Lecture - 03

Analysis and Design of One-way Slab System (Part II)

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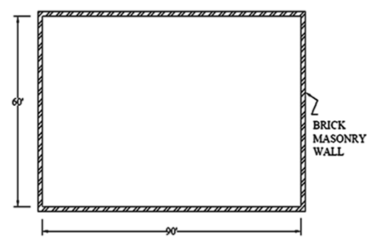
Topics

- Design Problem: Option 2 for Design of Hall
- References

Design Problem

- Design slab, beams, girder, columns and footings of a 90′ × 60′ Hall. Minimum obstruction to mobility inside the Hall requires that only two columns can be allowed inside the Hall. Height of the hall is 20′.
  - Concrete compressive strength (f_c') = 3 ksi.
  - Steel yield strength (f_y) = 40 ksi.



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Design Problem

- **Structural Arrangement**

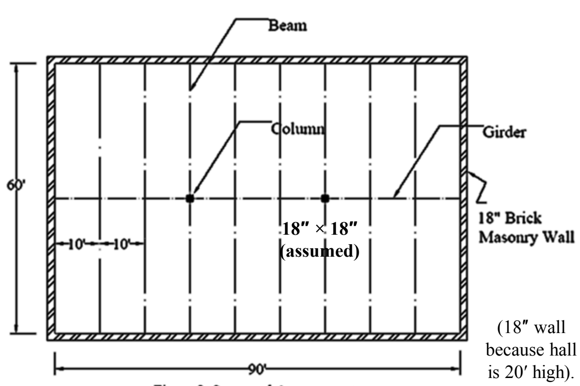
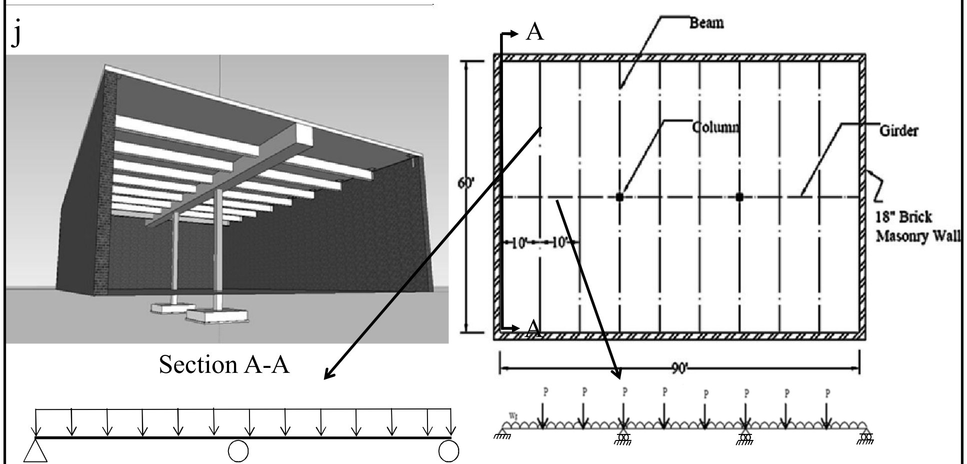


Figure 2: Structural Arrangement.

(18″ wall because hall is 20′ high).

Design Problem

- **Structural Arrangement**



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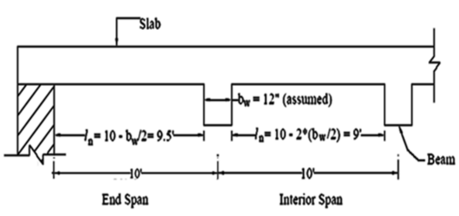
Design Problem

- **Slab Design**
  - Step No. 01: Sizes

TABLE 9.5(a)—MINIMUM THICKNESS OF NONPRESTRESSED BEAMS OR ONE-WAY SLABS UNLESS DEFLECTIONS ARE COMPUTED

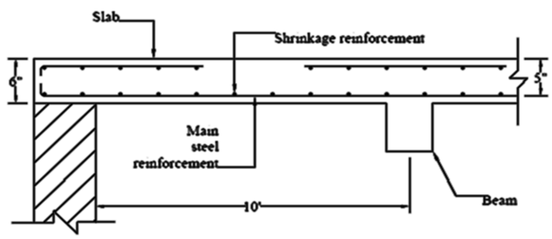
	Minimum thickness, *h*			
	Simply supported	One end continuous	Both ends continuous	Cantilever
Member	Members not supporting or attached to partitions or other construction likely to be damaged by large deflections.			
Solid one-way slabs	$\ell/20$	$\ell/24$	$\ell/28$	$\ell/10$
Beams or ribbed one-way slabs	$\ell/16$	$\ell/18.5$	$\ell/21$	$\ell/8$



$l = l_n = 9.5'$ $l = l_n = 9'$

Design Problem

- **Slab Design**
  - Step No. 01: Sizes
    - h = l/24 × (0.4+f_y/100000) = 3.8″ (Minimum by ACI for end span) [l = 9.5′]
    - h = l/28 × (0.4+f_y/100000) = 3″ (Minimum by ACI for interior span) [l = 9′]
    - End span governs. Finally take assumed h = 6″.
    - Effective depth (d) = h_f – 0.75 – (3/8)/2 = 5″ (for #3 main bar)



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Design Problem

- **Slab Design**
  - Step No. 02: Loads

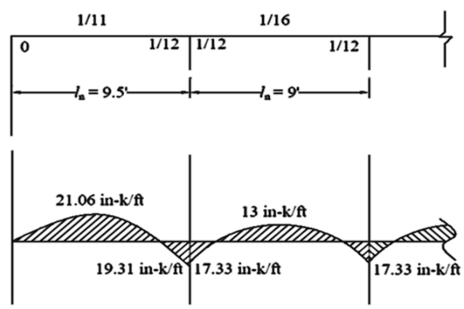
Table: Dead Loads.			
Material	Thickness (in)	γ (kcf)	Load = thickness × γ (ksf)
Slab	6	0.15	(6/12) × 0.15 = 0.075
Mud	3	0.12	(3/12) × 0.12 = 0.03
Tile	2	0.12	(2/12) × 0.12= 0.02
		Total	0.125 ksf

Factored Load (w_u) = 1.2D.L + 1.6L = 1.2 × 0.125 + 1.6 × 0.04 = 0.214 ksf

Design Problem

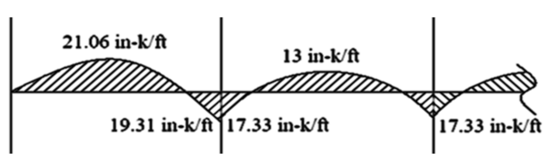
- **Slab Design**
  - Step No. 03: Analysis
    - Bending moment diagram for slab

M = coefficient × w_u × l_n^2



Design Problem

- **Slab Design**
  - Step No. 04: Design
    - Calculate moment capacity provided by minimum reinforcement in slab:
      - $A_{smin} = 0.002bh_f = 0.002 \times 12 \times 6 = 0.144$ in^2/ft
      - $\Phi M_n = \Phi A_{smin} f_y$ (d-a/2) = 0.9 × 0.144 × 40 × (5-0.188/2) = 25.4 in-k/ft
      - ΦM_n calculated from A_{smin} is > all moments calculated in Step No 3.
      - Therefore $A_s = A_{smin}$ = 0.144 in^2/ft (#3 @ 9.166″ c/c)
      - This will work for both positive and negative steel as A_{smin} governs.



