

Live Load = 40 psf

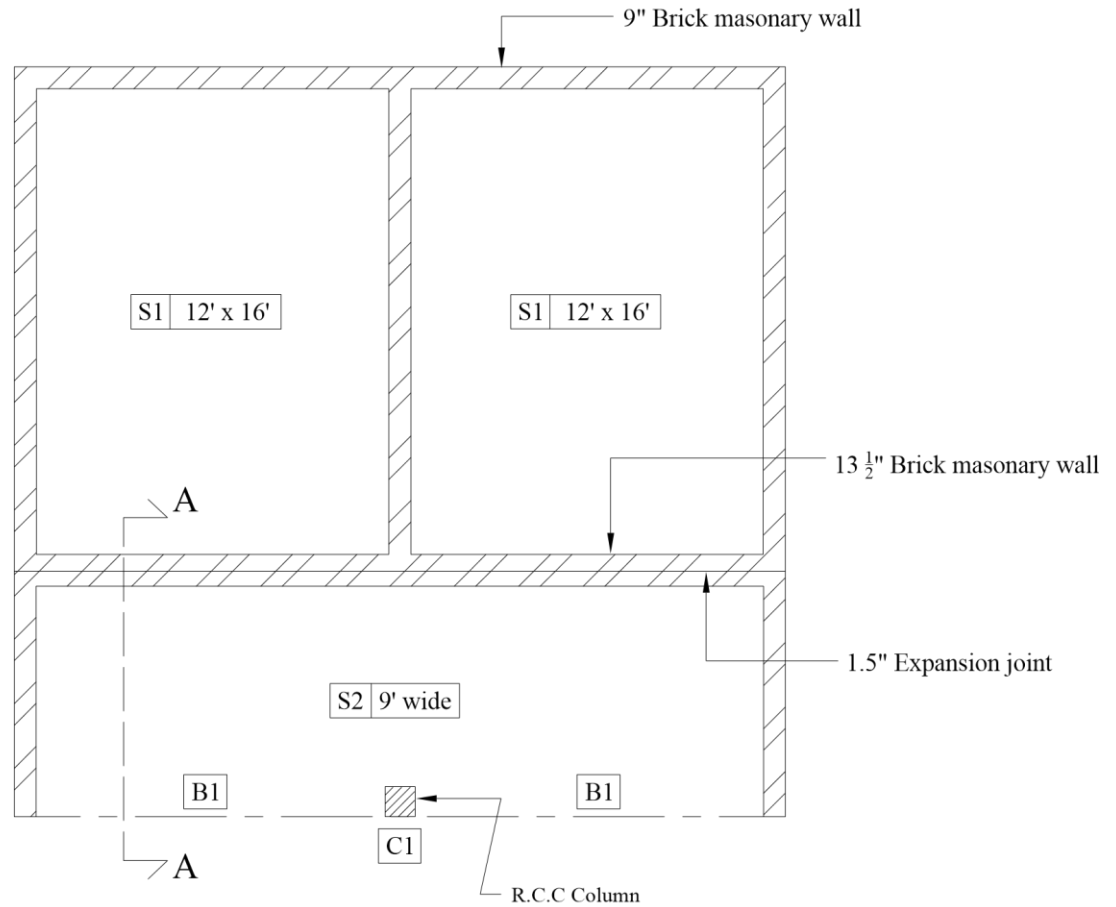


Figure 1: Slabs S_1 and S_2 to be designed.

Example 2: Design the roof slab, beam and column of house given in figure 1.

Concrete compressive strength (f'_c) = 3 ksi.

Steel yield strength (f_y) = 40 ksi.

Load on slab:

4" thick mud.

2" thick brick tile.

Discussion: *Expansion Joints...*

Solution: -

(1) Design of slab "S₂":

Step No 1: Sizes. $l_b/l_a = 24.75/8 =$

$3.09 > 2$ "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 supports = 9.0625'

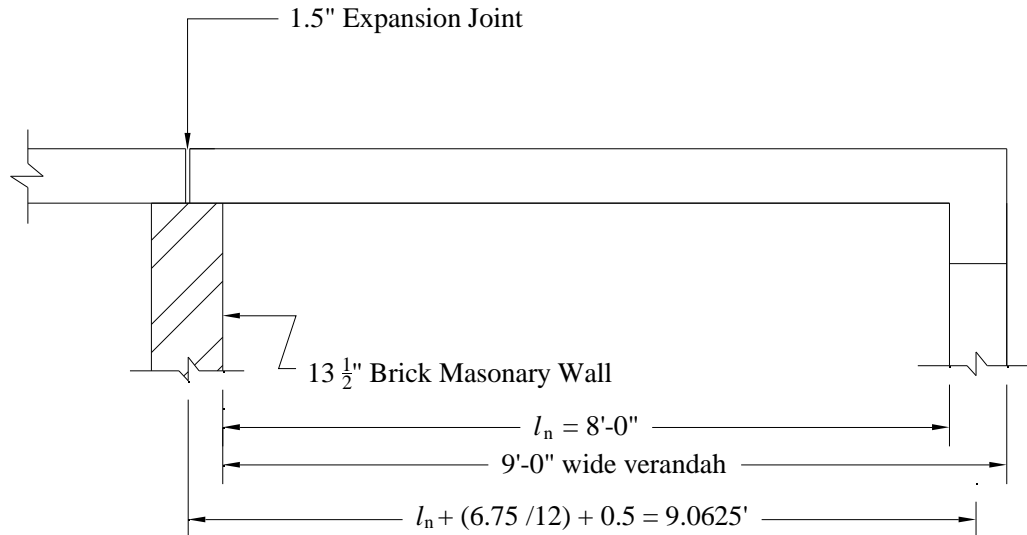


Figure 2: Section A-A (see figure 1 above).

Therefore $l = 8.42'$

Slab thickness (h_f) = $(l/20) \times (0.4 + f_y/100000)$ [for $f_y < 60000$ psi]

$$= (8.42/20) \times (0.4 + 40000/100000) \times 12$$

= 4.04" (Minimum requirement of ACI 9.5.2.1). Therefore

take $h_f = 5''$

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

Step No 2: Loading.

Table 1.1: Dead Loads.			
Material	Thickness (in)	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	$0.12 \times (2/12) = 0.02$

$$\text{Service Dead Load (D.L)} = 0.0625 + 0.04 + 0.02$$

$$= 0.1225 \text{ ksf}$$

$$\text{Service Live Load (L.L)} = 40 \text{ psf or } 0.04 \text{ ksf}$$

$$\text{Factored Load (w}_u\text{)} = 1.2\text{D.L} + 1.6\text{L.L}$$

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

$$= 0.211 \text{ ksf}$$

Step No 3: Analysis.

$$M_u = w_u l^2 / 8 \quad (l = \text{span length of slab})$$

$$M_u = 0.211 \times (8.42)^2 / 8$$

$$= 1.87 \text{ ft-k/ft} = 22.44 \text{ in-k/ft}$$

Step No 4: Design.

$$A_{smin} = 0.002bh_f \text{ (for } f_y \text{ 40 ksi, ACI}$$

$$10.5.4) = 0.002 \times 12 \times 5 =$$

$$0.12 \text{ in}^2 \quad a = A_{smin} f_y / (0.85 f_c 'b)$$

$$= 0.12 \times 40 / (0.85 \times 3 \times 12) = 0.156 \text{ in}$$

$$\Phi M_{n(min)} = \Phi A_{smin} f_y (d - a/2)$$

$$= 0.9 \times 0.12 \times 40 \times (4 - 0.156/2)$$

$$= 16.94 \text{ in-k} < M_u$$

Therefore,

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

Take $a = 0.2d$

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

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

- $a = 0.173 \times 40 / (0.85 \times 3 \times 12) = 0.226 \text{ in}$

$$A_s = 22.44 / \{ 0.9 \times 40 \times (4 - (0.226/2)) \}$$

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

- $a = 0.160 \times 40 / (0.85 \times 3 \times 12) = 0.209 \text{ in}$ $A_s = 22.44 / \{ 0.9 \times 40 \times (4 - (0.209/2)) \}$
 $= 0.160 \text{ in}^2, \text{ O.K.}$

Using $1/2'' \Phi$ (#4) {#13, 13 mm}, with bar area $A_b = 0.20 \text{ in}^2$

Spacing = Area of one bar (A_b)/ A_s

$$= [0.20 \text{ (in}^2\text{)}/0.160 \text{ (in}^2\text{/ft)}] \times 12 = 15 \text{ in}$$

Using $3/8'' \Phi$ (#3) {#10, 10 mm}, with bar area $A_b = 0.11 \text{ in}^2$

Spacing = Area of one bar (A_b)/ A_s

$$= [0.11 \text{ (in}^2\text{)}/0.160 \text{ (in}^2\text{/ft)}] \times 12 = 7.5'' \approx 6''$$

Finally use #3 @ 6" c/c (#10 @ 150 mm c/c).

