Arch 324
Structures II

Winter 2026 Recitation 004

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Recitation 004

Welcome to session 6!

Preliminary Reports Due!!

- Quick Lecture Recap
- Homework #6 Steel Column Analysis
- Lab: Steel Columns

Feel free to ask questions anytime

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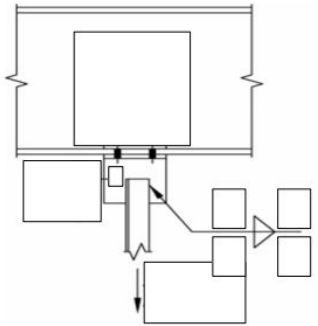
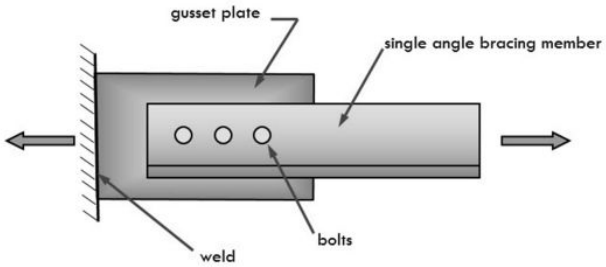
Lecture: Steel Column Design (2/16)

Steel Connections

Failure modes – Limit States

- Fasteners (bolts or welds)
  - shear
  - tension
  - bearing
- Connecting elements (plates or tees)
  - tension
  - block shear
  - tear out
- Supporting or supported members





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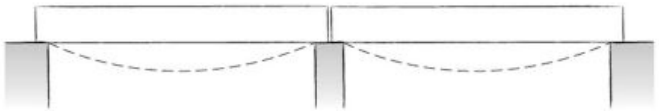
Lecture: Continuous Beams (2/28)

Continuous Beams

- Continuous over one or more supports
  - Most common in monolithic concrete
  - Steel: continuous or with moment connections
  - Wood: as continuous beams, e.g. long Glulam spans
- Statically indeterminate
  - Cannot be solved by the three equations of statics alone
  - Internal forces (shear & moment) as well as reactions are affected by movement or settlement of the supports



two spans - simply supported

two spans - continuous

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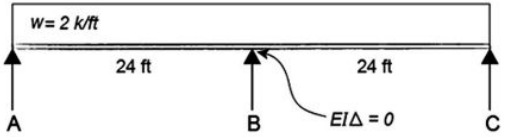
Lecture: Continuous Beams (2/28)

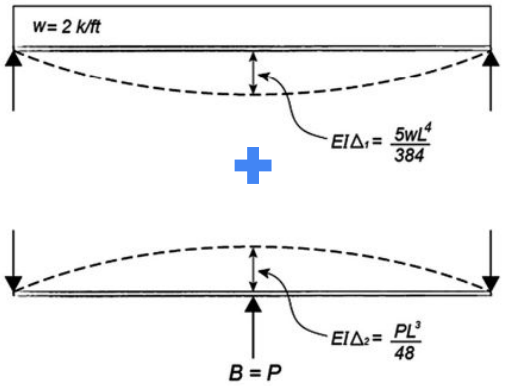
Deflection Method

- Two continuous, symmetric spans
- Symmetric Load



Procedure:

1. Remove the center support and calculate the center deflection for each load case as a simple span.
2. Remove the applied loads and replace the center support. Set the deflection equation for this case (center point load) equal to the deflection from step 1.
3. Solve the resulting equation for the center reaction force. (upward point load)
4. Calculate the remaining two end reactions.
5. Draw shear and moment diagrams as usual.



$$EI\Delta_1 + EI\Delta_2 = 0$$

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Lecture: Continuous Beams (2/28)