Design Problem

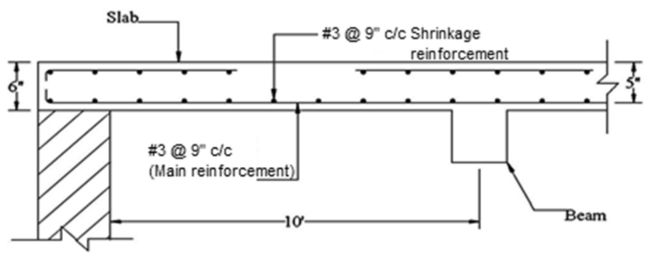
- **Slab Design**
  - Step No. 04: Design
    - Main reinforcement spacing:
      - Maximum spacing for main steel reinforcement in one way slab according to ACI 7.6.5 is minimum of:
        - $3h_f = 3 \times 6 = 18''$
        - 18″
      - Finally use, #3 @ 9″ c/c.

Design Problem

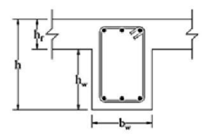
- **Slab Design**
  - Step No. 04: Design
    - Shrinkage steel or temperature steel (A_{st}):
      - $A_{st} = 0.002bh_f$ $A_{st} = 0.002 \times 12 \times 6 = 0.144$ in^2/ft
      - Shrinkage reinforcement is same as main reinforcement, because:
      - $A_{st} = A_{smin} = 0.144$ in^2
    - Maximum spacing for temperature steel reinforcement in one way slab according to ACI 7.12.2.2 is minimum of:
      - $5h_f = 5 \times 6 = 30''$ OR 18″
    - Therefore 9″ spacing is O.K.

Design Problem

- **Slab Design**
  - Step No. 05: Drafting
    - Main reinforcement = #3 @ 9″ c/c (positive & negative)
    - Shrinkage reinforcement = #3 @ 9″ c/c
    - Supporting bars = #3 @ 18″ c/c



Design Problem



- **Beam Design**
  - Step No. 01: Sizes
    - Minimum thickness of beam (simply supported) = h_{min} = l/18.5

      l = clear span (l_n) + depth of member (beam) ≤ c/c distance between supports
    - Let depth of beam = 2′

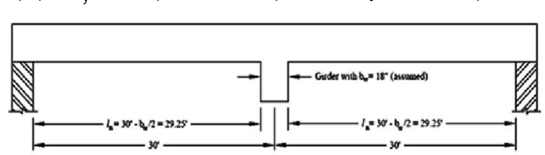
      l_n + depth of beam = 29.25′ + 2′ = 31.25′

      c/c distance between beam supports = 30 + (9/12) = 30.75′
    - Therefore l = 30.75′

      Depth (**h**) = (30.75/18.5)×(0.4+f_y/100000)×12= **15.95″** (Minimum by ACI 9.5.2.2).
    - Take h = 2′ = 24″

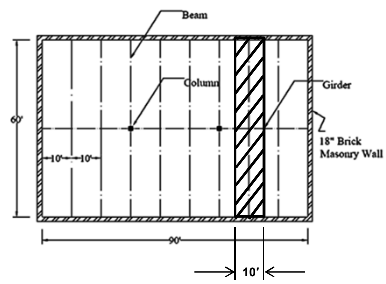
      d = h – 3 = 21″

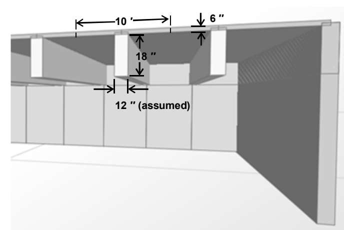
      b_w = 12″ (assumed)



Design Problem

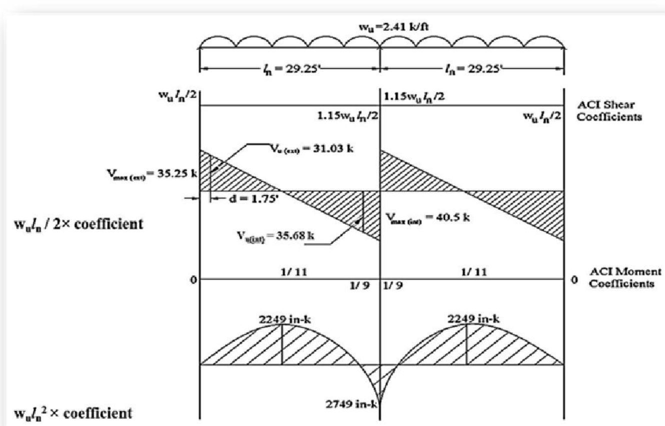
- **Beam Design**
  - Step No 02: Loads
    - Load on beam will be equal to
    - Factored load on beam from slab + factored self weight of beam web
    - Factored load on slab = 0. 214 ksf
    - Load on beam from slab = 0. 214 ksf x 10 = 2.14 k/ft
    - Factored Self load of beam web =
    - = 1.2 x (18 × 12/144) × 0.15 = 0.27 k/ft
    - Total load on beam = 2.14 + 0.27 = 2.41 k/ft





Design Problem

- **Beam Design**
  - Step No. 03: Analysis



Design Problem

- **Beam Design**
  - Step No. 04: Design
    - Design for flexure (for positive moment)
      - Step (a): According to ACI 8.12, b_{eff} for T-beam is minimum of:
        - $16h_f + b_w = 16 \times 6 + 12 = 108''$
        - (c/c span of beam)/4 =(30.75′/4) × 12 =92.25″
        - c/c spacing between beams =10′ × 12 =120″
      - So $b_{eff} = 92.25''$

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Design Problem

- **Beam Design**
  - Step No. 04: Design
    - Design for flexure (for positive moment)
      - Step (b): Check if beam is to be designed as rectangular beam or T-beam.
      - Assume $a = h_f = 6''$ and calculate A_s:

        $A_s = M_u / \{\Phi f_y (d–a/2)\} = 2249 / \{0.9 \times 40 \times (21–6/2)\} = 3.47$ in^2
      - Re-calculate "a":

        $a = A_s f_y / (0.85 f_c' b_{eff}) = 3.47 \times 40 / (0.85 \times 3 \times 92.25) = 0.6'' < h_f$

        Therefore design beam as rectangular beam.
      - After trials $A_s = 3.01$ in^2 {$A_{smax} = 5.11$ in^2 ;$A_{smin} = 1.26$ in^2}
      - Therefore $A_s = 3.01$ in^2 {4 #8 bars}

Design Problem

- **Beam Design**
  - Step No. 04: Design
    - Design for flexure (for interior negative moment)
      - b_w = 12″ instead of b_{eff} for calculation of "a" because of flange in tension.
      - M_u = 2749 in-kip; h = 24″; d = 21″
      - Let a = 0.2d = 0.2 × 21″ = 4.2″