Shrinkage steel or temperature steel (A_{st}):

$$A_{st} = 0.002bh_f$$

$$A_{st} = 0.002 \times 12 \times 5 = 0.12 \text{ in}^2$$

Using 3/8" Φ (#3) {#10, 10 mm}, with bar area $A_b = 0.11 \text{ in}^2$

$$\text{Spacing} = \text{Area of one bar } (A_b) / A_{smin}$$

$$= (0.11/0.12) \times 12 = 11" \text{ c/c}$$

Finally use #3 @ 9" c/c (#10 @ 225 mm c/c).

- Maximum spacing for main steel in one way slab according to ACI 7.6.5 is minimum of:

i) $3h_f = 3 \times 5 = 15"$

ii) 18"

Therefore 6" spacing is O.K.

- Maximum spacing for shrinkage steel in one way slab according to ACI 7.12.2 is minimum of:

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

ii) 18"

Therefore 9" spacing is O.K.

(2) Design of slab "S₁":

Step No 1: Sizes. $l_b/l_a = 16/12 =$

$$1.33 < 2 \text{ "two way slab"}$$

Minimum depth of two way slab is given by

formula, $h_{min} = \text{perimeter}/180$

$$= 2 \times (16 + 12) \times 12/180 = 3.73 \text{ in}$$

Assume $h = 5"$

Step No 2: Loads.

Factored Load (w_u) = $w_{u, dl} + w_{u, ll}$

$$w_u = 1.2D.L + 1.6L.L$$

$$w_u = 1.2 \times 0.1225 + 1.6 \times 0.04 \text{ (see table 1.1 above)}$$

$$= 0.147 + 0.064 = 0.211 \text{ ksf}$$

Step No 3: Analysis.

The precise determination of moments in two-way slabs with various conditions of continuity at the supported edges is mathematically formidable and not suited to design practice. For this reason, various

simplified methods have been adopted for determining moments, shears, and reactions of such slabs.

According to the 1995 ACI Code, all two-way reinforced concrete slab systems, including edge supported slabs, flat slabs, and flat plates, are to be analyzed and designed according to one unified method, such as Direct Design Method and Equivalent Frame Method. However, the complexity of the generalized approach, particularly for systems that do not meet the requirements permitting analysis by “direct design method” of the present code, has led many engineers to continue to use the design method of the 1963 ACI Code for the special case of two-way slabs supported on four sides of each slab panel by relatively deep, stiff, edge beams.

This method has been used extensively since 1963 for slabs supported at the edges by walls, steel beams, or monolithic concrete beams having a total depth not less than about 3 times the slab thickness. While it was not a part of the 1977 or later ACI Codes, its continued use is permissible under the ACI 318-95 code provision (13.5.1) that a slab system may be designed by any procedure satisfying conditions of equilibrium and geometric compatibility, if it is shown that the design strength at every section is at least equal to the requires strength, and that serviceability requirements are met.

The method makes use of tables of moment coefficients for a variety of conditions. These coefficients are based on elastic analysis but also account for inelastic redistribution. In consequence, the design moment in either direction is smaller by an appropriate amount than the elastic maximum moment in that direction. The moments in the middle strip in the two directions are computed from:

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

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

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

$$M_{b, \text{ neg}} = C_{b, \text{ neg}} W_u l_a^2$$

Where C_a, C_b = tabulated moment coefficients as given in Appendix A

w_u = Ultimate uniform load, psf l_a, l_b = length of clear spans in short and long directions respectively.

Therefore, for the design problem under

discussion, $m = l_a/l_b$

$$= 12/16 = 0.75$$

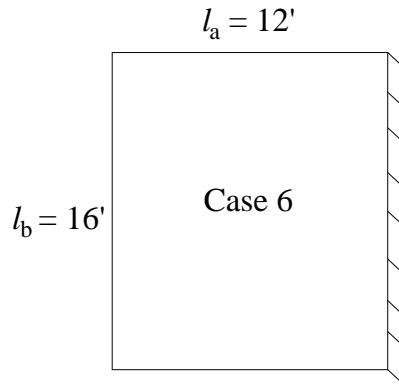


Figure 3: Two way slab (S_2)

Table 1.2: Moment coefficients for slab					
Case # 6 [$m = 0.75$]					
Coefficients for negative moments in slabs		Coefficients for dead load positive moments in slabs		Coefficients for live load positive moments in slabs	
$C_{a,neg}$	$C_{b,neg}$	$C_{a,dl}$	$C_{b,dl}$	$C_{a,ll}$	$C_{b,ll}$
0.088	0	0.048	0.012	0.055	0.016
Refer to tables 12.3 to 12.6, of Nilson 12th Ed.					

$$M_{a, neg} = C_{a, neg} \times w_u \times l_a^2$$

$$= 0.088 \times 0.211 \times 12^2 = 2.67 \text{ ft-k} = 32.04 \text{ in-k}$$

$$M_{b, neg} = C_{b, neg} \times w_u \times l_b^2 = 0 \times 0.211 \times 16^2 = 0 \text{ ft-k}$$

$$M_{a, pos, dl} = C_{a, pos, dl} \times w_{u, dl} \times l_a^2$$

$$= 0.048 \times 0.147 \times 12^2 = 1.016 \text{ ft-k} = 12.19 \text{ in-k}$$

$$M_{b, pos, dl} = C_{b, pos, dl} \times w_{u, dl} \times l_b^2$$

$$= 0.012 \times 0.147 \times 16^2 = 0.45 \text{ ft-k} = 5.42 \text{ in-k}$$

$$M_{a, pos, ll} = C_{a, pos, ll} \times w_{u, ll} \times l_a^2$$

$$= 0.055 \times 0.064 \times 12^2 = 0.51 \text{ ft-k} = 6.12 \text{ in-k}$$

$$M_{b, pos, ll} = C_{b, pos, ll} \times w_{u, ll} \times l_b^2$$

$$= 0.016 \times 0.064 \times 16^2 = 0.262 \text{ ft-k} = 3.144 \text{ in-k}$$

Therefore, finally we have,

$$M_{a, \text{neg}} = 2.67 \text{ ft-k} = 32.04 \text{ in-k}$$

$$M_{b, \text{neg}} = 0 \text{ ft-k}$$

$$M_{a, \text{pos}, (dl + ll)} = 1.016 + 0.51 = 1.53 \text{ ft-k} = 18.36 \text{ in-k}$$

$$M_{b, \text{pos}, (dl + ll)} = 0.45 + 0.262 = 0.712 \text{ ft-k} = 8.544 \text{ in-k}$$

Step No 4: Design.

$$A_{s\text{min}} = 0.002bh_f = 0.002 \times 12 \times 5 =$$

$$0.12 \text{ in}^2 \quad a = A_{s\text{min}}f_y / (0.85f_c'b)$$

$$= 0.12 \times 40 / (0.85 \times 3 \times 12) = 0.156 \text{ in}$$

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

$$= 0.9 \times 0.12 \times 40 \times (4 - 0.156/2) = 16.94 \text{ in-k (capacity provided}$$

by $A_{s\text{min}}$).

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

$$M_{b, \text{pos}, (dl + ll)} = 0.712 \text{ ft-k} = 8.544 \text{ in-k} < \Phi M_{n(\text{min})}$$

Therefore, $A_{s\text{min}} = 0.12 \text{ in}^2$ governs.

