Example 1(a): Design slab and beams of a 90′ × 60′ Hall. The height of Hall is 20′.

Concrete compressive strength (f_c') = 3 ksi.

Steel yield strength (f_y) = 40 ksi.

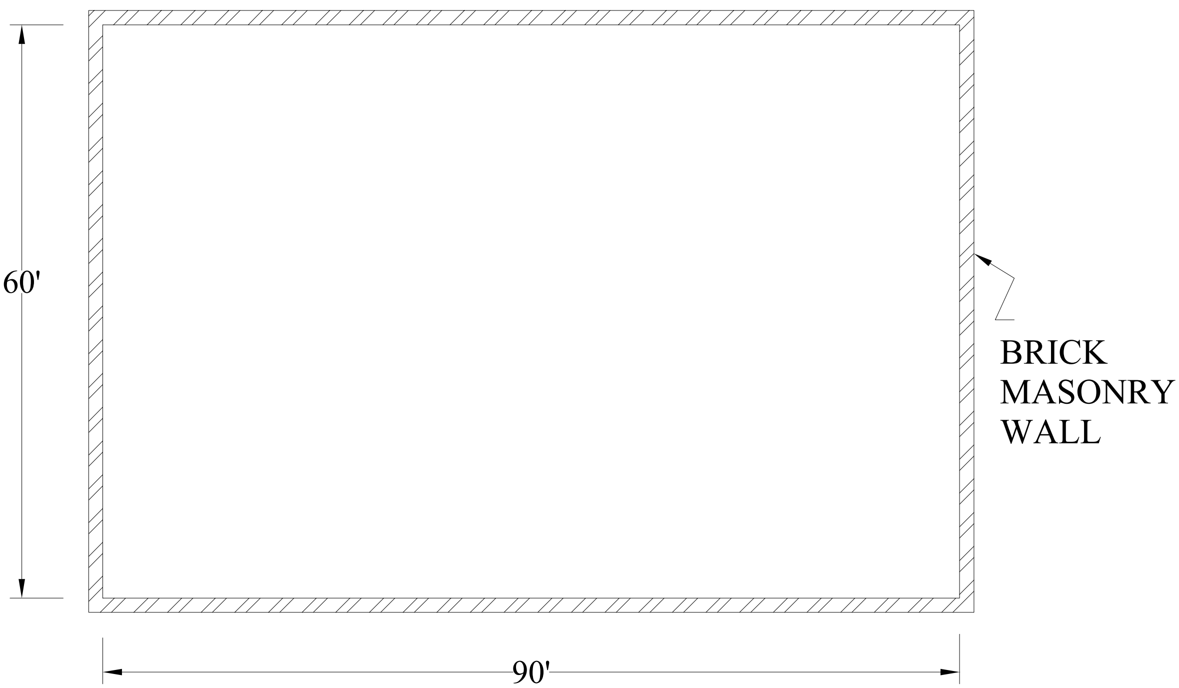


Figure 1: 90′ × 60′ Hall.

Solution: Assume structural configuration. Take time to reach to a reasonable arrangement of beams, girders and columns. It depends on experience. Several alternatives are possible.

First option for structural arrangement of the given Hall, figure 2:

- Beams spaced at 10′ c/c running along 60′ side of Hall.
- As height of Hall is 20′, assume 18″ thick brick masonry walls.

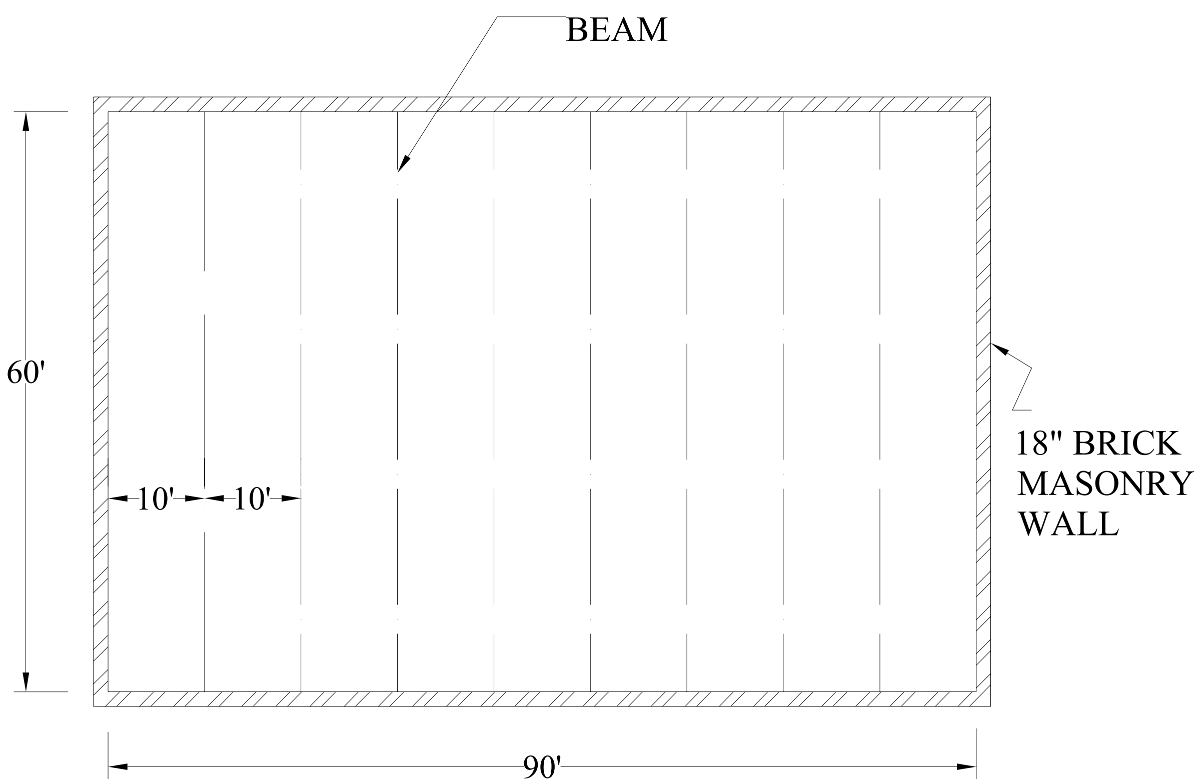


Figure 2: Structural Arrangement (90′ × 60′ Hall).

Discussion: *Various structural configurations…*

Discussion: *Beam as thickened slab portions…*

(1) SLAB DESIGN:

Step No 1: Sizes.

- Minimum thickness of continuous one way slab as given under ACI 9.5.2, table 9.5 (a) is:

Table 2.1: ACI formulae for continuous one way slab thickness, ACI 9.5.2	
Case	Slab thickness (in)
End span (one end continuous)	$l/24$
Interior span (both ends continuous)	$l/28$
(i) l = Span length in inches. *(ii)For f_y other than 60,000 psi, the values from above formulae shall be multiplied by $(0.4 + f_y/100000)$.*	

Span length "*l*" of slab is defined in ACI 8.7

Span length (*l*):

- According to ACI 8.7.1: *Span length of members not built integrally with support shall be considered as the clear span plus depth of the member, but need not exceed distance between center of supports.*
- According to ACI 8.7.4: *Span lengths for slabs built integrally with supports can be taken equal to clear span, if clear span of slab is not more than 10′.*
- ACI 8.7.1 applies to end span.
- ACI 8.7.4 applies to other spans.

Assuming the thickness of slab equal to 6″. Span length for end span of slab will be equal to clear span plus depth of member (slab), but need not exceed center to center distance between supports.

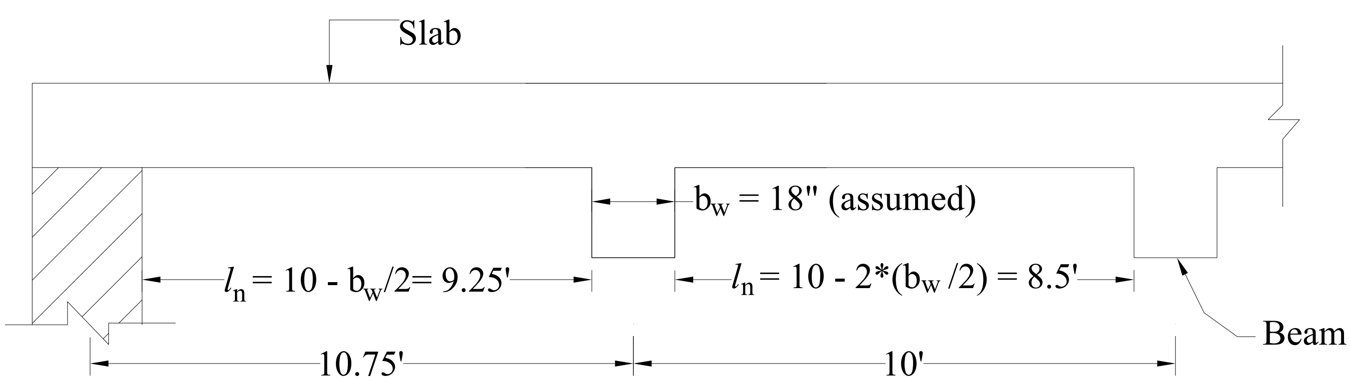


Figure 3: c/c & clear spans of slab.

Table 2.2: Span length of slab (figure 3)				
Case	c/c distance	Clear span (l_n)	l_n + depth of slab (ACI 8.7.1)	Span length(l)
End span (one end continuous)	10.75′	9.25'	9.25 + 0.5 = 9.75'	9.75′
Interior spans (both ends continuous)	10′	8.5'	n/a	8.5′

Table 2.3: Slab thickness calculation according to ACI 9.5.2.		
Span	Formula for thickness	Thickness of slab (in)
End span (one end continuous)	$l/24 \times (0.4+f_y/100000)$	(9.75/24)×(0.4 +40000/100000) ×12=3.9″
Interior span (both ends continuous)	$l/28 \times (0.4+f_y/100000)$	(8.5/28)×(0.4 + 40000/100000)×12=2.9″
l = Span length in inches.		