        $A_s = M_u / \{\Phi f_y (d–a/2)\}$ =2749/ {0.9 × 40 × (21–4.2/2)} = 3.65 in^2
      - Re-calculate "a":

        $a = A_s f_y / (0.85 f_c' b_{eff})$ =3.65× 40/ (0.85 × 3 × 12) = 4.77″
      - After trials A_s = 4.17 in^2 {A_{smax} = 5.11 in^2 ;A_{smin} = 1.26 in^2}
      - Therefore A_s = 4.17 in^2 {6 #8 bars}

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Design Problem

- **Beam Design**
  - Step No. 04: Design
    - Design for shear

Location	V_u (kip)	$\Phi V_c = \Phi 2\sqrt{f'_c}\, b_w d$ (kips)	Reinforcement required?	$s_d = \Phi A_v f_y d/(V_u - \Phi V_c)$	$s_{max, ACI}$	Governing s
Exterior	31.03	20.7	Yes	13″	10.5″	10.5″
Interior	35.68	20.7	Yes	9″	10.5″	9″

- $S_{max, ACI}$ is least of $A_v f_y/(50 b_w)$, d/2 , 24″, $A_v f_y / 0.75\sqrt{f'_c}\, b_w$

Design Problem

- **Beam Design**
  - Step No. 04: Design
    - Design for shear
      - Other checks
        - Check for depth of beam:
          - $\Phi V_s \le \Phi 8 \sqrt{f'_c}\, b_w d$ (ACI 11.4.7.9)
          - $\Phi 8 \sqrt{f'_c}\, b_w d = 0.75 \times 8 \times \sqrt{3000} \times 12 \times 21/1000 = 82.4$ kips
          - $\Phi V_s = (\Phi A_v f_y d)/s_d$

            $= (0.75 \times 0.22 \times 40 \times 21)/9 = 15.4$ kip < 82.4 kip, O.K.
          - So depth is O.K. If not, increase depth of beam.

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Design Problem

- **Beam Design**
  - Step No. 04: Design
    - Design for shear
      - Other checks
        - Check if “$\Phi V_s \le \Phi 4 \sqrt{f'_c}\, b_w d$” {ACI 11.4.5.3}:
          - If “$\Phi V_s \le \Phi 4 \sqrt{f'_c}\, b_w d$”, the maximum spacing ($s_{max}$) is O.K. Otherwise reduce spacing by one half.
          - $\Phi 4 \sqrt{f'_c}\, b_w d = 0.75 \times 4 \times \sqrt{3000} \times 12 \times 21/1000 = 41.4$ kips
          - $\Phi V_s = (\Phi A_v f_y d)/s_d$

            $= (0.75 \times 0.22 \times 40 \times 21)/9 = 15.4$ kip < 41.4 kip, O.K.

Design Problem

- **Beam Design**
  - Step No. 04: Design
    - Design for shear
      - Reinforcement provision
        - It will be practically more feasible to provide # 3, 2 legged @ 9″ c/c throughout, starting at $s_d/2 = 9/2 = 4.5″$ from the face of the support at both ends.

Design Problem

- **Girder Design**
  - Step No. 01: Sizes

TABLE 9.5(a)—MINIMUM THICKNESS OF NONPRESTRESSED BEAMS OR ONE-WAY SLABS UNLESS DEFLECTIONS ARE COMPUTED

	Minimum thickness, *h*			
	Simply supported	One end continuous	Both ends continuous	Cantilever
Member	Members not supporting or attached to partitions or other construction likely to be damaged by large deflections.			
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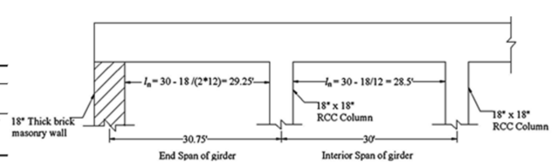
Beams or ribbed one-way slabs	ℓ/16	ℓ/18.5	ℓ/21	ℓ/8



End span:

$l = l_n + h \le l_{c/c}$
Assume h = 3′ = 36″
$l = 29.25 + 3 = 32.25' > l_{c/c}$
Therefore, $\boldsymbol{l = l_{c/c} = 30.75'}$

Interior span:

$l = l_n + h \le l_{c/c}$
Assume h = 3′ = 36″
$l = 28.5 + 3 = 31.5' > l_{c/c}$
Therefore, $\boldsymbol{l = l_{c/c} = 30'}$

Design Problem

- **Girder Design**
  - Step No. 01: Sizes
    - Minimum thickness of beam (simply supported) = h_{min} = l/18.5

      l = 30.75′

      Depth (**h**) = (30.75/18.5)×(0.4+f_y/100000)×12= **15.95″** (Minimum by ACI 9.5.2.2).
    - Take h = 3′ = 36″

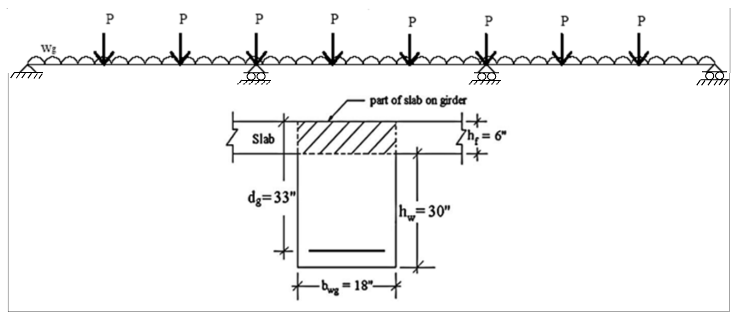
      d = h – 3 = 33″

      b_w = 18″ (assumed)

Design Problem

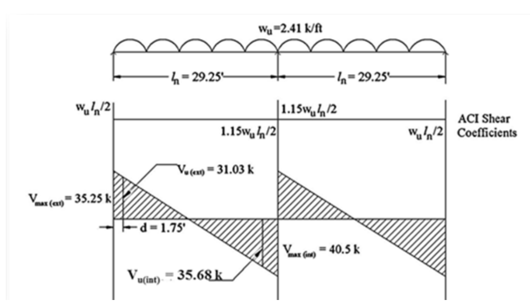
Design Problem

- **Girder Design**
  - Step No. 02: Loads
    - Beams load can be approximated as point loads on girder. The uniformly distributed load on girder is coming from self weight of girder rib plus weight of slab directly resting on girder.



