CE 331: Reinforced Concrete Design

Spring 2022

Dr. Bayrak

Final Report

May 1, 2022

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Executive Summary

This was a semester long project that went along with the coursework of CE 331 Reinforced Concrete Design. We were tasked with designing the beams and columns of a six-story building. We submitted interim reports and got feedback. This report includes the final designs along with the interim reports. Our final design was 16-inch square columns for the exterior columns and 23-inch square columns for the interior columns. The beams designed have an effective width of 97.5 inches and a depth of 30 inches. ACI 318-319 and Dr. Bayrak guided the design process and helped us reach a complete design.

Introduction

The goal of this project was to design the beams and columns of the first floor of a six-story building. The buildings frame can be seen in Figure 1, and it will be constructed using reinforced concrete. The uniform (unfactored) service dead load on the frame was 1.6 kips/ft (including the self-weight of the members) and the live load on the frame was 2.75 kips/ft. The concrete used in the design has a $f_c{}'$ of 5500 psi and a f_y of 60 ksi. To develop a final design for columns 1-4 and beams 1-3 we started with a preliminary design of the beams and columns. Then we designed the beams for flexure and shear. Then we determined the bar cut off in the beams. Lastly, we designed the columns and made final drawings.

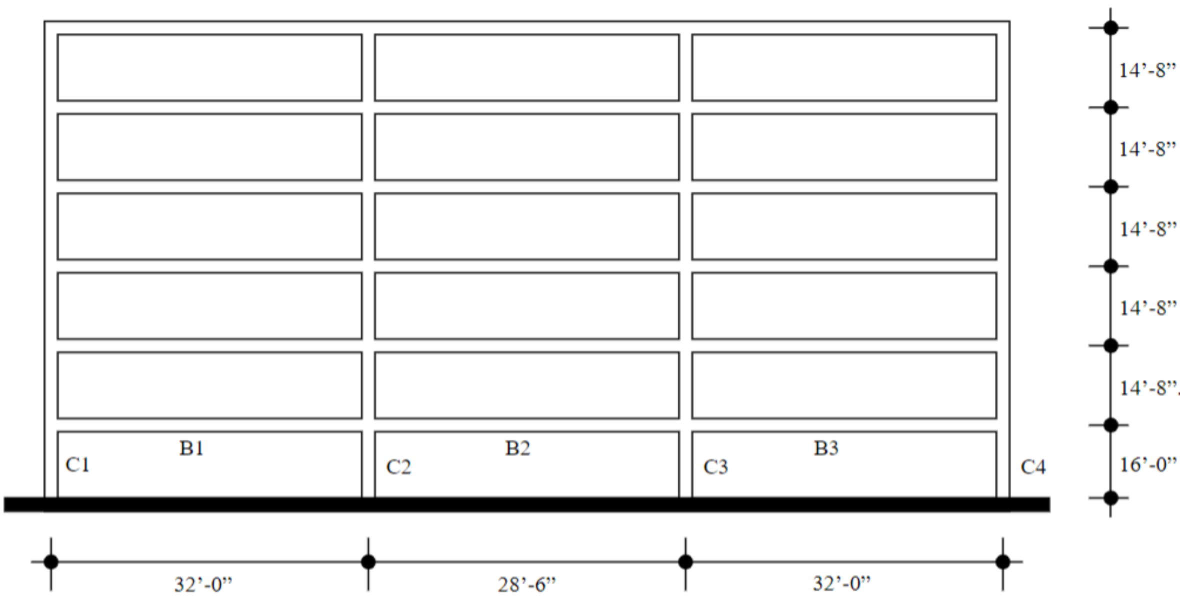


Figure 1: Buildings Frame

Design

Beam Cross Sections:

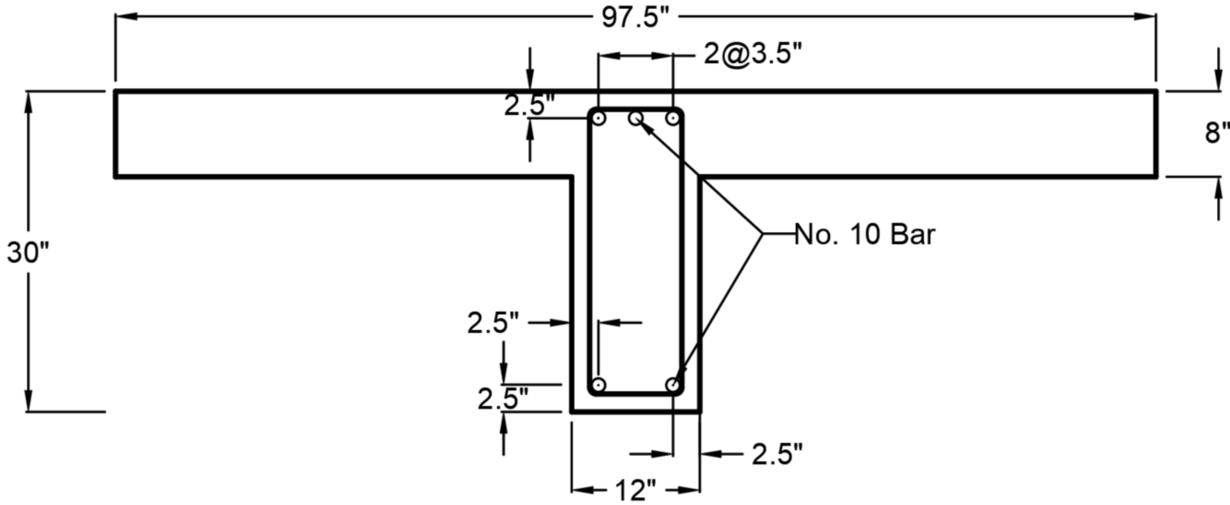


Figure 2: Beam Cross Section A

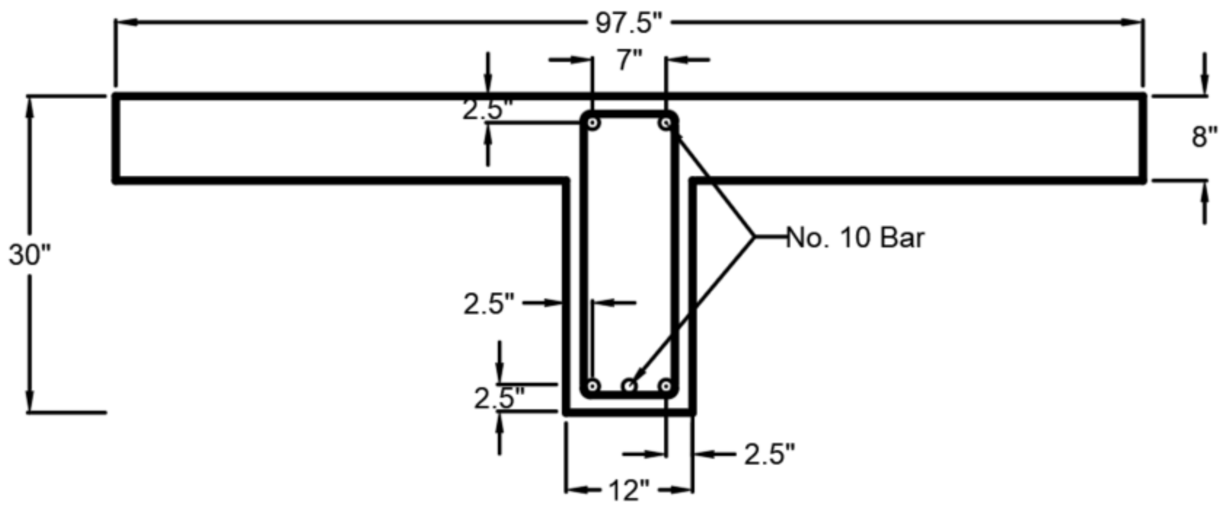


Figure 3: Beam Cross Section B

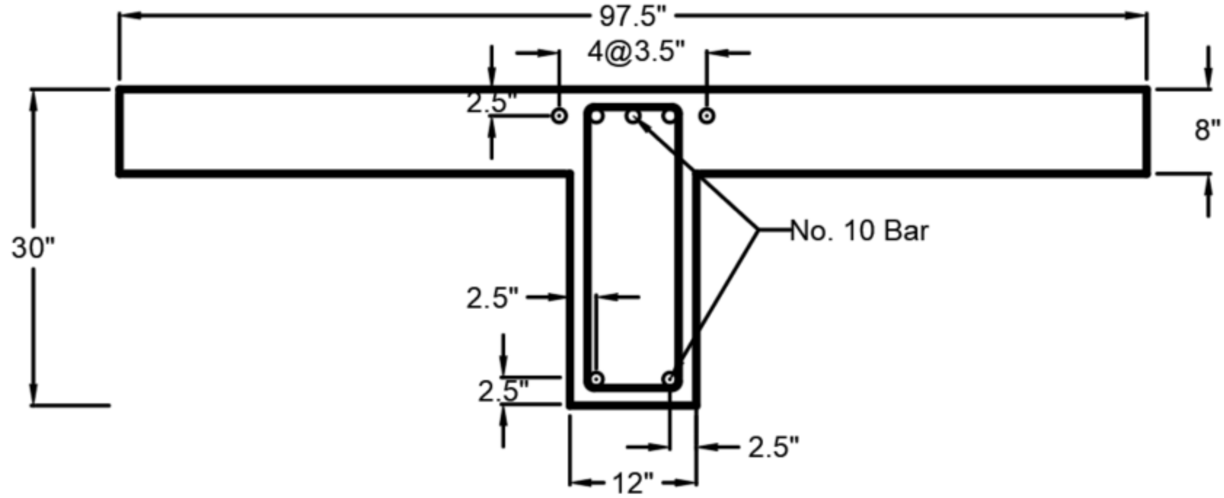


Figure 4: Beam Cross Section C

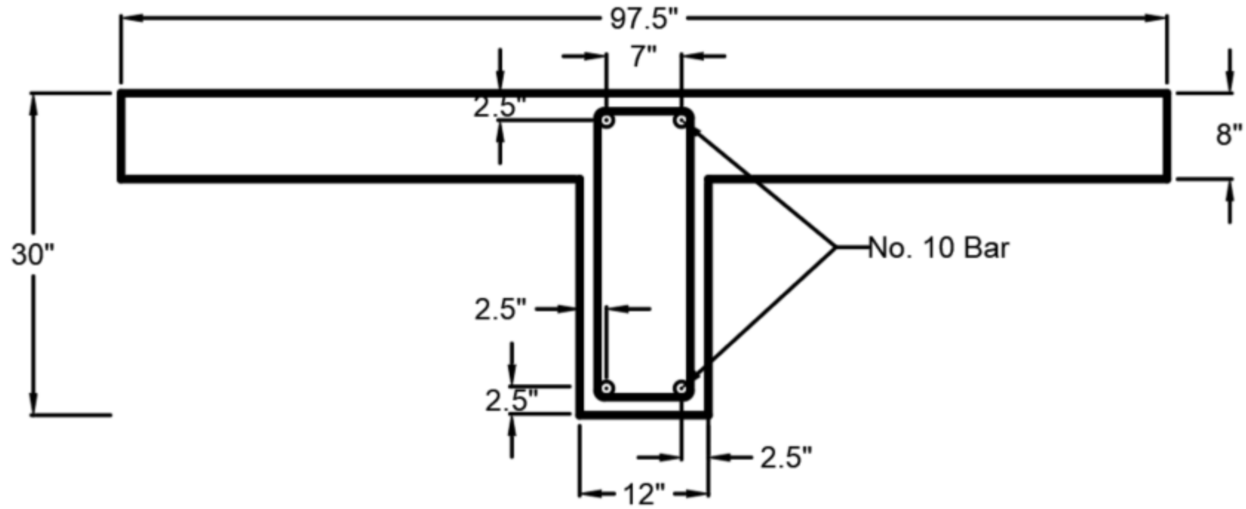


Figure 5: Beam Cross Section D

Beam 1:

