**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.

<u>Load on slab:</u>

4″ thick mud.

2″ thick brick tile.

Live Load = 40 psf

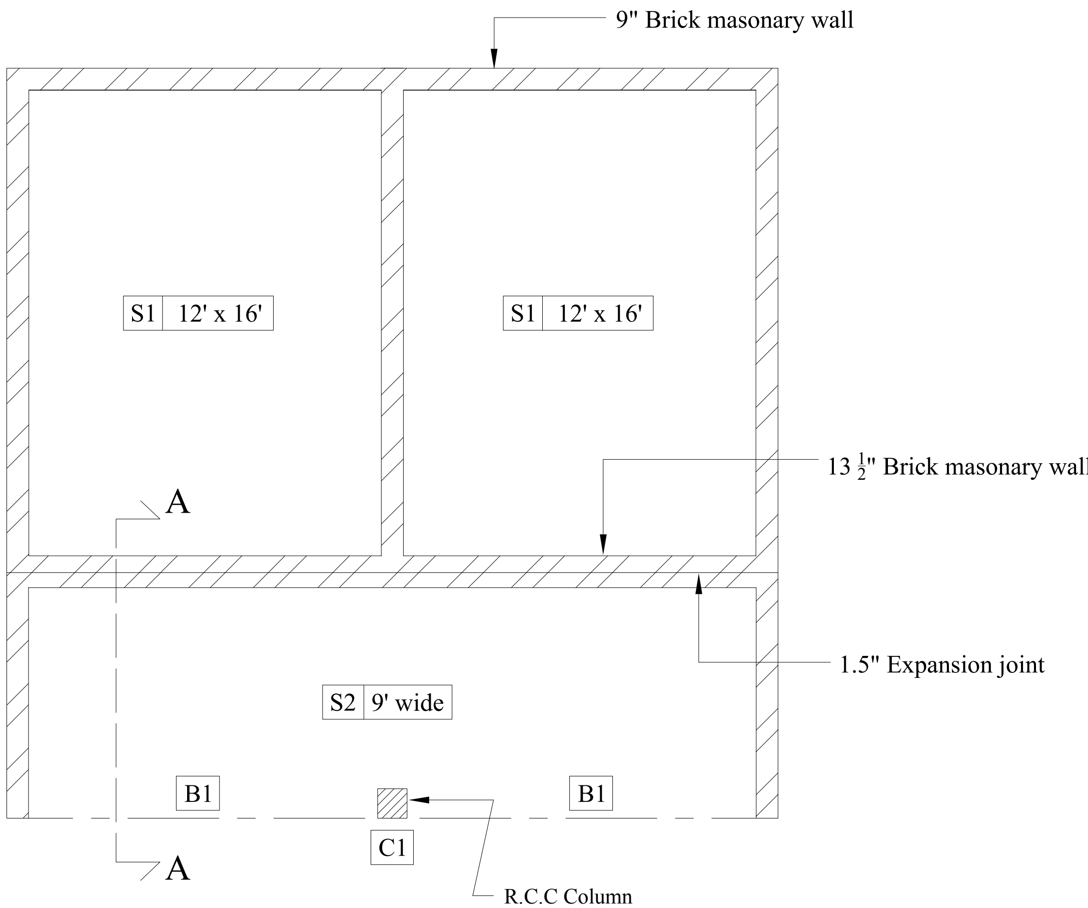



Figure 1: Slabs $S_1$ and $S_2$ to be designed.

**Discussion:** *Expansion Joints…*

**Solution: -**

(1) Design of slab "$S_2$":

**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′

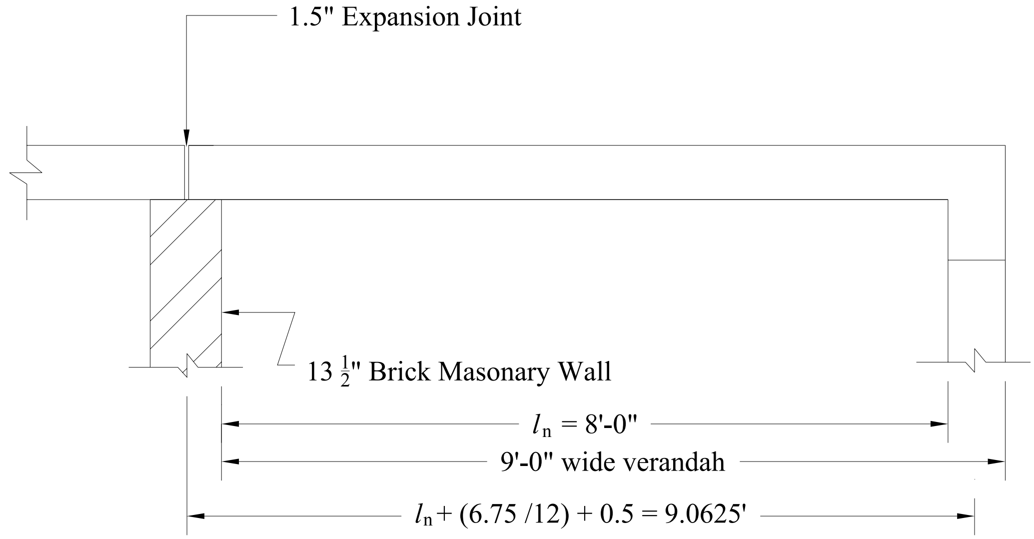



Figure 2: Section A-A (see figure 1 above).

