TAUBMAN COLLEGE
ARCHITECTURE & URBAN PLANNING
UNIVERSITY OF MICHIGAN

# Arch324
# STRUCTURES II

Winter 2025
Recitation

FACULTY: Prof. Peter von Bülow
Mohsen Vatandoost

# Welcome to Recitation session 02/28


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Office: Room 3122


hours:

Fri: 11:30 – 12:30

Mon, Wed: 11:00 - 12:00

walk-ins welcome!

Click here to make an appointment

Please feel free to ask questions.

# Welcome to Recitation session 02/25

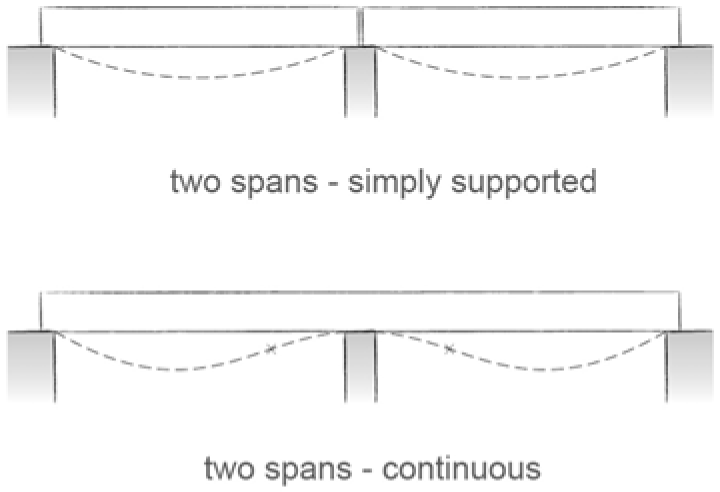
Outline:

- Quick **Recap** of the week
- Provide the solution for the assignment **(Homework 6)**
- Answering student’s questions
- Lab: **---**
- **Tower Project:** feedbacks on Preliminary report will be posted shortly.

Please feel free to ask questions.

Recap of the week

# Continuous beams




Methods for solving internal forces in Continuous beams:

- Deflection Method
- Slope Method
- Three-Moment Theorem

**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

# Recap of the week

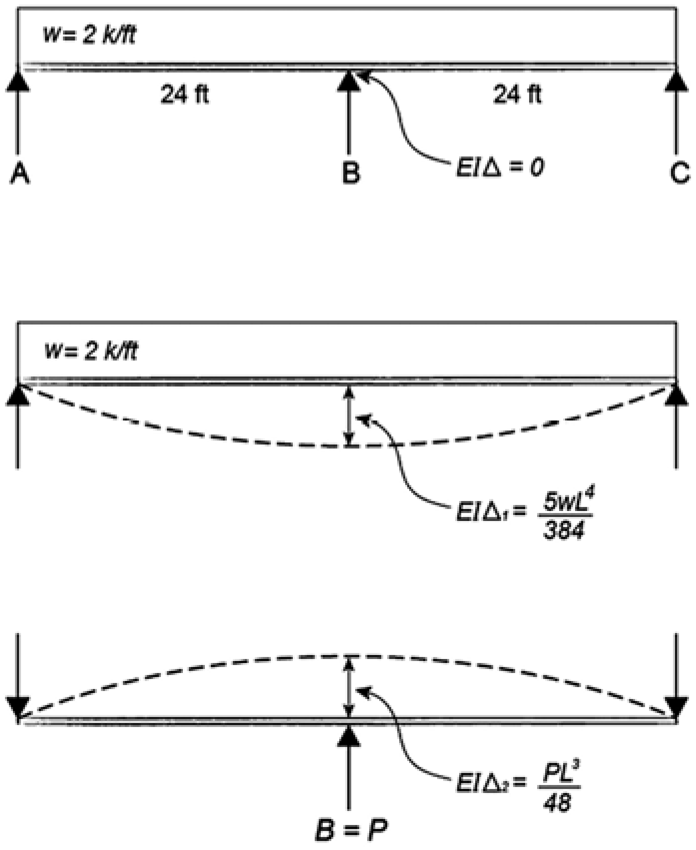
# Continuous beams

## 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.




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

# Recap of the week

# Continuous beams

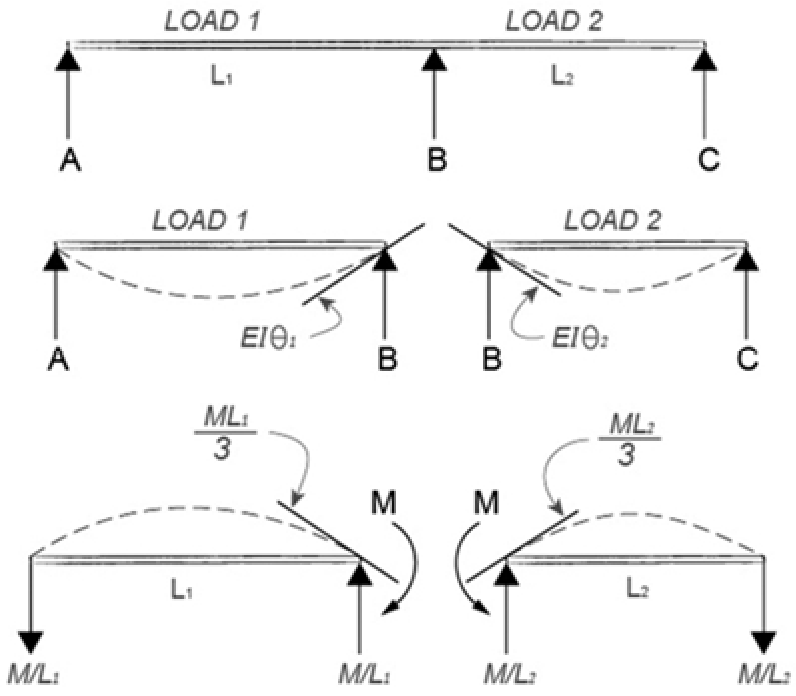
## Slope Method

- 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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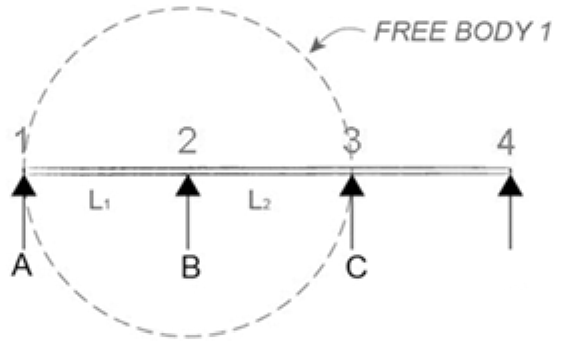
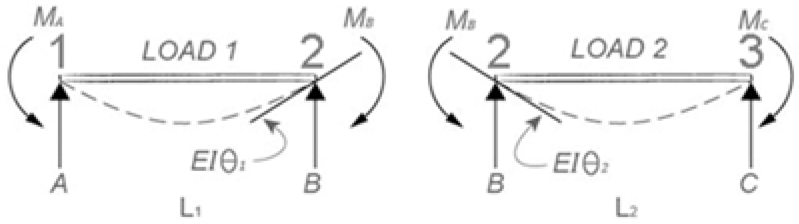
Recap of the week

# Continuous beams

## Three-Moment Theorem

- Any number of spans
- Symmetric or non-symmetric




$$M_A L_1 + 2M_B(L_1 + L_2) + M_C L_2 = 6[EI\Theta_1 + EI\Theta_2]$$

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# Provide the solution for the assignment – **HW6**

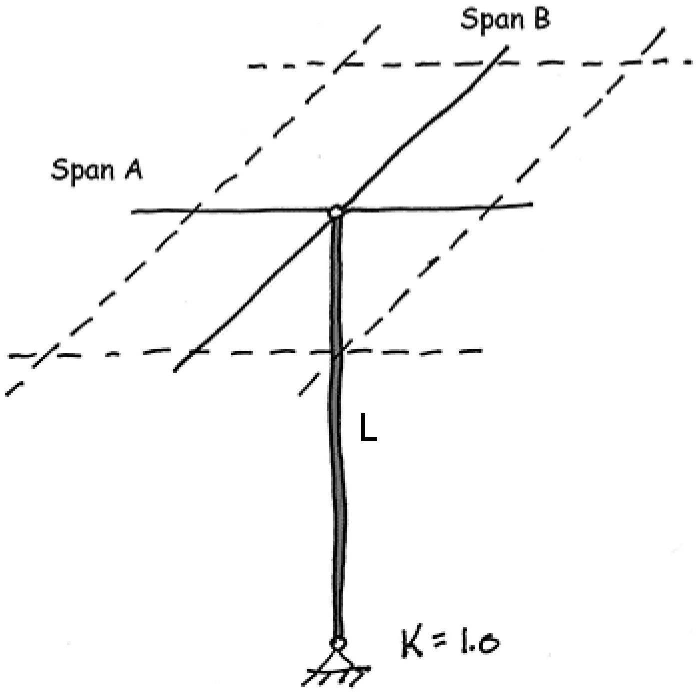
- Problem:

### 6. Steel Column Analysis

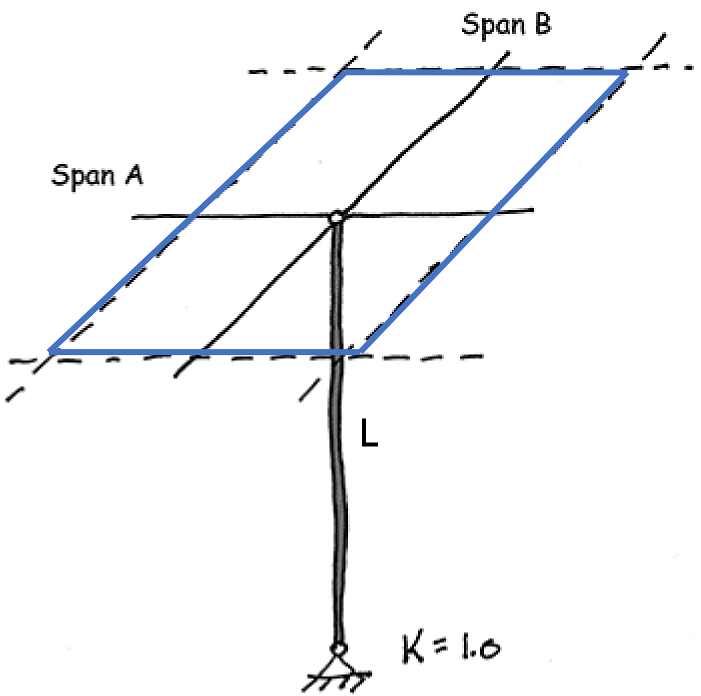
For the given axially loaded steel W-section, determine the maximum floor live load capacity, P LL. Assume the column is pinned top and bottom: K = 1.0, and there is no intermediate bracing. Use AISC-LRFD steel equations to determine phi Pn and the load. E = 29000 ksi.

DATASET: 1 -2- -3-

| | |
|---|---|
| W-section | W8X35 |
| Fy | 50 KSI |
| Span A | 34 FT |
| Span B | 46 FT |
| Height L | 17 FT |
| Floor Dead Load | 18 PSF |



# Provide the solution for the assignment – **HW6**




| # | Question | Your Response | |
|---|---|---|---|
| 1 | Total unfactored floor dead load on the column | | KIPS |
| 2 | Controlling slenderness ratio | | |
| 3 | Transition slenderness value, 4.71(E/Fy)^.5 | | |
| 4 | Euler stress, Fe | | KSI |
| 5 | Critical stress, Fcr | | KSI |
| 6 | Nominal strength, Pn | | KIPS |
| 7 | Factored nominal strength, phi Pn | | KIPS |
| 8 | UN-factored live load on column (actual total LL) | | KIPS |
| 9 | Actual unfactored floor live load | | PSF |

# Provide the solution for the assignment – **HW6**

Total unfactored floor dead load on the column

Tributary Area: Span A x Span B = 34 x 46 = 1564 ft²

Dead load: 18 psf

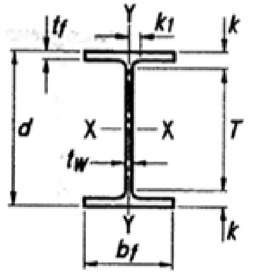
Total floor dead load: 18 x 1564 = 28152 lb x $\frac{1}{1000}$ = 28.152 Kips

