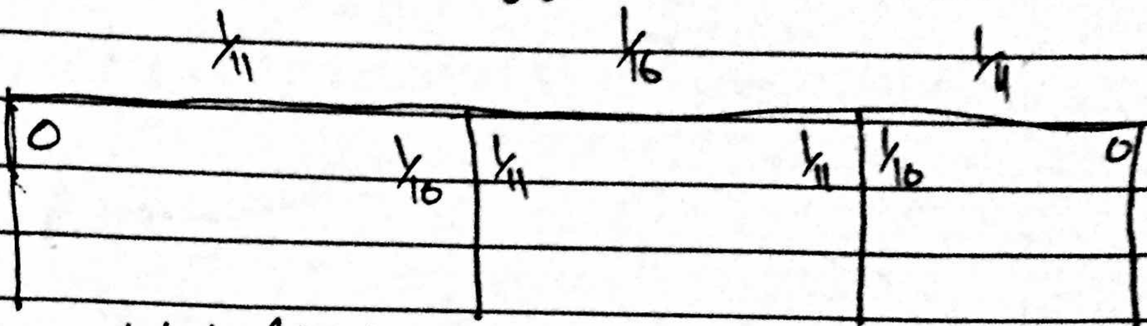
Factored load = 1.2 D.L + 1.6 L.L

= 1.2 * (0.935) + 1.6 (0.2)

= 1.44 K/ft

Date:

ACI moment Coefficients are: (13)



$W_u = 1.44$ K/ft.

+ve moment @ exterior Spans i.e

$M_u = \frac{1}{11} * (1.44)(11.75)^2 * 12 = 216.9$ k-in

+ve moment @ interior Span

$= \frac{1}{16}(1.44)(12)^2 \times 12 = 155.52$ k-in

-ve moment @ interior Supports i.e

$= \frac{1}{10}(1.44)(11.75)^2 \times 12 = 238.6$ k-in

and

$= \frac{1}{11}(1.44)(12)^2 \times 12 = 226.2$ k-in

Flexure Design:

For +ve moment:

Date:

beff is minimum of

(i) $6h_f + b_w = 6*6.5 + 12 = 51''$

(ii) b_w + Span length of beam/12

$= 12 + \frac{12.625*12}{12} = 24.625''$

So

$b_{eff} = 24.625''$

Assume $a = h_f = 6.5''$

$\Rightarrow A_s = \frac{M_u}{\phi f_y (d - a/2)} = \frac{155.52}{0.9*40*(15 - 6.5/2)} = 0.37 in^2$

$\Rightarrow a = 0.234'' < h_f$, so design as rectangular beam.

Now $A_{s,min} = \rho_{min} * b_w * d$

$= 0.005*12*15 = 0.9 in^2$

$\Rightarrow a = 0.57 in \Rightarrow \phi M_n = 0.9*0.9*40(d - a/2)$

$\Rightarrow \phi M_n = 476.7$ k-in $> M_u$ (all) ok

So, for all the moments, minimum steel area should be provided

Darsi Notes

Use 3, #5 bars ($A_s = 0.93 in^2$)

for +ve reinforcement.

Design for -ve moment:

$M_u = 236.8$ k-in

$b_w = 12''$, $d = 15''$

Let $a = 0.2 * d = 0.2 * 15 = 3''$

$\Rightarrow A_s = 0.45$ in

Trying $A_{smin} = 0.0025 * 12 * 15 = 0.9 in^2$

$\phi M_n > M_u$ ok.

So use 3, #5 bars ($A_s = 0.93 in^2$)

for -ve moment @

supports.