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 = γ × thickness (ksf) |
|---|---|---|---|
| Slab | 5 | 0.15 | 0.15 × (5/12) = 0.0625 |
| Mud | 4 | 0.12 | 0.12 × (4/12) = 0.04 |
| Brick Tile | 2 | 0.12 | 0.12 × (2/12) = 0.02 |

Service Dead Load (D.L) = 0.0625 + 0.04 + 0.02

= 0.1225 ksf

Service Live Load (L.L) = 40 psf or 0.04 ksf

Factored Load ($w_u$) = 1.2D.L + 1.6L.L

= 1.2 × 0.1225 + 1.6 × 0.04

= 0.211 ksf

**Step No 3: Analysis.**

$M_u = w_u l^2/8$ ($l$ = span length of slab)

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

= 1.87 ft-k/ft = 22.44 in-k/ft

**Step No 4: Design.**

$A_{smin} = 0.002bh_f$ (for $f_y$ 40 ksi, ACI 10.5.4)

$= 0.002 \times 12 \times 5 = 0.12$ in$^2$

$a = A_{smin}f_y/ (0.85f_c'b)$

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

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

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

$= 16.94$ in-k $< M_u$

Therefore,

- $A_s = M_u/ \{\Phi f_y (d – a/2)\}$

  Take a = 0.2d

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

  $A_s = 0.173$ in$^2$

- a = 0.173 x 40/ (0.85 x 3 x 12) = 0.226 in

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

  = 0.160 in$^2$

- $a = 0.160 \times 40/ (0.85 \times 3 \times 12) = 0.209$ in

  $A_s = 22.44/ \{0.9 \times 40 \times (4 – (0.209/2))\}$

  = 0.160 in$^2$, O.K.

Using ½″ Φ (#4) {#13, 13 mm}, with bar area $A_b = 0.20$ in$^2$

Spacing =Area of one bar ($A_b$)/$A_s$

= [0.20 (in$^2$)/0.160 (in$^2$/ft)] × 12 = 15 in

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

Spacing = Area of one bar ($A_b$)/$A_s$

= [0.11(in$^2$)/0.160(in$^2$/ft)] × 12 = 7.5″ ≈ 6″

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

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

$A_{st} = 0.002bh_f$

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

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

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

= (0.11/0.12) × 12 = 11″ 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 × 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 × 5 =25″
  ii) 18″
  Therefore 9″ spacing is O.K.

(2) Design of slab "$S_1$":

**Step No 1: Sizes.**

$l_b/l_a$ = 16/12 = 1.33 < 2 "two way slab"

Minimum depth of two way slab is given by formula,

$h_{min}$ = perimeter/180

= 2 × (16 + 12) × 12/180 = 3.73 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 × 0.1225 + 1.6 × 0.04 (see table 1.1 above)

= 0.147 + 0.064 = 0.211 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:

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

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

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

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

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,

$m = l_a/l_b$

$= 12/16 = 0.75$

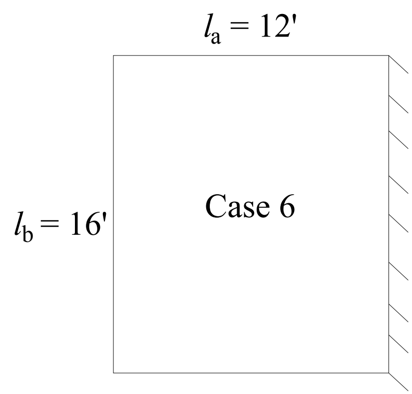



Figure 3: Two way slab ($S_2$)

| 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. | | | | | |

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

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

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

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

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

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

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

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

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

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

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

Therefore, finally we have,

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

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

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

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

**Step No 4: Design.**

$A_{smin} = 0.002bh_f = 0.002 \times 12 \times 5 = 0.12$ in$^2$

$a = A_{smin}f_y / (0.85f_c'b)$

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

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

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

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

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

Therefore, $A_{smin} = 0.12$ in$^2$ governs.

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

Spacing = (0.11/0.12) × 12 = 11″

Maximum spacing according to ACI 13.3.2 for two way slab 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."

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

Let $a = 0.2d = 0.2 \times 4 = 0.8$ in

$A_s$ = 1.53 × 12/ {0.9 × 40 × (4 – (0.8/2))} = 0.146 $in^2$

a = 0.146 × 40/ (0.85 × 3 × 12) = 0.191 in

$A_s$ = 1.53 × 12/ {0.9 × 40 × (4 – 0.191/2)} = 0.131 $in^2$

a = 0.131 × 40/ (0.85 × 3 × 12) = 0.171 in

$A_s$ = 1.53 × 12/ {0.9 × 40 × (4 – 0.30/2)} = 0.131 $in^2$, O.K

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

Spacing = 0.11 × 12/0.131 = 10.07″ ≈ 9″ c/c

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

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

Let a = 0.2d = 0.2 × 4 = 0.8 in

$A_s$ = 2.67 × 12/ {0.9 × 40 × (4 – (0.8/2))} = 0.24 $in^2$

a = 0.24 × 40/ (0.85 × 3 × 12) = 0.31 in

$A_s$ = 2.67 × 12/ {0.9 × 40 × (4 – 0.31/2)} = 0.23 $in^2$

a = 0.23 × 40/ (0.85 × 3 × 12) = 0.30 in

$A_s$ = 2.67 × 12/ {0.9 × 40 × (4 – 0.30/2)} = 0.23 $in^2$, O.K.