Using 3/8" Φ (#3) {#10, 10 mm}, with bar area $A_b = 0.11 \text{ in}^2$

$$\text{Spacing} = (0.11/0.12) \times 12 = 11"$$

Maximum spacing according to ACI 13.3.2 for two way slab

$$\text{is: } 2h_f = 2 \times 5 = 10"$$

Therefore maximum spacing of 10" governs.

Finally use #3 @ 9" c/c (#10 @ 225 mm c/c).

"Provide #3 @ 9" c/c as negative reinforcement along the longer direction."

$$M_{a, \text{pos}, (dl + ll)} = 1.53 \text{ ft-k} = 18.36 \text{ in-k} > \Phi$$

$$M_n \text{ Let } a = 0.2d = 0.2 \times 4 = 0.8 \text{ in}$$

$$A_s = 1.53 \times 12 / \{0.9 \times 40 \times (4 - (0.8/2))\} =$$

$$0.146 \text{ in}^2 \quad a = 0.146 \times 40 / (0.85 \times 3 \times 12) =$$

$$0.191 \text{ in}$$

$$A_s = 1.53 \times 12 / \{0.9 \times 40 \times (4 - 0.191/2)\} =$$

$$0.131 \text{ in}^2 \quad a = 0.131 \times 40 / (0.85 \times 3 \times 12) =$$

$$0.171 \text{ in}$$

$$A_s = 1.53 \times 12 / \{0.9 \times 40 \times (4 - 0.30/2)\} = 0.131 \text{ in}^2, \text{ O.K}$$

Using 3/8" Φ (#3) {#10, 10 mm}, with bar area $A_b = 0.11 \text{ in}^2$ Spacing = $0.11 \times 12/0.131 = 10.07'' \approx 9'' \text{ c/c}$
 Finally use #3 @ 9" c/c (#10 @ 225 mm c/c).

$M_{a, \text{neg}} = 2.67 \text{ ft-k} = 32.04 \text{ in-k}$
 Let $a = 0.2d = 0.2 \times 4 = 0.8 \text{ in}$
 $A_s = 2.67 \times 12 / \{0.9 \times 40 \times (4 - (0.8/2))\} = 0.24 \text{ in}^2$
 $a = 0.24 \times 40 / (0.85 \times 3 \times 12) = 0.31 \text{ in}$

$A_s = 2.67 \times 12 / \{0.9 \times 40 \times (4 - 0.31/2)\} = 0.23 \text{ in}^2$
 $a = 0.23 \times 40 / (0.85 \times 3 \times 12) = 0.30 \text{ in}$

$A_s = 2.67 \times 12 / \{0.9 \times 40 \times (4 - 0.30/2)\} = 0.23 \text{ in}^2$, O.K.
 Using 3/8" Φ (#3) {#10, 10 mm}, with bar area $A_b = 0.11 \text{ in}^2$ Spacing = $0.11 \times 12/0.23 = 5.7'' \approx 4.5'' \text{ c/c}$
 Finally use #3 @ 4.5" c/c (#10 @ 110 mm c/c).

(3) BEAM DESIGN (2 span, continuous): Data Given:

Exterior supports = 9" brick masonry wall.
 $f'_c = 3 \text{ ksi}$
 $f_y = 40 \text{ ksi}$
 Column dimensions = 12" \times

12" Step No 1: Sizes.

According to ACI 9.5.2.1, table 9.5 (a):
 Minimum thickness of beam with one end continuous = $h_{\text{min}} = l/18.5$ $l =$ clear span (l_n) + depth of member (beam) \leq c/c distance between supports [ACI 8.7].

Table 1.3: Clear Spans of beam.	
Case	Clear span (l_n)
End span (one end continuous)	$12.375 - (12/12)/2 = 11.875'$

Let depth of beam = 18"

$$l_n + \text{depth of beam} = 11.875' + (18/12) = 13.375'$$

$$\text{c/c distance between beam supports} = 12.375 + (4.5/12) = 12.75'$$

Therefore $l = 12.75'$

$$\text{Depth (h)} = (12.75/18.5) \times (0.4 + 40000/100000) \times$$

$$12 = 6.62" \text{ (Minimum requirement of}$$

ACI 9.5.2.1). Take $h = 1.5' = 18"$

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

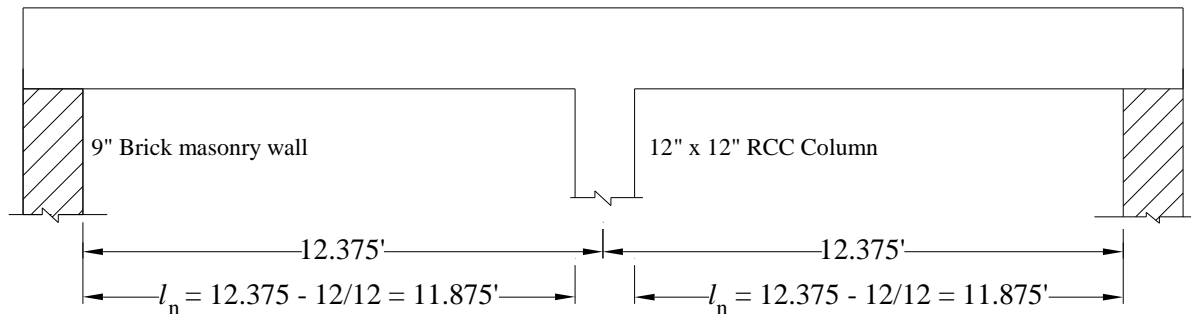


Figure 4: c/c distance & clear span of Beam.

Step No 2: Loads.

$$\text{Service Dead Load (D.L)} = 0.0625 + 0.04 + 0.02 = 0.1225 \text{ ksf (Table 1.1)}$$

$$\text{Service Live Load (L.L)} = 40 \text{ psf or } 0.04 \text{ ksf}$$

Beam is supporting 5' slab. Therefore load per running foot will be as follows:

$$\text{Service Dead Load from slab} = 0.1225 \times 5 = 0.6125 \text{ k/ft}$$

$$\text{Service Dead Load from beam's self weight} = h_w b_w \gamma_c$$

$$= (13 \times 12/144) \times 0.15 =$$

$$0.1625 \text{ k/ft}$$

$$\text{Total Dead Load} = 0.6125 + 0.1625 = 0.775 \text{ k/ft}$$

$$\text{Service Live Load} = 0.04 \times 5 = 0.2 \text{ k/ft } w_u = 1.2\text{D.L}$$

$$+ 1.6\text{L.L} = 1.2 \times 0.775 + 1.6 \times 0.20 = 1.25 \text{ k/ft}$$

Step No 3: Analysis.

Refer to ACI 8.3.3 or page 396, Nelson 13th Ed, for ACI moment and shear coefficients.

1) AT INTERIOR SUPPORT:

$$\text{Negative moment (M}_{\text{neg}}) = \text{Coefficient} \times (w_u l_n^2)$$

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

$$= 19.59 \text{ ft-k} = 235.08 \text{ in-k}$$

2) AT MID SPAN:

$$\begin{aligned} \text{Positive moment } (M_{\text{pos}}) &= \text{Coefficient} \times (w_u l_n^2) \\ &= (1/11) \times \{1.25 \times (11.875)^2\} \\ &= 16.02 \text{ ft-k} = 192.24 \text{ in-k} \end{aligned}$$

$$V_{\text{int}} = 1.15w_u l_n/2 = 1.15 \times 1.25 \times 11.875/2 = 8.54 \text{ k} \quad V_{u(\text{int})} = 8.54 - 1.25 \times 1.25 = 6.97 \text{ k}$$

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

$$\begin{aligned} V_{\text{ext}} &= \\ w_u l_n/2 &= \\ 1.25 \times & \\ 11.875/2 & \\ = 7.42 \text{ k} & \end{aligned}$$