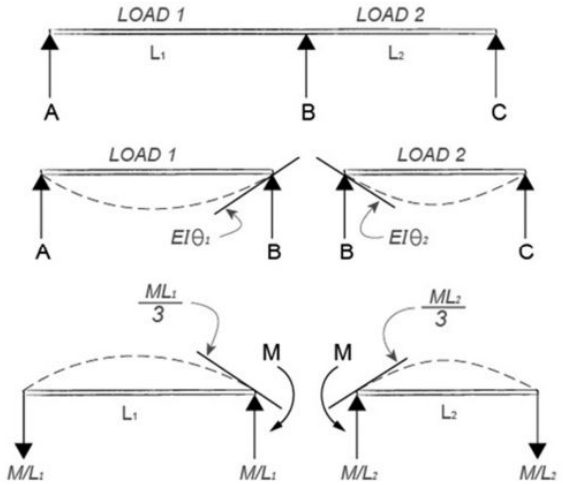
Slope Method

- Two continuous spans
- Non-symmetric loads and spans

Procedure:

1. Break the beam into two halves at the interior support and calculate the interior slopes of the two simple spans.
2. Use the Slope Equation to solve for the negative interior moment.
3. Find the reactions of each of the simple spans plus the M/L reactions caused by the interior moment.
4. Add all the reactions by superposition.
5. Draw the shear and moment diagrams as usual.

$$M = \frac{3}{L_1 + L_2}\left[EI\Theta_1 + EI\Theta_2\right]$$



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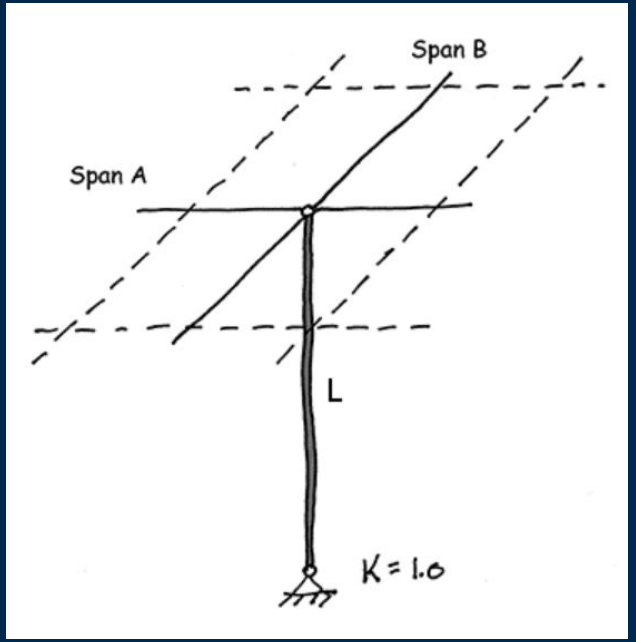
HW #6: Steel Column Analysis

For the given axially loaded steel W-section, determine the maximum floor live load capacity, P LL. Assume the column is pinned top and bottom: K = 1.0, and there is no intermediate bracing. Use AISC-LRFD steel equations to determine phi Pn and the load. E = 29000 ksi.

DATASET: 1 -2- -3-

W-section	W8X31
Fy	50 KSI
Span A	36 FT
Span B	34 FT
Height L	15 FT
Floor Dead Load	42 PSF



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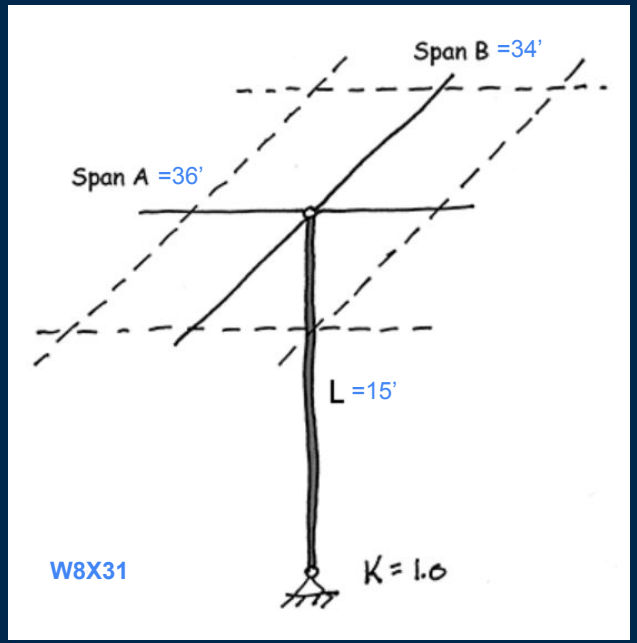
HW #6: Steel Column Analysis



given

1. Tributary Area = Span A × Span B
= 36' (34')
= 1,224 ft²

Total Floor Deadload = DL × tributary area
= 42 × 1,224
= 51,408 × 1/1000 ← convert!
= 51.408 kips ← #1