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

Spacing = 0.11 × 12/0.23 = 5.7″ ≈ 4.5″ 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 ksi

$f_y$ = 40 ksi

Column dimensions = 12″ × 12″

**Step No 1: Sizes.**

According to ACI 9.5.2.1, table 9.5 (a):

Minimum thickness of beam with one end continuous = $h_{min}$ = $l$/18.5

$l$ = clear span ($l_n$) + depth of member (beam) ≤ 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″

$l_n$ + depth of beam = 11.875′ + (18/12) = 13.375′

c/c distance between beam supports = 12.375 + (4.5/12) = 12.75′

Therefore $l$ = 12.75′

Depth (h) = (12.75/18.5) × (0.4 + 40000/100000) × 12

= 6.62″ (Minimum requirement of ACI 9.5.2.1).

Take h = 1.5′ = 18″

d = h – 3 = 15″

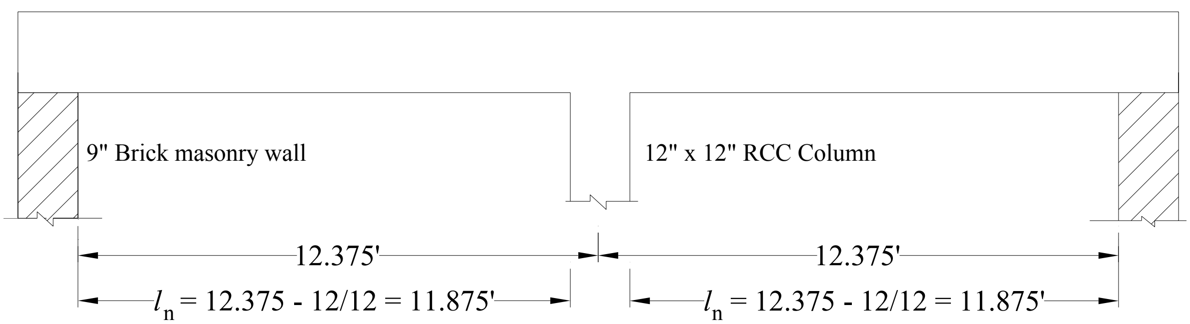



Figure 4: c/c distance & clear span of Beam.

**Step No 2: Loads.**

Service Dead Load (D.L) = 0.0625 + 0.04 + 0.02 = 0.1225 ksf (Table 1.1)

Service Live Load (L.L) = 40 psf or 0.04 ksf

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

Service Dead Load from slab = 0.1225 × 5 = 0.6125 k/ft

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

= (13 × 12/144) × 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

$w_u$ = 1.2D.L + 1.6L.L = 1.2 × 0.775 + 1.6 × 0.20 = 1.25 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:

Negative moment ($M_{neg}$) = Coefficient × ($w_u l_n^2$)

= (1/9) × {1.25 × $(11.875)^2$}

= 19.59 ft-k = 235.08 in-k

2) AT MID SPAN:

Positive moment ($M_{pos}$) = Coefficient x ($w_u l_n^2$)

= (1/11) × {1.25 × $(11.875)^2$}

= 16.02 ft-k = 192.24 in-k

$V_{int} = 1.15 w_u l_n/2 = 1.15 \times 1.25 \times 11.875/2 = 8.54$ k

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

$V_{ext} = w_u l_n/2 = 1.25 \times 11.875/2 = 7.42$ k

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

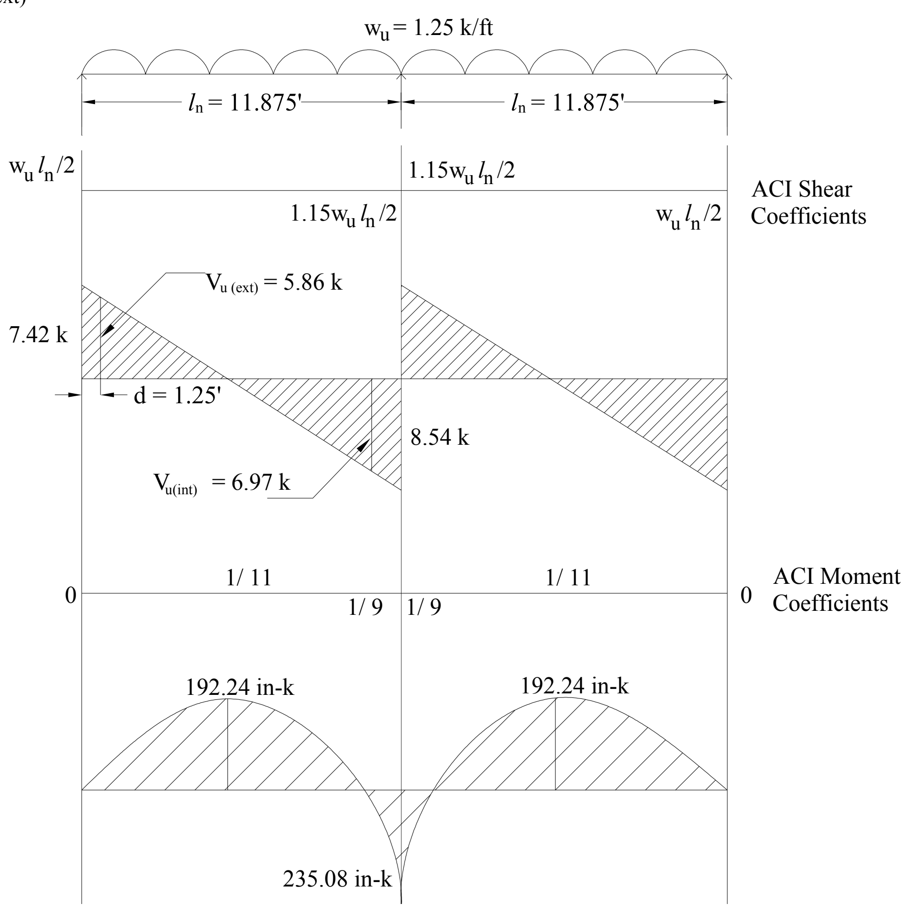



Figure 5: Approximate shear force and bending moment diagrams.