Therefore,

Slab thickness (h_f) = 3.9″ (Minimum requirement by ACI 9.5.2.1).

Though any depth of slab greater than 3.9″ can be taken as per ACI minimum requirement, we will use the same depth as assumed i.e. 6″

Effective depth (d) = h_f – 0.75 – (3/8)/2 = 5″ (for #3 main bars)

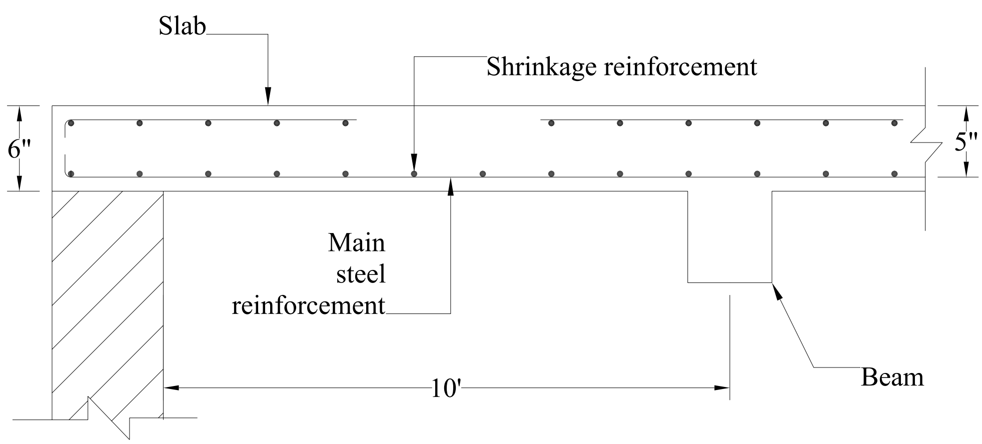


Figure 4: Effective depth of slab.

Step No 2: Loading.

Table 2.4: Dead Loads.			
Material	Thickness (in)	γ (kcf)	Load = γ x thickness (ksf)
Slab	6	0.15	(6/12) × 0.15 = 0.075
Mud	3	0.12	(3/12) × 0.12 = 0.03
Tile	2	0.12	(2/12) × 0.12= 0.02

Service Dead Load (D.L) = 0.075 + 0.03 + 0.02 = 0.125 ksf

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

Class Activity: *Calculate live load per square foot on the class room floor when it is fully occupied.*

Service Load (w_s) = D.L + L.L = 0.125 + 0.04 = 0.165 ksf

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

= 1.2 × 0.125 + 1.6 × 0.04 = 0.214 ksf

Step No 3: Analysis.

Our Slab system is:

- One-way,
- Clear spans less than 10′, and
- Exterior ends of slab are discontinuous and unrestrained.

Refer to ACI 8.3.3 or page 396, Nilson 13th Ed, following ACI moment coefficients apply:

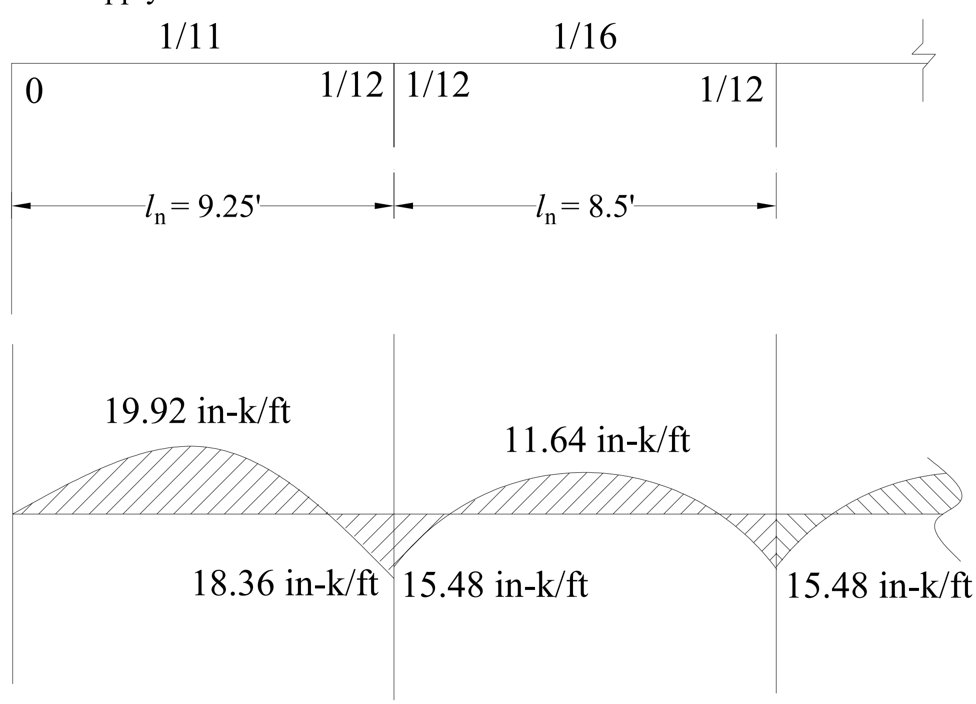


Figure 5: Bending Moment Diagram for slab.

(i) AT INTERIOR SUPPORT (left side of support):

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

$= (1/12) \times \{0.214 \times (9.25)^2\}$

$= 1.53$ ft-k/ft = 18.36 in-k/ft

(ii) AT INTERIOR SUPPORT (right side of support):

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

$= (1/12) \times \{0.214 \times (8.5)^2\}$

$= 1.29$ ft-k/ft = 15.48 in-k/ft

(iii)AT EXTERIOR MID SPAN:

Positive moment ($+M_{Mext}$) =Coefficient × ($w_u l_n^2$)

$= (1/11) \times \{0.214 \times (9.25)^2\}$

$= 1.66$ ft-k/ft = 19.92 in-k/ft

(iv)AT INTERIOR MID SPAN:

Positive moment ($+M_{Mint}$) =Coefficient × ($w_u l_n^2$)

$= (1/16) \times \{0.214 \times (8.5)^2\}$

$= 0.97$ ft-k/ft = 11.64 in-k/ft

Discussion: *ACI analysis vs actual conditions for beam support, concepts of hinge, roller supports etc.*

Step No 4: Design.

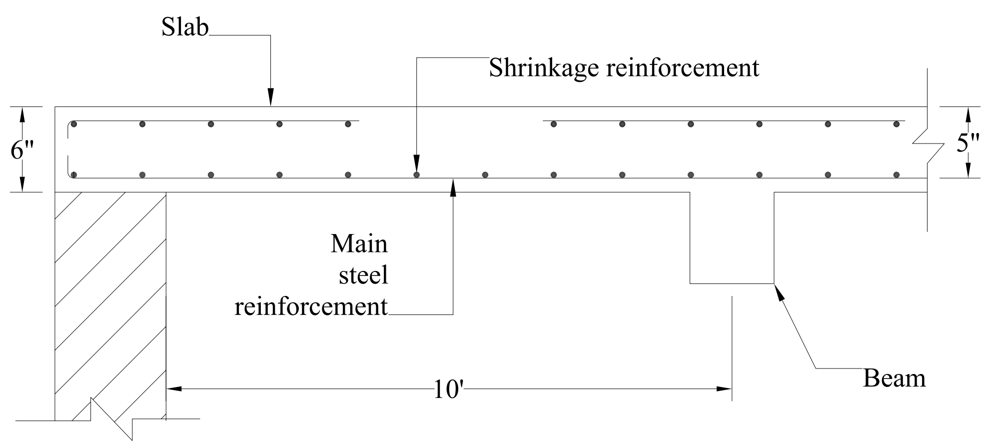


Figure 6: Reinforcement Placement in slab.

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

$= 0.002 \times 12 \times 6 = 0.144$ in^2/ft

$a = A_{smin}f_y/ (0.85f_c'b) = 0.144 \times 40/ (0.85 \times 3 \times 12) = 0.188''$

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

= 0.9 × 0.144 × 40 × (5-0.188/2) = 25.4 in-k/ft

Φ M_n calculated from A_{smin} is greater than all moments as calculated in Step No 3.

Therefore $A_s = A_{smin} = 0.144$ in^2/ft

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^2/0.144 in^2/ft) × 12 = 16.67 in

Discussion: *Bar numbers commonly used in slabs…?*

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^2/0.144 in^2/ft) × 12 = 9.16″ ≈ 9″

Finally use #3 @ 9″ c/c (#10 @ 225 mm c/c). This will work for both Positive and Negative steel as A_{smin} governs.

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

$A_{st} = 0.002bh_f$

$A_{st} = 0.002 \times 12 \times 6 = 0.144$ in^2/ft

Shrinkage reinforcement is same as main reinforcement, because:

$A_{st} = A_{smin} = 0.144$ in^2

- Maximum spacing for main steel reinforcement in one way slab according to ACI 7.6.5 is minimum of:
  (i) $3h_f = 3 \times 6 = 18$″
  (ii) 18″

Therefore 9″ spacing is O.K.

- Maximum spacing for temperature steel reinforcement in one way slab according to ACI 7.12.2.2 is minimum of:
  (i) $5h_f = 5 \times 6 = 30$″
  (ii) 18″

Therefore 9″ spacing is O.K.

(2) BEAM DESIGN (single span, simply supported):

Data Given:

Exterior supports of beam = 18″ brick masonry wall.

f_c'= 3 ksi; f_y = 40 ksi

Beams c/c spacing =10′

Step No 1: Sizes.

According to ACI 9.5.2.1, table 9.5 (a):

Minimum thickness of beam (simply supported) = h_{min} = $l/16$

l = clear span (l_n) + depth of member (beam) ≤ c/c distance between supports [ACI 8.7]

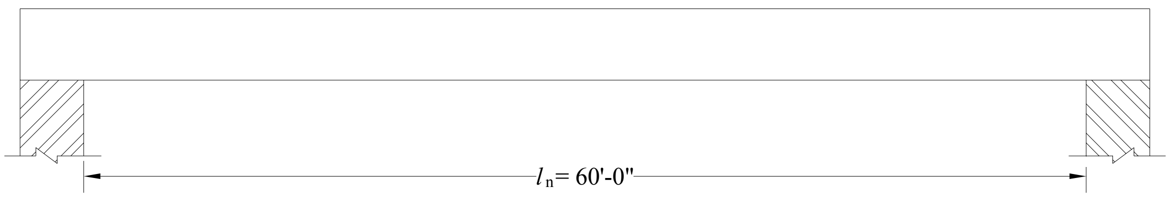


Figure 7: Clear span of Beam.