Date:

Shear Design:

$V_u = 1.15 \frac{w_u l}{2} = \frac{1.15(1.44)(11.75)}{2} = 9.8K$

and

$V_u = \frac{w_u l_n}{2} = 9.8K$ (struck through)

$= \frac{1.44 * 12}{2} = 8.64K$

Shear Capacity of beam i.e

$\phi V_c = \phi 2 \sqrt{f'c}\, b_w d$

$= 0.75 * 2 * \sqrt{3000} * 12 * 15 / 1000$

$= 14.79K > V_u$

So, Minimum Reinforcement given by ACI will be applicable i.e

using #3 bars with $A_v = 2(0.11) = 0.22$

(i) $\frac{A_v . f_y}{50 b_w} = \frac{0.22 * 40000}{50 * 12} = 14.67'' c/c$

(ii) $d/2 = 15/2 = 7.5'' c/c$

(iii) $24'' c/c$

Date:

So provide #3 @ 7.5" c/c (both legs) 17

for shear reinforcement throughout the beam length.

Design of Column:

i) Load on Column is

$= 9.8 + 8.64 = 18.44\ K$.

Assuming Column 12" x 12"

Nominal Strength of Column is

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

(ACI 10.3.6) for tied Column.

Let $A_{st} = 1\%\, A_g = 0.01 \times 12 \times 12 = 1.44\ in^2$ (min).

$\Rightarrow \phi P_n = 0.80 \times 0.65 \{0.85 \times 3 (142.56) + 1.44 \times 40\}$

$= 218.98K > P_u$

So, use $A_{st} = 1.44\ in^2$.

Use **4, #6 bars** with $A_s = 1.76\ in^2$

Date:

Tie bars:

using #3 bars as ties in column

As per A.C.I 7.10.5.1, Spacing is minimum of

(i) 16 × Dia of Main bar = $16 \times 6/8$ = 12" c/c.

(b) 48 × Dia of tie bars = $48 \times 3/8$ = 18" c/c

(C) Least Column dimension = 12" c/c.

So use #3, tie bars @ 12" c/c.

Design of Footing:

Column size = 12" × 12".

$f_c' = 3$ ksi, $f_y = 40$ ksi

Allowable pressure = $q_a = 2.204\ k/ft^2$.

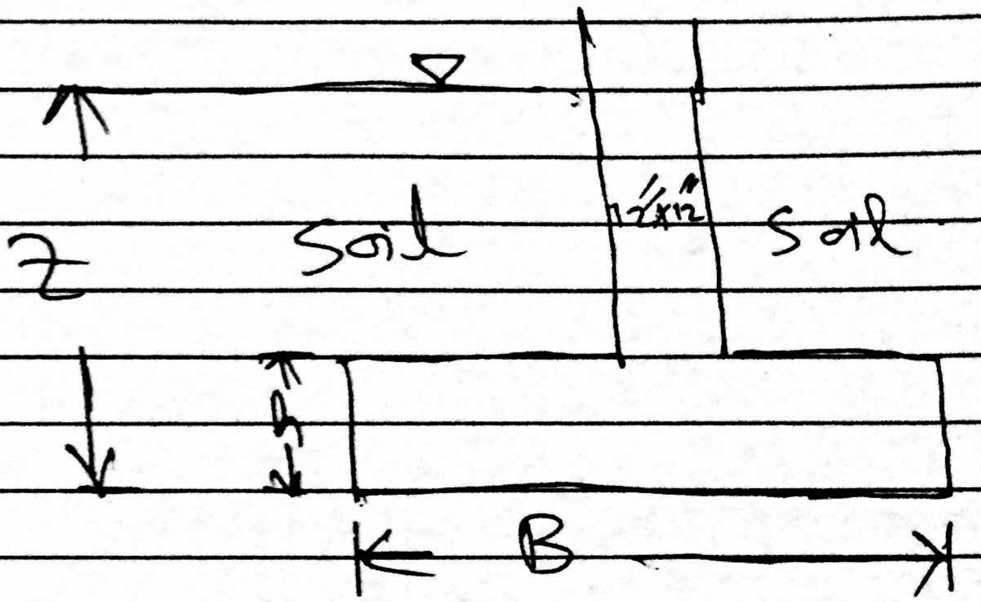
Factored load on Column = 18.44 k.

D.L of Column = $\frac{12 \times 12}{144} \times 11 \times 0.15 = 1.65$ k

Factored = 1.98 k.