**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) $b_w$ + Span length of beam/12 = 12 + (12.75 × 12) /12 = 24.75″

(iii) Clear distance to the next web = Not Applicable

So $b_{eff} = 24.75''$

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

Trial #1:

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

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

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

(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:

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

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

This value is close enough to the previously calculated value of "a" therefore, $A_s = 0.358 \text{ in}^2$, O.K.

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

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

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

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

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

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

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

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

No. of bars = $A_s/A_b$

$= 0.90/0.31 = 2.90 \approx 3$ 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.

$M_u = 235.08$ in-k

$b_w = 12''$

$h = 18''$

$d = 15''$

(i) Trial #1:

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

Let a = 0.2d

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

$a = 0.484 \times 40 / (0.85 \times 3 \times 12) = 0.632''$

(ii) Trial #2:

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

$a = 0.44 \times 40 / (0.85 \times 3 \times 12) = 0.58''$

(iii)Trial #3:

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

So $A_{smin}$ governs.

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

No. of bars = $A_s/A_b$

$= 0.90/0.31 = 2.90 \approx 3$ bars

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

(B) Shear Design for beam:

Step (a):

$d = 15'' = 1.25'$

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

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

Step (b):

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

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

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) $A_vf_y/(50b_w) = 0.22 \times 40000/(50 \times 12) = 14.67''$ c/c

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

(iii) $24''$ c/c

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

<u>Other checks:</u>

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

$\Phi V_s \le \Phi 8\sqrt{(f_c')}b_wd$

$\Phi 8\sqrt{(f_c')}b_wd = 0.75 \times 8 \times \sqrt{(3000)} \times 12 \times 15/1000 = 59.14$ k

$\Phi V_s = (\Phi A_vf_yd)/s_d$

$= (0.75 \times 0.22 \times 40 \times 15)/7.5 = 13.2 \text{ k} < 82.8$ k, O.K.

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

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

$\Phi V_s \le \Phi 4\sqrt{(f_c')}b_wd$. {ACI 11.5.4.3}

$\Phi 4\sqrt{(f_c')}b_wd = 0.75 \times 4 \times \sqrt{(3000)} \times 12 \times 15/1000 = 29.57$ k

$\Phi V_s = (\Phi A_vf_yd)/s_d$

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

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

<u>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.</u>

(4) <u>DESIGN OF COLUMN:</u>

**i) Load on column:**

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

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

$f_c' = 3$ ksi

$f_y = 40$ ksi

**ii) Design:**

Nominal strength $(\Phi P_n)$ of axially loaded column is:

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

Let $A_{st}$ = 1% of $A_g$ ($A_{st}$ is the main steel reinforcement area)

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

$= 218.98$ k > ($P_u$ = 17.08 k), O.K.

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

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

No. of bars = $A_s/A_b$

= 1.44/0.44 = 3.27 ≈ 4 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 × dia of main bar =16 × 3/4 =12″ c/c

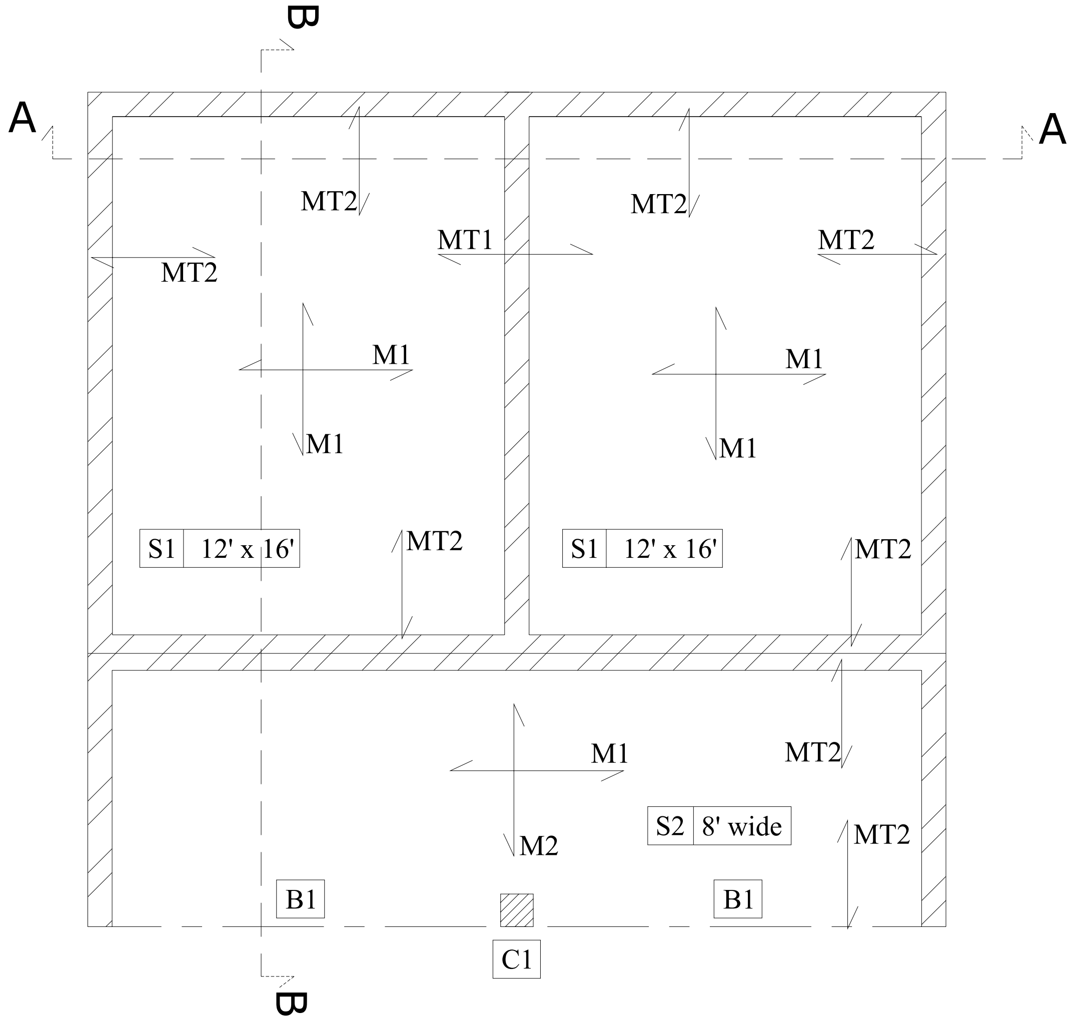
(b) 48 × dia of tie bar = 48 × (3/8) =18″ c/c