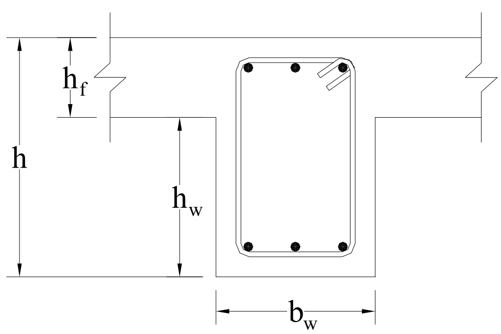


Figure 8: Beam cross-section.

Let depth of beam = 5′

l_n + depth of beam = 60′ + 5′ = 65′

c/c distance between beam supports = 60 + 2 × (9/12) = 61.5′

Therefore l = 61.5′

Depth (h) = (61.5/16) × (0.4+f_y/100000) × 12

= 36.9″ (Minimum requirement by ACI 9.5.2.1).

Though any depth of beam greater than 36.9″ can be taken as per ACI minimum requirement, we will use the same depth as assumed i.e. 60″

Take h = 5′ = 60″

d = h – 3 = 57″

Step No 2: Loads.

Service Dead Load (D.L) = 0.075 + 0.03 + 0.02 = 0.125 ksf (Table 2.3)

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

Beam is supporting 10′ slab. Therefore load per running foot will be as follows:

Service Dead Load from slab= 0.125 × 10 = 1.25 k/ft

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

= (54 × 18/144) × 0.15 =1.0125 k/ft

Total Service Dead Load = 1.25 + 1.0125 =2.2625 k/ft

Service Live Load = 0.04 × 10 = 0.4 k/ft

w_s = D.L +L.L = 1.0125 + 0.4 = 1.4125 k/ft

w_u =1.2D.L + 1.6L.L =1.2 × 2.2625 + 1.6 × 0.4 =3.355 k/ft

Step No 3: Analysis.

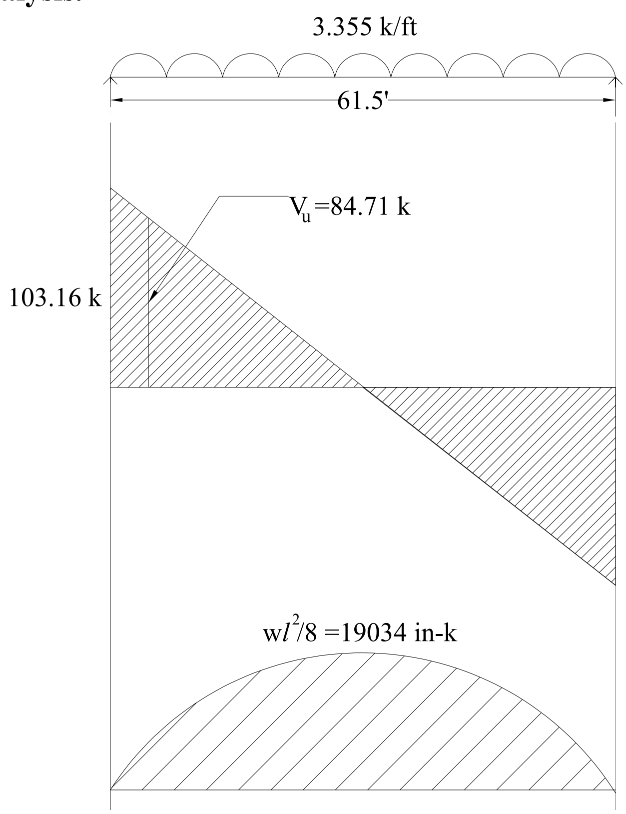


Figure 9: Shear Force & Bending Moment Diagrams.

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

$M_u = 3.355 \times 61.5^2/8 = 1586.18$ ft-k $=1586.18 \times 12 = 19034$ in-k

$d = 57'' = 4.75'$

$V_{max} = 103.16$ k

$V_u = 84.70$ k

Step No 4: Design of beam.

(A) Flexural Design:

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

(i) $16h_f + b_w = 16 \times 6 + 18 = 114''$

(ii) (c/c span of beam)/4 $=(61.5'/4) \times 12 = 184.5''$

(iii) c/c spacing between beams $=10' \times 12 = 120''$

So $b_{eff} = 114''$

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

Trial #1:

(i) Assume $a = h_f = 6''$

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

$A_s = 19034 / \{0.9 \times 40 \times (57-6/2)\} = 9.79$ in^2

(ii) Re-calculate "a":

$a = A_s f_y / (0.85 f_c' b_{eff})$

$a = 9.79 \times 40 / (0.85 \times 3 \times 114) = 1.34'' < h_f$

Therefore design beam as rectangular beam.

Trial #2:

$A_s = 19034 / \{0.9 \times 40 \times (57-1.34/2)\} = 9.38$ in^2

$a = 9.38 \times 40 / (0.85 \times 3 \times 114) = 1.29''$

This value is close enough to the previously calculated value of "a", therefore, $A_s = 9.38$ in^2, O.K.

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

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

$\rho_{max} = 0.85\beta_1 (f_c'/f_y) \{\varepsilon_u/(\varepsilon_u + \varepsilon_y)\}$

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

$A_{smax} = 0.0203 \times 18 \times 57 = 20.83$ in^2

$A_{smin} = \rho_{min} b_w d = (200/40000) \times 18 \times 57 = 5.13$ in^2

$A_{smin} < A_s < A_{smax}$, O.K.

Note that ρ_{min} & ρ_{max} can also be found using table A.4, Nelson 13th Ed.

<u>Beam (main reinforcement):</u>

$A_s = 9.38$ in^2

Using #8, 1″ Φ {#25, 25 mm}, with bar area $A_b = 0.79$ in^2

No. of bars = $A_s/A_b = 9.38/0.79 = 11.87 \approx 12$ bars

Use 12 #8 bars (12 #25 bars, 25 mm).

☞ <u>Check the capacity of designed beam:</u>

Area of 12 #8 Bars = 12 × 0.79 = 9.48 in^2

$a = A_s f_y / (0.85 f_c' b_{eff}) = 9.48 \times 40 / (0.85 \times 3 \times 114) = 1.30''$

$d' = 1.5 + (3/8) + 1 + (1/2) = 3.375''$

$d = h - d' = 60 - 3.375 = 56.625''$

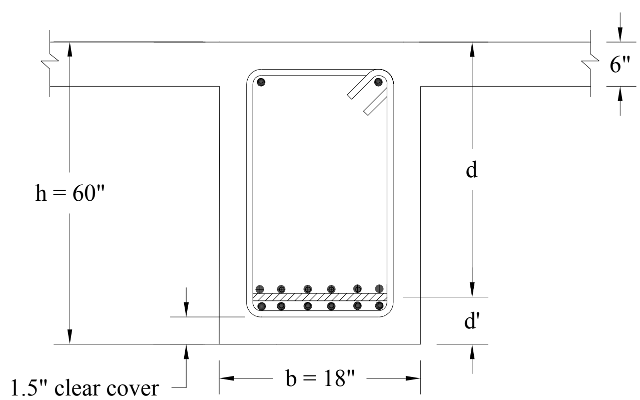


Figure 10: Beam section dimensions.

$M_d = \Phi A_s f_y (d - a/2) = 0.9 \times 9.48 \times 40 \times (56.625 - 1.30/2) = 19103.2$ in-k

$M_d > (M_u = 19034$ in-k), O.K.

☞ <u>Skin Reinforcement:</u>

According to ACI 10.6.7 "If the effective depth ***d*** of a beam or joist exceeds 36 in., longitudinal skin reinforcement shall be uniformly distributed along both side faces of the member for a distance ***d* /2** nearest the flexural tension reinforcement. The spacing s_{sk} between longitudinal bars or wires of the skin reinforcement shall not exceed the least of ***d***/6, 12 in., and $1000A_b/(d - 30)$. It shall be permitted to include such reinforcement in strength computations if a strain compatibility analysis is made to determine stress in the individual bars or wires. The total area of longitudinal skin reinforcement in both faces need not exceed one-half of the required flexural tensile reinforcement".

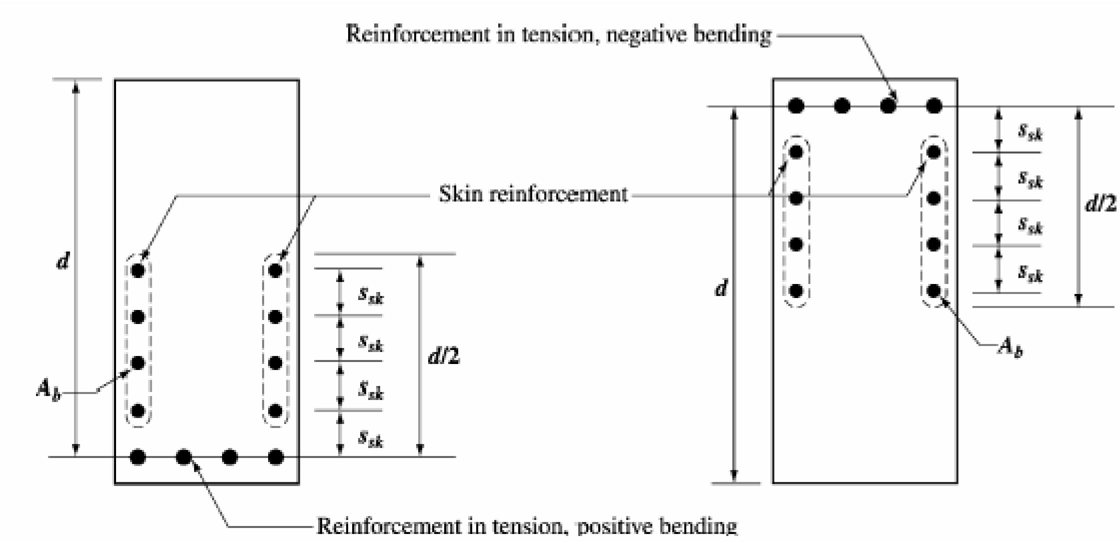


Figure 11: Skin reinforcement for beams and joists with d > 36 inches.