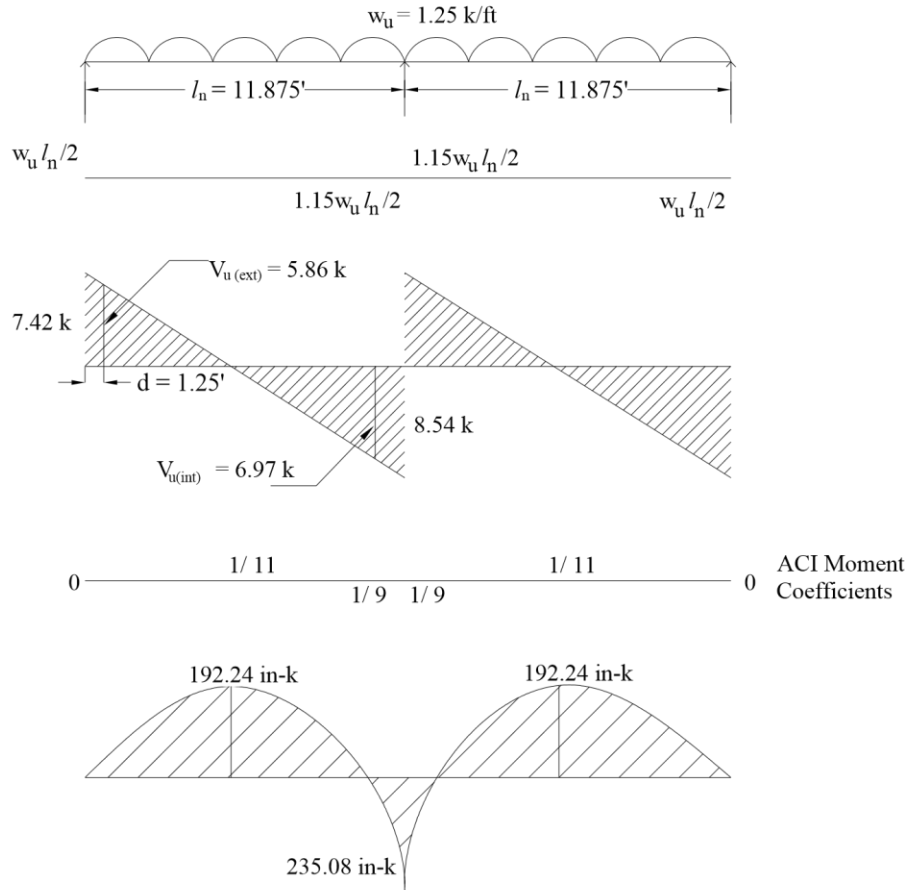


Figure 5: Approximate shear force and bending moment diagrams.
Coefficients

Step No 4: Design.

(A) Flexural Design:

(1) For Positive Moment:

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

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

(ii) $b_w + \text{Span length of beam}/12 = 12 + (12.75 \times 12) / 12 = 24.75''$ (iii) Clear distance to the next web = Not Applicable

$$\text{So } b_{eff} = 24.75''$$

Step (b): Check if beam is to be designed as rectangular beam or L-beam.

Trial #1:

(i). Assume $a = h_f = 5''$ $A_s =$

$$M_u / \{\Phi f_y (d - a/2)\}$$

$$A_s = 192.24 / \{0.9 \times 40 \times (15 - 5/2)\} = 0.427 \text{ in}^2 \text{ (ii). Re-calculate "a":}$$

$$a = A_s f_y / (0.85 f_c' b_{eff}) = 0.427 \times 40 / (0.85 \times 3 \times 24.75) = 0.271'' < h_f$$

Therefore design beam as rectangular.

Trial #2:

$$A_s = 192.24 / \{0.9 \times 40 \times (15 - 0.271/2)\} = 0.358 \text{ in}^2$$

$$= 0.358 \times 40 / (0.85 \times 3 \times 24.75) = 0.228''$$

This value is close enough to the previously calculated value of "a" therefore,

$$A_s = 0.358 \text{ in}^2, \text{ O.K.}$$

Step (c): Check for maximum and minimum reinforcement.

$$A_{smax} = \rho_{max} b_w d$$

$$\rho_{max} = 0.85 \times 0.85 \times (3/40) \times \{0.003 / (0.003 + 0.005)\} = 0.0203$$

$$A_{smax} = 0.0203 \times 12 \times 15 = 3.654 \text{ in}^2$$

$$A_{smin} = \rho_{min} b_w d$$

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

$$0.358 \text{ in}^2 < A_{smin}, \text{ so } A_{smin} \text{ governs.}$$

Using 5/8" Φ (#5) {#16, 16 mm}, with bar area $A_b = 0.31 \text{ in}^2$

$$\text{No. of bars} = A_s / A_b$$

$$= 0.90 / 0.31 = 2.90 \approx 3 \text{ bars Use}$$

4 #5 bars {4 #16 bars, 16 mm}.

(2) For Interior Negative Moment:

Step (a): Now we take $b_w = 12''$ instead of b_{eff} for calculation of "a" because of flange in tension.

$$M_u = 235.08 \text{ in-k}$$

$$b_w = 12'' \quad h = 18''$$

$$d = 15'' \quad (\text{i}) \text{ Trial}$$

#1:

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

$$\text{Let } a = 0.2d$$

$$A_s = 235.08 / [0.9 \times 40 \times \{15 - (0.2 \times 15)/2\}] = 0.484 \text{ in}^2$$

$$a = 0.484 \times 40 / (0.85 \times 3 \times 12) = 0.632'' \quad (\text{ii}) \text{ Trial \#2:}$$

$$A_s = 235.08 / \{0.9 \times 40 \times (15 - 0.632/2)\} = 0.44 \text{ in}^2$$

$$a = 0.44 \times 40 / (0.85 \times 3 \times 12) = 0.58'' \quad (\text{iii}) \text{ Trial \#3:}$$

$$A_s = 235.08 / \{0.9 \times 40 \times (15 - 0.58/2)\} = 0.44 \text{ in}^2 < A_{smin} \text{ So}$$

A_{smin} governs.

$$\text{Using } 5/8'' \Phi (\#5) \{ \#16, 16 \text{ mm} \}, \text{ with bar area } A_b = 0.31 \text{ in}^2$$

$$\text{No. of bars} = A_s / A_b$$

$$= 0.90 / 0.31 = 2.90 \approx 3 \text{ bars Use}$$

$$\underline{4 \#5 \text{ bars}} \{ 4 \# 16 \text{ bars, } 16 \text{ mm} \}.$$

(B) Shear Design for beam:

Step (a):

$$d = 15'' = 1.25'$$

$$V_{u \text{ (ext)}} = 5.86 \text{ k}$$

$$V_{u \text{ (int)}} = 6.97 \text{ k}$$

Step (b):

$$\Phi V_c = \Phi 2 \sqrt{f_c'} b_w d$$

$$= \{0.75 \times 2 \times \sqrt{(3000)} \times 12 \times 15\} / 1000 = 14.78 \text{ k} > V_{u \text{ (ext)}} \& V_{u \text{ (int)}}.$$

Theoretically, no shear reinforcement is required, but minimum will be provided. Maximum spacing and minimum reinforcement requirement as permitted by ACI

11.5.4 and 11.5.5.3 shall be minimum of:

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

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

$$(iii) 24'' \text{ c/c}$$

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

Other checks:

(a) Check for depth of beam {ACI 11.5.6.9}:

$$\Phi V_s \leq \Phi 8 \sqrt{f_c'} b_w d$$

$$\Phi 8 \sqrt{f_c'} b_w d = 0.75 \times 8 \times \sqrt{3000} \times 12 \times 15/1000 = 59.14 \text{ k}$$
$$\Phi V_s = (\Phi A_v f_y d) / s_d$$

$$= (0.75 \times 0.22 \times 40 \times 15) / 7.5 = 13.2 \text{ k} < 59.14 \text{ k}, \text{ O.K.}$$

So depth is O.K. If not, increase depth of beam.

(b) Check for spacing given under "maximum spacing requirement of ACI":

$$\Phi V_s \leq \Phi 4 \sqrt{f_c'} b_w d. \text{ {ACI 11.5.4.3}}$$

$$\Phi 4 \sqrt{f_c'} b_w d = 0.75 \times 4 \times \sqrt{3000} \times 12 \times 15/1000 = 29.57 \text{ k}$$
$$\Phi V_s = (\Phi A_v f_y d) / s_d$$

$$= (0.75 \times 0.22 \times 40 \times 21) / 7.5 = 13.2 \text{ k} < 29.57 \text{ k}, \text{ O.K.}$$

Therefore spacing given under "maximum spacing requirement of ACI" is O.K. Otherwise reduce spacing by half.