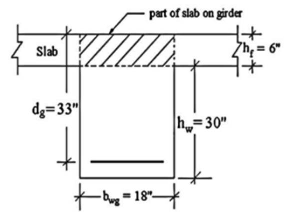
Design Problem

- **Girder Design**
  - Step No. 02: Loads
    - P is the point load on girder and is the reaction coming from the interior support of beam due to factored load.
    - P = 2 × 40.5 = 81 kips

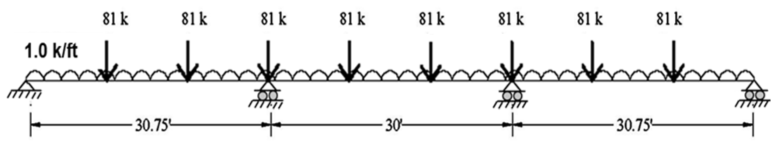


Design Problem

- **Girder Design**
  - Step No. 02: Loads



    - $(U.D.L)_{self\ wt}$ = Factored self weight of girder rib
      $=1.2h_{wg}b_{wg}\gamma_c$
      $=1.2 \times (30 \times 18 \times 0.15)/144 = 0.675$ k/ft
    - Part of slab on girder $\{(U.D.L)_{sg}\}$:
    - $(U.D.L)_{sg} = w_{u\ (on\ slab)} \times b_{wg} = 0.214 \times 18/12 = 0.321$ k/ft
    - Therefore $w_g = (U.D.L)_{self\ wt} + (U.D.L)_{sg} = 0.675 + 0.321 = 1.0$ k/ft

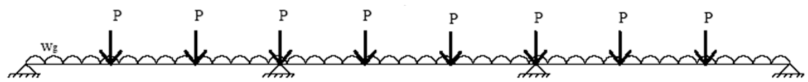


Design Problem

- **Applicability of ACI Approximate Analysis**
  - According to ACI 8.3.3, ACI approximate analysis (coefficient method) is subjected to following limitations
    - The structure has two or more spans
    - The spans are approximately equal, with the larger of the two adjacent spans not greater than the shorter by more than 20 percent
    - Loads are uniformly distributed, and the unfactored live load does not exceed 3 times the unfactored dead load
    - Members are prismatic

Design Problem

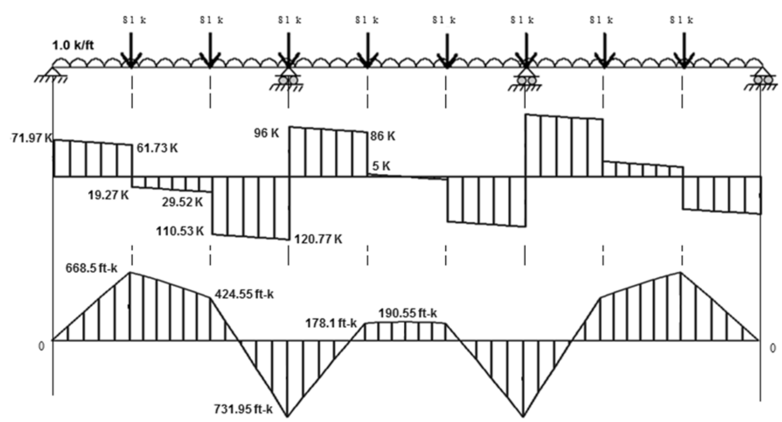
- **Applicability of ACI Approximate Analysis**
  - The ACI coefficient method cannot be applied to the girder as it is subjected to uniform loading as well as pointed loads.



  - Any method of elastic analysis e.g., slope deflection method, moment distribution method, flexibility method, stiffness method etc. may be used for the analysis of such cases.

Design Problem

- **Girder Design**
  - Step No. 02: Analysis



Design Problem

- **Girder Design**
  - Step No. 04: Design
    - Design for Flexure
      - According to ACI 8.12, b_{eff} is minimum of:
      - $16h_f + b_{wg} = 16 \times 6 + 18 = 114''$
      - (c/c span of girder)/4 = (30.75/4) × 12 = 92.25″
      - c/c spacing between girder = not applicable
      - $b_{eff} = 92.25''$

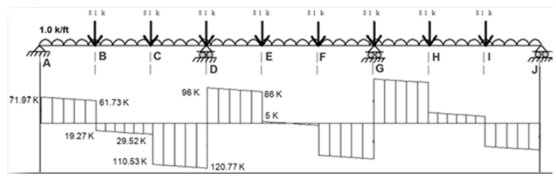
Section	M_u (kip-ft)	d (in.)	b (in.)	A_s (in^2)	A_{smin} (in^2)	A_{smax} (in^2)	Bar used	# of bars
Exterior +	668.5	33	92.25	6.87	2.97	11.88	#8	9 (5 +4)
Interior −	731.95	33	18	8.30	2.97	11.88	#8	12 (6 +6)
Interior +	190.55	33	92.25	1.93	2.97	11.88	#8	4

Design Problem

- **Girder Design**
  - Step No. 04: Design
    - Design for Shear: Shear design of girder is done by another approach
      - $d_g = 33'' = 2.75'$
      - $\Phi V_c = \Phi 2 \sqrt{f'_c}\, b_{wg} d_g = \{0.75 \times 2 \times \sqrt{3000} \times 18 \times 33\}/1000 = 48.8$ kip
      - Maximum spacing and minimum reinforcement requirement as permitted by ACI 11.4.5 and 11.4.6 shall be minimum of:
        - $A_v f_y/(50b_w) = 0.22 \times 40000/(50 \times 18) = 9.77'' \approx 9.5''$
        - $d_g/2 = 33/2 = 16.5''$
        - 24″
        - $A_v f_y/ 0.75 \sqrt{f'_c}\, b_w = 0.22 \times 40000/ \{(0.75 \times \sqrt{3000} \times 18\} = 11.90''$

Design Problem

- **Girder Design**
  - Step No. 04: Design
    - Design for Shear:

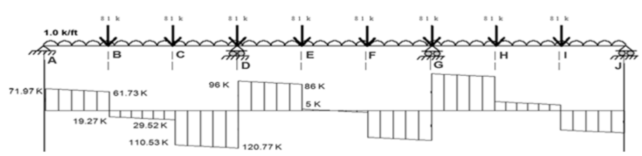


Shear design of girder is done by another approach

- Since maximum spacing allowed by ACI is 9.5″,
- $\Phi V_n = \Phi V_c + \Phi V_s$
- $\Phi V_s = (\Phi A_v f_y d_g)/s_{max}$ = (0.75 × 0.22 × 40 × 33/9.5) = 22.92 kip
- ΦV_n = 48.808 + 22.92 = 71.72 k > Max. Shear at **A** and **B** but < Max. Shear at **C**, **D** and **E**
- It means that maximum spacing of 9.5'' as permitted by ACI governs between point **A** and **C**.
- We will calculate spacing for shear reinforcement between point **C** and **E**.

Design Problem

- **Girder Design**
  - Step No. 04: Design
    - Design for Shear:



Location	V_{max} (kip)	V_u (@ d) (kip)	$\Phi V_c = \Phi 2\sqrt{f'_c} b_w d$ (kips)	$s_d = \Phi A_v f_y d/(V_u - \Phi V_c)$	$s_{max, ACI}$	S taken
CD	120.5	117.5	48.8	3″	9.5″	3″
DE	96	94	48.8	4.5″	9.5″	3″

- Final Spacing: Providing 3″ from **C** to **E** and **F** to **H**, and 9″ elsewhere.

Design Problem

- **Girder Design**
  - Step No. 04: Design
    - Design for Shear:
      - Other checks
        - Check for depth of girder:
          - **$\Phi V_s \leq \Phi 8 \sqrt{f'_c} b_w d$** (ACI 11.4.7.9)
          - $\Phi 8 \sqrt{f'_c} b_w d = 0.75 \times 8 \times \sqrt{3000} \times 18 \times 33/1000 = 195.20$ kips
          - $\Phi V_s = (\Phi A_v f_y d)/s_d$

            $= (0.75 \times 0.22 \times 40 \times 33)/3 = 72.60$ kip < 195.20 kip, O.K.
          - So depth is O.K. If not, increase depth of beam.