Total factored load

$= 1.98 + 18.44 = 20.42\ K$



Assume $h = 1.5' = 18''$
$d = 18 - 4 = 14''$

Assume depth of base of Footing from ground level $= 5'$

Now pressure of above design at 5' depth i.e.

$W = 100*(5-1.5) + 150(1.5)$

$= 575\ lb/ft^2$

Date:

Now effective bearing pressure to carry column's service load.

$q_e = q_a - W = 2204 - 575 = 1629 \ lb/ft^2$

Area of Footing:

$$A_{req} = \frac{\text{Service load on Column}}{q_e}$$

From previous calculations, Service load on column is.

Service load from slab = 0.~~[illegible]~~ k/ft^2.
$= 0.935 + 0.2 = 1.135 \ k/ft^2$

$$V_u = 1.15 \frac{(w_u l_n)}{2} = \frac{1.15 * (1.135)(11.75)}{2}$$

$= 7.7 \ K$.

and

$$\frac{w_u l_n}{2} = \frac{(1.135)(12)}{2} = 6.81 \ K.$$

Total Service load = 14.51 K.

Factored load = 20.42 K.

So, Ratio

~~Area~~ = Area = $\frac{14.51}{1.629}$ = 8.9 ft²

So

B × B = 9 ft²

~~Assume~~

⇒ B = 3 ft. 4'×4'

Area taken (3'~~×3'~~).

Loads:

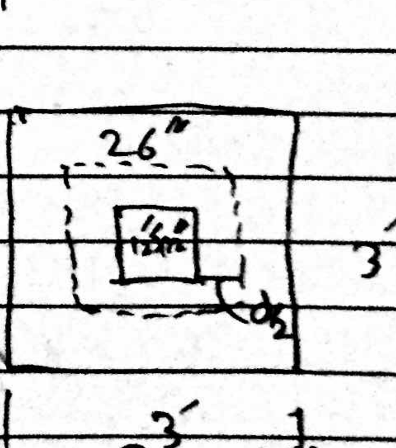
q_u = ~~Bearing~~ pressure for strength design of Footing

= $\frac{\text{Factored load}}{\text{Area}}$ = $\frac{20.42}{B_1 \times B_1}$

= ~~2.27~~ 1.28 K/ft².

Analysis:

(1) Punching Shear:



$V_{up} = q_u \times A - q_u(c+d)^2$

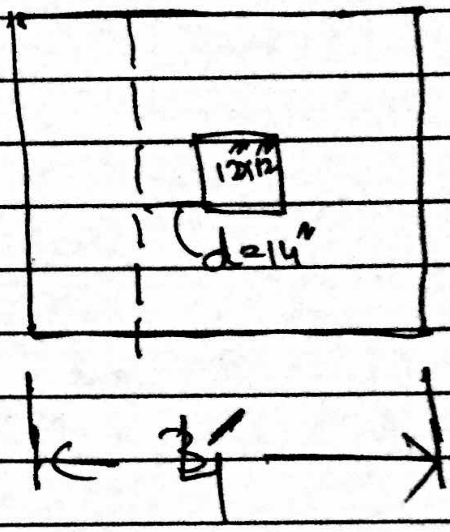
= ~~2.27~~ 1.28 × (9) − ~~2.27~~ 1.28 (12+14)²

$V_{up} =$ ~~...~~ $5.51\ K$

(ii) Beam Shear:

$V_{ub} = q_u\left\{\frac{(4-1)}{2} - \frac{14}{12}\right\}B$

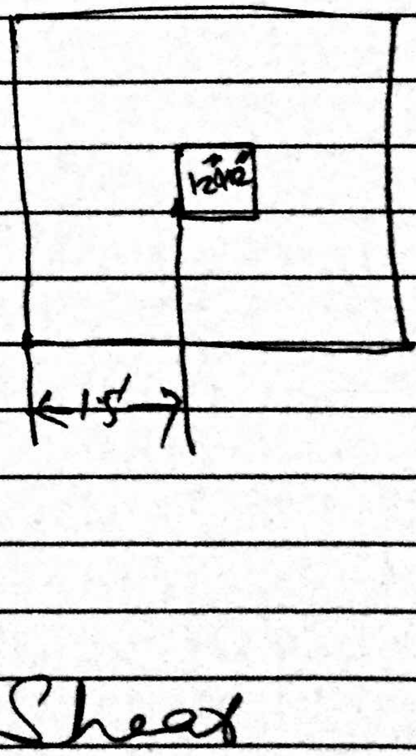
$= 1.71\ K$.