(c) Least column dimension =12″ c/c

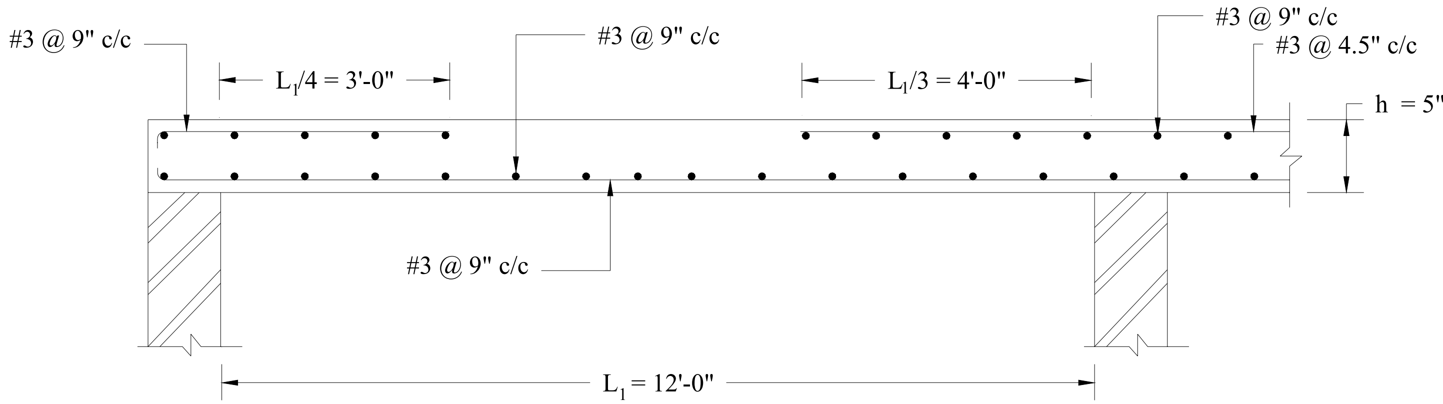
Finally use #3, tie bars @ 9″ c/c (#10, tie bars @ 225 mm c/c).

(5) Drafting:

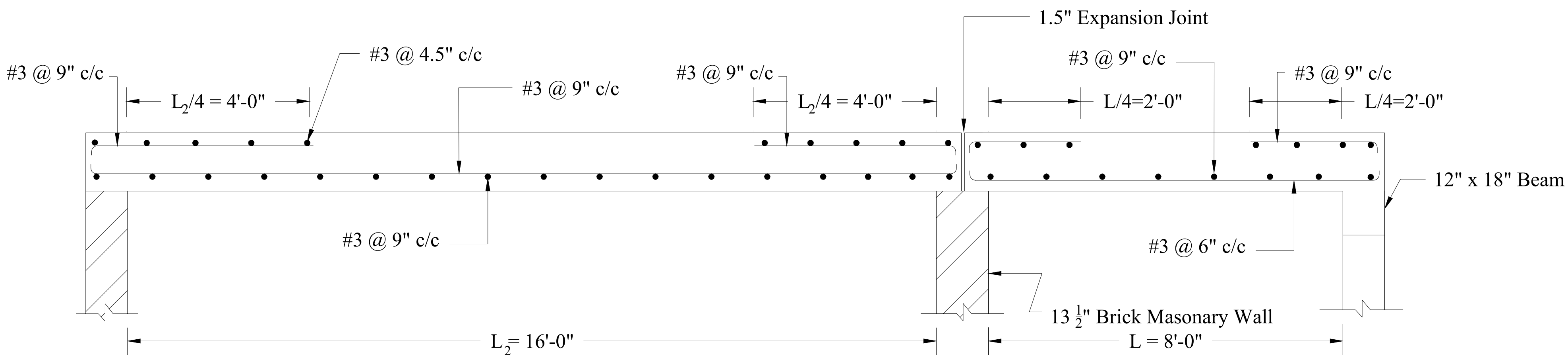
(A) Slab "S1" and "S2":