Maximum area of skin reinforcement allowed by ACI:

$A_{skin, max}$ = Main flexural reinforcement/2 = 9.38/2 = 4.69 in^2

Range up to which skin reinforcement is provided:

d/2 = 56.625/2 = 28.3125″

For #6 bar used in skin reinforcement,

s_{sk} is least of:

- d/6 = 56.625/6 = 9.44″
- 12″
- $1000A_b/(d - 30) = 1000 \times 0.44/(56.625 - 30) = 16.53''$

Therefore $s_{sk} = 9.44'' \approx 9''$

With this spacing, 3 bars on each face are required. And for #6 bar, the total area of skin reinforcement is:

$A_{skin} = 6 \times 0.44 = 2.64\ in^2 < A_{skin, max} = 4.69\ in^2$, O.K

(B) Shear Design:

$V_u = 84.71$ k

$\Phi V_c = \Phi 2\sqrt{(f_c')} b_w d = (0.75 \times 2 \times \sqrt{(3000)} \times 18 \times 57)/1000 = 84.29$ k

$\Phi V_c < V_u$ {Shear reinforcement is required}

$s_d = \Phi A_v f_y d/(V_u - \Phi V_c)$

Using #3, 2 legged stirrups with $A_v = 0.11 \times 2 = 0.22$ in^2}

$s_d = 0.75 \times 0.22 \times 40 \times 57/(84.71 - 84.29) = 895''$

Maximum spacing and minimum reinforcement requirement as permitted by ACI 11.5.4 and 11.5.5.3 shall be minimum of:

(i) $A_v f_y/(50 b_w) = 0.22 \times 40000/(50 \times 18) \approx 9.5''$

(ii) $d/2 = 57/2 = 28.5''$

(iii) $24''$

(iv) $A_v f_y/ 0.75\sqrt{(f_c')} b_w = 0.22 \times 40000/ \{(0.75 \times \sqrt{(3000)} \times 18\} = 11.90''$

<u>Other checks:</u>

(a) Check for depth of beam:

$\Phi V_s \leq \Phi 8\sqrt{(f_c')} b_w d$ (ACI 11.5.6.9)

$\Phi 8\sqrt{(f_c')} b_w d = 0.75 \times 8 \times \sqrt{(3000)} \times 18 \times 57/1000 = 337.18$ k

$\Phi V_s = (\Phi A_v f_y d)/s_d$

$= (0.75 \times 0.22 \times 40 \times 57)/9.5 = 39.6 \text{ k} < 337.18$ k, O.K.

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

(b) Check if "$\Phi V_s \leq \Phi 4\sqrt{(f_c')} b_w d$" {ACI 11.5.4.3}:

If "$\Phi V_s \leq \Phi 4\sqrt{(f_c')} b_w d$", the maximum spacing ($s_{max}$) is O.K. Otherwise reduce spacing by one half.

$\Phi 4\sqrt{(f_c')} b_w d = 0.75 \times 4 \times \sqrt{(3000)} \times 18 \times 57/1000 = 168.58$ k

$\Phi V_s = (\Phi A_v f_y d)/s_d$

$= (0.75 \times 0.22 \times 40 \times 57)/9.5 = 39.6 \text{ k} < 168.58$ k, O.K.

<u>Arrangement of stirrups in the beam:</u> With #3, 2 legged vertical stirrups @ 9.5″ c/c (maximum spacing and minimum reinforcement requirement as permitted by ACI), the shear capacity (ΦV_n) of the beam will be equal to:

$\Phi V_n = \Phi V_c + \Phi V_s$

$\Phi V_s = (\Phi A_v f_y d)/s_{max}$

$\Phi V_s = (0.75 \times 0.22 \times 40 \times 57/9.5) = 39.6$ k

Therefore $\Phi V_n = 84.29 + 39.6 = 123.89$ k > ($V_u = 87.22$ k)

It means that theoretically, from a section at a distance equal to s/2 up to a section where shear is equal to $\Phi V_c/2$, #3, 2 legged vertical stirrups @ 9.5″ c/c shall be provided. Beyond the value of $\Phi V_c/2$, no shear reinforcement is theoretically required. However # 3, 2 legged vertical stirrups @ 12″ c/c are recommended to hold the flexural reinforcement bars.

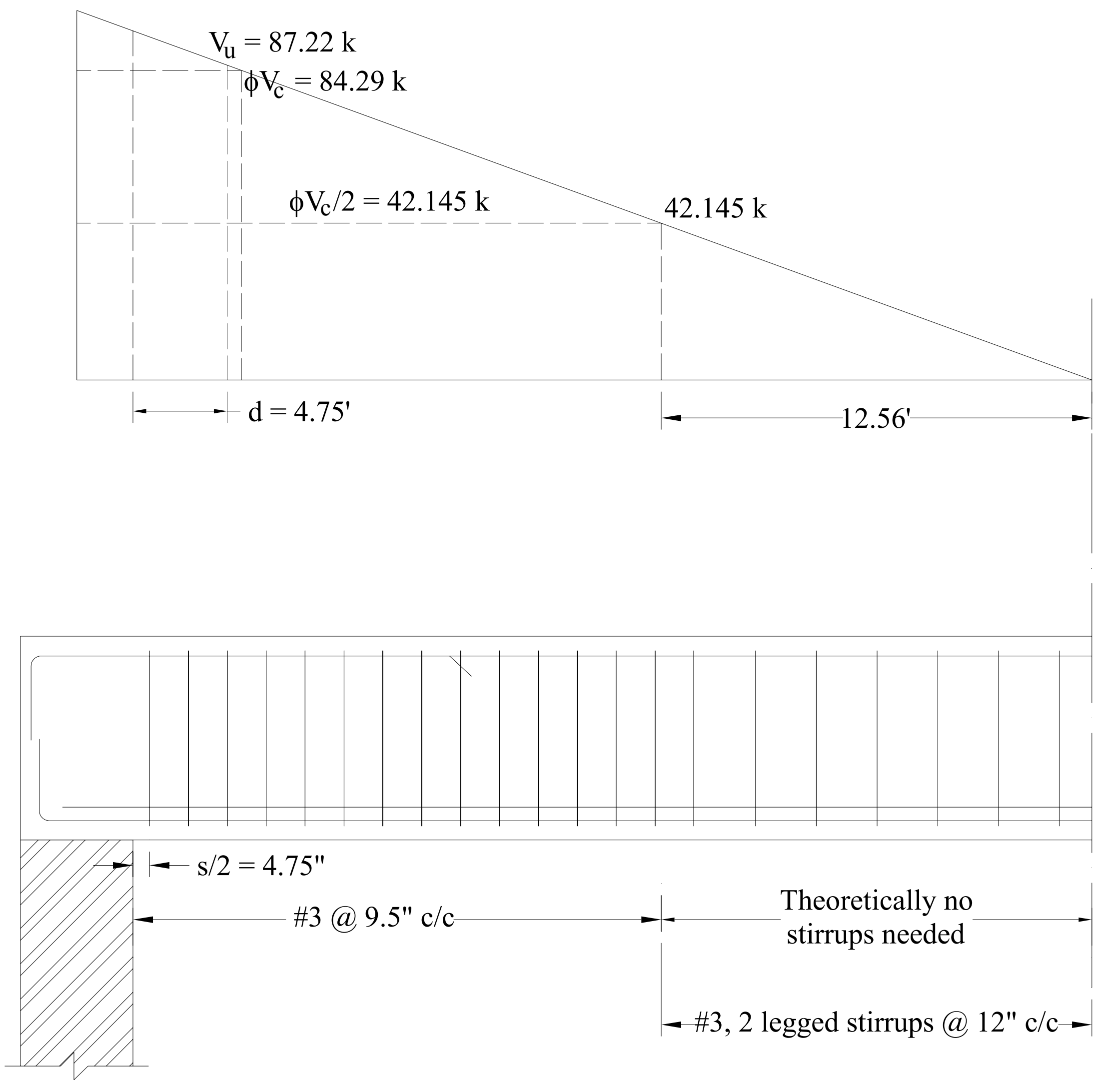
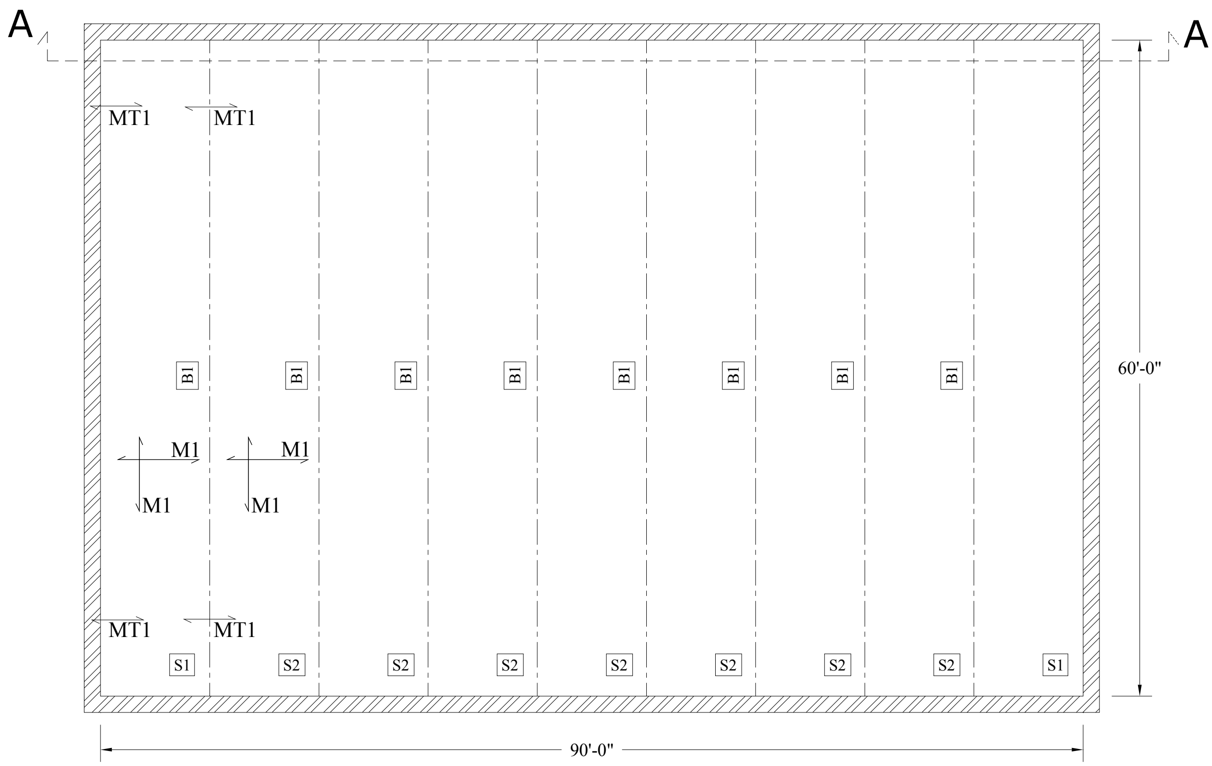