Q1

□ Slenderness ratio: $\lambda = \frac{KL}{r}$

W8x35 ⟶ Table 1-1a

$r_x = 3.51''$
$r_y = 2.03''$
$A_g = 10.3\ in^2$




## 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 $k_{des}$ | k $k_{det}$ | $k_1$ | T | Workable Gage |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 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 $\frac{b_f}{2t_f}$ | $\frac{h}{t_w}$ | Axis X-X I | S | r | Z | Axis Y-Y I | S | r | Z | $r_{ts}$ | $h_o$ | Torsional Properties J | $C_w$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 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 |

# Provide the solution for the assignment – **HW6**

slenderness ratio

Effective lenght factor

$$\lambda_u = \frac{KL}{r_u} = \frac{1 \times 17 \times 12 \text{ in}}{3.51 \text{ in}} = 58.1196 < 200$$

$$\lambda_y = \frac{KL}{r_y} = \frac{1 \times 17 \times 12}{2.03} = \underline{100.49} < 200$$

Q2

maximum is governing the desig

☐ Transition slenderness value: (short or long column?)

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000 \text{ ksi}}{50 \text{ ksi}}} = 113.43$$

Q3

Long/Short column?

☐ Euler stress, $F_e$ :

$$\frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{(3.1415)^2\, 29000 \text{ ksi}}{(100.49)^2} = 28.3417 \text{ ksi}$$

Q4

# Provide the solution for the assignment – **HW6**

□ Critical Stress, $F_{cr}$ :

short column:

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y = \left[0.658^{\frac{50}{28.34}}\right] 50 = 23.8938 \text{ ksi}$$

→ 0.4778

long column: $F_{cr} = 0.877 F_e$

Q5

□ Nominal Strength, $P_n$

$$P_n = \underset{ksi}{F_{cr}} \cdot \underset{in^2}{A_g} = 23.8938 \times 10.3 = 246.10 \text{ kips}$$

Q5

$$\phi P_n = 0.9 \times 246.10 = 221.4955$$

Q6

# Provide the solution for the assignment – **HW6**

$P_u \leq \phi P_n$

$P_u = 1.2 P_D + 1.6 P_L \leq 221.4955 \longrightarrow P_L = 117.32$ kips

$P_D$ → 28.152

Q6

Actual unfactored floor live load:

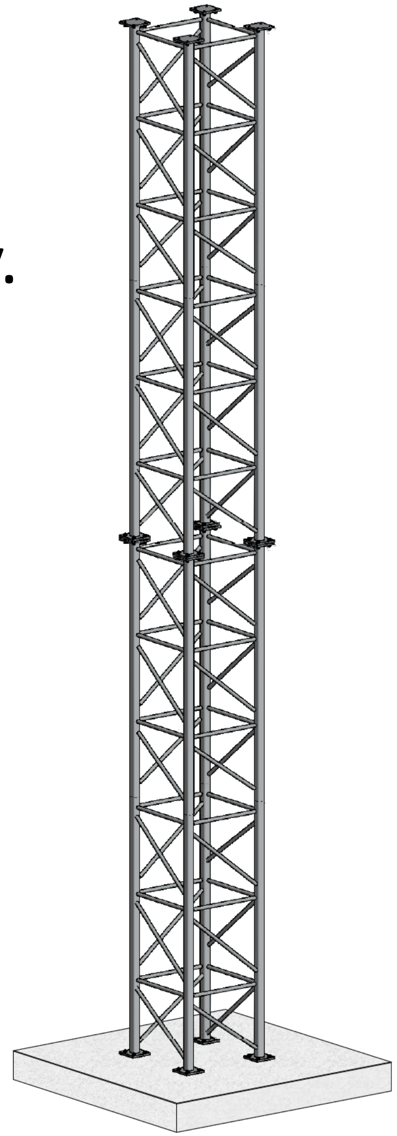
$\frac{P_L}{\text{Tributary Area}} = \frac{117.32 \times 1000 \ \text{lb}}{34 \times 46 \ \text{in}^2} = 75.012 \text{psf}$

Q7

# Tower Project: How to start

Feedbacks on Preliminary report will be posted shortly.

Tower Test : March 23

# Thank you.

# Enjoy your break!

# Any question?

Please feel free to ask questions.



Contact: Mohsenv@umich.edu