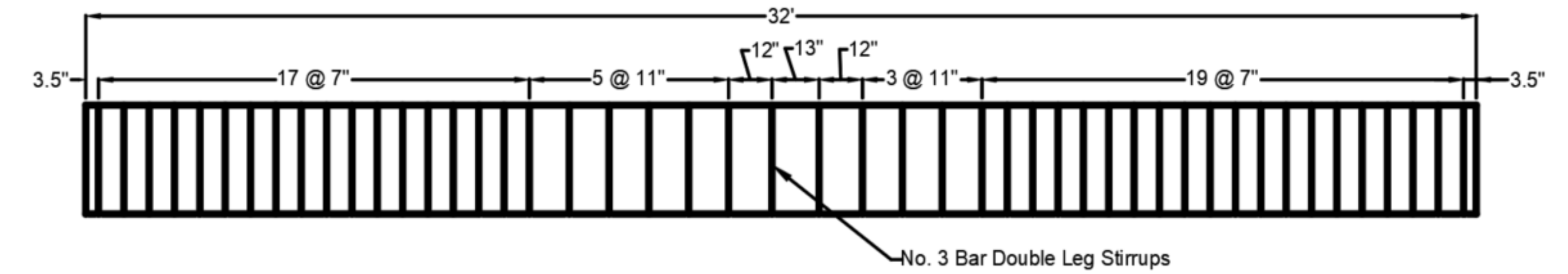


Figure 6: Beam 1 Stirrups Design

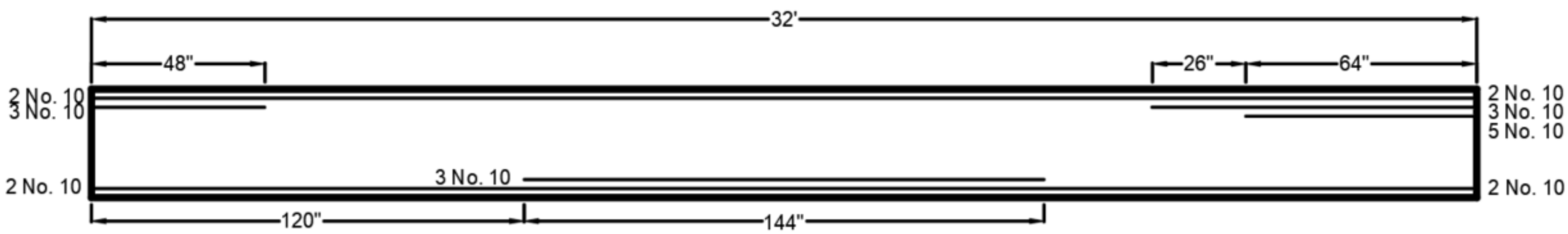


Figure 7: Beam 1 Bar Cut Offs

Beam 2:

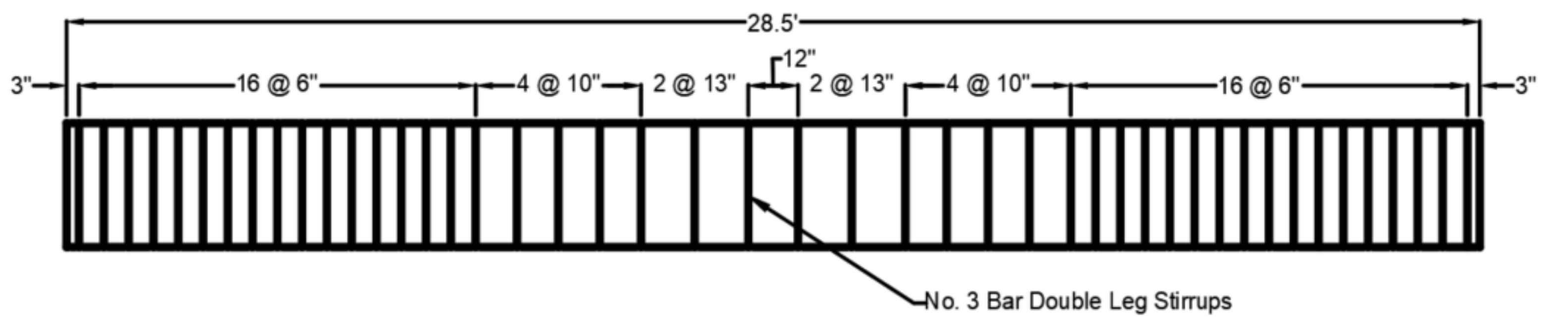


Figure 8: Beam 2 Stirrups Design

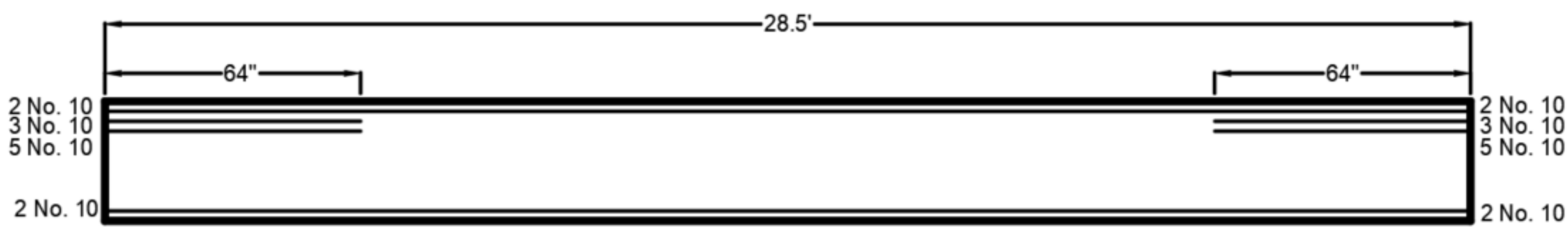


Figure 9: Beam 2 Bar Cut Offs

Beam 3:

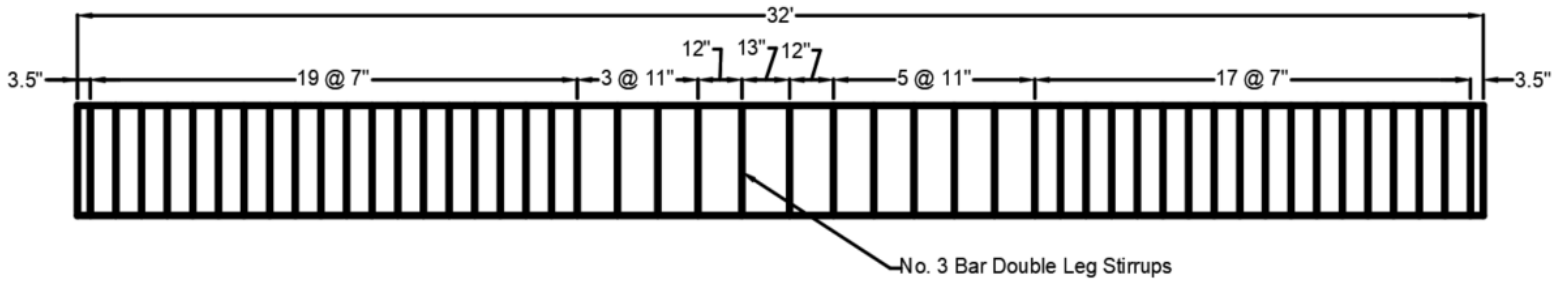


Figure 10: Beam 3 Stirrups Design

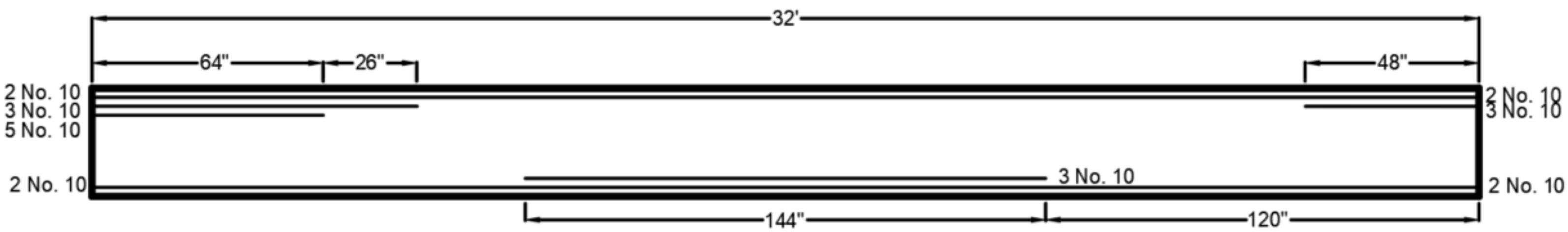


Figure 11: Beam 3 Bar Cut Offs

Column Designs

Column 1 and Column 4:

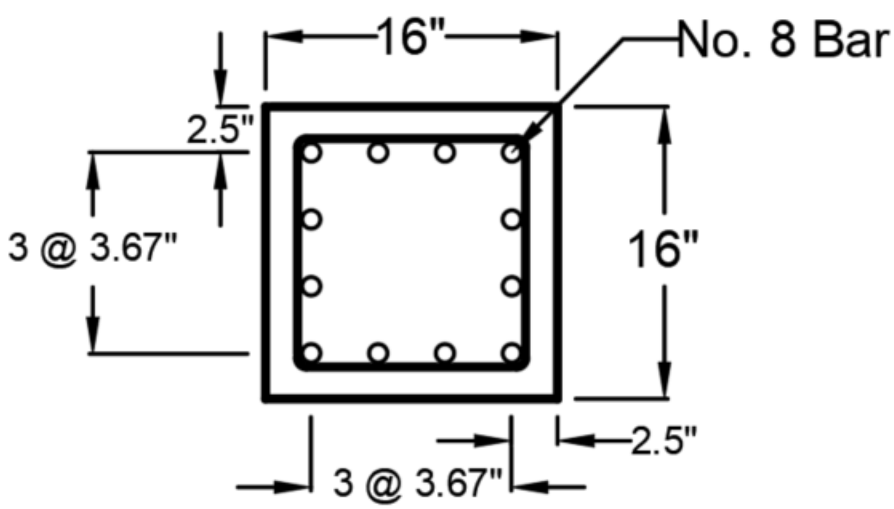


Figure 12: Column 1 and 4 Cross Sections

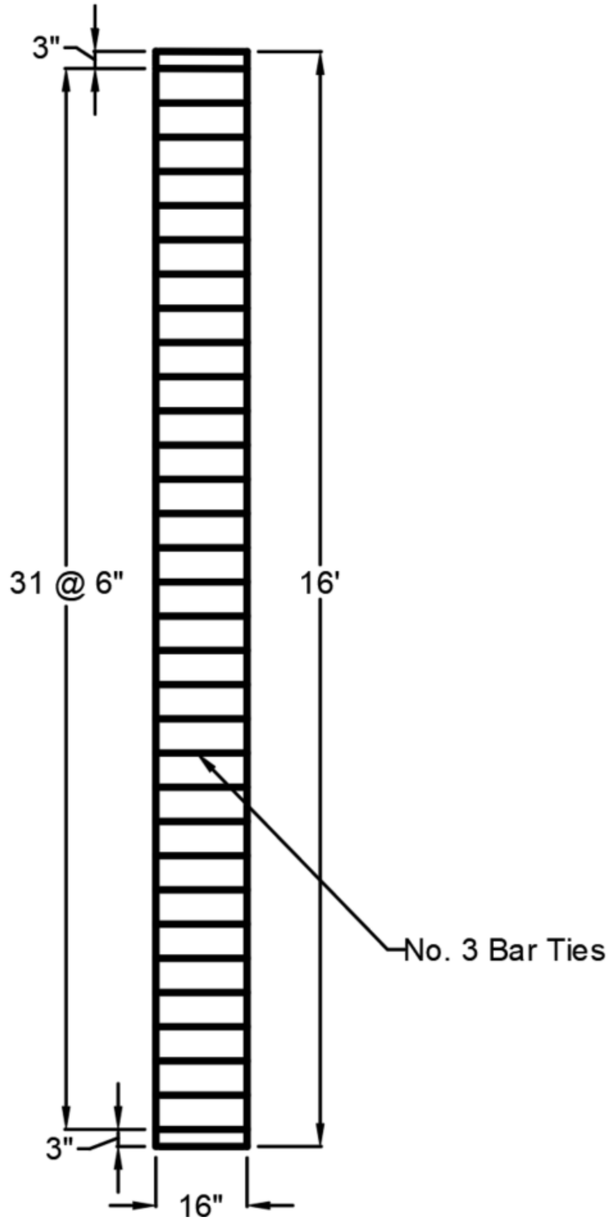


Figure 13: Column 1 and 4 Bar Ties

Column 2 and 3:

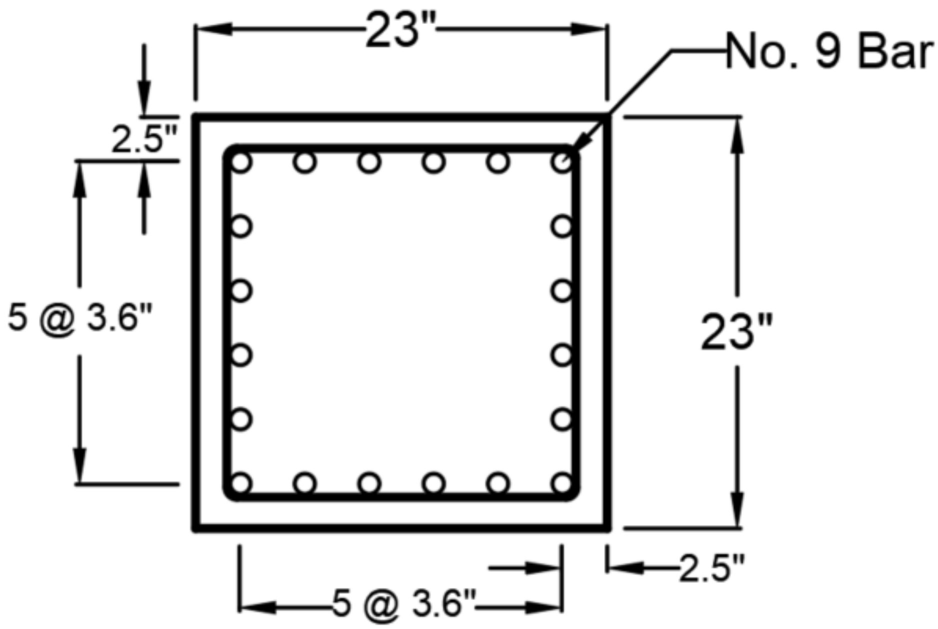


Figure 14: Column 2 and 3 Cross Section

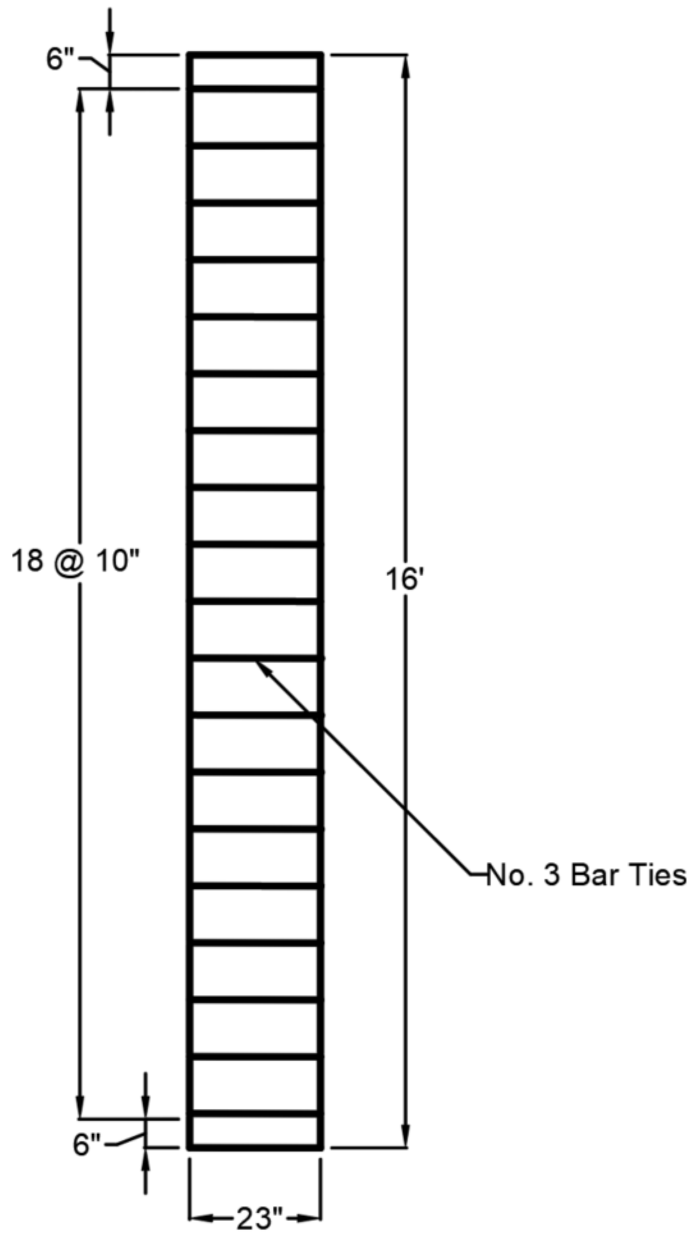


Figure 15: Column 2 and 3 Bar Ties

Appendix A: Preliminary Design of Beams and Columns

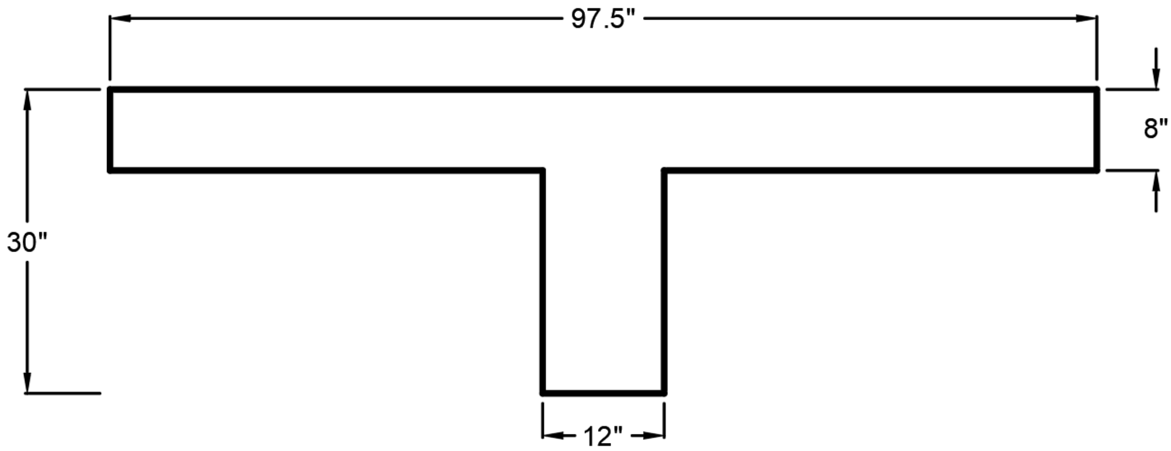


Figure 16: Preliminary Design of Beams 1,2, and 3

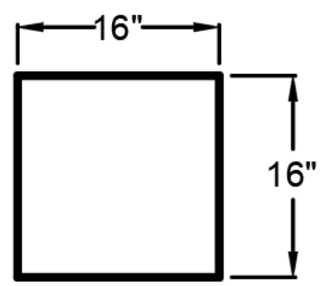


Figure 17: Preliminary Design of Columns 1 and 4

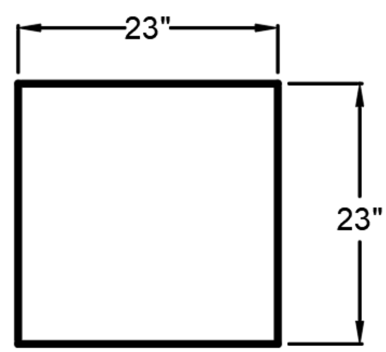
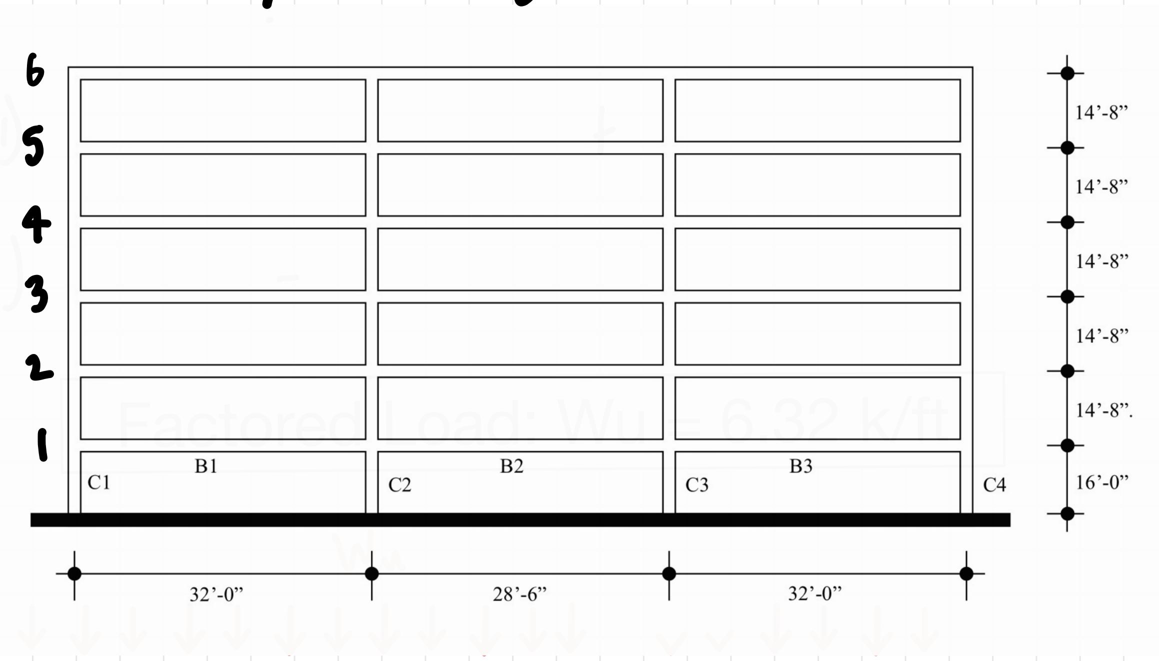
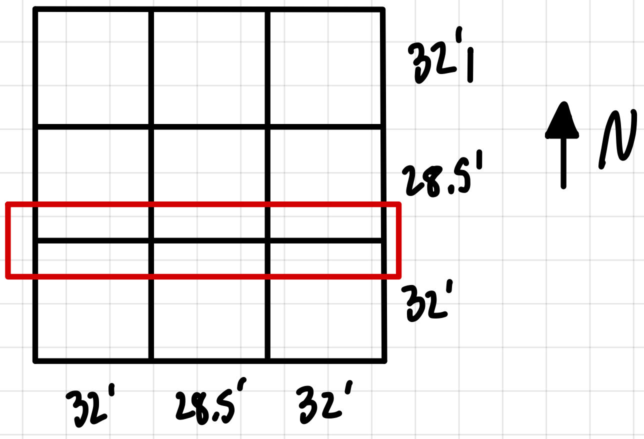


Figure 18: Preliminary Design of Columns 2 and 3

Preliminary Design Calculations:



3D appearance:



Slab thickness: 8"

$f_c' = 5500$ psi

$f_y = 60$ ksi

service dead load: 1.6 kips/ft

live load: 2.75 kips/ft

Dead Load = 1.6 kips/ft Live Load = 2.75 kips/ft

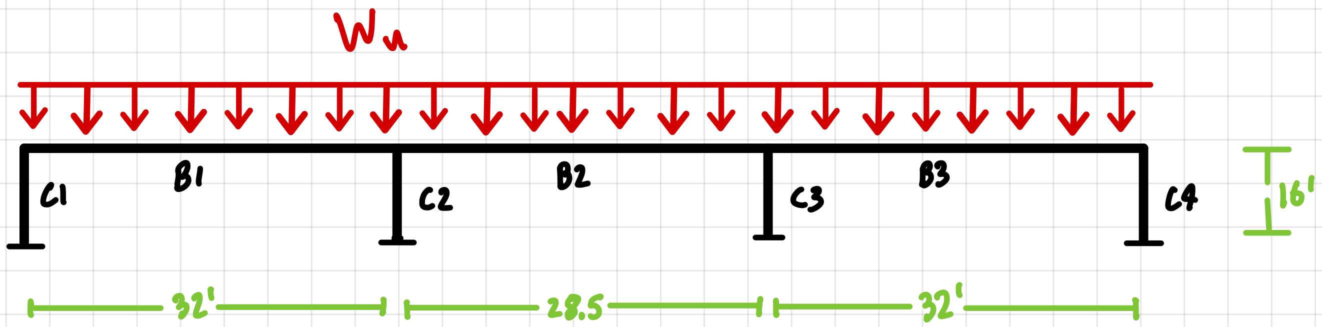
Load Combinations

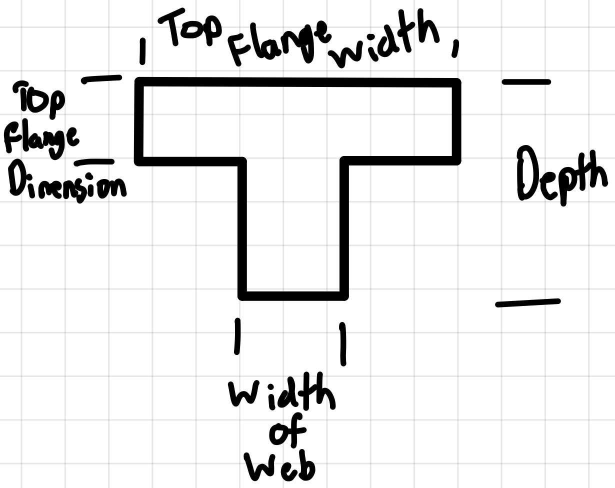
1) $1.2(1.6) = 1.92\ ^{k}/_{ft}$

2) $1.2(1.6) + 1.6(2.75) = \underline{6.32\ ^{k}/_{ft}}$ governs

$\boxed{\text{Factored Load: } W_u = 6.32 \text{ k/ft}}$

FBD:





Preliminary

$\approx \frac{\text{span}}{12}$ $\quad \frac{1}{2} \rightarrow \frac{1}{3}$ of depth

Beam	Span	Depth	Width of Web	Top Flange Width	Top Flange Depth
B1, B3	32'	30"	12"	97.5"	8"
B2	28.5'	30"	12"	97.5"	8"

effective flange width:

Table 6.3.2.1—Dimensional limits for effective overhanging flange width for T-beams

Flange location	Effective overhanging flange width, beyond face of web	
Each side of web	Least of:	$8h$
		$s_w/2$
		$l_n/8$

$$\frac{b - b_w}{2} \leq \quad 8h = 64'' \quad s_w/2 = \frac{28.5''}{2} = 171'' \quad l_n/8 = \frac{28.5}{8} = 42.75''$$

$$b = 2(42.75) + 12'' = 97.5''$$

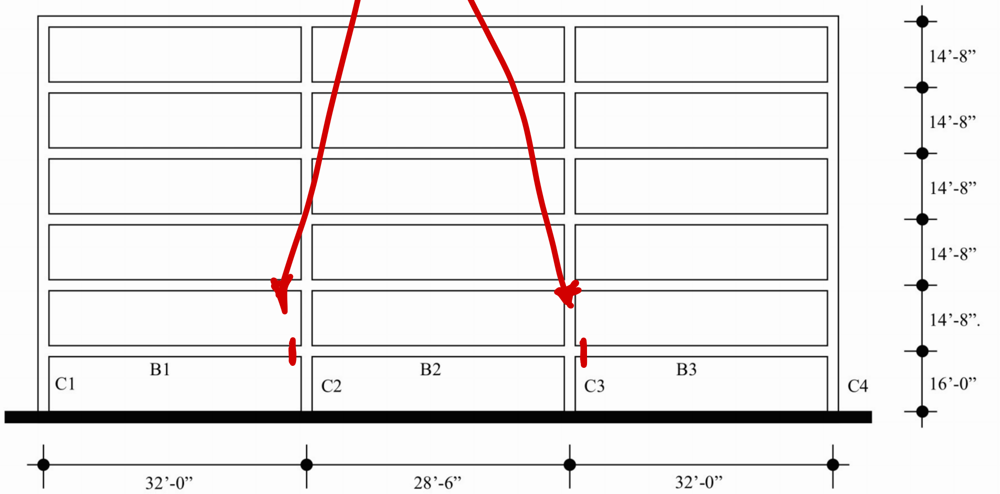
web thickness, b_w

1:2 1:3 ratio of beam depth : column width

Column:

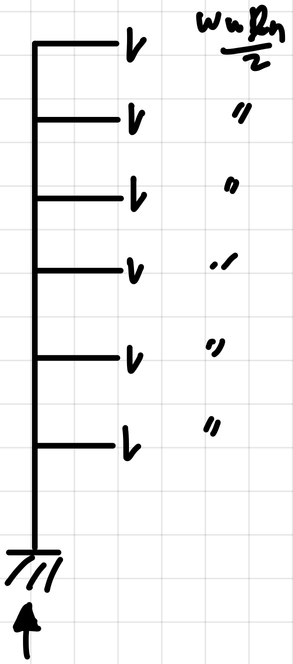
Table 6.5.4—Approximate shears for nonprestressed continuous beams and one-way slabs

	Location	V_u
C1, C4 →	Exterior face of first interior support	$1.15 w_u \ell_n / 2$
C2, C3 →	Face of all other supports	$w_u \ell_n / 2$



Column tree

Exterior:



$$P_{u_{ext}} = 6\left[\frac{6.32\,^{k}/_{ft}\,(32')}{2}\right] \doteq 606.72\,k$$

$P_u = \sum$ Factored Beam shears shown above

ACI 318-319 Ch. 6.5

a) prismatic
b) uniform loads
c) $L \leq 3D$
d) at least two spans

✓

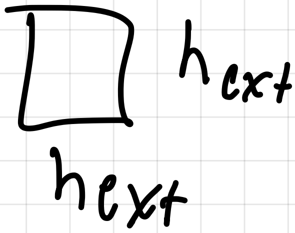
Exterior:

Demand $\leq$ capacity

$$P_u \leq \underset{\substack{\text{or} \\ 0.45}}{0.4}\,(A_c f_c')$$

$$A_c \geq \frac{P_u}{0.45 f_c'}$$

Sq. col



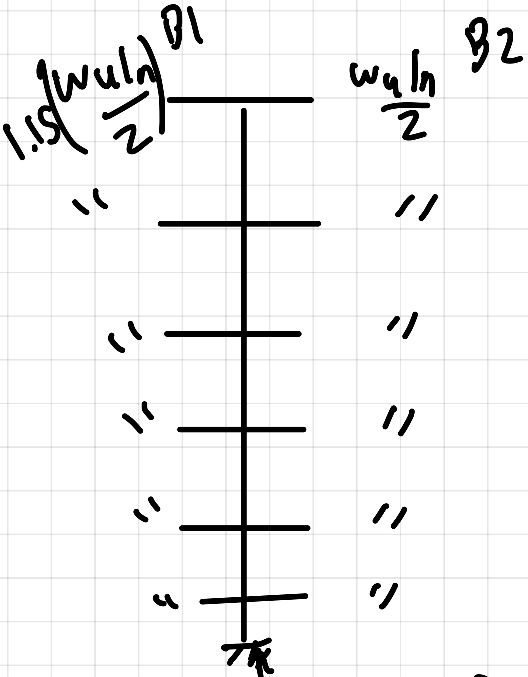
$$x \geq \sqrt{\frac{P_u}{0.45 f_c'}} = 15.66$$

$$\sqrt{\frac{P_u}{0.4 f_c'}} = \sqrt{\frac{606.72}{0.4(5.5)}} \quad 16.6$$

$\boxed{h_{ext} = 16''}$

16″ x 16″

Interior:



$$P_{u_{int}} = \underset{\substack{\text{or} \\ 0.45}}{0.4}\, f_c' (h_{int})^2$$

$$P_{u_{int}} = 6\left[1.15\left(\frac{6.32(32)}{2}\right) + 1\left(\frac{6.32(28.5)}{2}\right)\right]$$

$$P_{u_{int}} = 1238.088\,k$$

$$h_{int} = \sqrt{\frac{P_{u_{int}}}{0.4 \text{ or } 0.45\,(5.5)}} = \begin{matrix} 23.7 \\ \text{or} \\ 22.4 \end{matrix}$$