Bending Moment

$M_u = 1.28 \times 4 \times 1.5 \times \frac{1.5}{2}$

$= 69.12\ K\text{-}in$



Design:

Design for Punching Shear

$V_{up} = 5.51\ K$.

Date:

Punching Shear Capacity:

$\phi V_c = \phi 4 \sqrt{f'_c}\, b_o \cdot d$ $\qquad d = 14''$

$b_o = 26''$

$= 0.75 * 4 * \sqrt{3000} * 26 * 14$

$= 59.81K > V_{up}$ OK.

(ii) Design for Beam Shear:

Beam Shear Capacity:

$\phi V_c = \phi\, 2 \sqrt{f'_c} * B * d$

$= 0.75 * 2 * \sqrt{3000} * 4 * 12 * 14 / 1000$

$= 55.2K > V_{ub}$ OK.

Flexure Design:

Check minimum Reinforcement:

$A_{s\,min} = 3\sqrt{f'_c}/f_y\, B*d \geq \left(200/f_y\right) B*d$

$= \left(3 \times \sqrt{3000} \times 4 \times 12 \times 14\right)/40{,}000 \geq \left(\frac{200}{40{,}000}\right)(4 \times 12 \times 14)$

$= 2.76 in^2 \ngeq 3.36 in^2$

Date:

~~Let~~ $A_{s\,min} = 3.36\ in^2$.

$a = \frac{3.36 * 40}{0.85 * 3 * 4 * 12} = 1.09"$

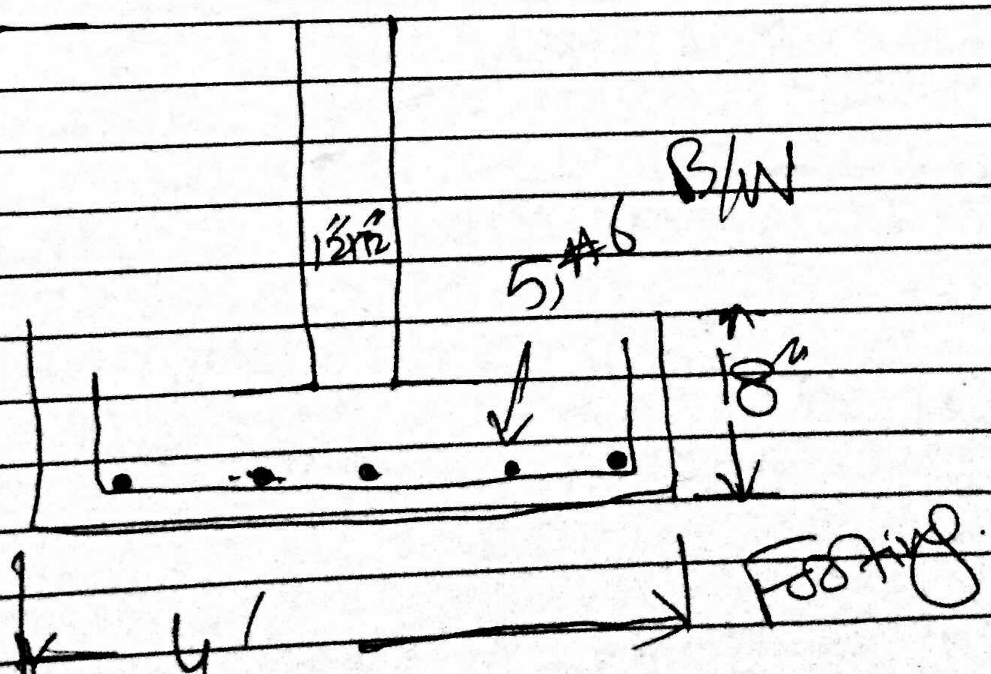
$\Rightarrow \phi M_n = 0.9 * 3.36 * 40{,}000 (14 - 1.09/2)$

