House area = 29'×60'

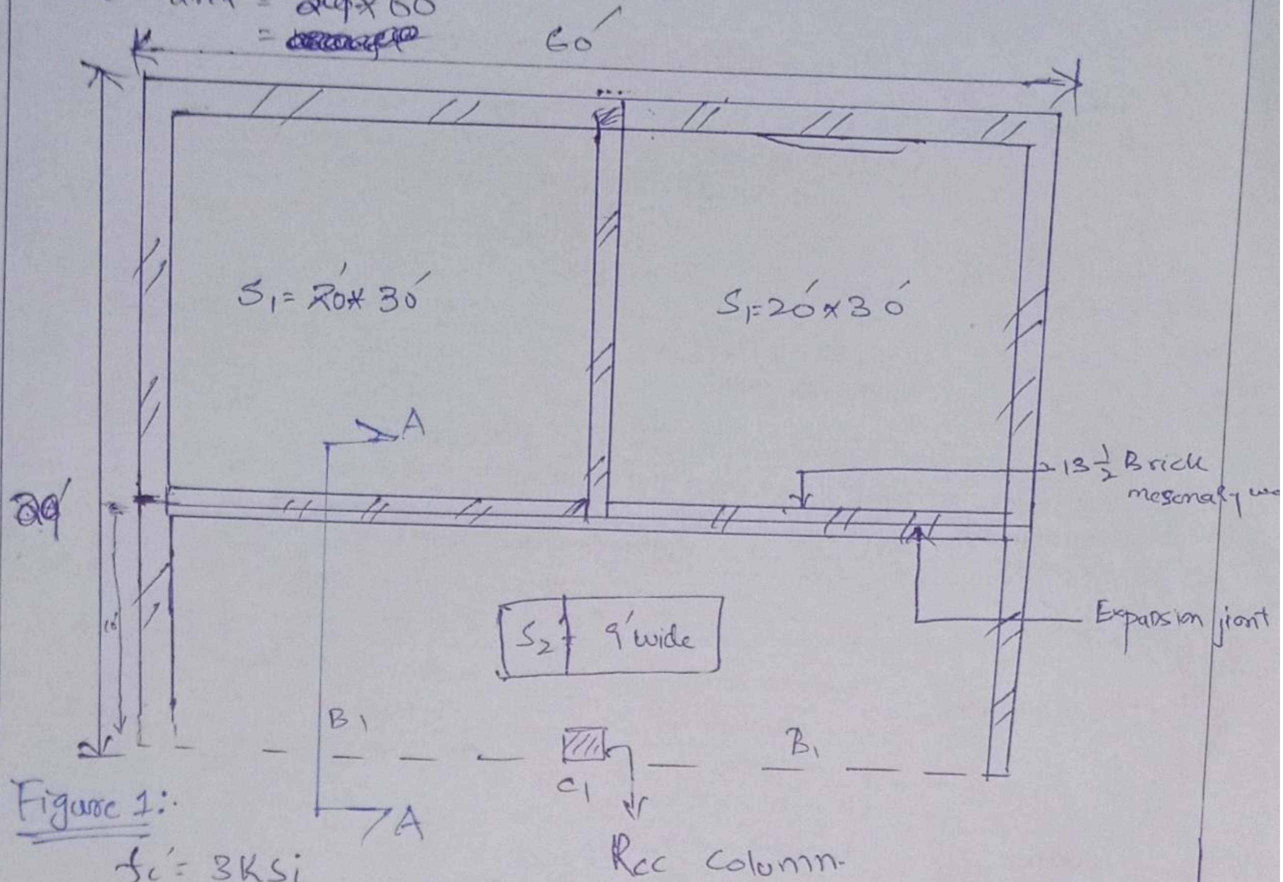

Figure 1:

$f_c' = 3\,ksi$

$f_y = 40\,ksi$ live load = 40 psf (Table 1.7)

load on slab

4" thick mud

2" thick brick tile.

Solution :-

① **Design of slab "S_2" :**

Step No 1: Sizes. $l_b/l_a = \dfrac{40.75}{8} = 5.09 > 2$

$5.09 > 2 \Rightarrow$ One way slab

Assume 5" slab

Span length for end span according to ACI 8.7 is minimum of : (i) $L = l_n + h_f = 8 + (5/12) = 8.42'$

(ii) C/C distance between support = 9.0625

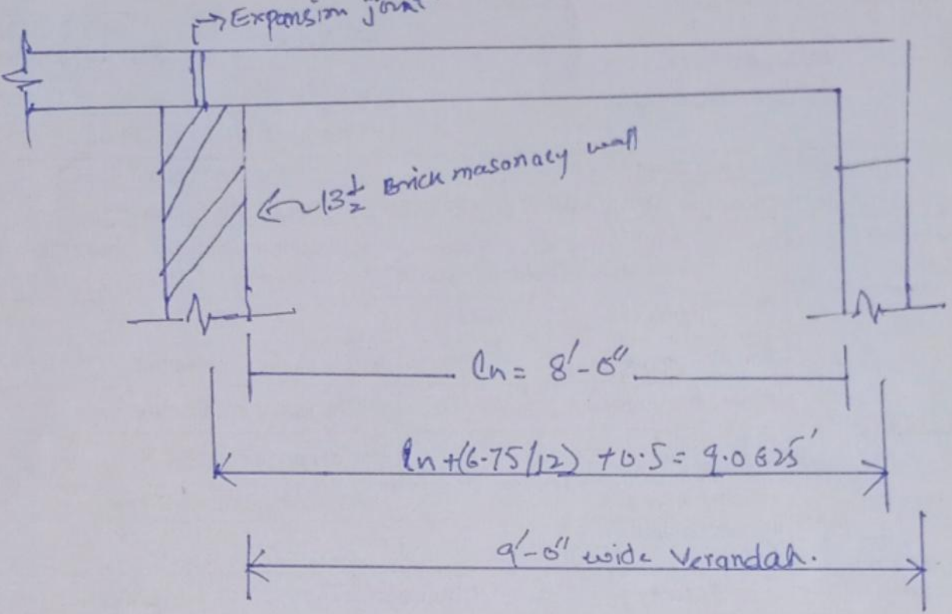

Figure 2: Section A-A

There fore $l = 8.42'$

Slab thickness $(h_f) = (l/20) * (0.4 + f_y/100000)$ For $f_y < 60000$ Psi

$= \frac{8.42}{20} * (0.4 + 40000/100000) * 12$

$= 4.04''$ (Minimum requirement of ACI 9.5.2.1)

There fore

$h_f = 5''$

$d = h_f - 0.75 (3/8) 2 = 4''$

Step#02. Loading

Table 1.1 Dead loads

Material	Thickness	γ (kcf)	load = $\gamma \times$ Thickness (ksf)
Slab	5	0.15	$0.15 \times 5/12 = 0.0625$
Mud	4	0.12	$0.12 \times 4/12 = 0.04$
Brick Tile	2	0.12	$0.12 \times 2/12 = 0.02$

Service D.L $= 0.0625 + 0.04 + 0.02$

$= 0.1225$ ksf

Service live load $= 40$ psf $= 0.04$ ksf

Factored load (w_u) $= 1.2DL + 1.6LL$

$= 1.2 \times 0.1225 + 1.6 \times 0.04$

$= 0.211$ ksf

Step #03 Analysis

$$M_u = \frac{w_u l^2}{8}$$

$$M_u = \frac{0.211 \times 8.42}{8} = 1.87 \text{ ft-k/ft}$$

$$M_u = 22.44 \text{ in-k/ft}$$

Step #04 : Design.

$A_{s\,min} = 0.002bh_f$ (For $f_y = 40$ ksi, ACI 10.5.4).