Provide # 3, 2 legged @ 7.5" c/c {#10, 2 legged stirrups @ 190 mm c/c} throughout, starting at $s_d/2 = 7.5/2 = 3.75"$ from the face of the support.

(4) DESIGN OF COLUMN:

i) **Load on column:**

$$P_u = 2V_{int} = 2 \times 8.54 = 17.08 \text{ k (Reaction at interior support of beam)}$$

$$\text{Gross area of column cross-section } (A_g) = 12 \times 12 = 144 \text{ in}^2 \quad f_c' = 3 \text{ ksi}$$

$$f_y = 40 \text{ ksi}$$

ii) **Design:**

Nominal strength (ΦP_n) of axially loaded column is:

$$\Phi P_n = 0.80 \Phi \{0.85 f_c' (A_g - A_{st}) + A_{st} f_y\} \quad \text{{for tied column, ACI 10.3.6}}$$

$$\text{Let } A_{st} = 1\% \text{ of } A_g \text{ (} A_{st} \text{ is the main steel reinforcement area)} \quad \Phi P_n = 0.80$$

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

$$218.98 \text{ k} > (P_u = 17.08 \text{ k}), \text{ O.K. } A_{st} = 0.01 \times 144 = 1.44 \text{ in}^2$$

Using $3/4"$ Φ (#6) {# 19, 19 mm}, with bar area $A_b = 0.44 \text{ in}^2$

$$\text{No. of bars} = A_s / A_b$$

$$= 1.44 / 0.44 = 3.27 \approx 4 \text{ bars Use}$$

4 #6 bars {4 #19 bars, 19 mm}.

Tie bars:

Using $3/8"$ Φ (#3) {#10, 10 mm} tie bars for $3/4"$ Φ (#6) {#19, 19 mm} main bars

(ACI 7.10.5),

Spacing for Tie bars according to ACI 7.10.5.1 is minimum of:

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

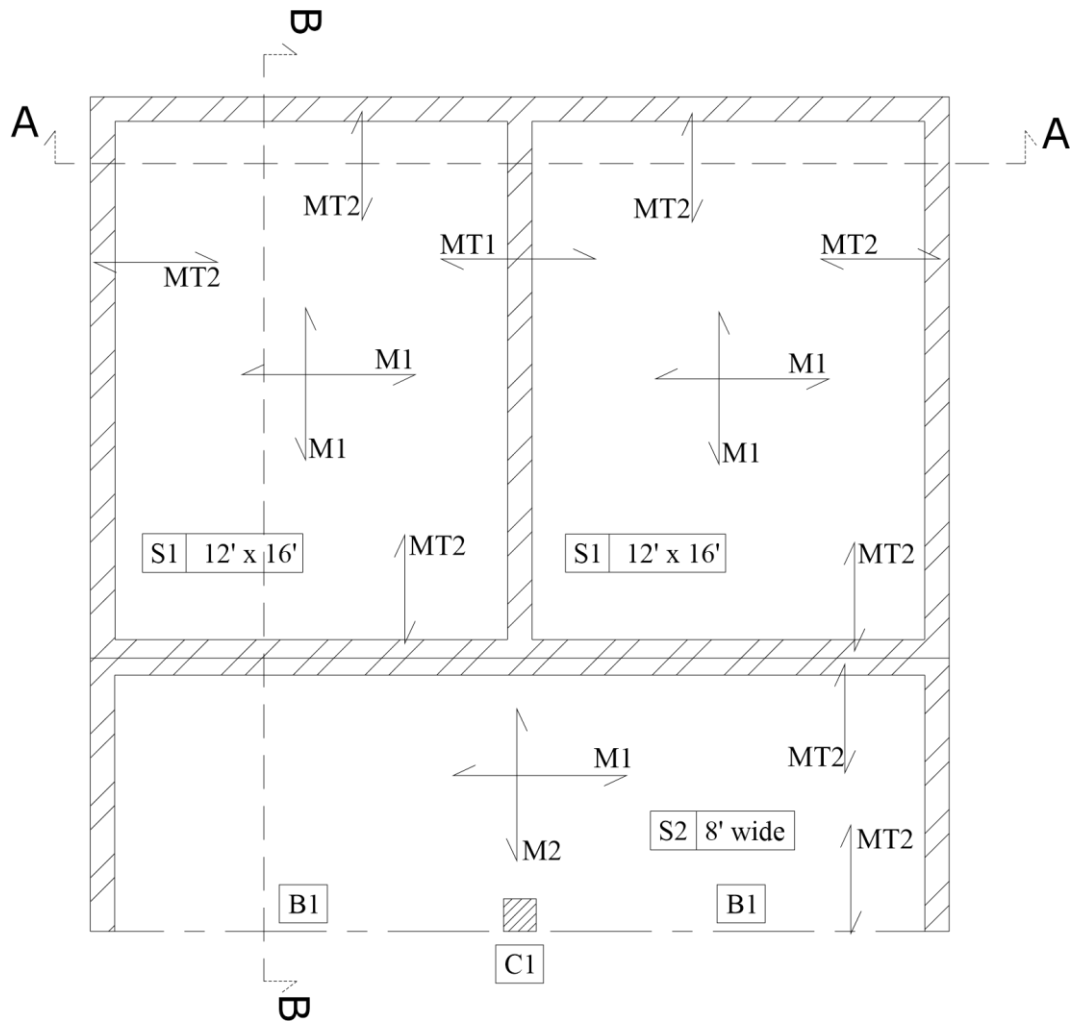
(b) $48 \times \text{dia of tie bar} = 48 \times (3/8) = 18'' \text{ c/c}$

(c) Least column dimension = $12'' \text{ c/c}$

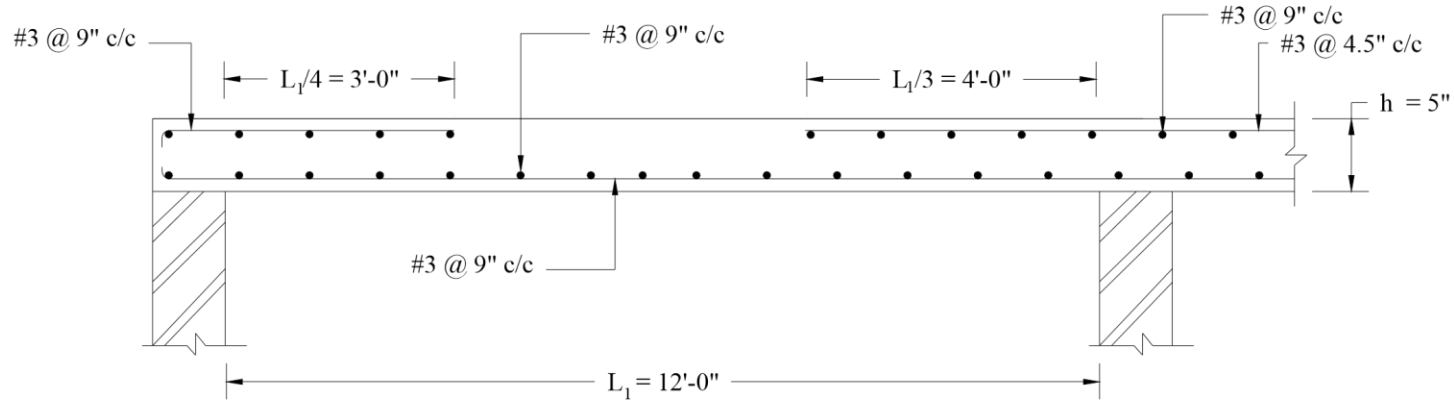
Finally use #3, tie bars @ 9'' c/c (#10, tie bars @ 225 mm c/c).