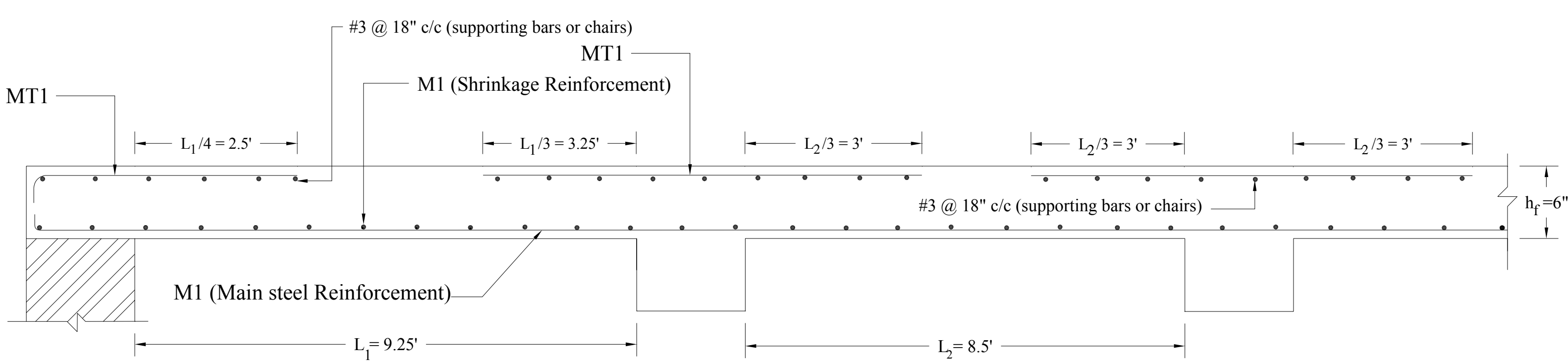
Figure 12: Stirrups arrangement.

(3) DRAFTING:

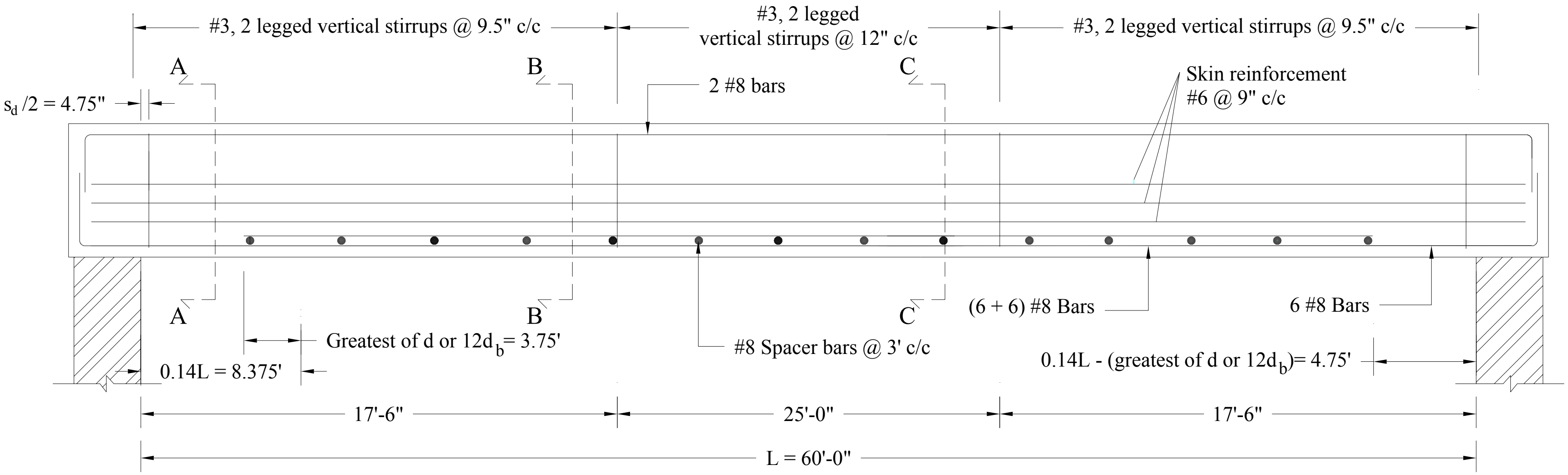
(I) Slab (S1 and S2):



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



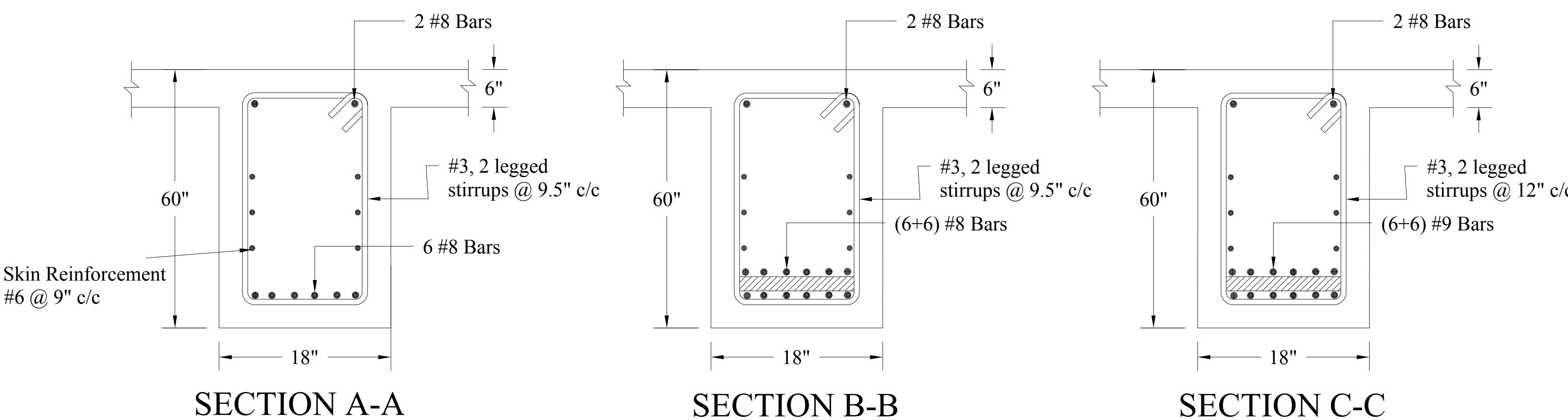
Section A-A. Refer to figure 5.15, chapter 5, Nelson 13th Ed for bar cutoff.



Beam (B1) Details

(a) Use graph A2 to find location of points where bars can be bent up or cutoff for simply supported beams uniformly loaded.

(b) Approximate locations of points where bars can be bent up or cotoff for continuous beams uniformly loaded and built integrally with their supports according to the coefficients in ACI code.



Appendix A

Comparison of ACI coefficients analysis with analysis of SAP2000 (finite element method based software): Assumptions made in SAP model are,

a. Brick masonry walls are modeled as hinged support.

b. Slab is modeled as shell element.

c. Beams are modeled as frame elements.

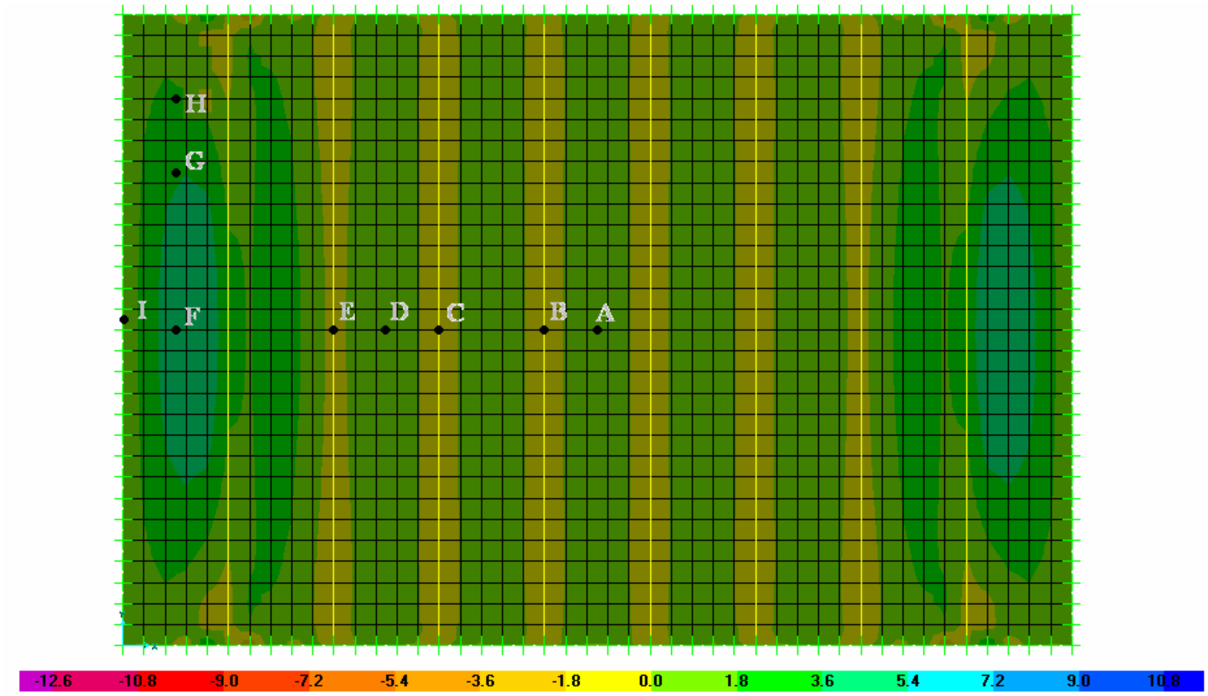


Figure 13: Plan view of hall showing variation in slab moment (kip-in/in). Marked points show the locations picked for comparison purpose.