$= 0.002 \times 12 \times 15 = 0.12 \text{ in}^2$

$$a = \frac{A_{s\,min} \times f_y}{0.85 f_c' b} = \frac{0.12 \times 40}{0.85 \times 3 \times 12} = 0.156 \text{ in}$$

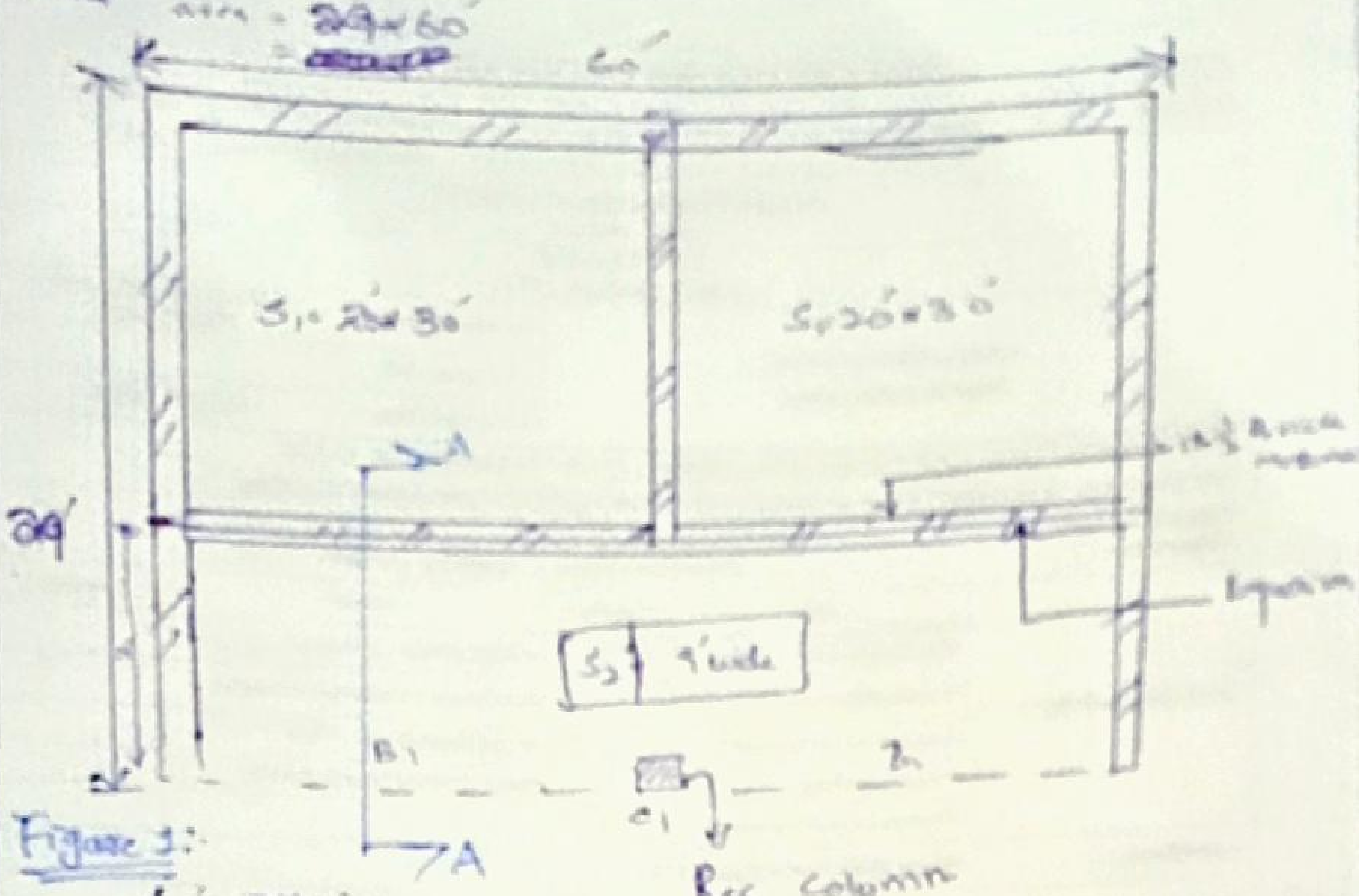

Figure 1:

$f_c' = 3Ksi$

$f_y = 40Ksi$ Live Load = 40psf (Table 1.1)

load on slab

4" thick mud

2" thick brick tile.

Solution:

① Design of slab "S2":

Step No 1. Sizes. $l_b/l_a = \dfrac{40.75}{8} = 5.09 > 2$

$5.09 > 2 \Rightarrow$ One way slab

Assume 5" slab

$\phi M_{n(min)} = \phi A_{s\,min} f_y (d - a/2)$

$= 0.9 * 0.12 * 40 * (40 - \frac{0.156}{2})$

$= 16.94 < M_u$

There fore

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

Take $a = 0.2d$

$A_s = 22.44 / 0.9 \times 40 \times (4 - (0.2 * 4)/2)$

$A_s = 0.173 \text{ in}^2$

$a = 0.173 * 40 / (0.85 * 3 * 12)$

$a = 0.226 \text{ in}$

$A_s = 22.44 / 0.9 \times 40 * (40 - \frac{0.226}{2})$

$A_s = 0.160 \text{ in}^2$

$a = \frac{0.160 * 40}{0.85 \times 3 \times 12} = 0.209 \text{ in}$

$A_s = \frac{22.44}{0.9 * 40 (4 - \frac{0.209}{2})}$

$A_s = 0.160 \text{in}^2 \longrightarrow$ OK -

using $3/8"$ ϕ (#3), with bar area ($A_b = 0.11 \text{in}^2$).

Spacing = $\frac{\text{Area of one bar } (A_b)}{A_s}$

$= \frac{0.11 \, in^2}{0.160 \, in^2/ft} * 12 = 7.5'' \approx 6''$

$\boxed{\text{We can use } \#3 @ 6'' \, c/c}$

Shrinkage Steel or temprature Steel. (A_{st})

$A_{st} = 0.002 \, b h_f$

$= 0.002 * 12 * 5 = 0.12 \, in^2$

Using $3/8''$ - ($A_b = 0.11 \, in^2$)

Spacing = A_b / A_s = $\frac{0.11}{0.12} * 12 = 11' \, c/c$

$\boxed{\text{We Can use } \#3 @ 9' \, c/c}$

Maximum spacing for main Steel Area in one ways according ACI 7.12.2 is minimum of

i) $5 h_f = 5 \times 5 = 25''$

ii) $18''$

Therefore $6''$ Spacing is OK.

Maximum Spacing for Shrinkage / Temprature Steel in one way slab. according ACI 7.12.2 is minimum of

i) $5h_f = 5 \times 5 = 25''$

ii) $18''$

Therefore $9''$ is ok.

(2) Design of slab "S_1":

Step # No. 1: $S_{izes} = l_b / l_a = \frac{30}{20} = 1.5 < 2$

$1.5 < 2 \Rightarrow$ Two way slab

Minimum depth of two way slab is

$$h_{min} = \frac{\text{Perimeter}}{180}$$

$$= \frac{2(20+30) * 12}{180}$$

$$h_{min} = 6.66 \text{ in}$$

$$\boxed{\text{Assume } h = 7 \text{ in}}$$

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Step #02. Loads:

$W_u = 1.2 \times 0.1225 + 1.6 \times 0.04$

$\boxed{W_u = 0.211\ ksf}$

Moments:

According ACI-318-95 code provision (13.15.1) that a slab system may be designed by any procedure. Following approved method we will use

$M_{a,pos(dl+ll)} = M_{a,pos\,dl} + M_{a\,pos\,l.l}$
$= C_{a,pos\,dl} \times W_{u\,dl} \times l_a^2 + C_{a,pos\,ll} \times W_{u,ll} \times l_a^2$

$M_{b\,pos\,(dl+ll)} = M_{b,pos\,dl} + M_{b\,pos\,l.l}$
$= C_{b,pos\,dl} \times W_{u\,dl} \times l_a^2 + C_{b\,pos,ll} \times W_{u,ll} \times l_b$

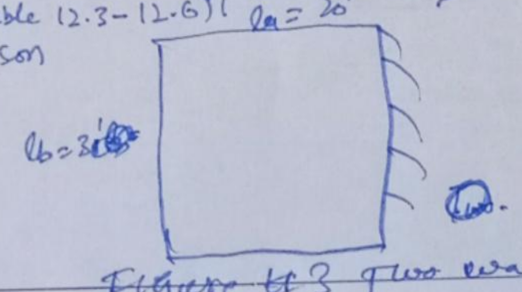
$M_{a,neg} = C_{a\,neg} \cdot W_u l_a^2$

$M_{a\,neg} = C_{a\,neg}\,W_u l_b^2$

Where C_a, C_b = Tabulated Coefficient
Ref: table 12.3–12.6 Nelson

$m = \frac{l_a}{l_b} = \frac{20}{30}$

$\boxed{m = 0.66}$



~~Figure # 3 Two way slab~~

Refer to tables 12.3 to 12.6, of Nelson 12th Ed we will get coefficient values.