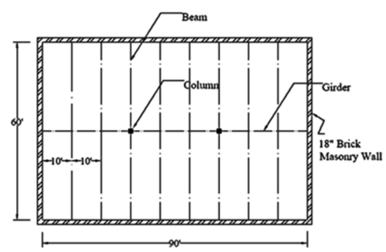
Design Problem

- **Girder Design**
  - Step No. 04: Design
    - Design for Shear:
      - Other checks
        - Check if "**$\Phi V_s \leq \Phi 4 \sqrt{f'_c} b_w d$**" {ACI 11.4.5.3}:
          - 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.
          - $\Phi 4 \sqrt{f'_c} b_w d = 0.75 \times 4 \times \sqrt{3000} \times 18 \times 33/1000 = 97.6$ kips
          - $\Phi V_s = (\Phi A_v f_y d)/s_d$

            $= (0.75 \times 0.22 \times 40 \times 33)/3 = 72.6$ kip < 97.6 kip, O.K.

Design Problem

- **Column Design**
  - Gross area of column cross-section (A_g) = 18 × 18 =324 in^2
  - f_c' =3 ksi; f_y =40 ksi
  - Load on column:
    - P_u = 297.8 k (Reaction at interior support of girder due to factored load)



Design Problem

- **Column Design**
  - Main Reinforcement Design:
    - Nominal strength (ΦP_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)
    - ΦP_n = 0.80 × 0.65 × {0.85 × 3 × (324 – 0.01 × 324) + 0.01 × 324 × 40}

      = 492k > (P_u = 297.775 k), O.K.
    - A_{st} = 0.01 × 324 =3.24 in^2
    - Using 3/4″ Φ(#6) , with bar area A_b =0.44 in^2
    - No. of bars = A_s/A_b = 3.24/0.44 = 7.36 ≈ 8 bars
    - Use 8 #6 bars

Design Problem

- **Column Design**
  - Tie Bars:
    - Using 3/8″ Φ (#3) tie bars for 3/4″ Φ (#6) main bars (ACI 7.10.5),
    - Spacing for Tie bars according to ACI 7.10.5.2 is minimum of:
    - 16 × dia of main bar =16 × 3/4 =12″ c/c
    - 48 × dia of tie bar = 48 × (3/8) =18″ c/c
    - Least column dimension =18″ c/c
    - Finally use #3, tie bars @ 9″ c/c

Design Problem

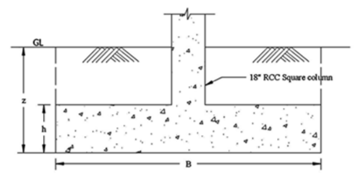
- **Footing Design**
  - Data Given:
    - Column size = 18″ × 18″
    - f_c' =3 ksi
    - f_y = 40 ksi
    - q_a = 2.204 k/ft^2
    - Factored load on column = 297.775 kips (Reaction at the support)
    - Service load on column = 234 kips (Reaction at the support due to service load)

Design Problem

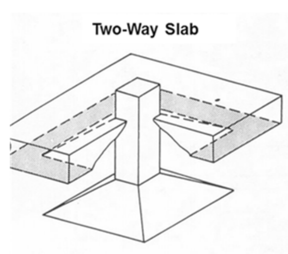
- **Footing Design**
  - Sizes:
    - Assume h = 18 in.
    - d_{avg} = h – clear cover – one bar dia

      = 18 – 3 – 1(for #8 bar) = 14 in.
    - Assume depth of the base of footing from ground level (z) = 5′
    - Weight of fill and concrete footing, W= $\gamma_{fill}(z - h) + \gamma_c h$

      =100 × (5 – 1.5) +150 × (1.5) = 575 psf = 0.575 ksf



Design Problem



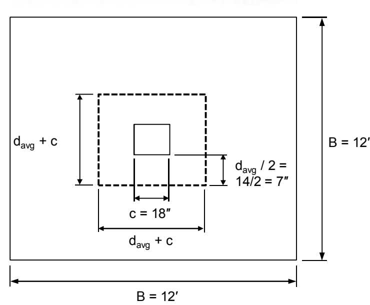
- **Footing Design**
  - Sizes:
    - Effective bearing capacity, $q_e = q_a - W$

      = 2.204 – 0.575 = 1.629 ksf
    - Bearing area, A_{req} = Service Load/ q_e

      = 234/1.629 = 143.95 ft^2

      A_{req} = B x B = 143.65 ft^2 => B = 12 ft.
    - Critical Perimeter, $b_o = 4 \times (c + d_{avg})$

      = 4 × (18 + 14) =128 in



Design Problem

- **Footing Design**
  - Loads:
    - q_u (bearing pressure for strength design of footing):
      - q_u = factored load on column / A_{req} = 297.775/ (12 × 12) = 2.068 ksf

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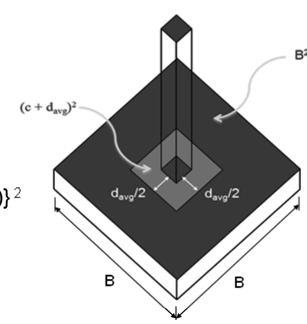
Design Problem

- **Footing Design**
  - Analysis:
    - Punching shear:
      - $V_{up} = q_u B^2 - q_u(c + d_{avg})^2$
      - $V_{up} = 2.068 \times 12^2 - 2.068 \times \{(18+14)/12\}^2$

        = 283.09 kip



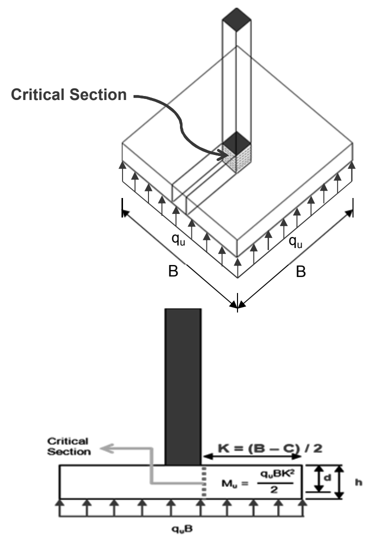
Design Problem

- **Footing Design**
  - Analysis:
    - Flexural Analysis:
    - $M_u = q_uBk^2/2$
    - $k = (B – c)/2 = (12 \times 12 – 18)/2$

      $= 63 \text{ in} = 5.25'$
    - $M_u = 2.068 \times 12 \times 5.25 \times 5.25/2$

      $= 342$ ft-k

      $= 4104$ in-kip



Critical Section for Flexure

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Design Problem

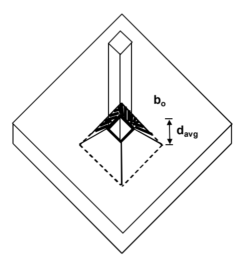
- **Footing Design**
  - Design:
    - Design for Punching Shear:
      - $V_{up} = 283.09$ kip

        Punching shear capacity (ΦV_{cp})