$\boxed{h_{int} = 23''}$

Appendix B: Design of Beams for Flexure

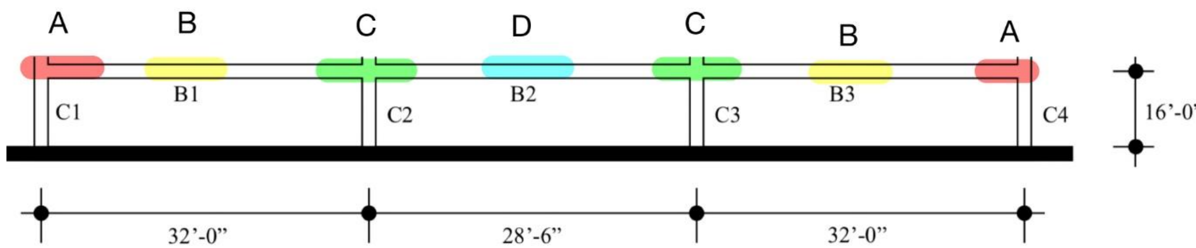


Figure 19: Flexural Sections in Beams

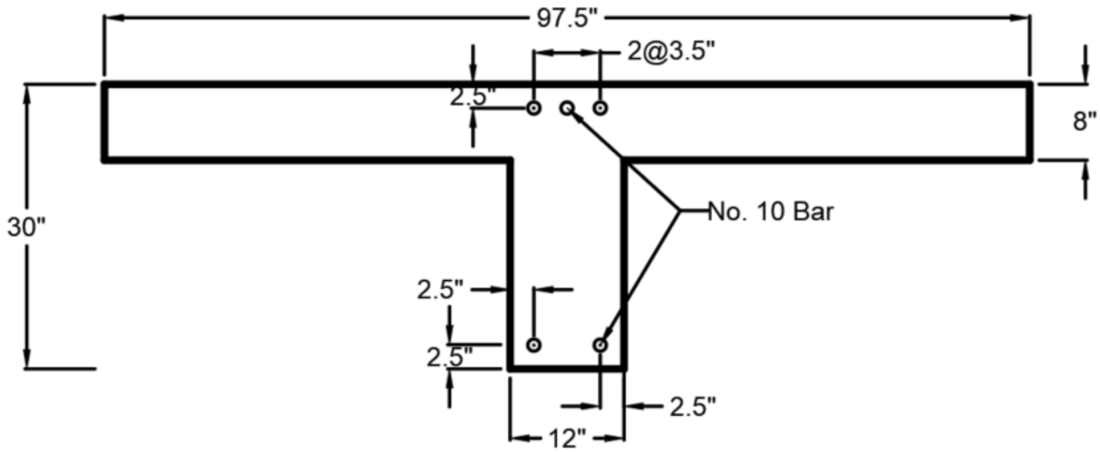


Figure 20: Section A Design of Beams for Flexure

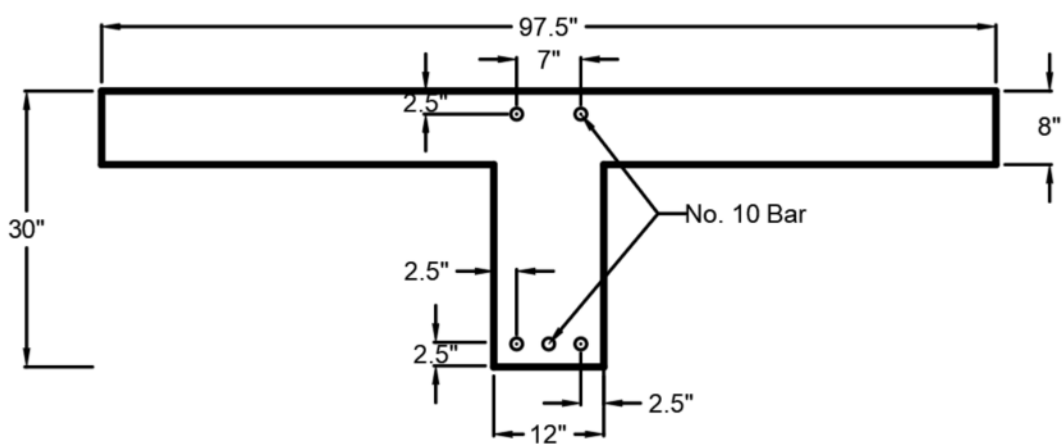


Figure 21: Section B Design of Beams for Flexure

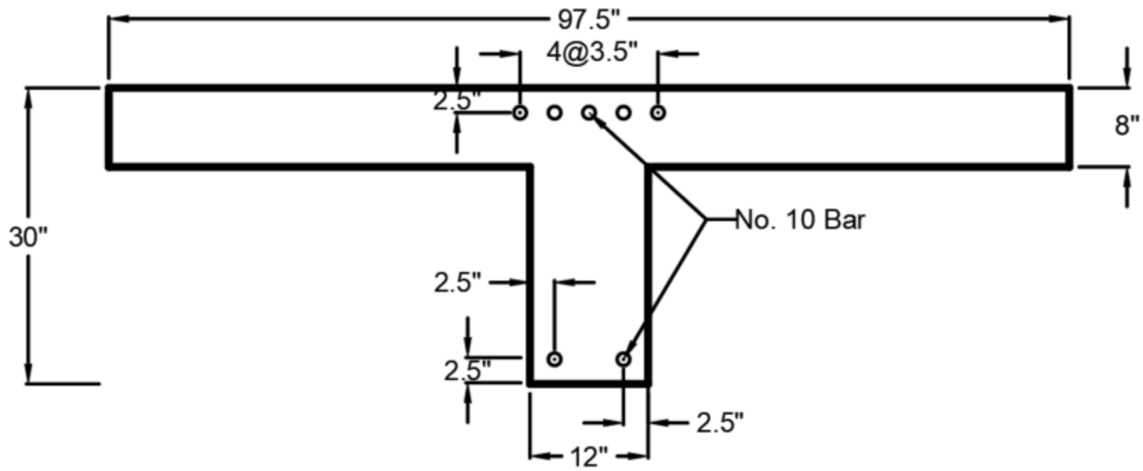


Figure 22: Section C Design of Beam for Flexure

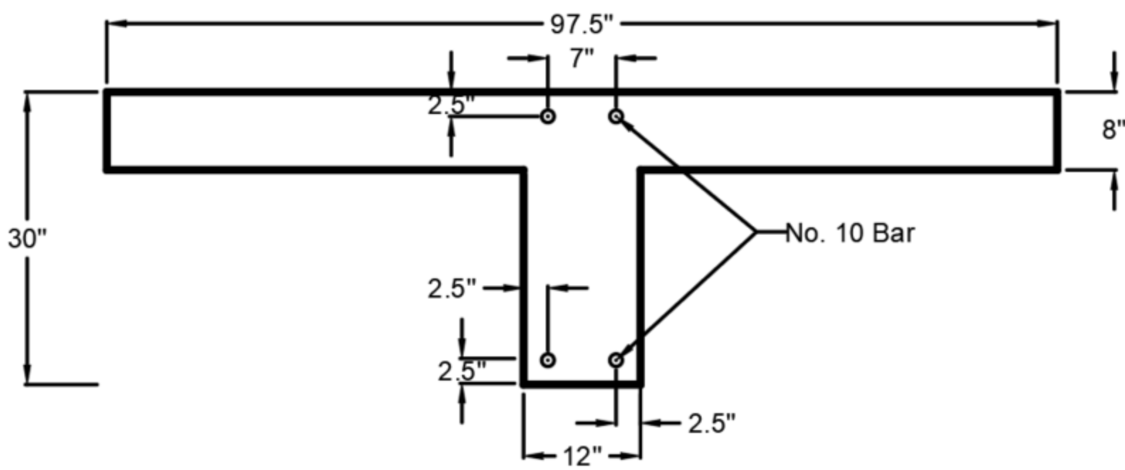


Figure 23: Section D Design of Beam for Flexure

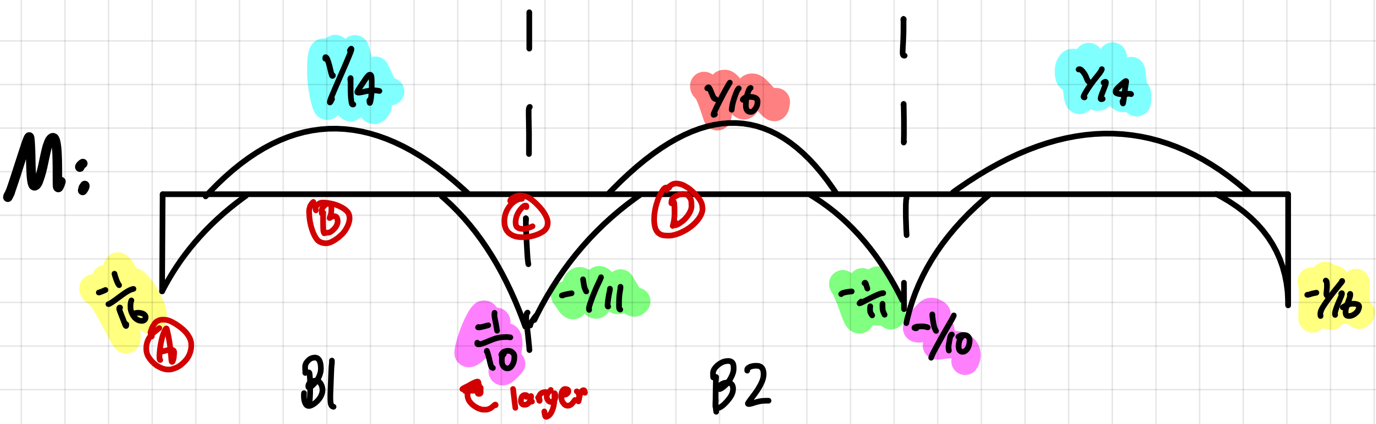
Design of Beams for flexure

Table 6.5.2

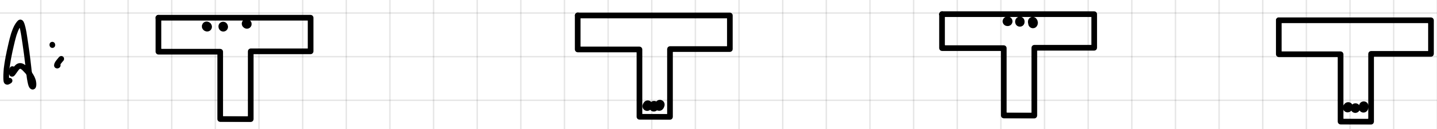
Table 6.5.2—Approximate moments for nonprestressed continuous beams and one-way slabs

Moment	Location	Condition	M_u
Positive	End span	Discontinuous end integral with support	$w_u \ell_n^2/14$
		Discontinuous end unrestrained	$w_u \ell_n^2/11$
	Interior spans	All	$w_u \ell_n^2/16$
Negative[1]	Interior face of exterior support	Member built integrally with supporting spandrel beam	$w_u \ell_n^2/24$
		Member built integrally with supporting column	$w_u \ell_n^2/16$
	Exterior face of first interior support	Two spans	$w_u \ell_n^2/9$
		More than two spans	$w_u \ell_n^2/10$
	Face of other supports	All	$w_u \ell_n^2/11$
	Face of all supports satisfying (a) or (b)	(a) slabs with spans not exceeding 10 ft (b) beams where ratio of sum of column stiffnesses to beam stiffness exceeds 8 at each end of span	$w_u \ell_n^2/12$

[1]To calculate negative moments, ℓ_n shall be the average of the adjacent clear span lengths.



• Discontinuity comes from different load combinations



required A_s → lect.
Appendix B

• top bar Bigger than bottom

2 bars on top → full way
2 bars bottom → full way

Dead Load = 1.6 kips/ft   Live Load = 2.75 kips/ft

Load Combinations

1) $1.2(1.6) = 1.92\ ^k/ft$

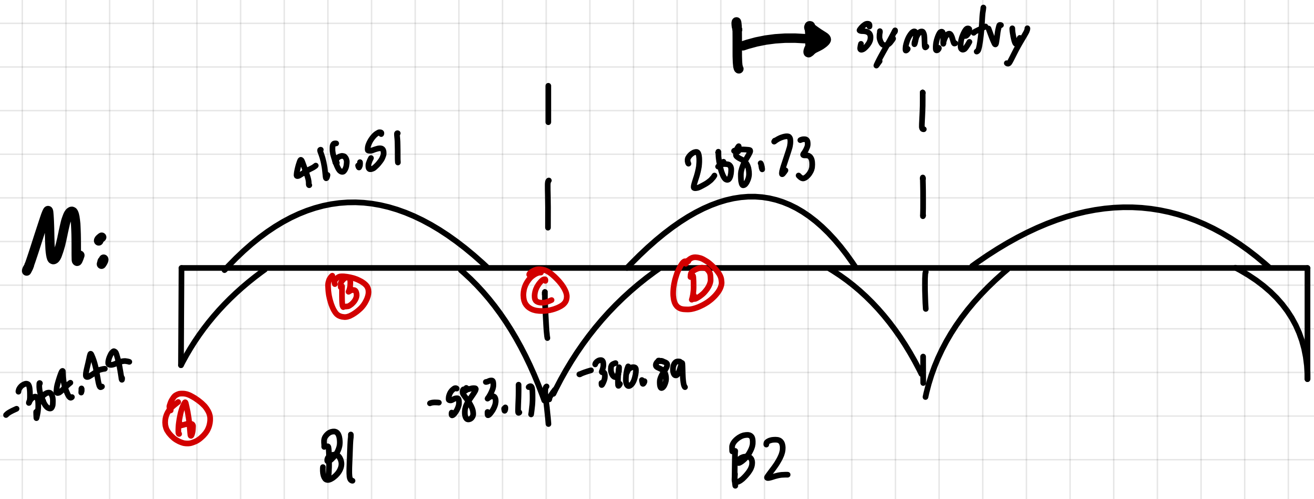
2) $1.2(1.6) + 1.6(2.75) = \underline{6.32\ ^k/ft}$ governs

$$\boxed{\text{Factored Load: } W_u = 6.32 \text{ k/ft}}$$

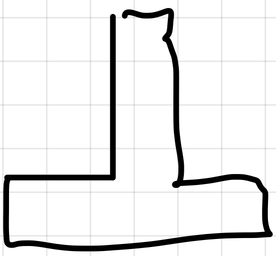
B1, B3 $\ell_n = 32' - \frac{1}{2}(16)'' - \frac{1}{2}(23)'' = 30.375'$

B2 $\ell_n = 28' - 23'' = 26.0833'$

B1, B3 : $w_u \ell_n^2 = 5831.1$ k·ft B2: 4299.75 k·ft



Beam Section C: $M_u = 583.11$ assuming $J = 0.9$



$$A_s = \frac{M_u}{\phi f_y J d} = \frac{583.11 \times 12}{0.9\,(60)\,0.9(27.5)} = 5.233$$

$d = 30 - 2.5$ (1 layer) or $d = 30 - 3.5 = 26.5$ (2 layer)

5 no. 10 bar one layer $A_s = 5 * 1.27 = 6.35 in^2$
D = 27.5

Check

- Height of rectangular stress block

$$a = \frac{A_s f_y}{0.85 f_c' b_w}$$

$$a = \frac{6.35(60)}{0.85(5.5)(12'')} = 6.79 \text{ in}$$

- Tension-controlled section

$$\epsilon_t \geq 0.005$$

$$\varepsilon_s = \frac{\left(30 - \frac{6.79}{0.775}\right)(0.003)}{6.79/0.775} = 0.007 \checkmark$$

  - If this is not tension-controlled section, consider compression reinforcement (A_s', effective only when it placed within rectangular stress block) or increase beam height.