$M_{a,neg} = C_{a,neg} * W_u * l_a^2$

$= 0.088 * 0.211 * 20^2$

$= 7.42 ft\text{-}K = 89\ in\text{-}K.$

$M_{b,neg} = C_{b,neg} * W_u * l_b^2$

$= 0 * 0.211 * 30^2 = 0\ in\text{-}K$

$M_{a,pos\ dl} = C_{a,pos\ dl} * W_{u,dl} * l_a^2$

$= 0.048 * 0.147 * 20^2 = 2.82\ in\text{-}K$

$= 33.86\ in\text{-}K$

$M_{b\ pos\ dl} = C_{b,pos,dl} * W_{u,dl} * l_b^2$

$= 0.012 * 0.147 * 30^2$

$= 1.5876\ ft\text{-}K$

$= 19.05\ in\text{-}K$

$M_{a,pos,ll} = C_{a,pos,ll} * W_u * l_a^2$

$= 0.055 * 0.064 * 20^2 = 1.47\ ft\text{-}K$

$= 17.64\ in\text{-}K$

$M_{b\ pos\ ll} = C_{b,pos,ll} * W_{u\ ll} * l_b^2$

$= 0.016 * 0.064 * 30^2 = 0.9216 \times 12 = 11\ in\text{-}K$

There fore finally, we have,

$M_{a\,neg} = 89$ in-K

$M_{b\,neg} = 0$ in-K

$M_{a,pos(dl+ll)} = 33.86 + 17.64 = 51.5$ in-K

$M_{b,pos(dl+ll)} = 30.05$ in-K.

Step # 04 : Design

$$A_{s\,min} = 0.002\, b\, h_f$$

$$= 0.002 * 12 * 7$$

$$= 0.168\ in^2$$

$$a = \frac{0.168 * 40}{0.85 \times 3 \times 12} = 0.21$$

$$\phi M_{n(min)} = \phi A_{s\,min}\, f_y\, (d - a/2)$$

$$= 0.9 * 0.168 \times 40\, (6 - 0.21/2)$$

$$= 35.65 \text{ in-K (capacity provided)}$$

$\phi M_{n(min)}$ is greater than $M_{b,pos,(dl+ll)}$ but less than $M_{a\,neg}$ and $M_{a\,pos(dl+ll)}$

$M_{b,pos,(dl+ll)} = 30.05$ in k $< \phi M_{n(min)}$

There fore $As_{min} = 0.168\ in^2$ Governs

Using 3/8" ($A_b = 0.11\ in^2$)

$$\text{Spacing} = \frac{0.11}{0.168} \times 12 = 7.85"$$

Maximum Spacing according to ACI 13.3.2

For Two way slab is: $2h_f = 2 \times 7 = 14"$

there for maximum Spacing 14" Governs

Finally we can use

#3@ 9 c/c

Provide #3@ 4/c as negative reinforcement a longer direction.

$M_{a,pos}\ (dl+ll) = 51.05 > \phi M_n$

let $a = 0.2d = 0.2 * 6 = 1.2$ in

$$A_s = \frac{51.05}{0.9 * 40\,(6 - \frac{1.2}{2})} = 0.26\ in^2$$

$$a = \frac{0.262 \times 40}{0.85 \times 3 \times 12} = 0.342 \text{ in}$$

$$A_s = \frac{51.05}{0.9 \times 40 \times (6 - \frac{0.342}{2})} = 0.243 \text{ in}^2$$

$$a = \frac{0.243 \times 40}{0.85 \times 3 \times 12} = 0.317$$

$$A_s = \frac{51.05}{0.9 \times 40 \times (6 - \frac{0.317}{2})} = 0.242 \text{ in}^2$$

$A_s = 0.242 \text{ in}^2 \rightarrow$ OK

By using 3/8" ($A_b = 0.11 \text{ in}^2$)

$$\text{Spacing} = \frac{0.11}{0.242} \times 12 = 5.45'' \approx 5''$$

We can use #3 @ 5 c/c

$M_{a,neg} = 7.42$ ft-k $= 89$ in-k

let $a = 0.2d = 0.2 \times 6 = 1.2$

$$A_s = \frac{89}{0.9 \times 40 (6 - 1.2/2)} = 0.457$$

$$a = \frac{0.457 \times 40}{0.85 \times 3 \times 12} = \text{in}$$

$$a = 0.598 \text{ in}$$

$$A_s = \frac{89}{0.9 * 40(6 - \frac{0.598}{2})} = 0.433 \text{in}^2$$

$$a = \frac{0.433 \times 40}{0.85 \times 3 \times 12} = 0.566 \text{in}$$

$$A_s = \frac{89}{0.9 \times 40(6 - \frac{0.566}{2})} = 0.432 \text{ in}^2$$

$$a = \frac{0.432 \times 40}{0.85 \times 3 \times 12} = 0.564 \text{ in}$$

$$A_s = \frac{89}{0.9 \times 40(6 - \frac{0.564}{2})}$$

$$\boxed{A_s = 0.432 \text{ in}^2} \Rightarrow \text{OK}$$

using 3/8" ($A_b = 0.11$)

$$\text{Spacing} = \frac{0.11}{0.432} \times 12 = 3.05 \text{ in} \simeq 3$$

Use #3 @ 3" c/c

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Beam design :

2 span Continous.

Exterior support = 9" brick masonary wall

$f_c' = 3ksi$

$f_y = 40ksi$

Colum = 12" x 12"

Step 1 :-

Sizes:

According ACI 9.5-21 Table 9.5 a:

$h_{min} = l/18.5$

l = clean span + depth of member (beam) < c/c distance b/w support.