| 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 | | | |

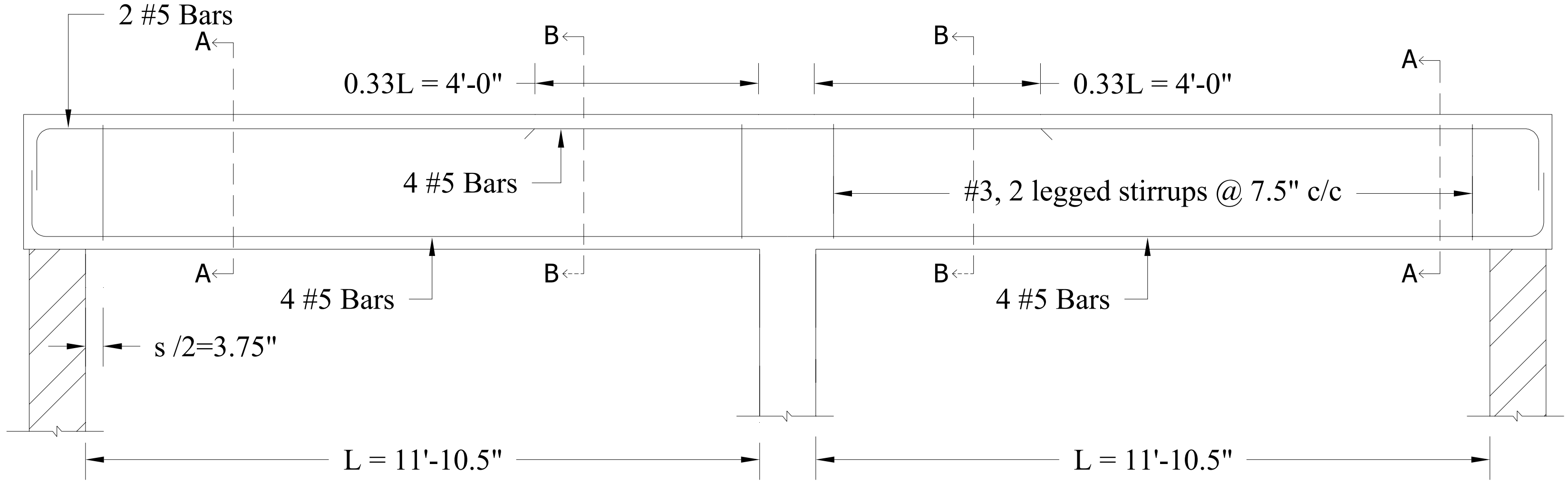



SECTION A-A




SECTION B-B

(B) Beam:

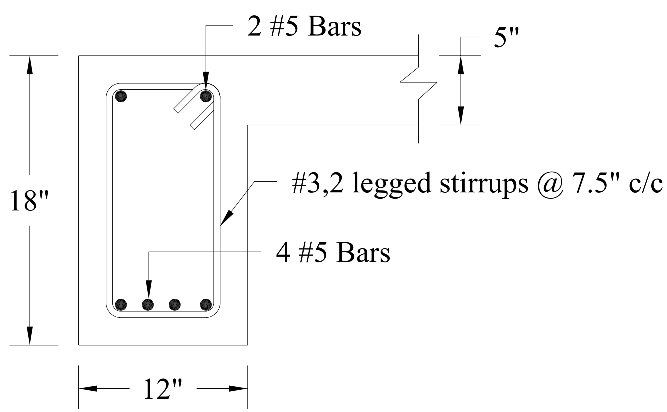



BEAM DETAIL

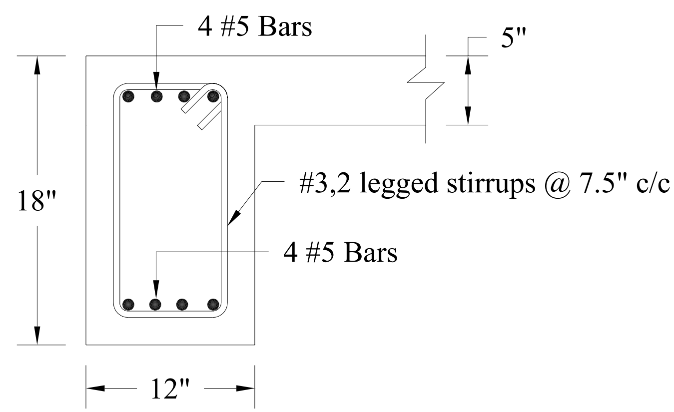
Notes: -

(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.