- Minimum reinforcement

$$A_{s,min} = \frac{3\sqrt{f_c'}}{f_y} b_w d$$

and not less than $200 b_w d / f_y$

$$A_{s,min} = \frac{3\sqrt{5,500}}{60000}(12'')(30'') = 1.335$$

$$A_{s,min} = \frac{200(12'')(30'')}{60} = 1.2$$

Beam Section B:

$\underline{416.51} = M_u$ assuming $J = 0.9$

$$A_s = \frac{M_u}{\phi f_y J d} = \frac{416.51 \times 12}{0.9(60)\,0.9(27.5)} = 3.74$$

3 no. 10 bars 1 layer
D = 27.5

$$a = \frac{3.74(60)}{0.85(5.5)(12)} \leq 8 \quad \checkmark$$

$$\varepsilon_s = \frac{\left(30 - \frac{4}{0.775}\right)(0.003)}{\left(\frac{4}{0.775}\right)} = 0.014 > 0.003 \quad \checkmark$$

$$A_{s_{min}} = 1.33 \quad \checkmark$$

Beam Section A: $M_u = 364.44$

$$A_s = \frac{364.44 \times 12}{0.9(60)0.9(27.5)} = 3.27$$

3 no. 10 Bar
1 layer

$A_s = 3.81$

$$a = \frac{3.81(60)}{0.85(5.5)(12)} = 4.07 \leq 8 \quad \checkmark$$

$$\varepsilon_s = \frac{\left(30 - \frac{4.07}{0.775}\right)(0.003)}{\frac{4.07}{0.775}} > 0.003 \quad \checkmark$$

$A_{s_{min}}$ ✓

Beam Section D:

$$M_u = 268.73 \qquad A_s = \frac{268.73 \times 12}{0.9(60)(0.9)(27.5)} = 2.413$$

2 no. 10 Bar
1 layer

$A_s = 2.54$

$$a = \frac{2.54(60)}{0.85(5.5)(12)} = 2.72 \leq 8 \quad \checkmark$$

$$\varepsilon_s = \frac{\left(30 - \frac{2.72}{0.775}\right)(0.003)}{\frac{2.72}{0.775}} > 0.003 \quad \checkmark$$

$A_{s_{min}}$ ✓

Appendix C: Beams for Shear

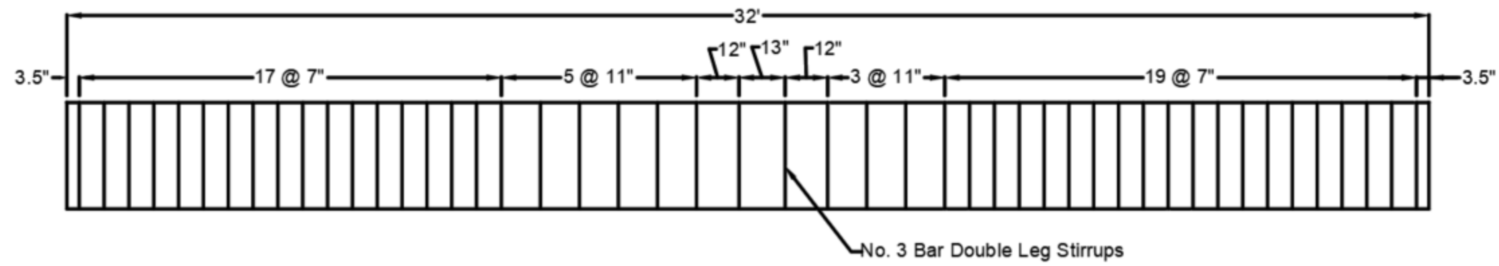


Figure 24: Beam 1 Design for shear

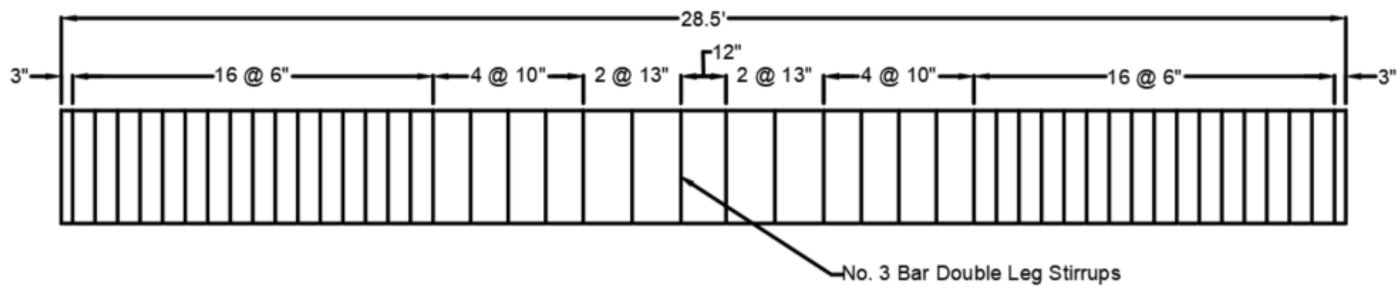


Figure 25: Beam 2 Design for Shear

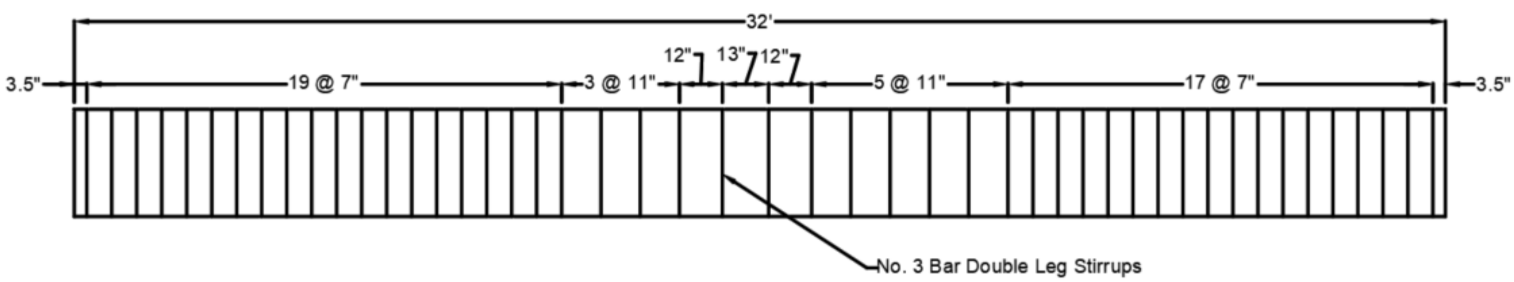


Figure 26: Beam 3 Design for Shear

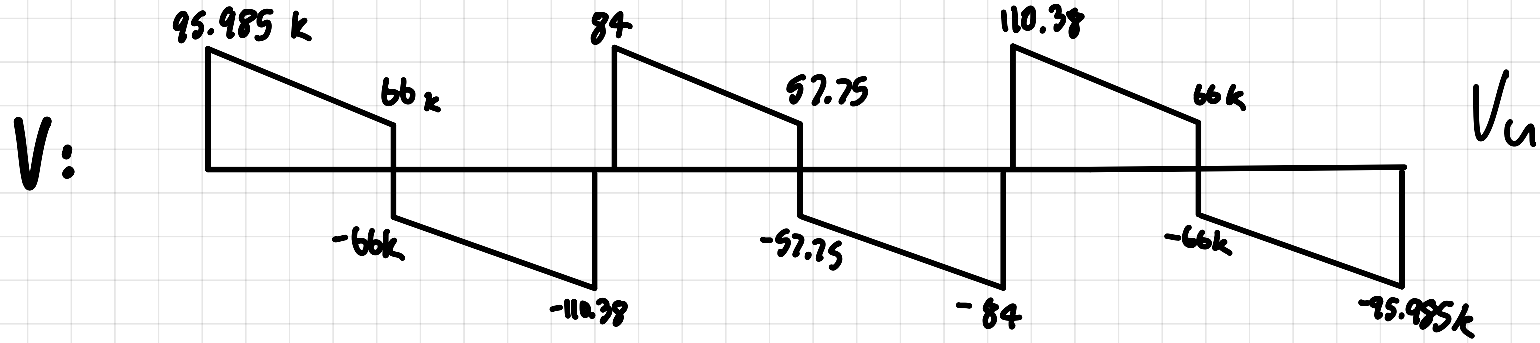
Beams for Shear

$w_{L_u} = 2.75$

$w_u = 6.32$

B3, B1: $l_n = 32 - \frac{1}{12}\cdot\frac{16}{2} - \frac{1}{12}\frac{23}{2} = 30.375'$ $\quad w_u l_n = 191.97$ k

B2: $l_n = 28.5 - \frac{23}{12} = 26.6'$ $\quad w_u l_n = 168$ k



B1:

location — shear demand

① left end $\quad V_u = \frac{6.32(30.375)}{2} - \frac{\left(\frac{6.32(30.375)}{2} - \frac{2.75(30.375)}{8}\right)}{\frac{30.375}{2}} 30'' = 81.91$

② midspan $\quad V_u = \frac{2.75(30.375)}{8} = 10.44$

③ right end $\quad V_u = 110.38 - \frac{110.38 - \frac{2.75(30.375)}{8}}{\frac{30.375}{2}} 30'' = 93.93$

B2:

location — shear demand

④ left + right $\quad V_u = 84 - \frac{84 - \frac{2.75(26.6)}{8}}{\frac{26.6}{2}} 30'' = 69.93$

⑤ midspan $\quad V_u = \frac{2.75(26.6)}{8} = 9.144$

B3:

Mirror of B1

Beam 1:

$V_c = 2\sqrt{f_c'}\, b_w d = 2\sqrt{5500}\ 12'' \ 30'' \times \frac{1}{1000} = 53.4\ k$

$\phi V_n = \phi(V_c + V_s)$

① $V_s = \frac{V_u}{\phi} - V_c = \frac{81.91}{0.75} - 53.4 = 55.8133$

② $V_s =$ none

③ $V_s = \frac{93.93}{0.75} - 53.4 = 40.53$

④ $V_s = \frac{69.93}{0.75} - 53.4 = 39.84$

⑤ $V_s =$ none

no. 3 double leg stirrups

assume $A_v = 0.22\ in^2$ 2 no. 3 bar

$s = \frac{0.22 in^2 \cdot 60{,}000 \cdot 30''}{55.8133} = 7.095 \approx 7''$

$V_s < 8\sqrt{5500}\ 12''\ 30'' = 213.586$ ✓

other $s = 11''$

normal shear beam since $V_s < 4\sqrt{f_c'}\, b_w d$

$s_{max} = \begin{cases} \frac{d}{2} = 15 \\ 24 \end{cases}$ $S_{max} = 15''$

$$A_{v,min} = 0.75\sqrt{5600}\,\frac{12''\,7''}{60{,}000} = 0.077 \checkmark$$

$$A_{v,min} = \frac{50(12'')(7'')}{60{,}000} = 0.07 \checkmark$$

Where to put these stirrups

left: $m = \dfrac{127980 - 13921.9}{30.375/2} = 7510$

$\dfrac{V_u}{\phi} = V_c$ $\quad x = 9.9312'$ $\qquad \dfrac{V_u}{\phi} = \dfrac{V_c}{2}$ $\quad x = 13.486'$

right: $m = \dfrac{147177 - 13921.9}{30.375/2} = 8774$

$\dfrac{V_u}{\phi} = V_c$ $\quad x = 10.68'$ $\qquad \dfrac{V_u}{\phi} = \dfrac{V_c}{2}$ $\quad x = 13.73'$

Beam 2: assume $A_v = 0.22\ in^2$ 2 no.3 bar

$$s = \frac{0.22in^2 \cdot 60{,}000 \cdot 30''}{69.93 \times 1000} = 5.66'' \approx 6''$$