Table. 3 clear Span

Case	Clear Span (l_n)
End span (one end cont)	$12.375 - (12/12)12 = 11.895$

depth of beam = 18"

l_n + depth of Beam = $11.875' + (18/12) = 13.375$

c/c distance b/w beam support = $12.375 + \frac{4.5}{12}$

= $12.75'$

There for $l = 12.75$

$$h = \frac{12.75}{18.5} \cancel{*} \left(0.4 + \frac{40000}{100000}\right) \times 12$$

$h_{min} = 6.62$ (Minimum requirements of ACI 9.5.2.1)

Take $h = 1.5' = 18"$

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

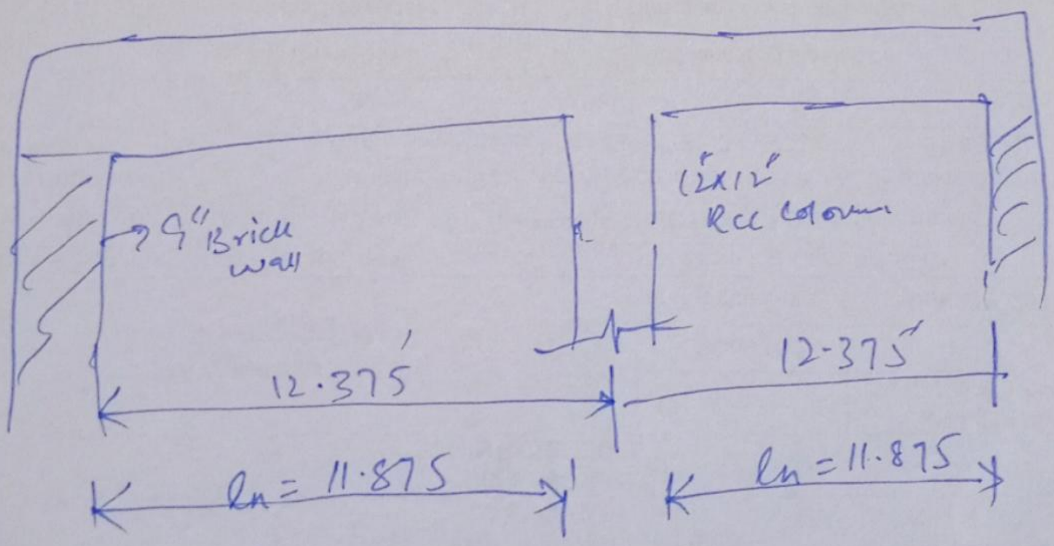

Figure # 4 c/c distance & clear Span of beam

Step # 02 Loads

Service dead load = 0.0625 + 0.04 + 0.02 = 0.1225 ksf

" live load = 40 psf = 0.04 ksf

Beam is supporting 5" slab, therefore

Service D.L from slab = 0.1225 × 5 = 0.6125 k/ft

Service DL from beam's self. weight

$= h_w b_w \gamma_c = (13 \times 12/144) \times 0.15 =$

$= 0.1625$ k/ft

Total Dead load = 0.6125 + 0.1625

= 0.775 k/ft

Service live load = 0.04 × 5 = 0.2 k/ft

$W_u = 1.2 * 0.775 + 1.6 * 0.2$

$\boxed{W_u = 1.2 \text{ k/ft}}$

Step #. 03

Refer to ACI 8.3.3, For ACI moment and Shear Coefficients.

(1) At interior support.

M_{neg} = Coefficient * $W_u l_n^2$

$= \frac{1}{9} \times \{ 1.25 \times (11.875)^2 \}$

$= 19.59$ ft-k $= 233.08$ in-k

(2) At Mid Span.

$M_{pos} = \frac{1}{11} * (1.25 * (11.875)^2)$

$= 16.02$ ft-k $= 192.24$ in-k

$V_{int} = \frac{1.15 W_u l_n}{2} = \frac{1.15 * 1.25 * 11.875}{2} = 8.54$ k

$V_{u(int)} = 8.54 - 1.25 \times 1.25 = 6.97$ k.

$V_{u(ext)} = 7.42 - 1.25 \times 1.25 = 5.86$ k.

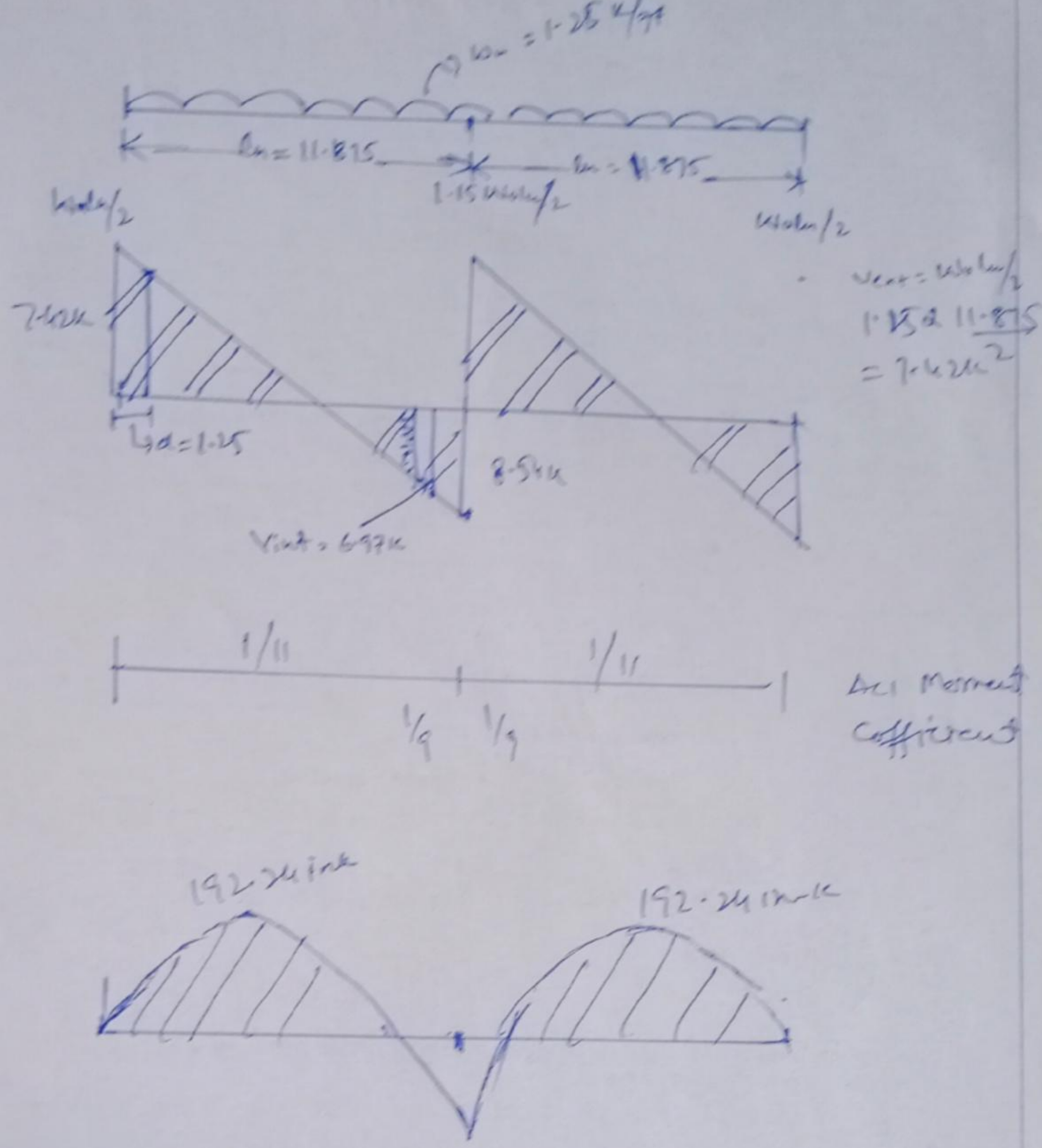

Figure 5: Approximate Shear force and bending moment diagram.

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Step # 04 Design.

(A) Flexure Design

① For Positive moment.

Step (a) Acc. to ACI 8.10, b_{eff} for L-beam is minimum of

(i) $6h_f + b_w = 6 \times 5 + 12 = 42''$

(ii) b_w + span length of beam/12 $= 12 + \left(\frac{12.75 \times 12}{12}\right)$

$= 24.75''$

(iii) clear distance next to web = N/A

So $b_{eff} = 24.75''$

Step (b)

Check if beam is to be design as rectangular or L-beam

Trial #1

Assume $a = h_f = 5''$

$$A_s = \frac{192.24}{0.9 \times 40 \times (15 - 5/2)} = 0.427 \text{ in}^2$$

$$a = \frac{0.427 \times 40}{0.85 \times 3 \times 24.75} = 0.271 < h_f$$

Therefore design beam is rectangular

Trial # 02

$$A_s = 0.358 \text{ in}^2$$

$$a = 0.228 \text{ in}^2$$

Value is close to previous so

$$A_s = 0.358 \text{ in}^2 \rightarrow ok$$

Step (c) check for maxi/minimum reinforcement

$$A_s = \rho_{max} b_w d$$

$$A_{s\,max} = 0.0203 \times 12 \times 15 = 3.654 \text{ in}^2$$

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

$$= 0.005 \times 12 \times 15 = 0.9 \text{ in}^2$$

$A_s = 0.358 < A_{s\,min}$ so $A_{s\,min}$ Govern

use #5 ($A_b = 0.31 \text{ in}^2$)

$$\text{No of bar} = \frac{A_s}{A_b} = \frac{0.9}{0.31} = 2.9 \approx 3 \text{ bars}$$

So 4#5 bar

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For Interior Negative moments

Step ④ Now take $b_w = 12$ instead of b_{eff} because flange in tension.