        $= \Phi 4 \sqrt{f'_c}\ b_od_{avg}$

        $= 0.75 \times 4 \times \sqrt{3000} \times 128 \times 14/1000$

        $= 294.45 \text{ k} > V_{up}$, O.K



Design Problem

- **Footing Design**
  - Design:
    - Design for Flexure:
      - M_u = 4104 in-kip

        a = $0.2d_{avg}$ = 0.2 × 14 = 2.8″

        $A_s = M_u / \{\Phi f_y(d_{avg} - a/2)\}$ = 4104/ {0.9 × 40 × (14 – 2.8/2)} = 9.05 in^2

        a = $A_s f_y / (0.85 f_c' B)$ = 9.05 × 40/ (0.85 × 3 × 12 × 12) = 0.99″

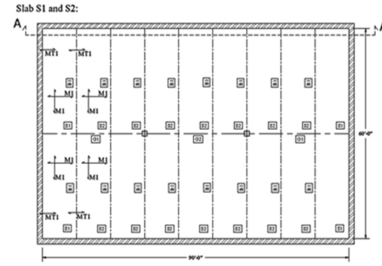
        After trials, A_s = 8.42 in^2 ($A_{smin} = 0.005Bd_{avg}$ = 10.08 in^2 so A_{smin} governs)

        Spacing = B × A_b/A_{smin} = 12 × 12 × 0.79/10.08 = 11.28 in c/c ≈ 11 in c/c
      - Use #8 @ 11″ c/c (Max. Spacing must not exceed 3h or 18″)

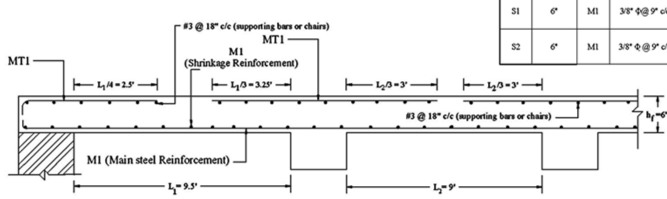
Design Problem

- **Drafting Details for Slab**

Slab S1 and S2:



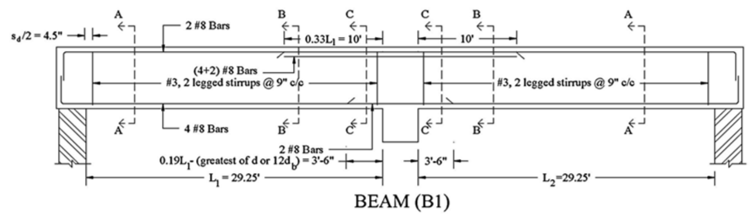
Panel	Depth (in)	Mark	Bottom Reinforcement	Mark	Top reinforcement	
S1	6"	M1	3/8" Φ@ 9" c/c	MT1	3/8" Φ @ 9" c/c	Non continuous End
S2	6"	M1	3/8" Φ @ 9" c/c	MT1	3/8" Φ @ 9" c/c	Continuous End



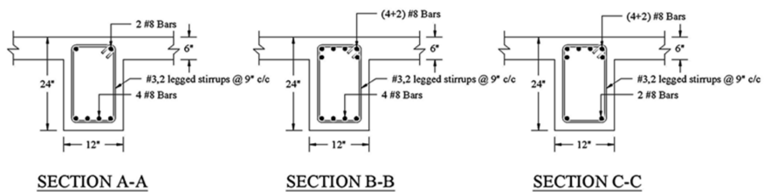
Section A-A. Refer to figure 5.15, chapter 5, Nelson 13th Ed for bar cutoff.

Design Problem

- **Drafting Details for Beam**



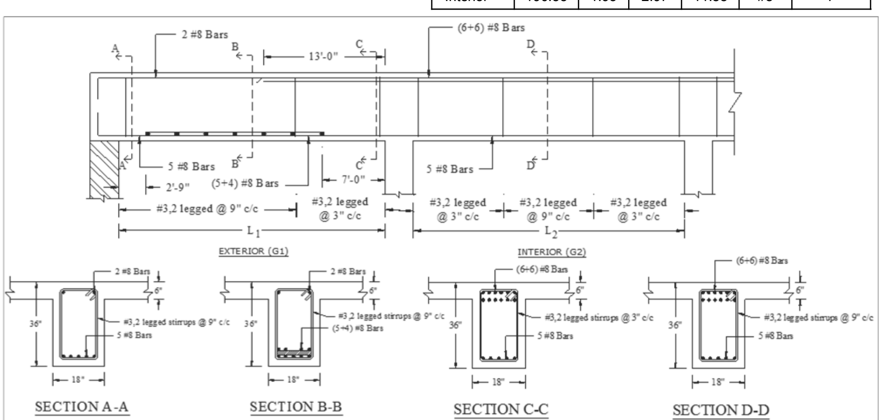
Notes: -
(1) Use graph A.3, Nelson 13th Ed for location of cut off for continuous beams.
(2) Use table A.7, Nelson 13th Ed for maximum number of bars as a single layer in beam stem.



Design Problem

- **Drafting Details for Girder**

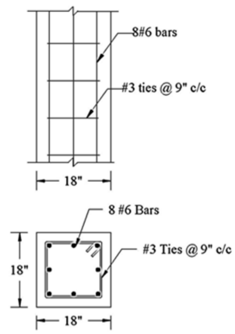
Section	M_u (kip-ft)	A_s (in^2)	A_{smin} (in^2)	A_{smax} (in^2)	Bar used	# of bars
Exterior +	668.5	6.87	2.97	11.88	#8	9 (5 +4)
Interior −	731.95	8.30	2.97	11.88	#8	12 (6 +6)
Interior +	190.55	1.93	2.97	11.88	#8	4*

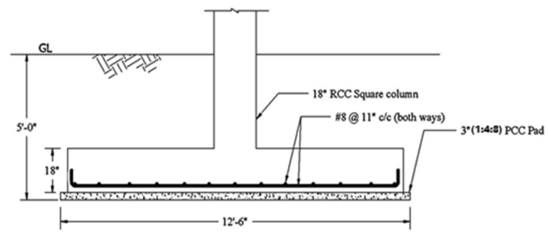


Note: at Sec C-C 4 bars are required from calculation but for practical feasibility we shall provide 5 bars as shown in the figure

Design Problem

- **Drafting Details for Column and Footing**





FOOTING DETAIL

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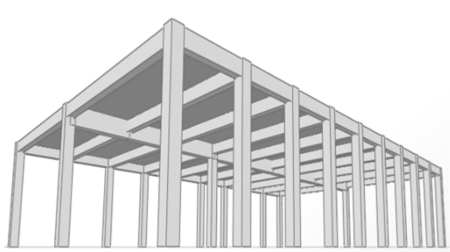
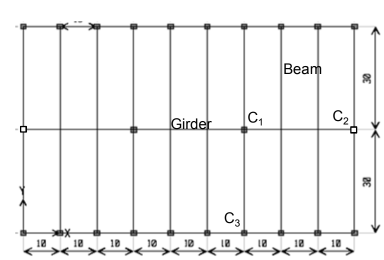
Design Problem