$$V_s < 8\sqrt{5500}\ 12''\ 30'' = 213.586$$

✓

other $s = 10''$

normal shear beam since $V_s < 4\sqrt{f_c'}\ b_w d$

$$s_{max} = \begin{cases} \frac{d}{2} = 15 \\ 24 \end{cases}$$

$s < s_{max}$

$s_{max} = 15''$

$$A_{v,min} = 0.75\sqrt{5500}\ \frac{12''\ 6''}{60{,}000} = 0.0668$$

$$s = \frac{0.0668 \cdot 60{,}000 \cdot 30}{69.91 \times 1000}$$

where to put minimum shear reinforcement

symmetrical: left: $m = \dfrac{112024.66 - 12191.66}{\frac{26.6}{2}} = 7510$

$\dfrac{V_u}{\phi} = V_c$ $x = 7.8133'$

$\dfrac{V_u}{\phi} = \dfrac{V_c}{2}$ $x = 11.37'$

Appendix D: Bar Cut-offs

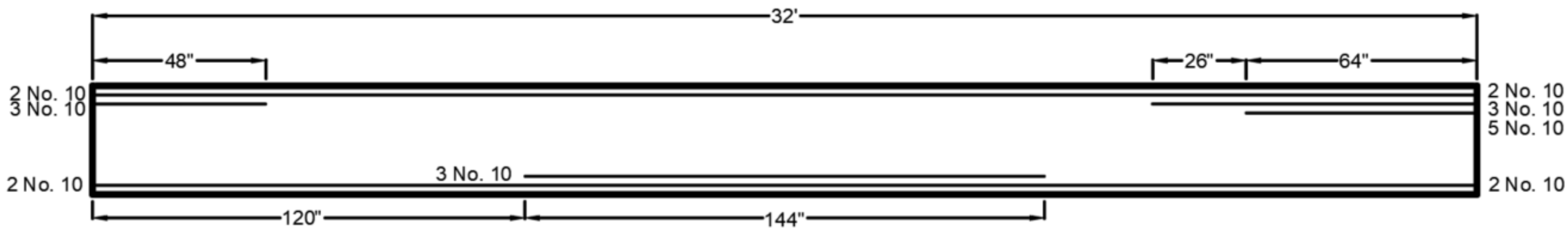


Figure 27: Beam 1 Bar Cut-offs

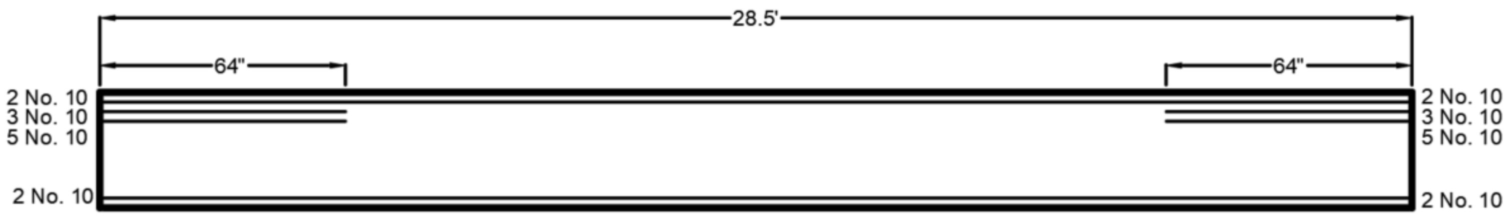


Figure 28: Beam 2 Bar Cut-offs

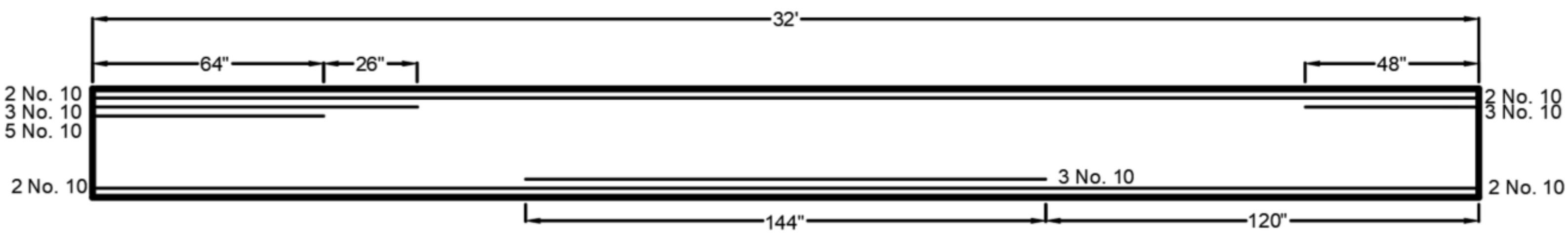


Figure 29: Beam 3 Bar Cut-offs

Bar Cut-off

$f_y = 60\,ksi$
$f_c' = 5.5\,ksi$

$$\ell_d = \left(\frac{3}{40} \frac{f_y}{\lambda \sqrt{f_c'}} \frac{\psi_t \psi_e \psi_s \psi_g}{\left(\frac{c_b + K_{tr}}{d_b} \right)} \right) d_b \qquad (25.4.2.4a)$$

$\psi_t = 1.3 \qquad \psi_e = 1.0 \qquad \psi_s = 1.0 \qquad \psi_g = 1.0$

$\lambda = 1.0$

in which the confinement term $(c_b + K_{tr})/d_b$ shall not exceed 2.5, and

$$K_{tr} = \frac{40 A_{tr}}{sn} \qquad (25.4.2.4b)$$

transverse reinforcement

spacing

# of bars being developed

$C_b \rightarrow$ lesser of:
1) distance from center of a bar to nearest conc. surface
2) ½ the center to center spacing of bars being developed

Beam Section A:

$C_{cover} = 2.5''$

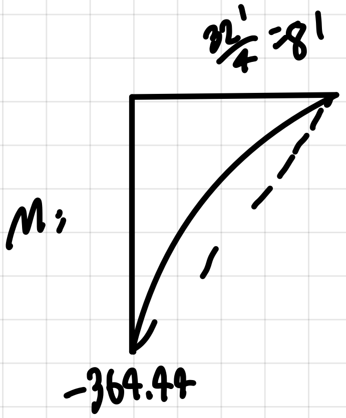
$K_{tr} = \frac{40(0.11 \times 2)}{7''(3)} = 0.42 \qquad C_{interior} = \frac{1}{2}(7'') = 3.5''$

$$\ell d = \frac{3}{40} \cdot \frac{60{,}000}{1\sqrt{5{,}500}} \cdot \frac{1.3(1)(1)(1)}{\frac{2.5+0.42}{1.27}} \cdot 1.27 = 43.57''$$

greater of $12d_b$ or d $\quad d = 30''$

2 no. 10 bar moment strength: $\quad A_s = 2.54$

$$\phi M_n = \phi A_s f_y Jd = 0.9(2.54)(60)(0.9)(27.5)/12 = 282.8925$$



$$x = \frac{364.44 - 282.9}{\left(\frac{364.44}{8}\right)} = 17.9''$$

$$17.9'' + 30'' = 48'' > l_d$$

d

cut off center bar at $x = 48''$

Beam section B:

$$C_{cover} = 2.5''$$

$$C_{interior} = \frac{1}{2}(3.5) = 1.75''$$

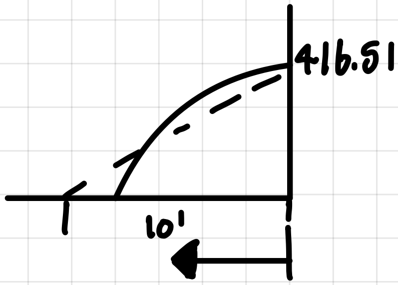
$$K_{tr} = \frac{40(0.22)}{11''(3)} = 0.33$$

$$l_d = \frac{3}{40} \cdot \frac{60{,}000}{\sqrt{5500}} \cdot \frac{1.3(1)(1)(1)}{\frac{1.75 + 0.33}{1.27}} (1.27) = 61.17''$$

$d = 30''$

2 no. 10 bar moment strength = 282.9 k

M:

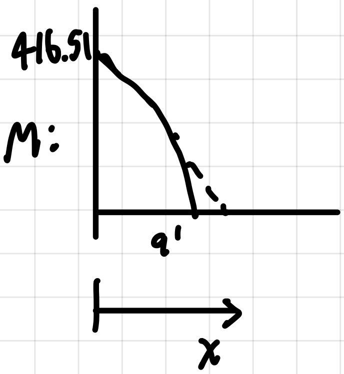


$$x = \frac{416.51-282.9}{\left(\frac{416.51}{10'}\right)} = 38.5''$$

$$38.5'' + 30'' = 69'' > \ell_d$$

at 10' add center bar to bottom of beam

M:



$$x = \frac{416.51-282.9}{\frac{416.51}{9}} = 34.65''$$

$$34.65'' + 30'' = 65'' > \ell_d$$

at 22' remove center bar on bottom of beam

Beam section C:

Beam 1 part

$c_{cover} = 2.5''$

$c_{interior} = \frac{1}{2}(3.5) = 1.75$

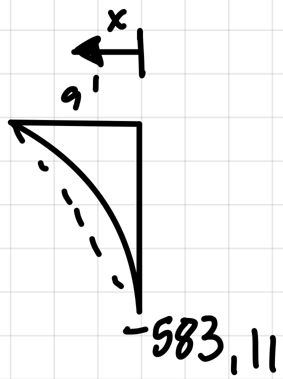
$$K_{tr} = \frac{40(0.11\times2)}{7''(5)} = 0.25$$

$$\ell_d = \frac{3}{40}\cdot\frac{60{,}000}{1\sqrt{5500}}\cdot\frac{1.3(1)(1)(1)}{\frac{1.75+0.25}{1.27}}\cdot 1.27 = 64''$$

$d = 30''$

3 no. 10 bar moment strength:

$A_s = 3.81 \qquad \phi M_n = 0.9(60)(3.81)(0.9)(27.5)/12 = 424.34$



$$x = \frac{583.11 - 424.34}{\frac{583.11}{9'}} = 29.4''$$

$29.4'' + 30'' = 59.4'' < l_d$

$l_d = 64''$

64" from interior column transition from 3 → 5 bars on top part of beam

2 no. 10 bar moment strength: 282.9k

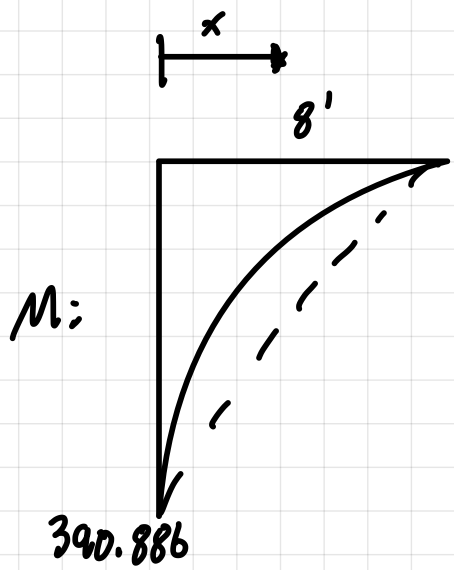
$$x = \frac{583.11 - 282.9}{\frac{583.11}{9}} = 55.6''$$

$55.6'' + 30'' = 85.6'' > l_d$

90" from interior column transistion from 2 → 3 bars on top part of beam

Beam 2 part: $K_{tr} = \frac{40(0.22)}{6''(5)} = 0.29$

$$l_d = \frac{3}{40} \cdot \frac{60{,}000}{1\sqrt{5500}} \cdot \frac{1.3(1)(1)(1)}{\frac{1.75 + 0.29}{1.27}} \cdot 1.27 = 62.36''$$



3 bars enough strength

use l_d

$$x = \frac{390.886 - 282.9}{\frac{390.886}{9}} = 30''$$

$30'' + 30'' < l_d$

use l_d

at 64" transistion from 5 → 2 bars

Beam section D

no bar cutoff

Appendix E: Column Design

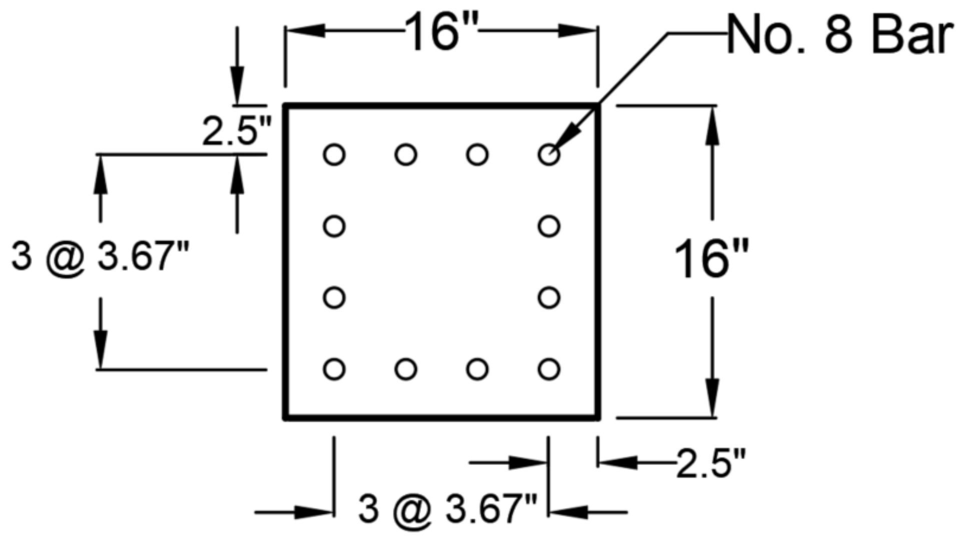


Figure 30: Column Design of Column 1 and 4

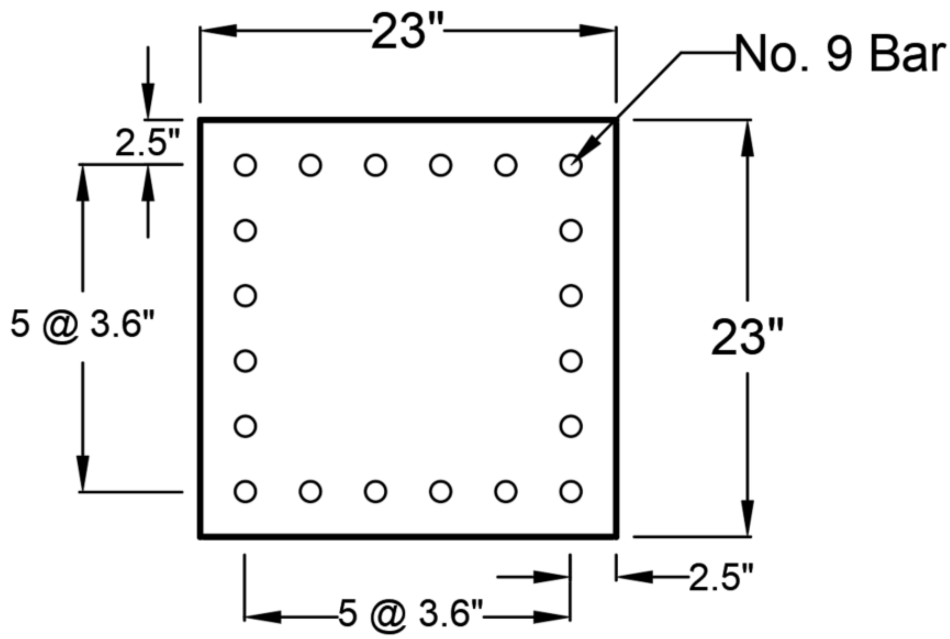


Figure 31: Column Design of Column 2 and 3

Column Design

$w_u = 6.32$
$w_{dead} = 1.6$
$w_{live} = 2.75$

$l_{n_1} = 32 - \frac{16}{2} \cdot \frac{1}{12} - \frac{23}{2} \cdot \frac{1}{12} = 30.375'$