$M_u = 235.08$ in K

Trial #01 Let $a = 0.2d$

$$A_s = \frac{235.08}{0.9 \times 40 (15 - (0.2 \times 15/2))}$$

$A_s = 0.484$ in2

$$a = \frac{0.484 \times 40}{0.85 \times 3 \times 12} = 0.632$$

Trial #02

$A_s = 0.44 \text{ in}^2$

$a = 0.58$ in

Trial #03

$A_s = 0.44 \text{ in}^2 < A_{s\,min}$ So

$A_{s\,min}$ Governs

using #5 ($A_b = 0.31$)

$$\text{No of bar} = \frac{0.9}{0.31} = 2.91 \simeq 3 \text{ bar}$$

use 4 #5 bars

(B) Shear design:

Step # (a)

$d = 15'' = 1.25'$

$V_{(ext)} = 5.86\,k$

$V_{(int)} = 6.97\,K$

Step (b)

$\phi V_c = \phi 2\sqrt{f_c}\, b_w d$

$\phi V_c = 0.75 \times 2\sqrt{(3000)} \times 12 \times 15/1000$

$\phi V_c = 14.78\,K > V_{u(ext)}\ \&\ V_{u(INT)}$

Theoretically no reinforcement is needed but we will provide.

Max spacing & minimum reinforcement permitted by ACI 11.5.4 & 11.5.5.3 shall minimum of

(i) $A_v f_y / 50 b_w = \dfrac{0.22 \times 40000}{50 \times 12} = 14.67''\ c/c$

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

(iii) $24\ c/c$

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

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there fore spacing given under "max. spacing requirements of ACI" is ok

Provide #3, 2 legged @ 7.5" c/c's throughout, starting at $S_d/2 = 7.5/2$ $= 3.75"$ from face of the support

(9) Design Column:

load on Column:

$$P_u = 2V_{int} = 2 \times 8.54 = 17.08\,K$$

Gross Area of Colum X-section $= A_g = 12 \times 12$

$$A_g = 144\, in^2$$

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

(11) Design:

Nominal strength (ϕP_n)

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

let $A_{st} = 1\%$ of A_g

$\phi P_n = 0.80 \times 0.65 \times \{0.85 \times 3 (144 - 0.01 \times 144) + 0.01 \times 144 \times 40\}$

$\phi P_n = 218.98\ K > (P_u = 17.08K)$, $\Rightarrow$ ok

$A_{st}\ 0.01 \times 144 = 1.44\ in^2$

using $3/4\ \phi$ (#6) with $A_b = 0.44 in^2$

No. of bars $= \frac{1.44}{0.44} = 3.27 \simeq 4$ bar

use 4#6 bar

Tie bars :-

using $3/8''\ \phi$ (#3), tie bar for $3/4''$ (# main ba

(ACI 7.10.5)

Spacing for Tie bars According to ACI 7.10.5.1 is minimum of

(a) 16 × dia of main bar $= 16 \times 3/4 = 12''$ c/c

(b) 48 × dia of tie bar $= 48 \times 3/8 = 18''$ c/c

(c) least column dia $= 12''$ c/c

Finall we can use

#3 tie bar @ 9" c/c

Footing design :-

Design of wall footing

Dead load = 0.1225 ksf

live load = 0.04 ksf

Assume the data

Bottom of the footing is to be 4' below the final grade,

Soil weights = 100 lb/ft³

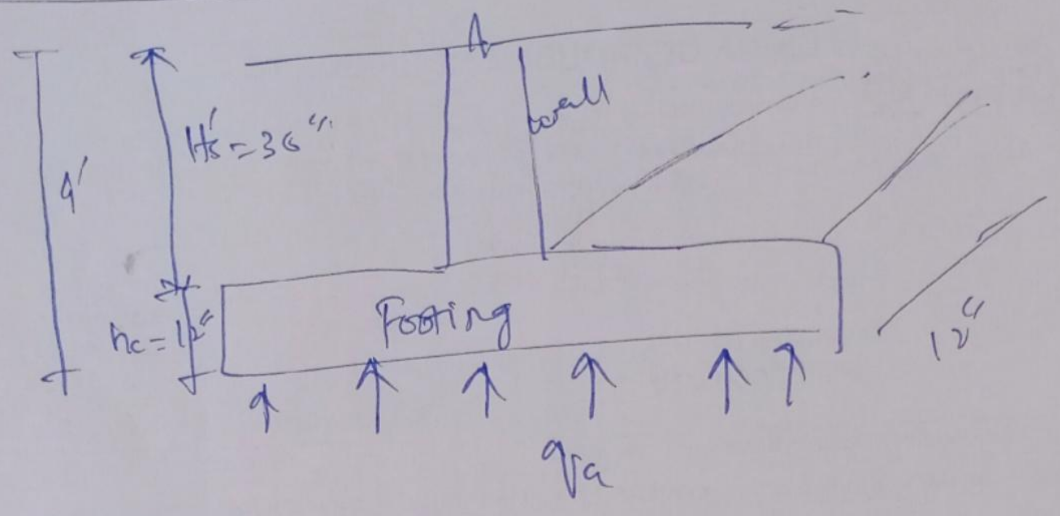
Allowable Soil pressure = $q_a = 4$ ksf

$f_y = 60$ ksi, $f_c = 3$ ksi

$\gamma_{conc} = 150$ lb/ft³

$h_c = 12$

$d = 12 - 3.5 = 8$ in. 3.5 → clear cover



<u>Solution:</u>

1. Step #01 Effective Soil Pressure "q_e"

$$q_e = q_a - h_c\gamma_c - H_s'\gamma_s = 4000 - \left(\frac{12'' \times 150}{12}\right) - 3(100)$$

$$q_e = 3550 \text{ Psf} = 3.55 \text{ Ksf}$$

<u>Step # 02</u> Width of footing required

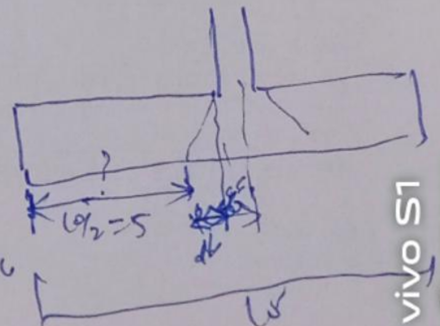
$$w = \frac{D+L}{q_e} = \frac{20+15}{3.55}$$

$$w = 9.86' \text{ Say } 10 ft'$$

<u>Step # 3:</u>

$$d = \frac{V_u}{\phi 2\sqrt{f_c'}\, bd}$$

$$V_u = \left(\frac{10}{2} - \frac{6}{2} - \frac{8.5}{12}\right) q_u$$



q_u = ultimate Bearing capacity

$q_u = 1.2\,DL + 1.6\,L.L.$

$q_u = 1.2 \times 0.1225 + 1.6 \times 0.04$

$$\boxed{q_u = 0.211\ ksf}$$

Now $V_u = \left(\frac{10}{2} - \frac{6}{12} - \frac{8.5}{12}\right) * 4.8$

$V_u = 18.2k = 18200\,lb$

$$d = \frac{V_u}{\phi 2\sqrt{f_c'}\, b_w} = \frac{18200}{0.75 * 2\sqrt{3000} \times 12}$$

$d = 18.46''$

$h = d + cover = 18.46' + 3.5' = 21.96$

$21.96\ in > 12\ in$

Actual > Assumed

Not OK

So.