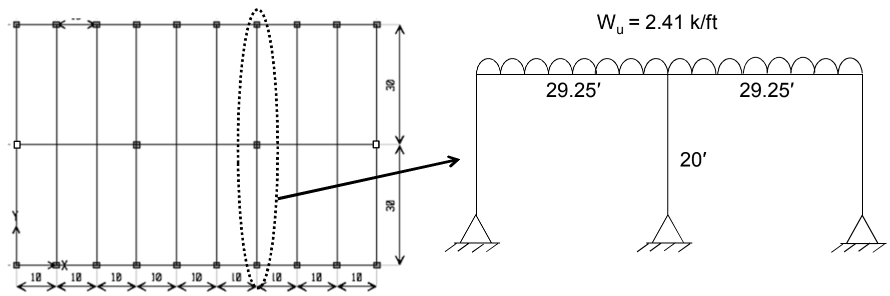
- In the subsequent slides the same hall has been analyzed and designed for beams and girders supported on columns instead of walls.
- Structural Plan:





Design Problem

- **Frame Analysis for Beam**
  - 2D frame can be detached from a 3D system as follows:



Column Size = 18″ × 18″

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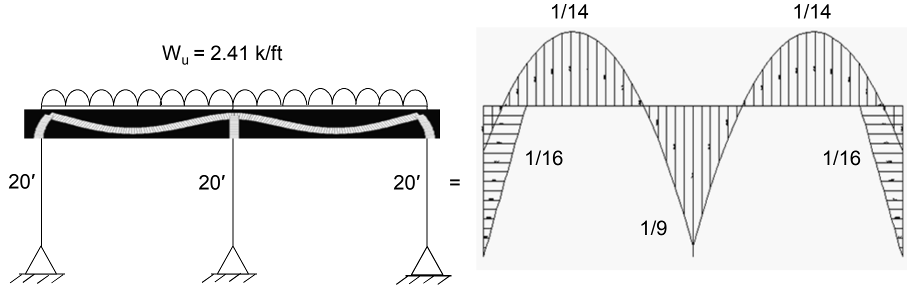
Design Problem

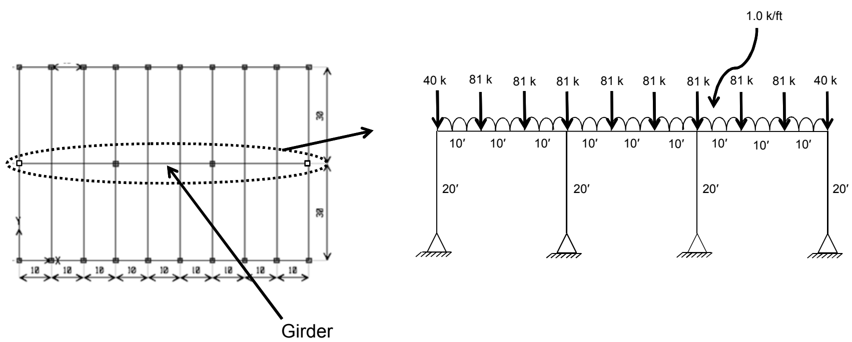
- **Frame Analysis for Beam**
  - Using ACI moment coefficients for analyzing the frame:



- Note: Interior support conditions for the beam (if supported on column, or on roller) does not effect analysis results.

Design Problem

- **Frame Analysis for Girder**
  - 2D frame can be detached from a 3D system in the following manner:



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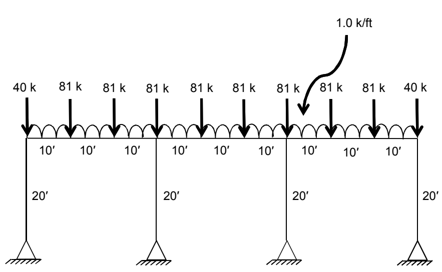
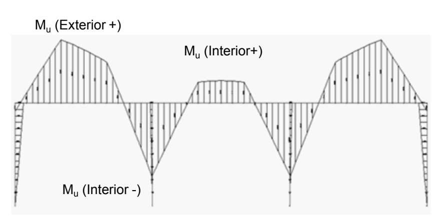
Design Problem

- **Frame Analysis for Girder**
  - Analysis of Girder



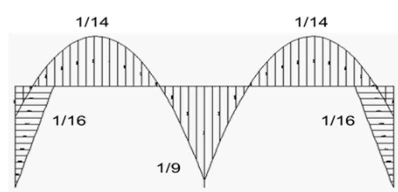


Design Problem

- **Slab Design**
  - Design of slab for beams supported on column will be same as that of beams supported on walls.
    - Main reinforcement = #3 @ 9″ c/c (positive & negative)
    - Shrinkage reinforcement = #3 @ 9″ c/c
    - Supporting bars = #3 @ 18″ c/c

Design Problem

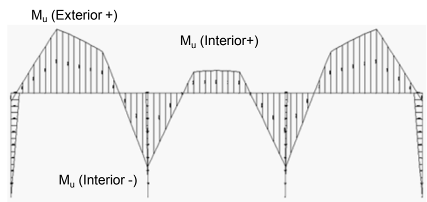
- **Beam Design**
  - M_u (+ve) = 1767 in-kips
  - $M_{u,ext}$ (-ve) = 1546 in-kips
  - $M_{u,int}$ (-ve) = 2749 in-kips
  - A_s (+ve) = 2.36 in^2 (3 #8 bars)
  - $A_{s,ext}$ (-ve) = 2.19 in^2 (3 #8 bars)
  - $A_{s,int}$ (-ve) = 4.18 in^2 (6 #8 bars)



- The shear reinforcement is not affected and hence it will be the same as in the previous case (beams supported on walls)

Design Problem

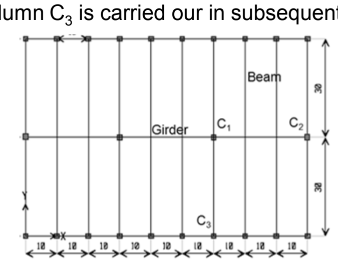
- **Girder Design**



Section	M_u (kip-ft)	d (in.)	b (in.)	A_s (in^2)	A_{smin} (in^2)	A_{smax} (in^2)	Bar used	# of bars
Exterior +	614.64	33	92.25	6.31	2.97	11.88	#8	8
Interior −	726.03	33	18	8.23	2.97	11.88	#7 + #8	6 + 6
Interior +	217.96	33	92.25	2.21	2.97	11.88	#8	4

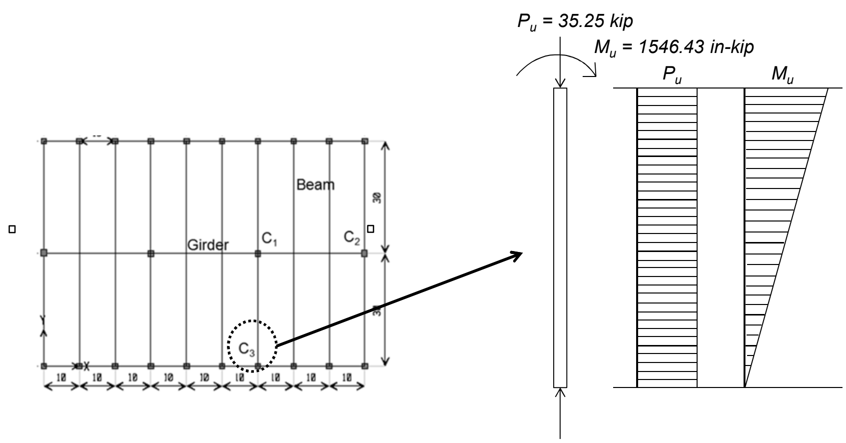
Design Problem

- **Column Design**
  - Design of C_1 (already done on slides 39 to 41)
    - 8 #6 bars, #3 ties @ 9″ c/c
  - Design of C_2 (do it yourself)
  - Design of C_3
    - Design of column C_3 is carried our in subsequent slides.