$l_{n_2} = 28.5 - \frac{23}{12} = 26.6'$

Column 1 and 4 load cases

1) $P_u = \dfrac{6 \times 6.32 \times 30.375}{2} = 575.91$

$M_u = \dfrac{6.32(30.375)^2}{16} \times \dfrac{\frac{1}{18}}{\frac{1}{18} + \frac{1}{15}} = 165.66$

2) $P_u = 6 \times 1.2 \times 1.6 \times \dfrac{30.375}{2} = 174.96$

$M_u = \dfrac{1.2(1.6)(30.375^2)}{16} \times \dfrac{\frac{1}{18}}{\frac{1}{18} + \frac{1}{15}} = 50.33$

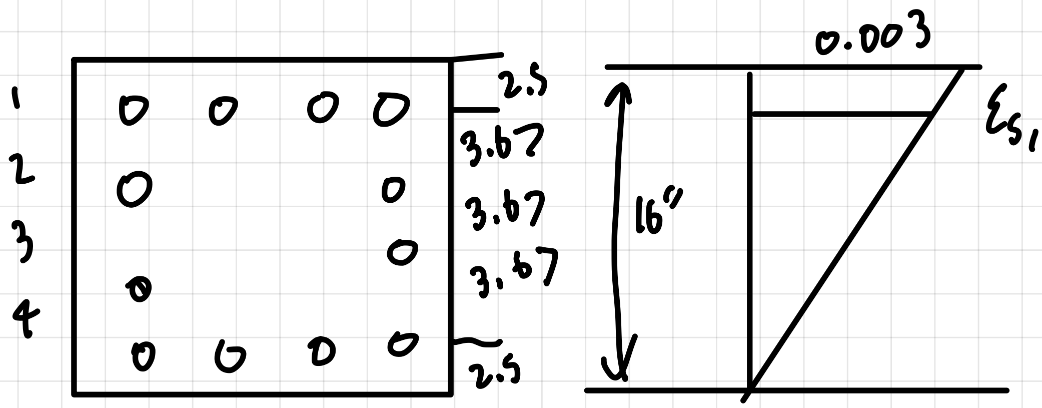
3) $P_u = (6 \times 1.2 \times 1.6 + 1.6(2.75))\left(\dfrac{30.375}{2}\right) = 241.785$

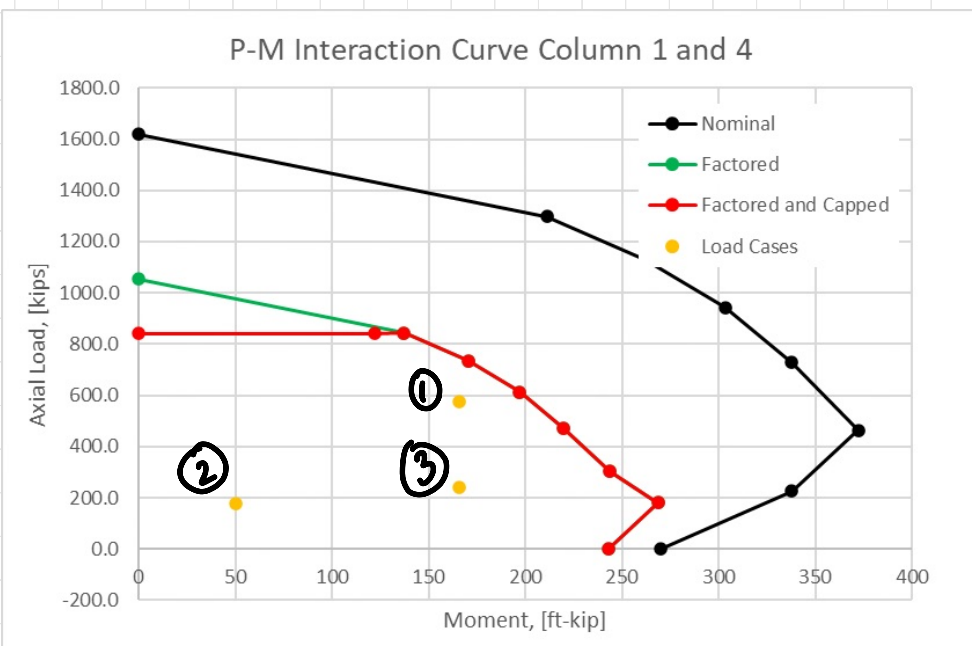
$M_u = \dfrac{6.32(30.375)^2}{16} \times \dfrac{\frac{1}{18}}{\frac{1}{18} + \frac{1}{15}} = 165.66$

Column 1 and 4:

$A_s \approx 0.04 \times (13)^2 = 10.24 \text{ in}^2$

12 no.8 bars $\qquad A_s = 9.48$





load case

1 good

2 good

3 good

No. 3 Ties

$s \le 16(1)$ and $48\left(\frac{3}{8}\right)$ and h and $\frac{d}{2}$

16 and 18 and 16 $\quad \frac{16-2.5}{2} = 6.75$

$s = 6"$

Column 2 and 3 load cases:

$l_{n_1} = 30.375 \qquad l_{n_2} = 26.6$

1) $P_u = 6 \times 6.32\,(1.15(30.375) + 26.6)/2 = 1166.6$

$$M_u = \frac{\frac{1}{18}}{\frac{1}{18}+\frac{1}{15}} \times \left(\frac{6.32\,(30.375)^2}{10} - \frac{6.32\,(26.6)^2}{11}\right) = 80.265$$

2) $P_u = 6 \times 6.32 \times 1.15 \times 30.375/2 + 6 \times 1.2 \times 1.6 \times \frac{26.6}{2} + 5 \times 1.6 \times 2.75 \times \frac{26.6}{2} = 1108.1$

$$M_u = \frac{\frac{1}{18}}{\frac{1}{18}+\frac{1}{15}} \times \left(\frac{6.32\,(30.375)^2}{10} - \frac{1.2(1.6)\,(26.6)^2}{11}\right) = 208.9$$

3) $P_u = 6 \times (1.2 \times 1.6)(1.15 \times 30.375 + 26.6)/2 + (1.6 \times 2.75)\,1.15 \times \frac{30.375}{2} = 431.3$

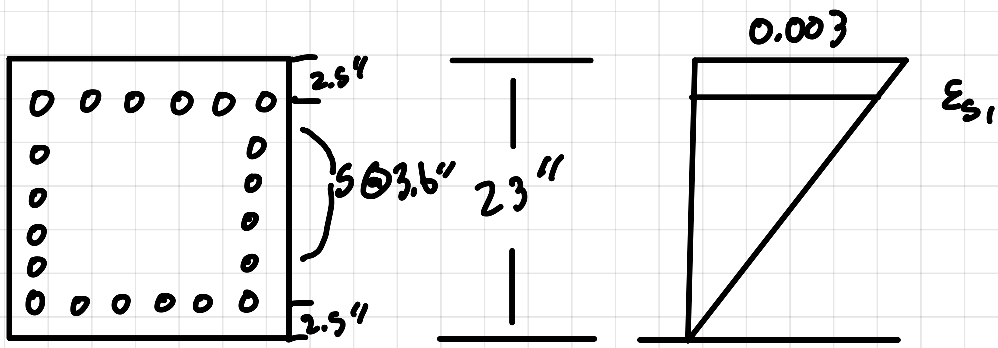
$$M_u = \frac{\frac{1}{18}}{\frac{1}{18}+\frac{1}{15}} \times \left(\frac{6.32\,(30.375)^2}{10} - \frac{1.2(1.6)\,(26.6)^2}{11}\right) = 208.9$$

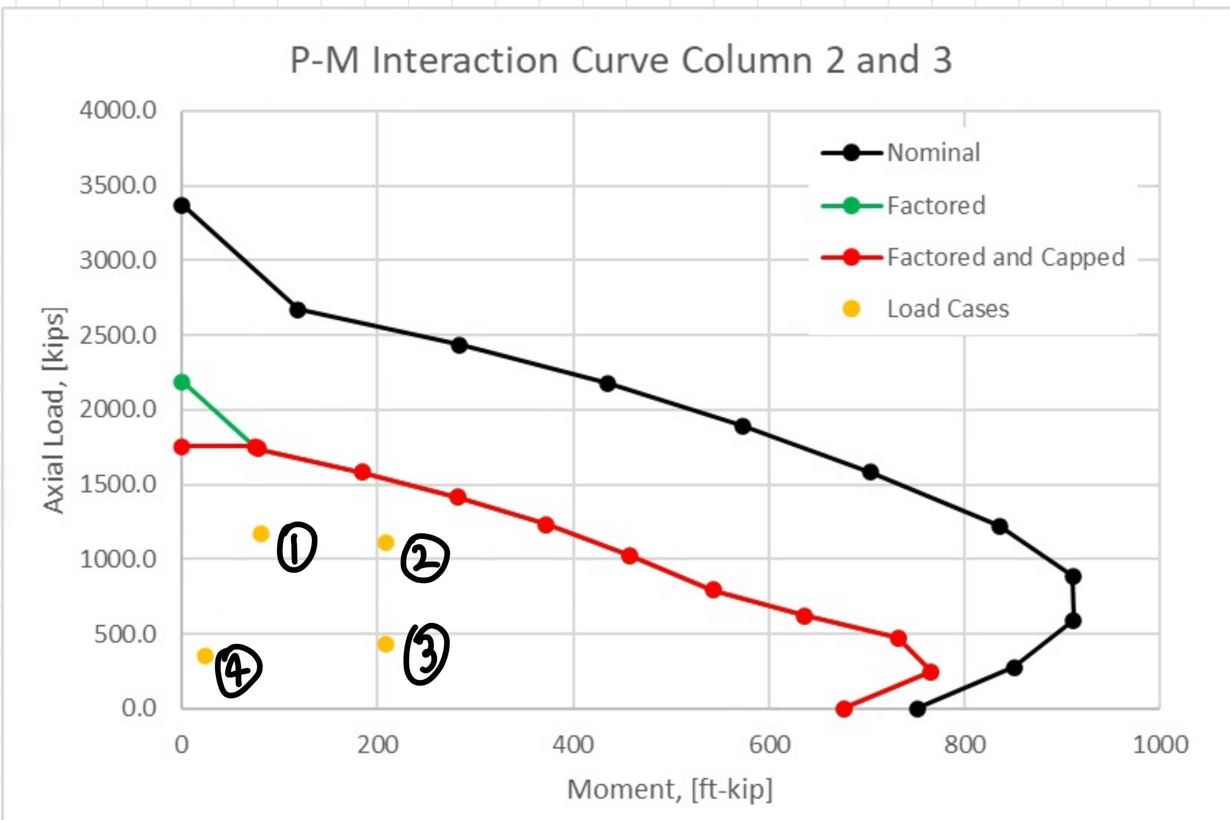
4) $P_u = 6 \times 1.2 \times 1.6\,(1.15 \times 30.375 + 26.6)/2 = 354.42$

$$M_u = \frac{\frac{1}{18}}{\frac{1}{18}+\frac{1}{15}} \times \left(\frac{1.2 \times 1.6\,(30.375)^2}{10} - \frac{1.2(1.6)\,(26.6)^2}{11}\right) = 24.38437$$

$A_s \approx 0.04 \times (23)^2 = 21.16$

20 no. 9 bars $\quad A_s = 20 \, in^2$





Load case:

1 good

2 good

3 good

4 good

no. 3 Ties $\quad s = 10''$

$$s \leq \frac{23-2.5}{2} = 10.25''$$