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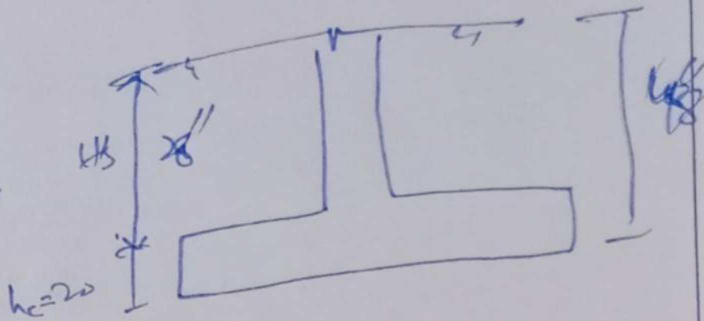
Again 20" footing



$h = 20$
$d = 20 - 3.5 = 16.5$"

Step #01 :

$$q_e = 4000 - \left(\frac{20}{12} \times 150\right) - \left(\frac{28}{12} * 100\right)$$

$$\boxed{q_e = 3.517 \text{ Ksf}}$$

Step # 02 width of footing

$$W = \frac{20+15}{3.517} = 9.95' \text{ say } 10'$$

Step #03 Depth required for Shear

$$V_u = \left(\frac{10}{2} - \frac{6}{12} - \frac{16.5}{12}\right) q_u \qquad q_u = 0.211 \text{ Ksf}$$

$$V_u = \left(\frac{10}{2} - \frac{6}{12} - \frac{16.5}{12}\right) * 0.211 \text{ Ksf}$$

$$\boxed{V_u = 0.65 \text{ K}}$$

$$d = \frac{V_u}{\phi 2\sqrt{f'c}\, b_w} = \frac{15000}{0.75 \times 2 \times \sqrt{3000} \times 12} = 15.21''$$

$h = 15.21 + 3.5 = 18.71$ use 20' total depth

$h = 20'$
$d = 16.5'$

Step#4 Steel Area (Main).

Cantilever length $= \frac{10}{2} - \frac{6}{12} = 4.5'$

$M_u = 4.5 \times 4.8 \times \frac{4.5}{2}$

$M_u = 48.6$ K-ft

$$\frac{M_u}{\phi b d^2} = \frac{48.6 \times 1000 \times 12}{0.9 \times 12 (16.5)^2} = 198.3 \text{ Psi}$$

Refering to Appendix Table A.12

when $\frac{M_u}{\phi b d} = 198.3$

Then by Interpolation $\rho = 0.0034$

$A_s = \rho bd$

$= 0.00345 \times 12 \times 16.5$

$\boxed{A_s = 0.68\ in^2}$

using #7 bar @ 10" c/c

$\boxed{A_{s\,selected} = 0.72\ in^2}$

Step # 5: Temprature steel

$A_s = \rho bd = 0.0018 \times 12 \times 20$

$A_s = 0.432\ in^2$

#5 @ 8" c/c

Step #6 Development length.

$\psi_t = \psi_e = \psi_s = \lambda = 1$

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

So $\frac{l_d}{d_b} = 32.86$

$\frac{l_d}{d_b} \times \frac{A_{s\,req}}{A_{s\,selected}} = 32.86 \times \frac{0.68}{0.72} = 31.0$

So $\boxed{l_d = 28''}$

Square Footing for Column

D.L = 0.1225 Ksf Lets Height = 12'

Column weight = $\frac{12}{12} \times \frac{12}{12} \times 12' \times 0.15' = 1.8$ Ksf

Total $\boxed{\text{Dead Load} = 1.9225 \text{ Ksf}}$

Live load = 0.04 Ksf

$\gamma_s = 100\ lb/ft^3$

$f_y = 40000$ psi = 4 Ksi

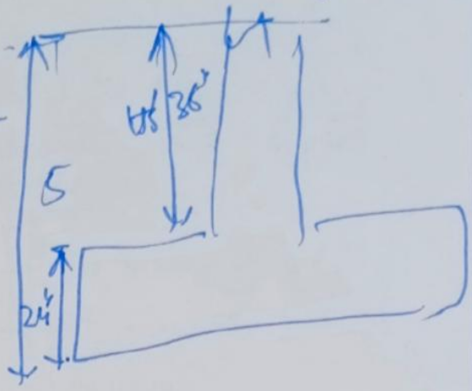
$f'_c = 3000$ psi = 3 Ksi

$q = 5000$ Psf = 5 Ksf

$\gamma_c = 150\ lbs/ft^3$

$h_c = 24"$ $d = 19.5$

$H_s = 36"$



Sol:

Step(1) $q_e = 5000 - (\frac{24}{12} \times 150) - (\frac{36}{12}) \times 100$

$q_e = 4400\ psf = 4.40\ ksf$

Step #(2) Area of footing $= \frac{P_{DL} + P_{LL}}{q_e}$

$= \frac{1.922 + 0.84}{4.4}$

$=$ 2

Step #(3) Ultimate Bearing capacity

$q = 6.12$

Step# 04

$$\boxed{V_{u2} = 4420 \text{ kb}}$$

① $d = \frac{V_{u2}}{\phi 4 \sqrt{f_c'}\, b} = 18.95 < 19.5$ ok

~~Step #05~~

② $d_2 = \frac{V_u}{\phi \left(\frac{\alpha_s d}{b_o} + 2\right) \sqrt{f_c'}\, b_o} = 10.12 < 19.5$ ok

Step# 05 Depth required

$d = 13.71 < 19.5 \rightarrow$ ok

~~Use $h = 24''$ ⇒ Total depth~~

Step# 05

Moment $= 4047$ ft-k

Step. # 06.

Area of Steel $= \rho b d$

$= 6.95$ in

Use 9 #8 bars in both direction

Development length:

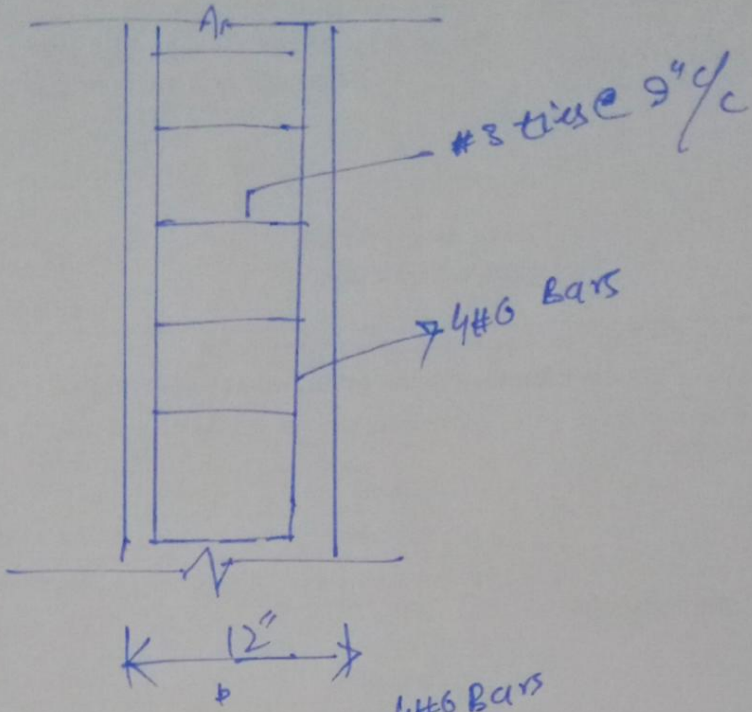
$$\frac{l_b}{d_b} \frac{A_{s\,req}}{A_{s\,se}} = 32.86 \times \frac{6.95}{7.07} = 32.30$$