(5) Drafting:

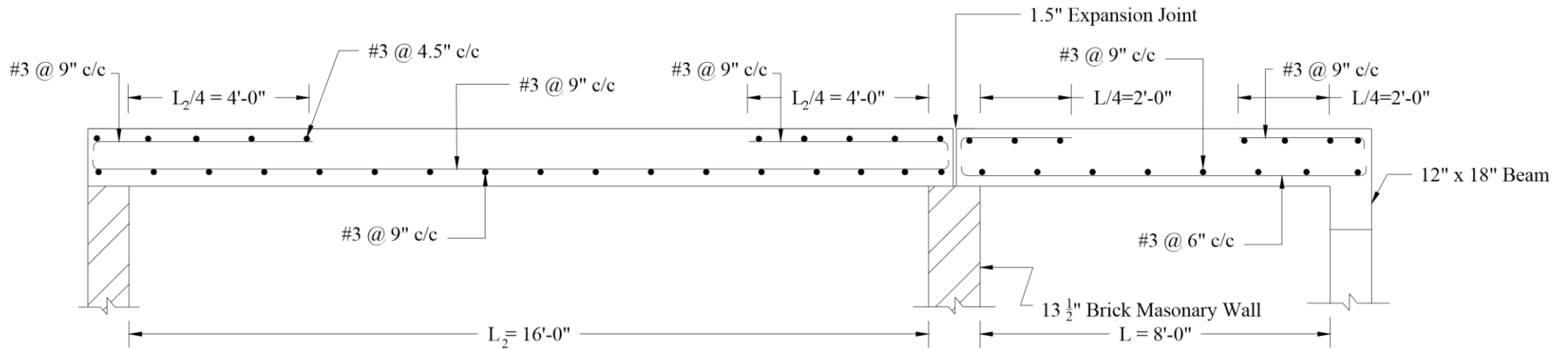
(A) Slab "S1" and "S2":



Panel	Depth (in)	Mark	Bottom Reinforcement	Mark	Top reinforcement	
S1	5"	M1	3/8" ϕ @ 9" c/c	MT1	3/8" ϕ @ 4.5" c/c	Continuous End
				MT2	3/8" ϕ @ 9" c/c	Non continuous Ends
S2	5"	M2	3/8" ϕ @ 6" c/c	MT2	3/8" ϕ @ 9" c/c	Non Continuous End
		M1	3/8" ϕ @ 9" c/c			

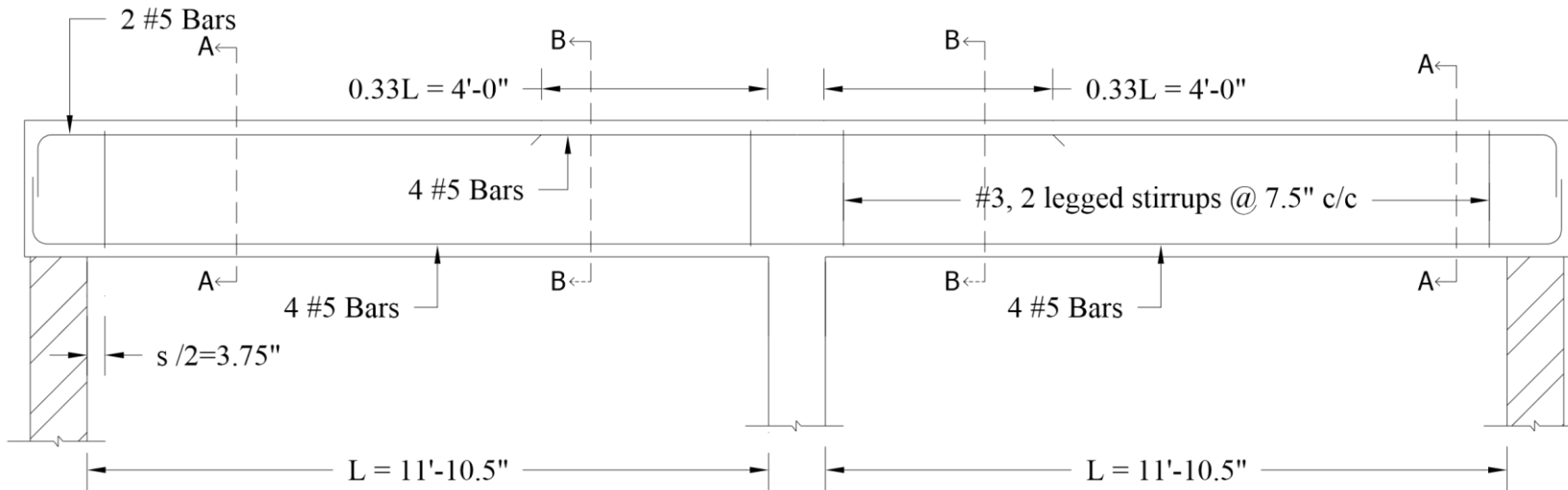


SECTION A-A



SECTION B-B

(B) Beam:

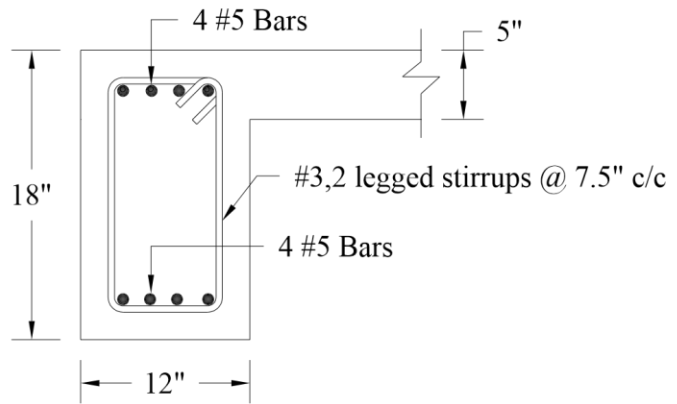
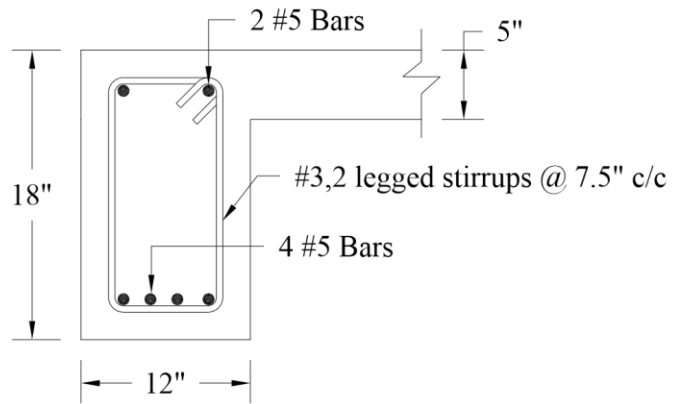


BEAM DETAIL



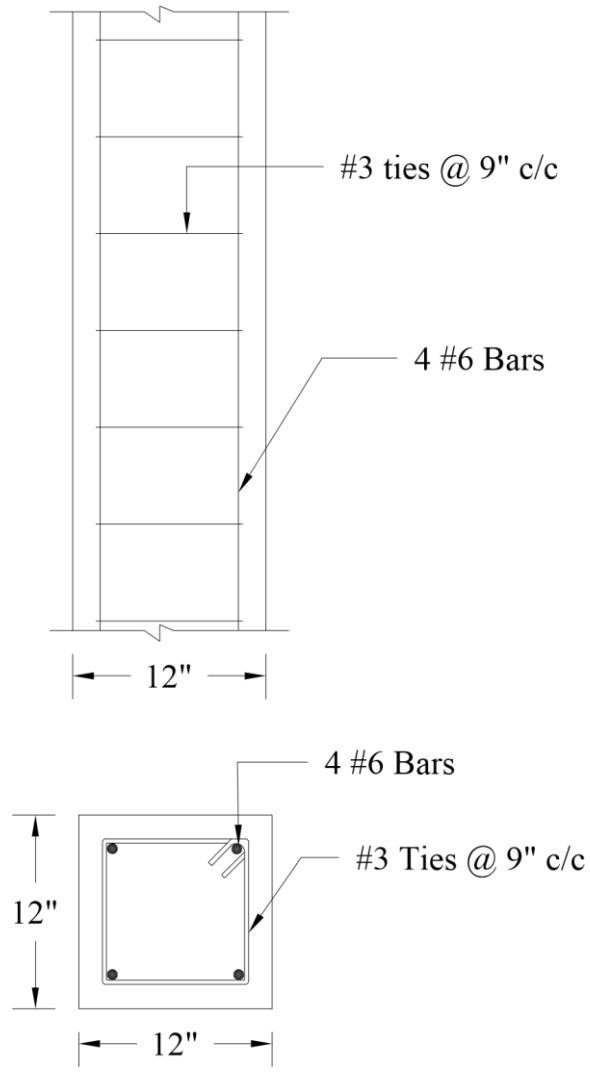
Notes: -

- (1) Use graph A.3, Nelson 13th Ed for location of cut off for continuous beams..
- (2) Use table A.7, Nelson 13th Ed for maximum number of bars as a single layer in beam stem.