Table 2.5: Slab moments				
	ACI 318-02 (kip-in/in)	See figure 9	SAP Results (k-in/in)	Percentage Difference
M (at wall)	0	M_I	0.02	-
M (at ext. mid span)	1.66	M_F	4.59	64
		M_G	3.40	51
		M_H	1.55	- 7
M (at int. support)	1.53	M_B	1.47	-4
		M_C	1.48	-4
		M_E	0.617	-60
M (at int. mid span)	0.97	M_A	1.04	7
		M_D	1.2	19

Table 2.6: Simply supported beam moment.			
	ACI 318-02	SAP Results	Percentage Difference
$M_{,mid\ span}$ (k-in)	19034	18987.253	0.25

Conclusions:

- There is more variation between SAP and ACI in slab moments.
- Less variation in beam moment.

Appendix B

Relevant Pictures

Figure 14: Supporting chairs for slab reinforcement.

Figure 15: Reinforcement in slab.

Figure 16: Flexure and shear reinforcement in a beam.

Figure 16: Local arrangement for bar bending.

Appendix C

Minimum uniformly distributed live load:

Representative values of minimum live loads to be used in a wide variety of buildings are found in *Minimum Design Loads for Buildings and Other Structures, SEI/ASCE 7-02, American Society of Civil Engineers*, a portion of which is reproduced in table C1. The table gives uniformly distributed live loads for various types of occupancies; these include impact provisions where necessary. These loads are expected maxima and considerably exceed average values.

Table C1: Minimum uniformly distributed live loads.

Occupancy or Use	Live Load, psf		Live Load, psf
Apartments (see residential)		**Dining rooms and restaurants**	100
Access floor systems		**Dwellings (see residential)**	
Office use	50	**Fire escapes**	100
Computer use	100	On single-family dwellings only	40
Armories and drill rooms	150	**Garages (passenger cars only)**	40
Assembly areas and theaters		Trucks and buses *(see foot note)*	
Fixed seats (fastened to floors)	60	**Grandstands (see stadium and arena bleachers)**	
Lobbies	100	**Gymnasiums, main floors, and balconies**	100
Movable seats	100	**Hospitals**	
Platforms (assembly)	100	Operating rooms, laboratories	60
Stage floors	150	Private rooms	40
Balconies (exterior)	100	Wards	40
On one and two family residences only, and not exceeding 100ft2	60	Corridors above first floor	80
Bowling alleys, poolrooms, and similar recreational areas	75	**Hotels (see residential)**	
Catwalks for maintenance access	40	**Libraries**	
Corridors		Reading rooms	60
First floor	100	Stack rooms	150
Other floors, same as occupancy served except as indicated		Corridors above first floor	80
Dance halls and ballrooms	100	**Manufacturing**	
Decks (patio and roof)		Light	125
Same as area served, or for the type of occupancy accommodated		Heavy	250

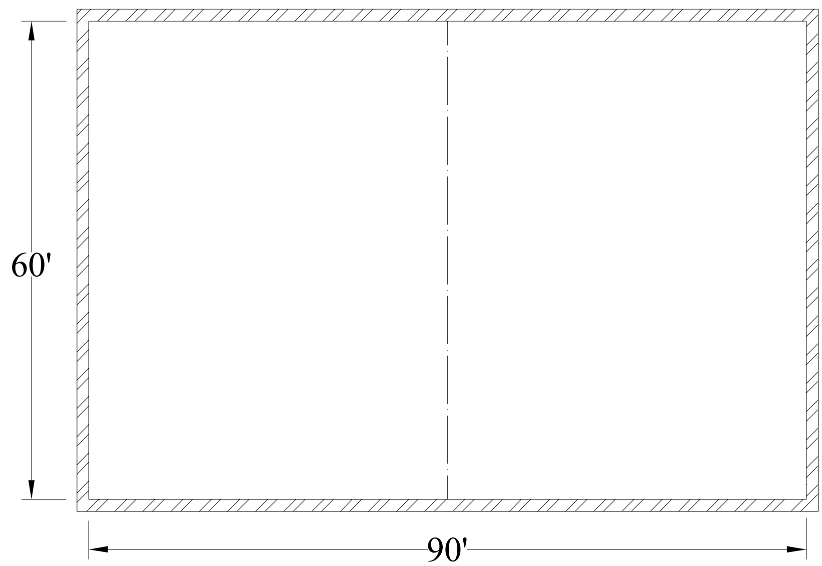
Table C1: (Continued)

Occupancy or Use	Live Load, psf	Occupancy or Use	Live Load, psf
Marquees and Canopies	75	**Sidewaks, vehicular driveways, and yards subject to trucking**	250
Office Buildings		**Stadium and arenas**	
File and computer rooms shall be designed for heavier loads based on anticipated occupancy		Pleachers	100
Lobbies and first-floor corridors	100	Fixed seats (fastened to floors)	60
Offices	50	**Stairs and exitways**	100
Corridors above first floor	80	One and two-family residences only	40
Penal institutions		**Storage areas above ceilings**	20
Cell blocks	40	**Storage warehouses (shall be designed for heavier loads if required for anticipated storage)**	
Corridors	100	Light	125
Residential		Heavy	250
Dwellings (one and two-family)		**Stores**	
Uninhabitable attics without storage	10	Retail	
Uninhabitable attics with storage	20	*First floor*	100
Habitable attics and sleeping areas	30	*Upper floors*	73
All other areas except stairs and balconies	40	Wholesale, all floors	125
Hotels and multifamily houses		**Walkways and elevated platforms (other than exitways)**	60
Private rooms and corridors serving them	40	**Yards and terraces, pedestrians**	100
Public rooms and corridors serving them	100		
Reviewing stands, grandstands and bleachers	100		
Schools			
Classrooms	40		
Corridors above first floor	80		
First-floor corridors	100		

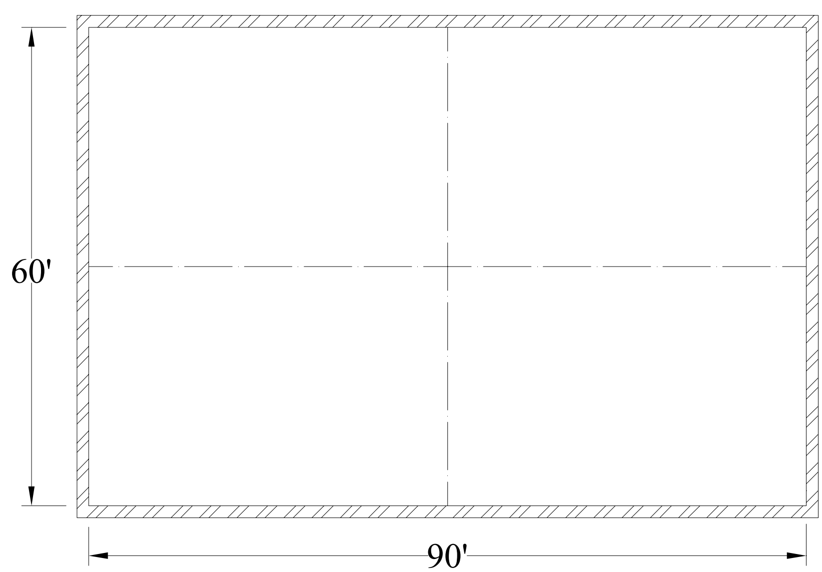
[a] *Garages accommodating trucks and buses shall be designed in accordance with an approved method that contains provisions for truck and bus loadings.*

Appendix D

Various Structural Configurations



90' x 60' Hall



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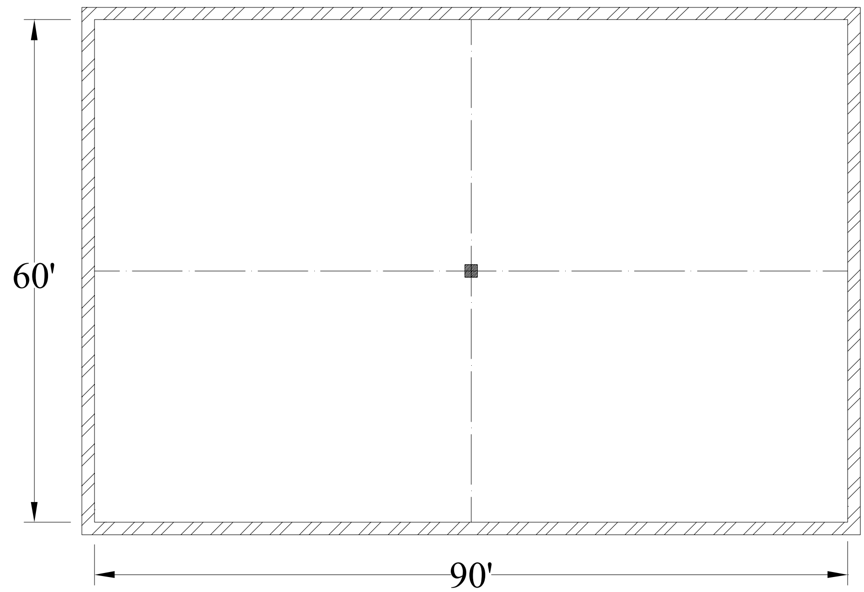