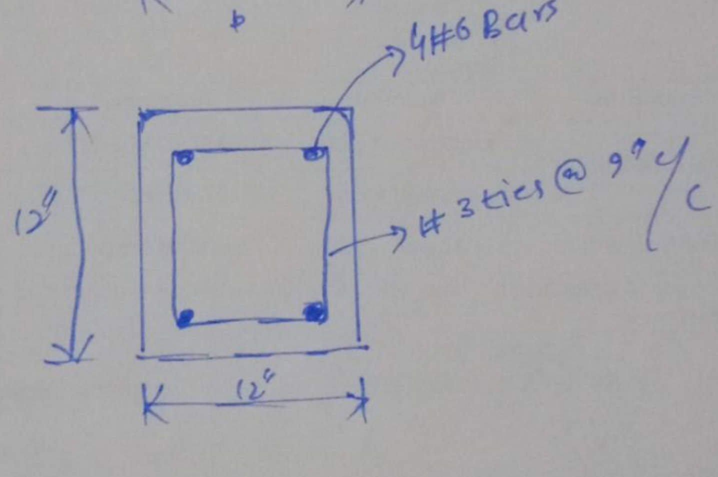
$$l_d = 32.30 \times d_b = 32.30 \times 1$$

$$l_d = 32''$$ ok

END

Ⓒ Column:





the END

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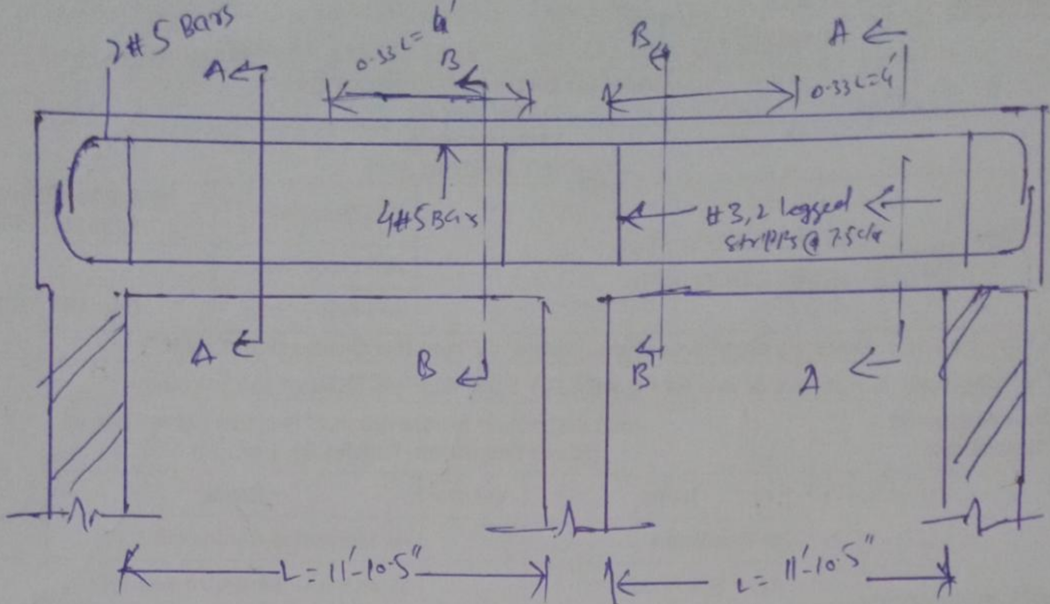
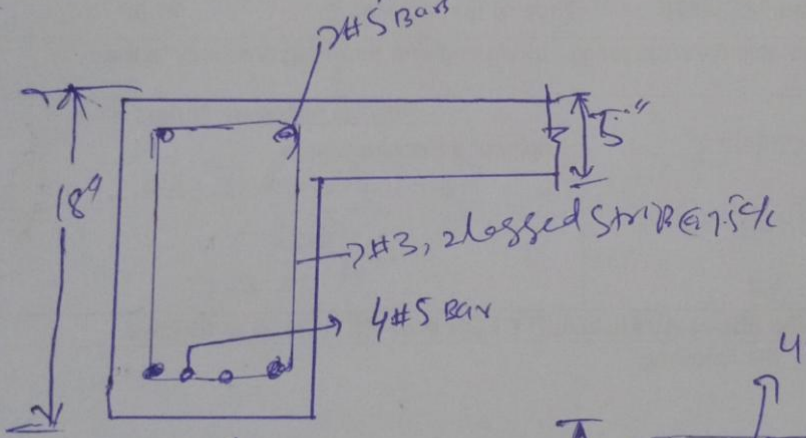
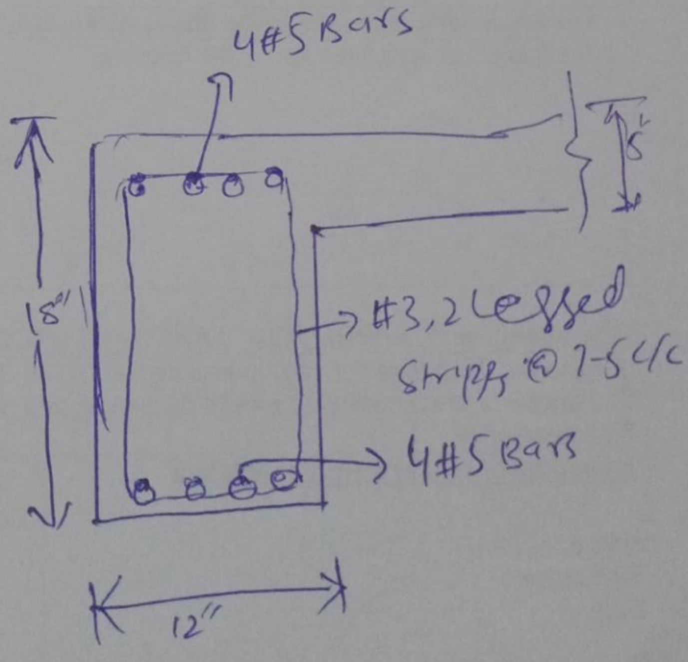
(B) Beam :