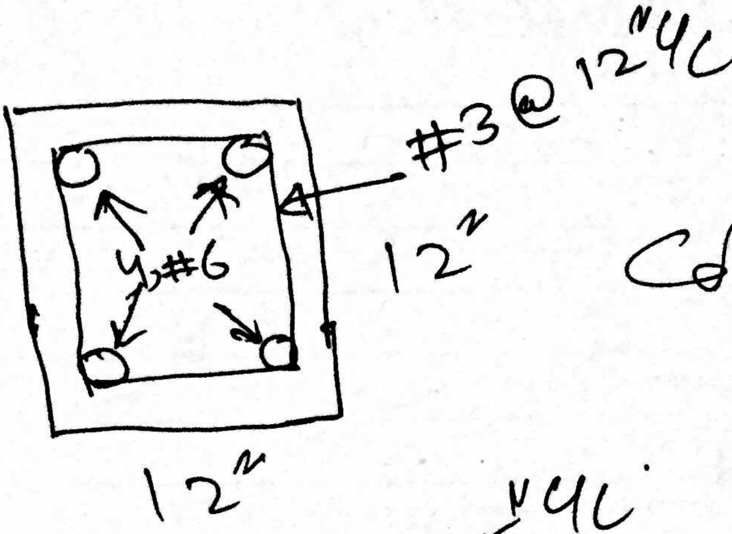
$= 1627\ k\text{-}in > M_u$

$A_s = 3.36\ in^2$.

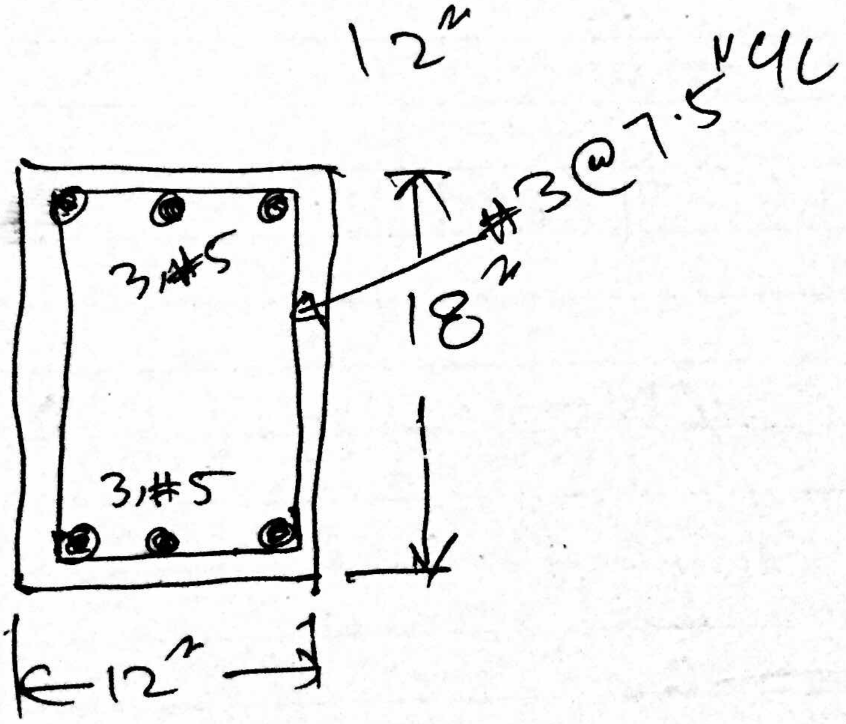
Use **5, #6 Bars** ($A_s = 3.75\ in^2$)

Both way.





Column:



Beam