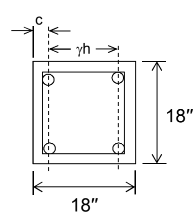
Design Problem

- **Column Design (C_3)**



Design Problem

- **Column Design (C_3): Using ACI Design Aids**
  - Main Reinforcement Design
    - Size:
      - 18 in. × 18 in.
    - Loads:
      - P_u = 35.25 kips
      - M_u = 1546.43 in-kips

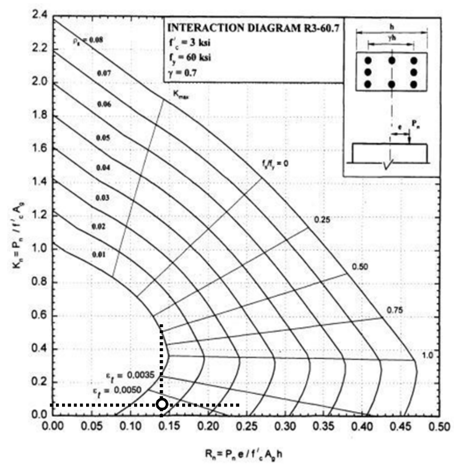


f'_c = 3 ksi, f_y = 60 ksi

- Calculate the ratio γ , for 2.5 in. cover: $\gamma = (18 - 2(2.5)) /18 = 0.72$
- Calculate K_n, $K_n = P_u/(\text{Ø}f'_cA_g) = 35.25/(0.65 \times 3 \times 324) = 0.06$
- Calculate R_n, $R_n = M_u/(\text{Ø}f'_cA_gh) = 1546.43/(0.65 \times 3 \times 324 \times 18) = 0.14$

Design Problem

- **Column Design (C_3)**
  - Main Reinforcement Design
    - For given material strength, the column strength interaction diagram gives the following reinforcement ratio:
      - ρ = 0.018
      - A_{st} = 0.018 × 324 = 5.83 in.^{2}
      - Using 8 #8 bars



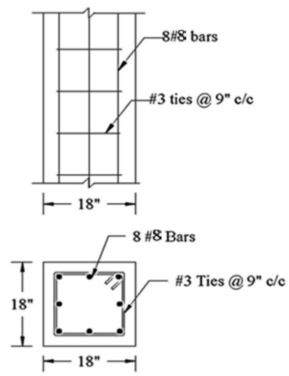
Strength Interaction Diagram (ACI Design Handbook)

Design Problem

- **Column Design (C_3)**
  - Tie Bars:
    - Using 3/8″ Φ (#3) tie bars for 3/4″ Φ (#6) main bars (ACI 7.10.5),
    - Spacing for Tie bars according to ACI 7.10.5.2 is minimum of:
    - 16 × dia of main bar =16 × 3/4 =12″ c/c
    - 48 × dia of tie bar = 48 × (3/8) =18″ c/c
    - Least column dimension =18″ c/c
    - Finally use #3, tie bars @ 9″ c/c

Design Problem

- **Column Design (C_3)**
  - Drafting



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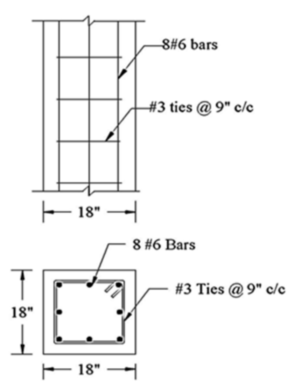
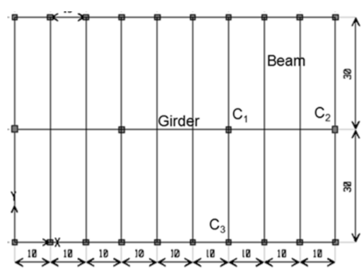
Design Problem

- **Column Design (C_1 and C_2)**
  - Drafting



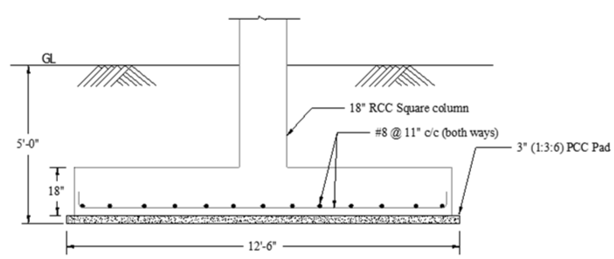
Design Problem

- **Footing Design**
  - Design of footing for the column C1 has already been done (refer to slides 42 to 49)
  - Do design of footing for column C_2 and C_3 yourself.
  - Details of reinforcement are presented in the subsequent slides.



Design Problem

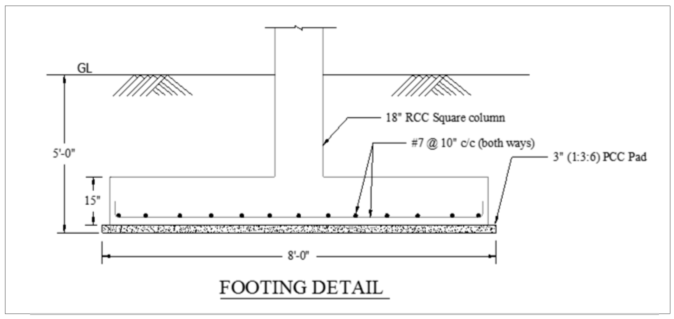
- **Footing Design for C_1**
  - Drafting



FOOTING DETAIL

Design Problem

- **Footing Design for C_2**
  - Drafting



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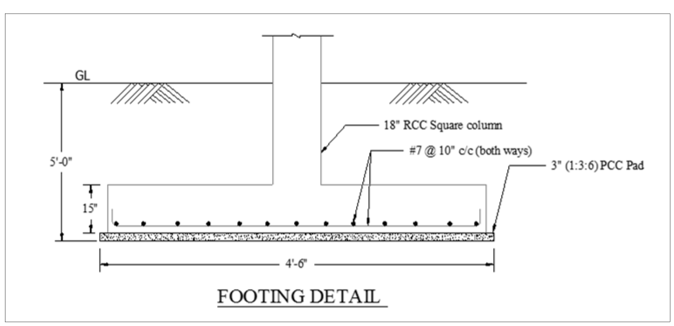
Design Problem

- **Footing Design for C_3**
  - Drafting



References

- ACI 318
- Design of Concrete Structures 13th Ed. by Nilson, Darwin and Dolan

The End