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HW #6: Steel Column Analysis

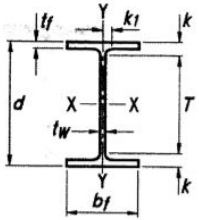

Table 1-1 (continued)
W-Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		$\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k		Distance k_1	Distance T	Distance Work-able Gage
											k_{des}	k_{det}			
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W8×67	19.7	9.00	9	0.570	9/16	5/16	8.28	8 1/4	0.935	15/16	1.33	1 5/8	15/16	5 3/4	5 1/2
×58	17.1	8.75	8 3/4	0.510	1/2	1/4	8.22	8 1/4	0.810	13/16	1.20	1 1/2	7/8		
×48	14.1	8.50	8 1/2	0.400	3/8	3/16	8.11	8 1/8	0.685	11/16	1.08	1 3/8	13/16		
×40	11.7	8.25	8 1/4	0.360	3/8	3/16	8.07	8 1/8	0.560	9/16	0.954	1 1/4	13/16		
×35	10.3	8.12	8 1/8	0.310	5/16	3/16	8.02	8	0.495	1/2	0.889	1 3/16	13/16		
×31[f]	9.13	8.00	8	0.285	5/16	3/16	8.00	8	0.435	7/16	0.829	1 1/8	3/4		

Table 1-1 (continued)
W-Shapes
Properties

W8-W4

Nominal Wt.	Compact Section Criteria $\frac{b_f}{2t_f}$	Compact Section Criteria $\frac{h}{t_w}$	Axis X-X I	Axis X-X S	Axis X-X r	Axis X-X Z	Axis Y-Y I	Axis Y-Y S	Axis Y-Y r	Axis Y-Y Z	r_{ts}	h_o	Torsional Properties J	Torsional Properties C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
67	4.43	11.1	272	60.4	3.72	70.1	88.6	21.4	2.12	32.7	2.43	8.07	5.05	1440
58	5.07	12.4	228	52.0	3.65	59.8	75.1	18.3	2.10	27.9	2.39	7.94	3.33	1180
48	5.92	15.9	184	43.2	3.61	49.0	60.9	15.0	2.08	22.9	2.35	7.82	1.96	931
40	7.21	17.6	146	35.5	3.53	39.8	49.1	12.2	2.04	18.5	2.31	7.69	1.12	726
35	8.10	20.5	127	31.2	3.51	34.7	42.6	10.6	2.03	16.1	2.28	7.63	0.769	619
31	9.19	22.3	110	27.5	3.47	30.4	37.1	9.27	2.02	14.1	2.26	7.57	0.536	530

My column: **W8X31**
Find r_x and r_y

Note A for later

$r_x = 3.47$

$r_y = 2.02$

$A = 9.13 in^2$

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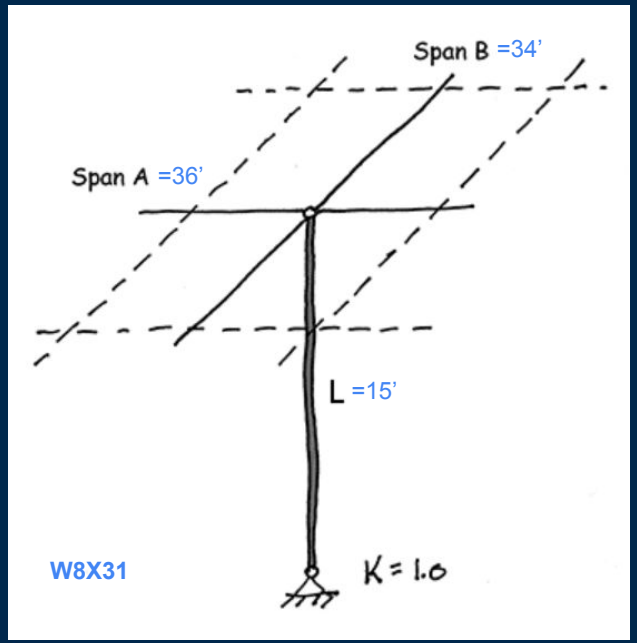
HW #6: Steel Column Analysis