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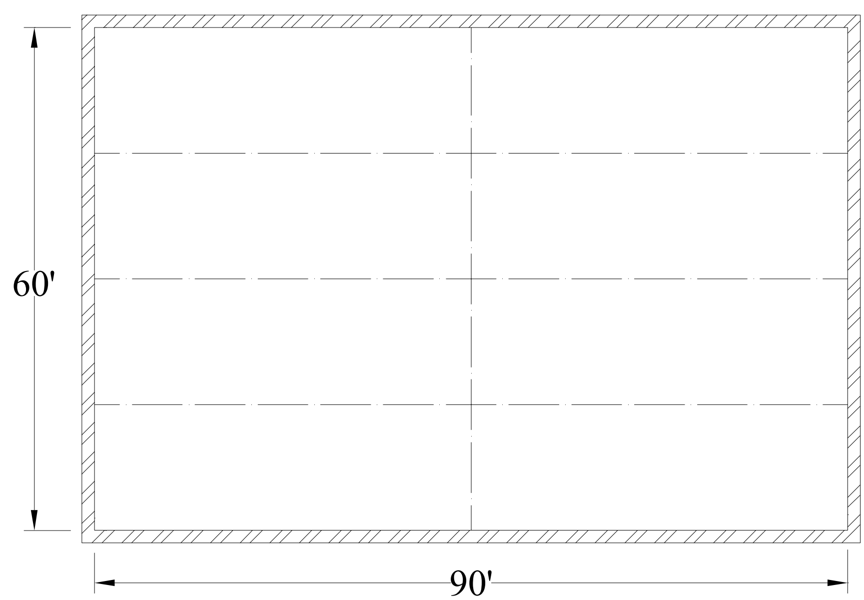

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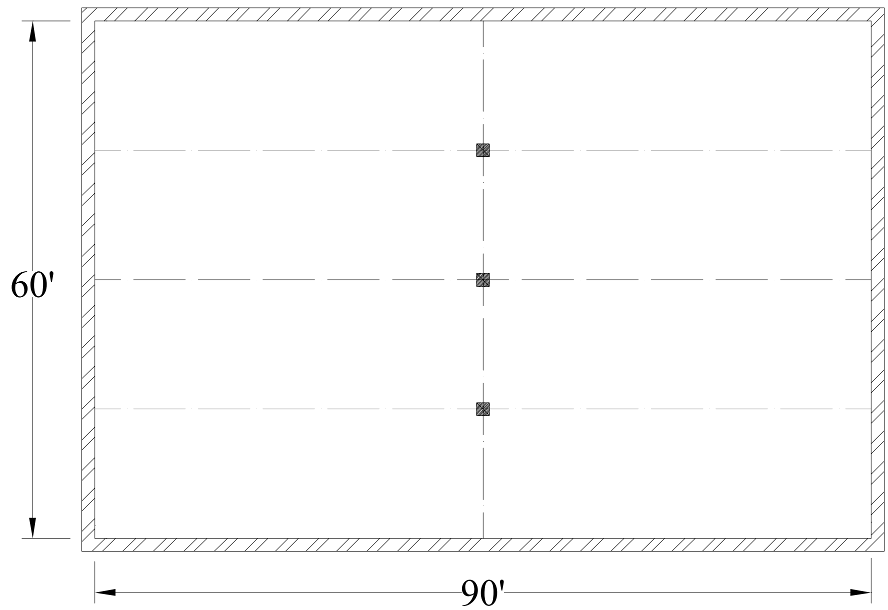

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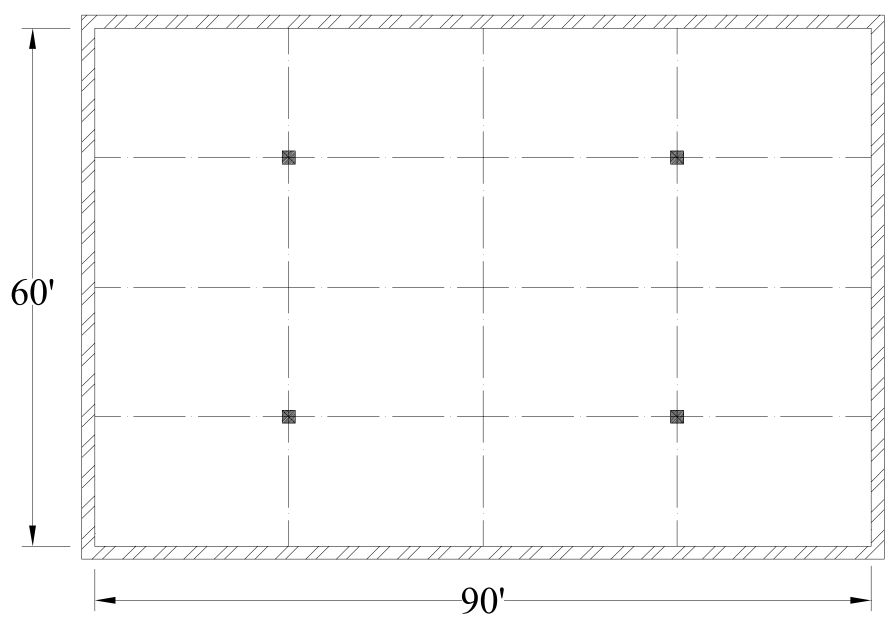

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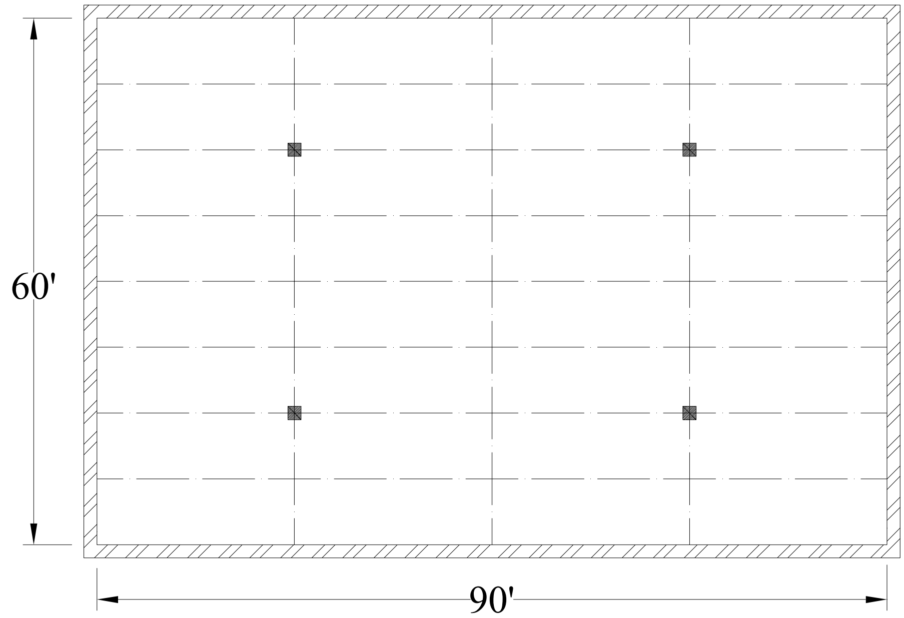

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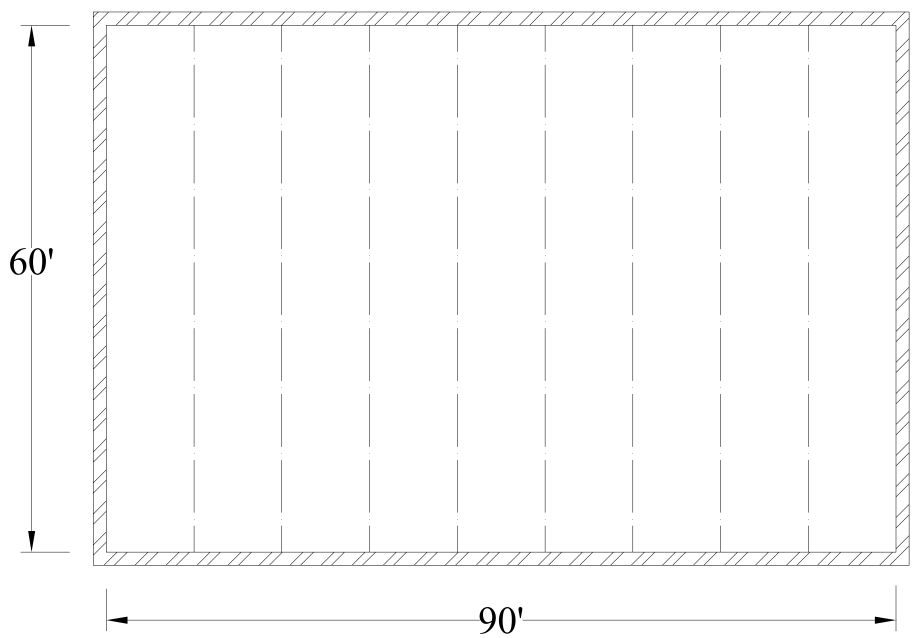