Beam detail

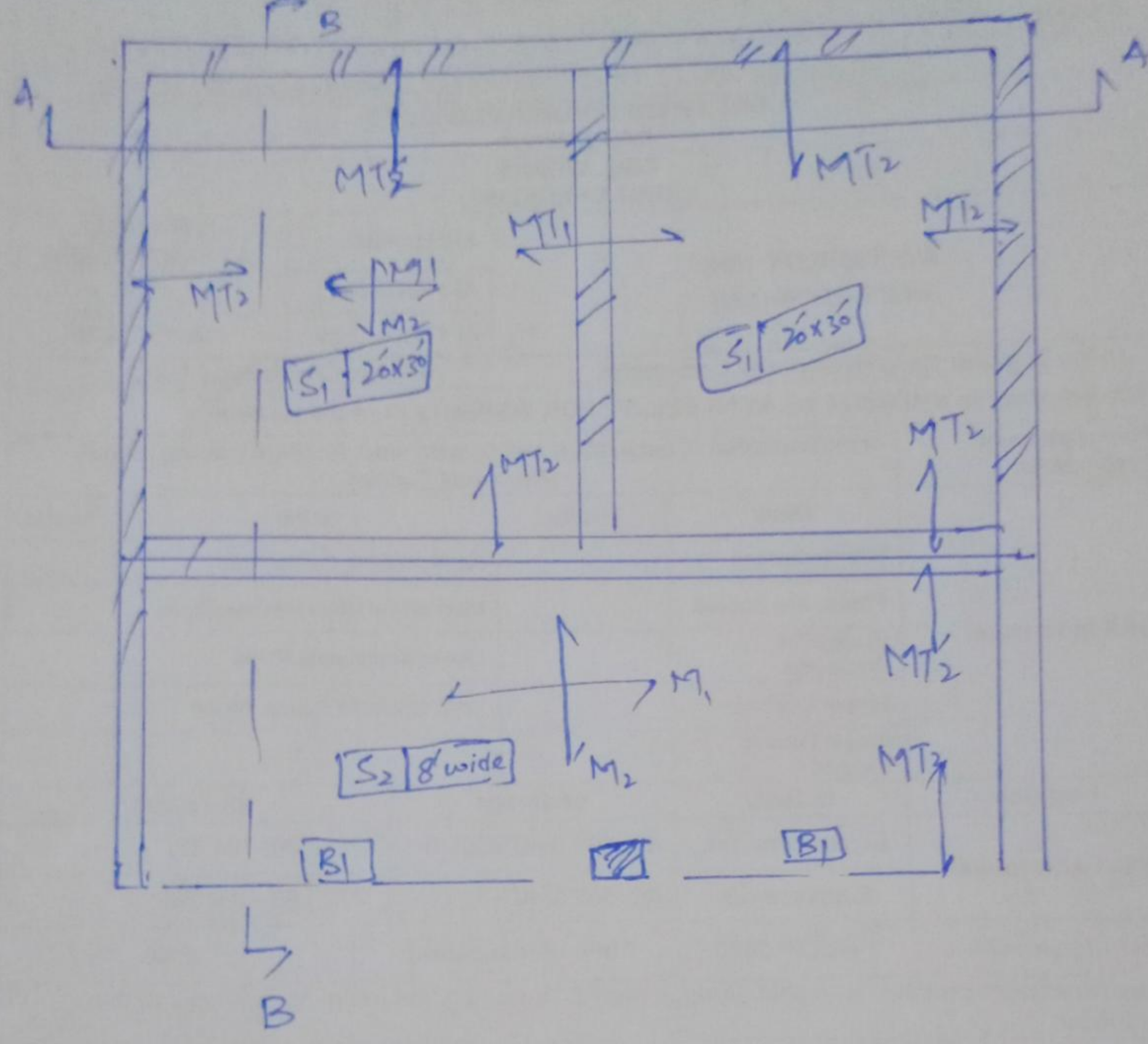


Section A-A

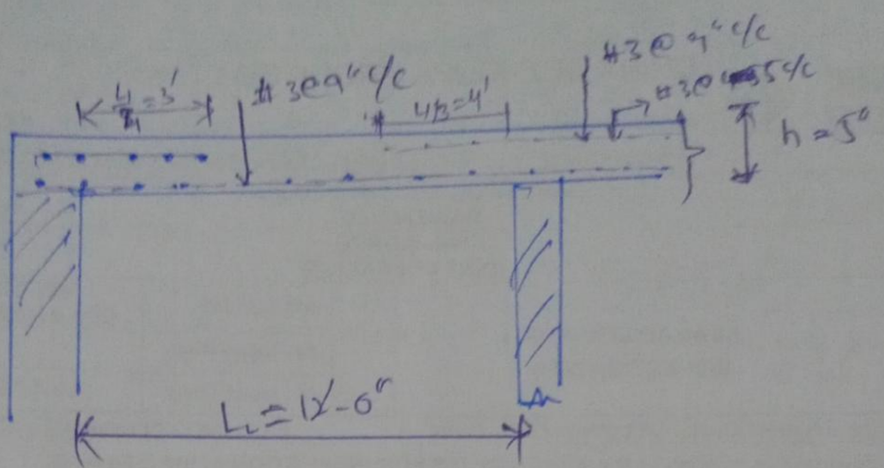


ID # 15345

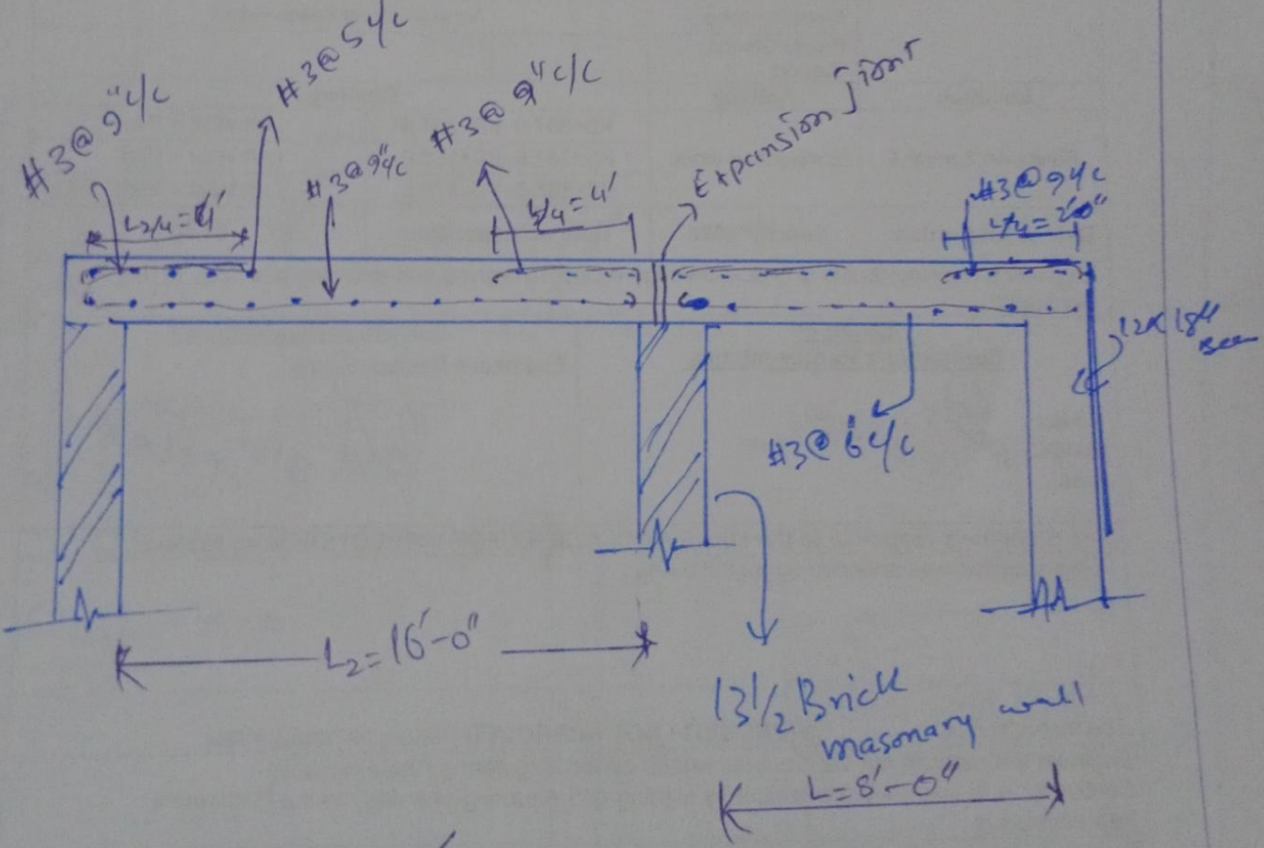
Drafting :- (A) Slab "S_1" and "S_2"



Panel	Depth (in)	Mark	Bottom Reinforcement	Mark	Top reinforcement	
S_1	7"	M_1	3/8" Φ @ 9" c/c	MT_1	3/8 Φ @ 8 c/c	Continuous End
				MT_2	3/8 Φ @ 9 c/c	Non continuous end
S_2	5"	M_2	3/8" Φ @ 6" c/c	MT_2	3/8 Φ @ 9" c/c	Non continuous End
		M_1	3/8" Φ @ 9" c/c			



Section A-A



Section - B - B