2. Slenderness ratio

$\lambda_x = \frac{KL}{r_x} = \frac{1(15' \times 12)}{3.47} = 51.87 < 200 \checkmark$

x-x: K=1 given; L=15' convert to in; r_x Table 1-1

y-y 89.11 > x-x 51.87

$\lambda_y = \frac{KL}{r_y} = \frac{1(15' \times 12)}{2.02} = 89.11 < 200 \checkmark$ ← #2

y-y: K=1 given; L=15' convert to in; r_y Table 1-1

3. Transition slenderness ratio

$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000}{50}} = 113.43$ ← #3

E, F_y given

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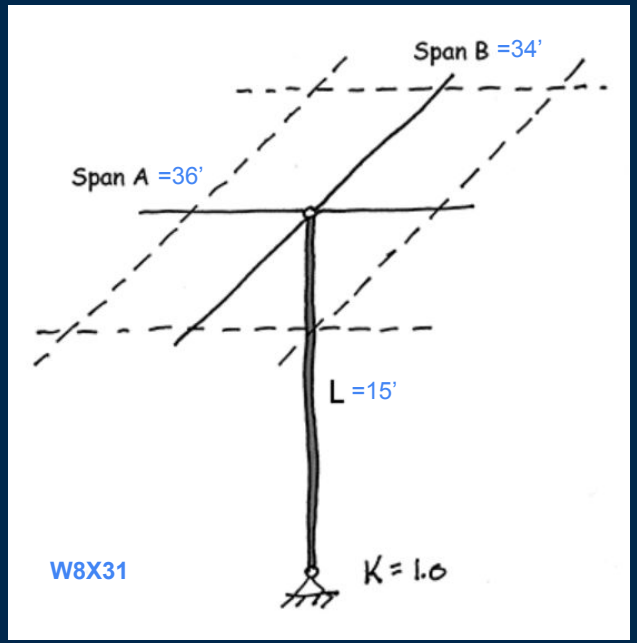
HW #6: Steel Column Analysis



4. Short or long column?

Transition s.r. vs s.r.

#3 – 113.43 > 89.11 – #2

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 (29{,}000)}{\left(\frac{1 \times 15 \times 12}{2.02}\right)^2} = \boxed{36.05 \text{ ksi}} \leftarrow \#4$$

use controlling r from #2

convert

5. Critical Stress

short column = $\left[0.658^{\frac{F_y}{F_e}}\right] F_y$

long column = $0.877 F_e$

$\left(0.658^{\frac{50}{36.05}}\right) 50 = \boxed{29.98 \text{ ksi}} \leftarrow \#5$

$= 0.5596$

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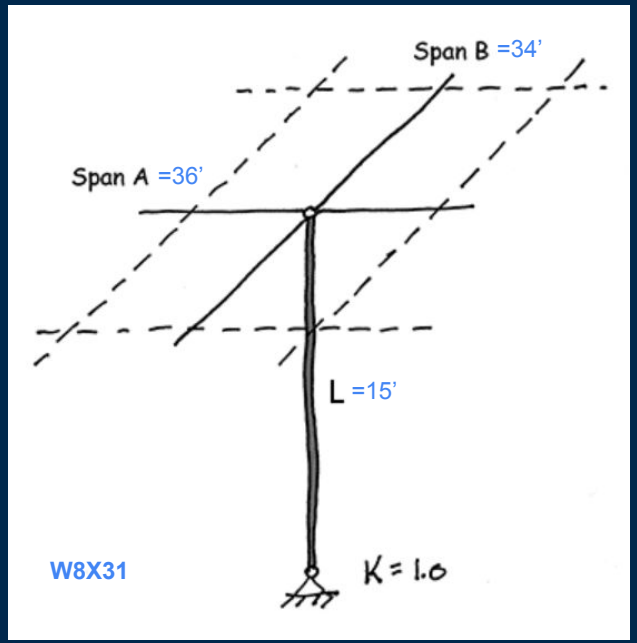
HW #6: Steel Column Analysis