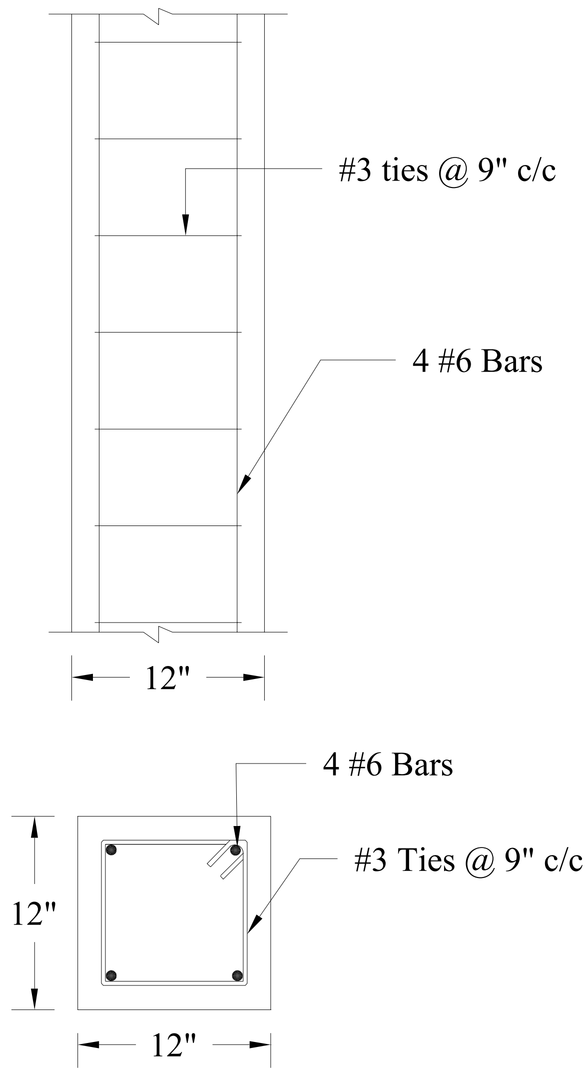



SECTION A-A




SECTION B-B

(C) Column:

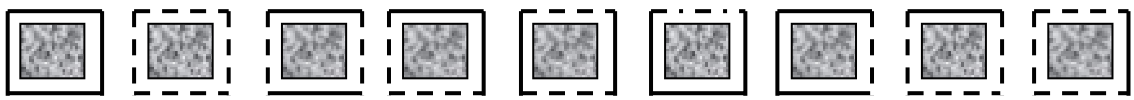



Column (C1) Detail

## Appendix A

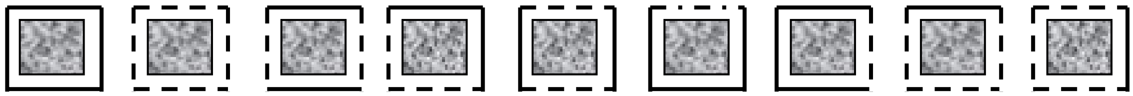
### Tables of moment coefficients in slab:

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



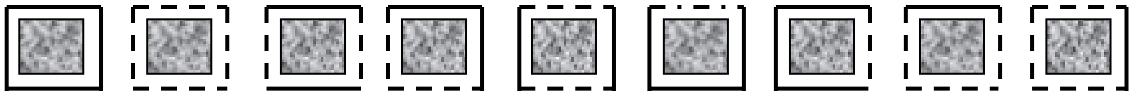
| Table A1: Coefficients ($C_{a, neg}$) for negative moment in slab along longer direction | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| m | Case1 | Case2 | Case3 | Case4 | Case5 | Case6 | Case7 | Case8 | Case9 |
| 0.50 | 0.000 | 0.086 | 0.000 | 0.094 | 0.090 | 0.097 | 0.000 | 0.089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

**<u>Assumptions made in SAP model:</u>**

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.

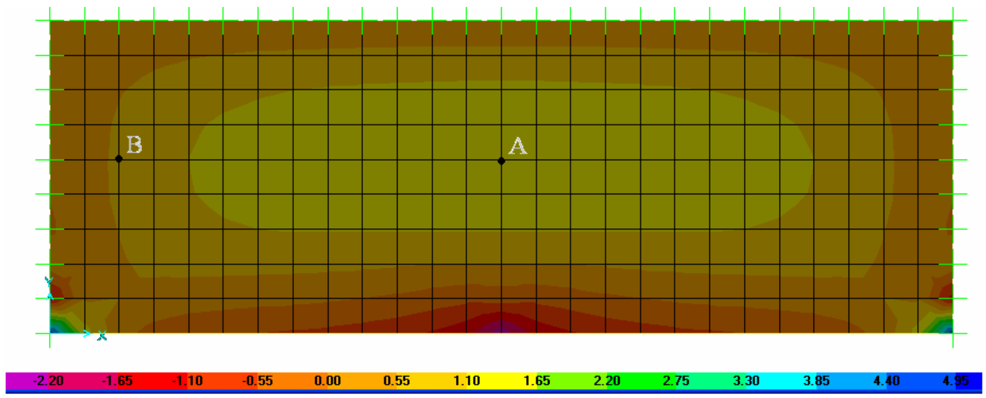



Figure B1**:** Plan of veranda of house with analyzed slab i.e. slab moment values (kip-in/in).Marked points shows the sections used for comparison purpose