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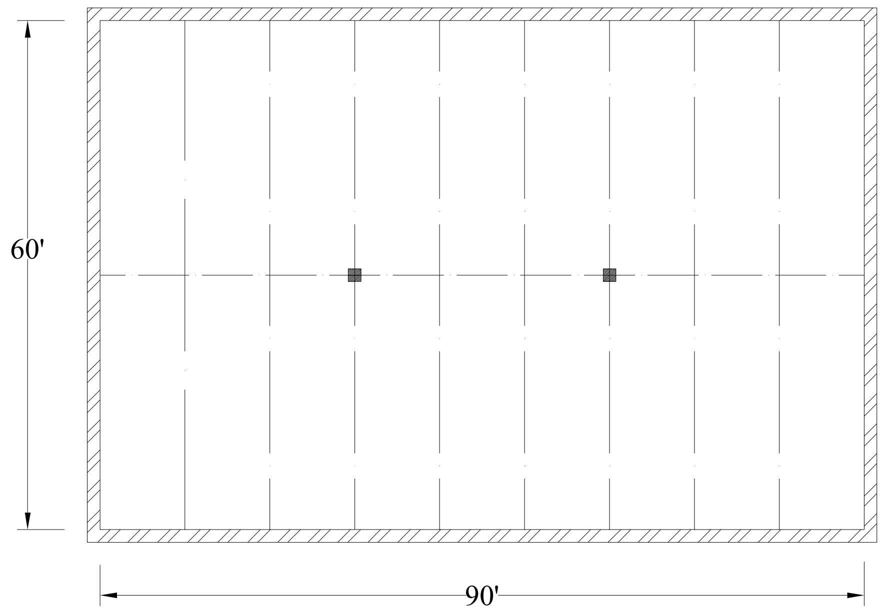

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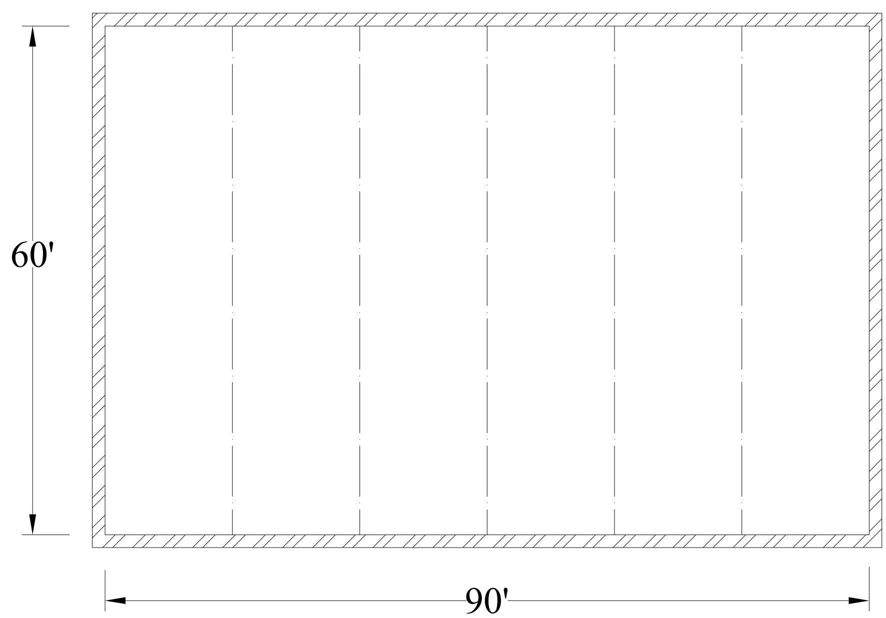
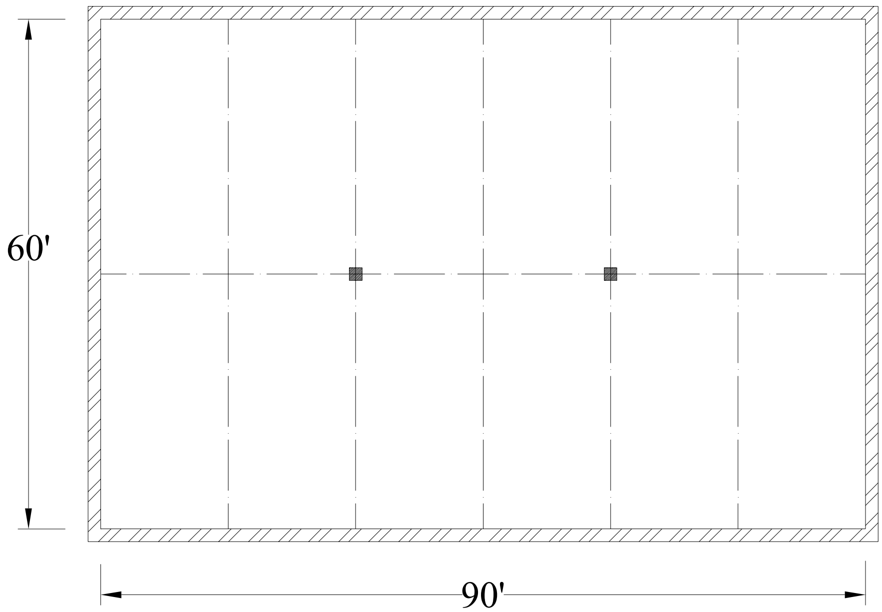
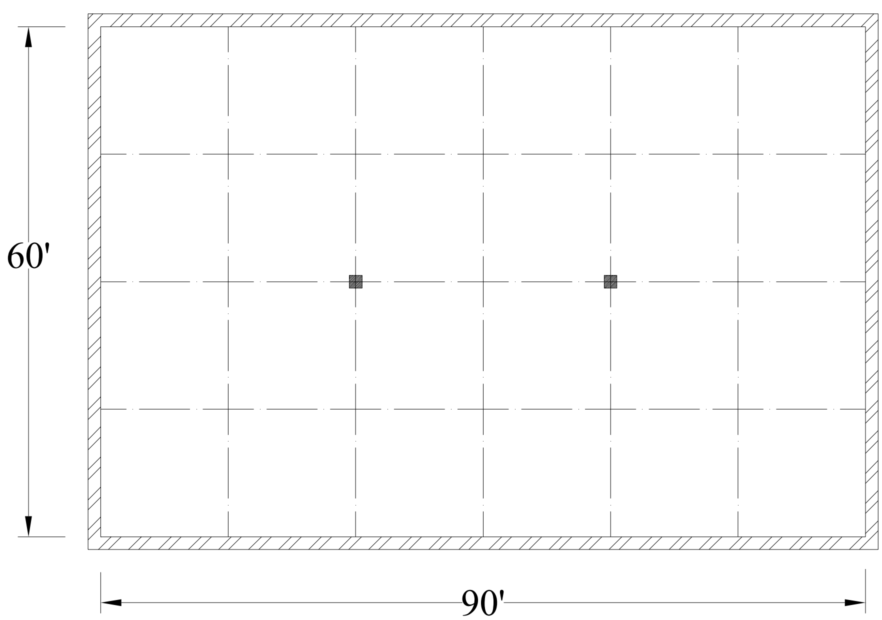


90' x 60' Hall



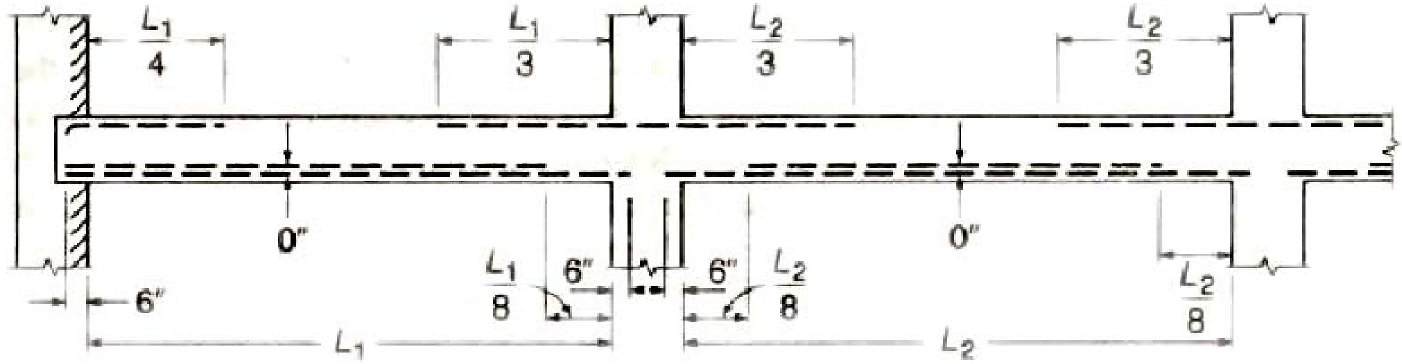
90' x 60' Hall



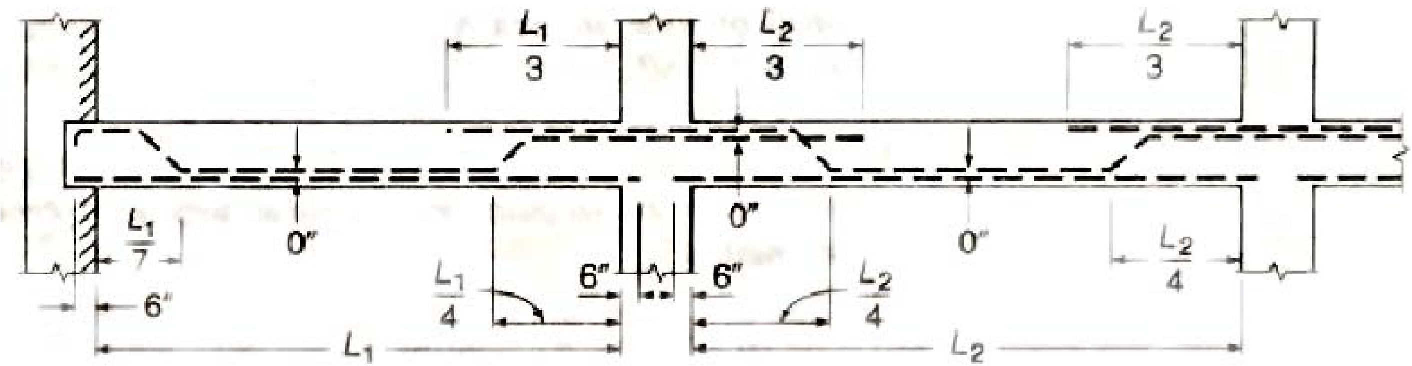
90' x 60' Hall

Appendix E

Cutoff or bend points for bars in approximately equal spans with uniformly distributed loads:



(a)



(b)

References

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