6. Nominal strength

$P_n = F_{cr} \times A_g = 27.98\ (9.13) = \boxed{255.46\ \text{Kips}}$ ← #6

#5 ↑ ksi ↑ in^2 ↑ Table 1-1

7. Factored nominal strength

$\phi P_n = 0.9\ (255.46) = \boxed{229.91\ \text{Kips}}$ ← #7

↑ #6

Table 1-1 (continued)

W-Shapes

Dimensions

Shape	Area, A	Depth, d	Web Thickness, t_w	Web $\frac{t_w}{2}$	Flange Width, b_f	Flange Thickness, t_f	Distance k k_{des}	Distance k k_{det}	Distance k_1	Distance T	Work-able Gage
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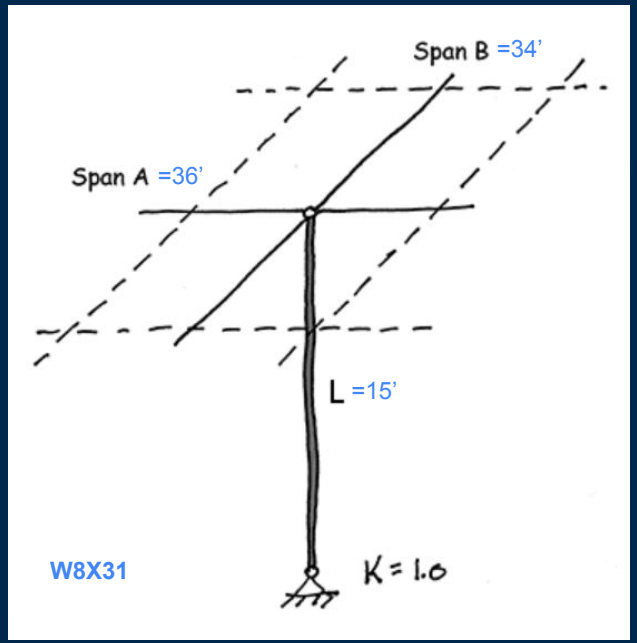

8. Un-factored live load (actual total)

$P_u = 1.2P_D + 1.6P_L$

$229.91 = 1.2(51.41) + 1.6P_L$

$229.91 = 61.692 + 1.6P_L$

$168.218 = 1.6P_L$

$P_L = \boxed{105.136 \text{ kips}}$ ← #8

9. Actual unfactored live load

$\frac{P_L}{\text{trib. area}} = \frac{105.136 \times 1000}{1,224} = \boxed{85.90 \text{ PSF}}$ ↑ #9

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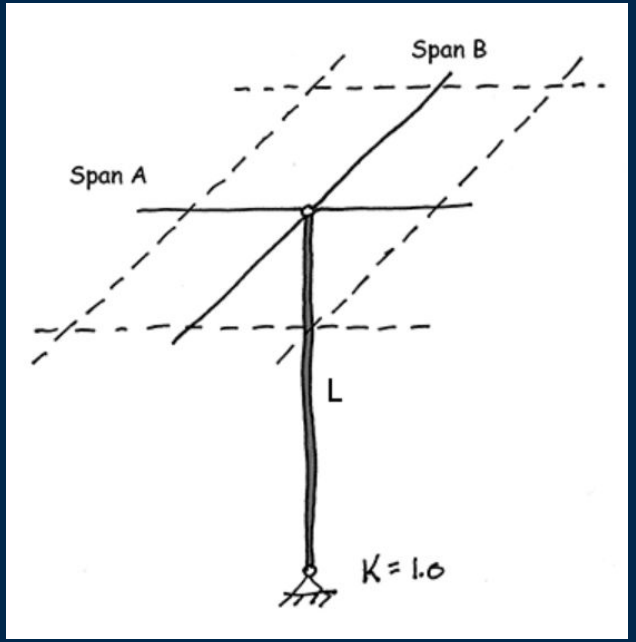

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Lab: Steel Columns

L = 13 ft. 4 in.

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