| 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 |

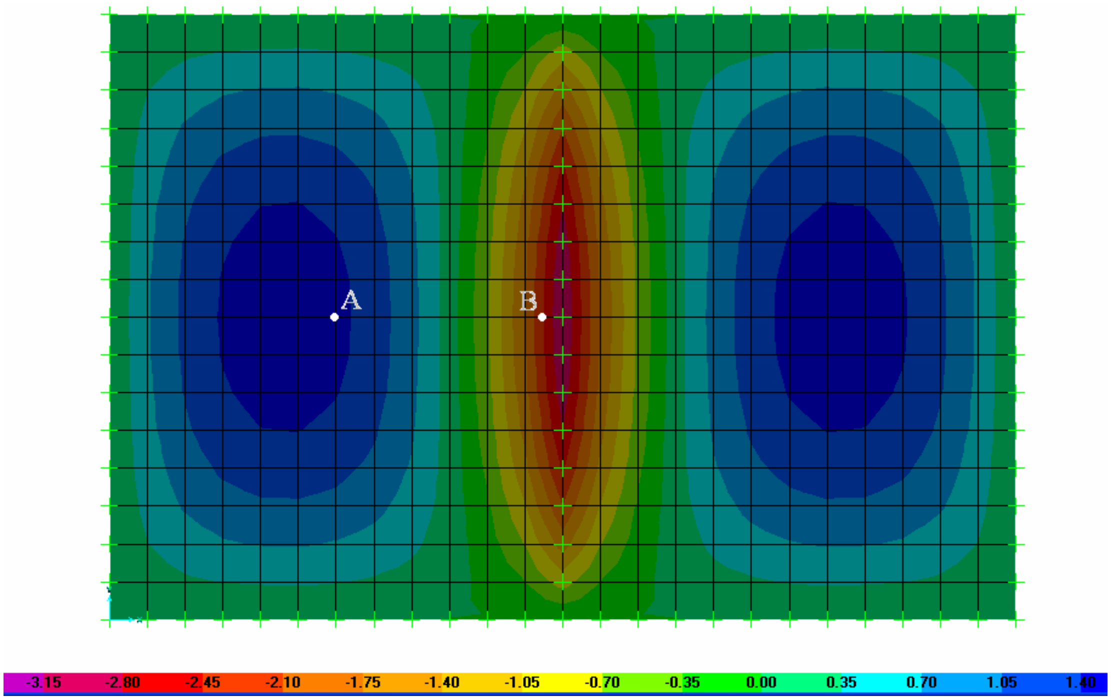



Figure B2: Plan of rooms of house with analyzed slab i.e. slabs moment values (kip-in/in). Marked points shows the sections used for comparison purpose (M11)

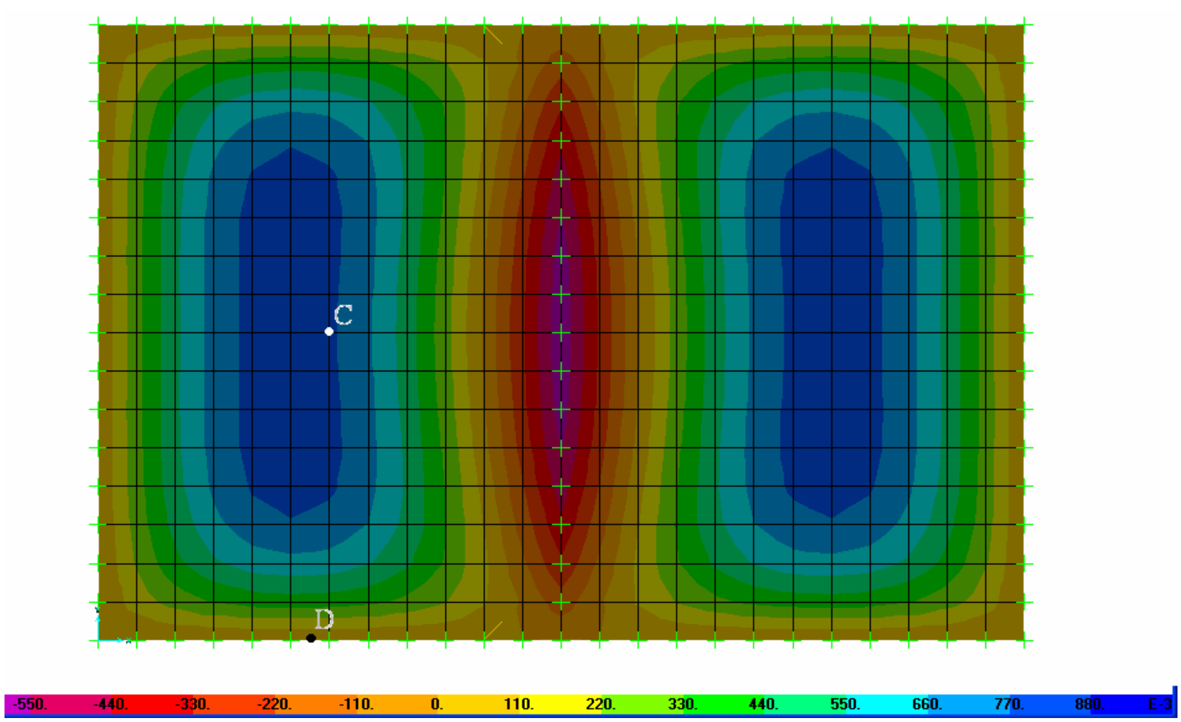



Figure B3: Plan of rooms of house with analyzed slab i.e. slab moment values (kip-in/in). Marked points shows the sections used for comparison purpose (M22)

| Table B2: Two-way slab of room, 12′ × 16′ (figure B2 & B3) | | | | |
|---|---|---|---|---|
| | ACI 318-99 | See fig | SAP Results | Percentage Difference |
| $M_{a,pos}$ (fig 04) | 1.53 | $M_A$ | 1.51 | 2 |
| $M_{b,pos}$ (fig 05) | 0.712 | $M_C$ | 0.75 | 5 |
| $M_{a,neg}$ (fig 04) | 2.67 | $M_B$ | 2.5 | -6 |
| $M_{b,neg}$ (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.*