SECTION A-A SECTION B-B

(C) Column:



Column (C1)
Detail

Appendix A

Tables of moment coefficients in slab:

NOTE: Horizontal sides of the figure represent longer side while vertical side represents shorter side of the slab.



Table A1: Coefficients ($C_{a, neg}$) for negative moment in slab along longer direction									
m	Case1	Case2	Case3	Case4	Case5	Case6	Case7	Case8	Case9
0.50	0.000	0.086	0.000	0.094	0.090	0.097	0.000	0.089	0.088

0.55	0.000	0.084	0.000	0.092	0.089	0.096	0.000	0.085	0.086
0.60	0.000	0.081	0.000	0.089	0.088	0.095	0.000	0.080	0.085
0.65	0.000	0.077	0.000	0.085	0.087	0.093	0.000	0.074	0.083
0.70	0.000	0.074	0.000	0.081	0.086	0.091	0.000	0.068	0.081
0.75	0.000	0.069	0.000	0.076	0.085	0.088	0.000	0.061	0.078
0.80	0.000	0.065	0.000	0.071	0.083	0.086	0.000	0.055	0.075
0.85	0.000	0.060	0.000	0.066	0.082	0.083	0.000	0.049	0.072
0.90	0.000	0.055	0.000	0.060	0.080	0.079	0.000	0.043	0.068
0.95	0.000	0.050	0.000	0.055	0.079	0.075	0.000	0.038	0.065
1.00	0.000	0.045	0.000	0.050	0.075	0.071	0.000	0.033	0.061

Table A2: Coefficients ($C_{b, neg}$) for negative moment in slab along shorter direction

m	Case1	Case2	Case3	Case4	Case5	Case6	Case7	Case8	Case9
0.50	0.000	0.006	0.022	0.006	0.000	0.000	0.014	0.010	0.003
0.55	0.000	0.007	0.028	0.008	0.000	0.000	0.019	0.014	0.005
0.60	0.000	0.010	0.035	0.011	0.000	0.000	0.024	0.018	0.006
0.65	0.000	0.014	0.043	0.015	0.000	0.000	0.031	0.024	0.008
0.70	0.000	0.017	0.050	0.019	0.000	0.000	0.038	0.029	0.011
0.75	0.000	0.022	0.056	0.024	0.000	0.000	0.044	0.036	0.014
0.80	0.000	0.027	0.061	0.029	0.000	0.000	0.051	0.041	0.017
0.85	0.000	0.031	0.065	0.034	0.000	0.000	0.057	0.046	0.021
0.90	0.000	0.037	0.070	0.040	0.000	0.000	0.062	0.052	0.025
0.95	0.000	0.041	0.072	0.045	0.000	0.000	0.067	0.056	0.029
1.00	0.000	0.045	0.076	0.050	0.000	0.000	0.071	0.061	0.033

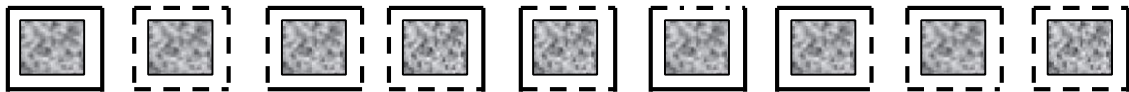


Table A3: Coefficients ($C_{a, pos, dl}$) for dead load positive moment in slab along longer direction

m	Case1	Case2	Case3	Case4	Case5	Case6	Case7	Case8	Case9
0.50	0.095	0.037	0.080	0.059	0.039	0.061	0.089	0.056	0.023
0.55	0.088	0.035	0.071	0.056	0.038	0.058	0.081	0.052	0.024
0.60	0.081	0.034	0.062	0.053	0.037	0.056	0.073	0.048	0.026

0.65	0.074	0.032	0.054	0.050	0.036	0.054	0.065	0.044	0.028
0.70	0.068	0.030	0.046	0.046	0.035	0.051	0.058	0.040	0.029
0.75	0.061	0.028	0.040	0.043	0.033	0.048	0.051	0.036	0.031
0.80	0.056	0.026	0.034	0.039	0.032	0.045	0.045	0.032	0.029
0.85	0.050	0.024	0.029	0.036	0.310	0.042	0.004	0.029	0.028
0.90	0.045	0.022	0.025	0.033	0.029	0.039	0.035	0.025	0.026
0.95	0.040	0.020	0.021	0.030	0.028	0.036	0.031	0.022	0.024
1.00	0.036	0.018	0.018	0.027	0.027	0.033	0.027	0.020	0.023

Table A4: Coefficients ($C_{b, dl}$) for dead load positive moment in slab along shorter direction

m	Case1	Case2	Case3	Case4	Case5	Case6	Case7	Case8	Case9
0.50	0.006	0.002	0.007	0.004	0.001	0.003	0.007	0.004	0.002
0.55	0.008	0.003	0.009	0.005	0.002	0.004	0.009	0.005	0.003
0.60	0.010	0.004	0.011	0.007	0.003	0.006	0.012	0.007	0.004
0.65	0.013	0.006	0.014	0.009	0.004	0.007	0.014	0.009	0.005
0.70	0.016	0.007	0.016	0.011	0.005	0.009	0.017	0.011	0.006
0.75	0.019	0.009	0.018	0.013	0.007	0.013	0.020	0.013	0.007
0.80	0.023	0.011	0.020	0.016	0.009	0.015	0.022	0.015	0.010
0.85	0.026	0.012	0.022	0.019	0.011	0.017	0.025	0.017	0.013
0.90	0.029	0.014	0.024	0.022	0.013	0.021	0.028	0.019	0.015
0.95	0.033	0.016	0.025	0.024	0.015	0.024	0.031	0.021	0.017
1.00	0.036	0.018	0.027	0.027	0.018	0.027	0.033	0.023	0.020



Table A5: Coefficients ($C_{a, ll}$) for live load positive moment in slab along longer direction

m	Case1	Case2	Case3	Case4	Case5	Case6	Case7	Case8	Case9
0.50	0.095	0.066	0.088	0.077	0.067	0.078	0.092	0.076	0.067
0.55	0.088	0.062	0.080	0.072	0.063	0.073	0.085	0.070	0.063

0.60	0.081	0.058	0.071	0.067	0.059	0.068	0.077	0.065	0.059
0.65	0.074	0.053	0.064	0.062	0.055	0.064	0.070	0.059	0.054
0.70	0.068	0.049	0.057	0.057	0.051	0.060	0.063	0.054	0.050
0.75	0.061	0.045	0.051	0.052	0.047	0.055	0.056	0.049	0.046
0.80	0.056	0.041	0.045	0.048	0.044	0.051	0.051	0.044	0.042
0.85	0.050	0.037	0.040	0.043	0.041	0.046	0.045	0.040	0.039
0.90	0.045	0.034	0.035	0.039	0.037	0.042	0.040	0.035	0.036
0.95	0.040	0.030	0.031	0.035	0.034	0.038	0.036	0.031	0.032
1.00	0.036	0.027	0.027	0.032	0.032	0.035	0.032	0.028	0.030

Table A6: Coefficients ($C_{b, II}$) for live load positive moment in slab along shorter direction

m	Case1	Case2	Case3	Case4	Case5	Case6	Case7	Case8	Case9
0.50	0.006	0.004	0.007	0.005	0.004	0.005	0.007	0.005	0.004
0.55	0.008	0.006	0.009	0.007	0.005	0.006	0.009	0.007	0.006
0.60	0.010	0.007	0.011	0.009	0.007	0.008	0.011	0.009	0.007
0.65	0.013	0.010	0.014	0.011	0.009	0.010	0.014	0.011	0.009
0.70	0.016	0.012	0.016	0.014	0.011	0.013	0.017	0.014	0.011
0.75	0.019	0.014	0.019	0.016	0.013	0.016	0.020	0.016	0.013
0.80	0.023	0.017	0.022	0.020	0.016	0.019	0.023	0.019	0.017
0.85	0.026	0.019	0.024	0.023	0.019	0.022	0.026	0.022	0.020
0.90	0.029	0.022	0.027	0.026	0.021	0.025	0.029	0.024	0.022
0.95	0.033	0.025	0.029	0.029	0.024	0.029	0.032	0.027	0.025
1.00	0.036	0.027	0.032	0.032	0.027	0.032	0.035	0.030	0.028

Appendix B

Comparison of analysis with Finite Element Method Based Software (SAP2000):

Assumptions made in SAP model:

- a. Brick masonry wall is modeled as hinged support.
- b. Slab is modeled as shell element.
- c. Beams are modeled as frame elements.
- d. Slab moments around the columns have not been considered for comparison purpose due to stress concentrations.

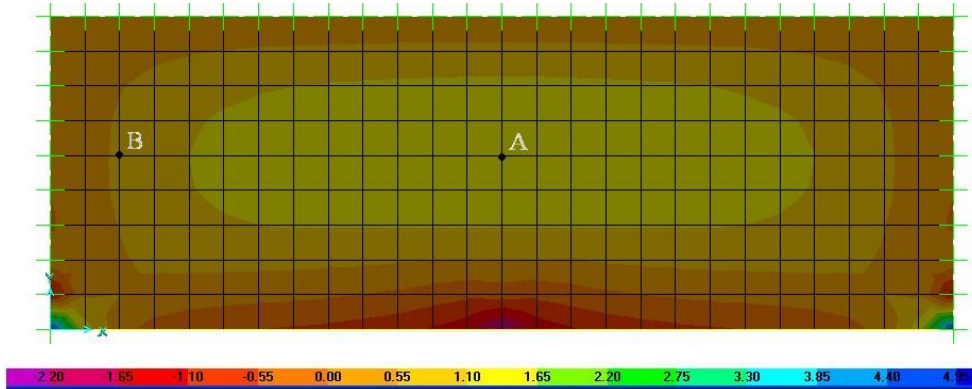


Figure B1: Plan of veranda of house with analyzed slab i.e. slab moment values (kipin/in).Marked points shows the sections used for comparison purpose

Table B1: Verandah 9' wide, figure B1.			
(1) Slab moments.			
	ACI 318-99	SAP Results	Percentage Difference
M (kip-in/in)	1.688	1.628 (at A)	- 4
		0.648 (at B)	- 62
(2) Beam moments.			
	ACI 318-99	SAP Results	Percentage Difference
M _{ext support} (k-in)	-	0	-
M _{mid span} (k-in)	192.24	159	-17
M _{int. support} (k-in)	235.08	269	13
(3) Column axial force and moments.			
	ACI 318-99	SAP Results	Percentage Difference
M (k-in)	-	109	-
Axial force (kip)	17.08	22	22

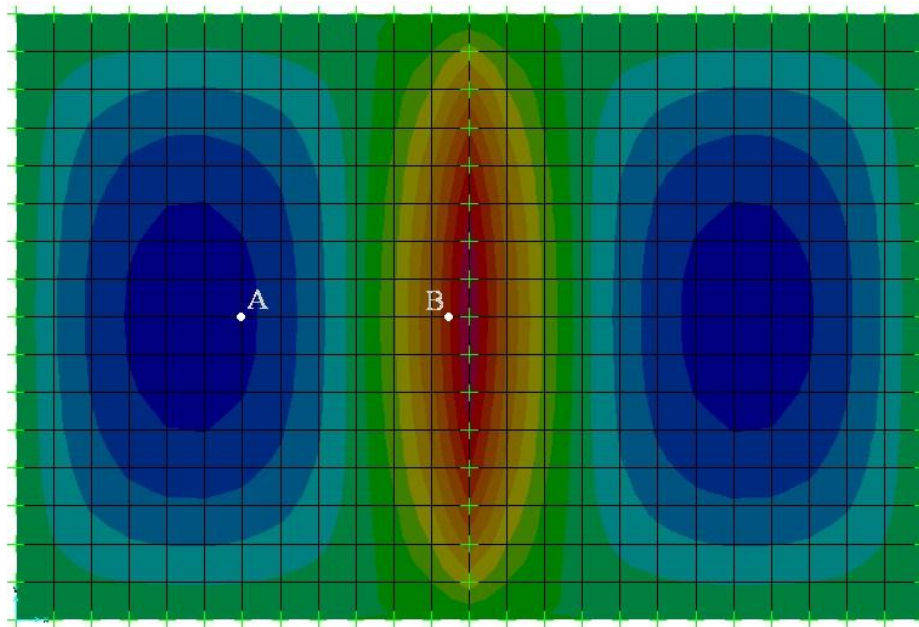


Figure B2: Plan of rooms of house with analyzed slab i.e. slabs moment values (kip-in/in).
Marked points shows the sections used for comparison purpose (M11)

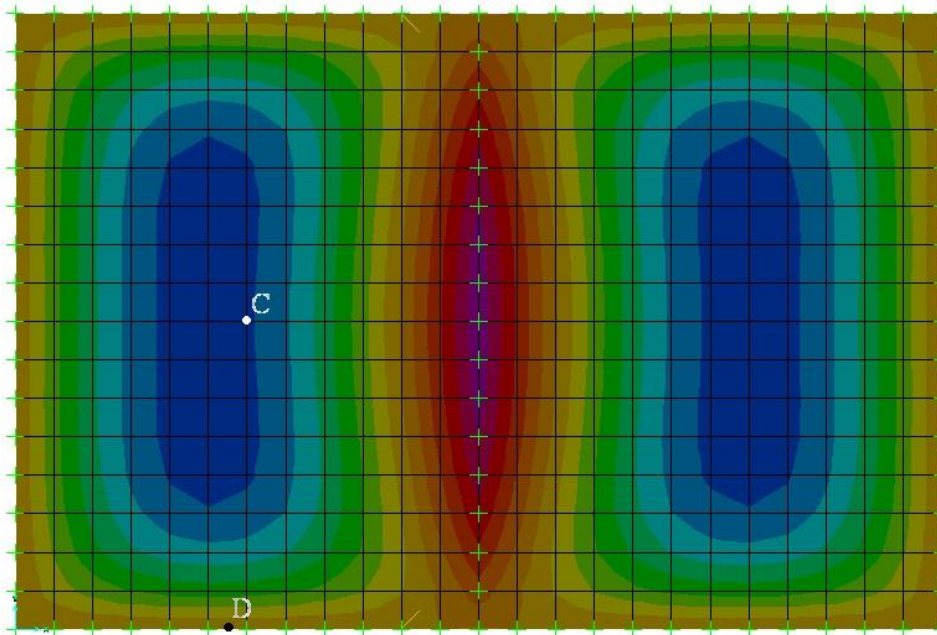


Figure B3: Plan of rooms of house with analyzed slab i.e. slab moment values (kip-in/in).
Marked points shows the sections used for comparison purpose (M22)

Table B2: Two-way slab of room, 12' × 16' (figure B2 & B3)				
	ACI 318-99	See fig	SAP Results	Percentage Difference

$M_{a, \text{pos}}$ (fig 04)	1.53	M_A	1.51	2
$M_{b, \text{pos}}$ (fig 05)	0.712	M_C	0.75	5
$M_{a, \text{neg}}$ (fig 04)	2.67	M_B	2.5	-6
$M_{b, \text{neg}}$ (fig 05)	-	M_D	0.02	-

References

- ⌚ *Design of Concrete Structures by Nilson, Darwin and Dolan (13th ed.)*
- ⌚ *Design of Concrete Structures by Nilson, Darwin and Dolan (12th ed.)*
- ⌚ *ACI 318-02/05.*