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Course Description

July 24, 2017 – Fundamentals of Stability for Steel Design: Behavior and Design of Beam-Columns

This session builds on Lecture 5, starting with an overview of the fundamental stability behavior and background to the AISC beam-column strength Eqs. H1-1a & H1-1b. Several design examples are presented highlighting (1) the important consideration of overall lateral stiffness in beam-column design and (2) the efficient application of Eqs. H1-1 using Table 6-1 of the AISC 14th Edition Manual. The discussion then focuses on the background and use of AISC Eq. H1-2 to account more realistically for the out-of-plane strength of I-section members loaded in axial compression and major-axis bending. A more advanced application of Table 6-1 based on Eq. H1-2 is presented. The session closes with an explanation and application of the C_b modifier provided in AISC Section H1.2, accounting for the beneficial effects of axial tension on I-section member LTB strength.



Learning Objectives

- Gain a broad understanding of the stability behavior of beam-column members and the technical basis for the AISC Chapter H design provisions
- Obtain simple estimates of the second-order amplification in typical building frames using lateral stiffness design criteria
- Apply AISC Manual Table 6-1 in a streamlined/efficient manner for proportioning of wide-flange section beam-columns
- Understand the basis for AISC Eq. H1-2
- Apply Eq. H1-2, with Table 6-1, to account for additional out-of-plane capacity not realized by Eqs. H1-1
- Understand the background to the C_b modifier in AISC Section H1.2
- Apply the C_b modifier in Section H1.2 to account for additional out-of-plane capacity of wide-flange members loaded in concurrent axial tension

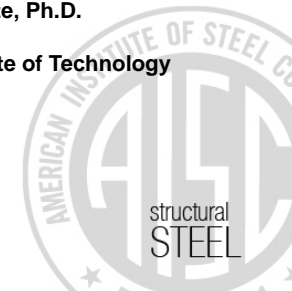


Fundamentals of Stability for Steel Design Session 6: Behavior and Design of Beam-Columns

July 24, 2017



Presented by
Donald W. White, Ph.D.
Professor
Georgia Institute of Technology



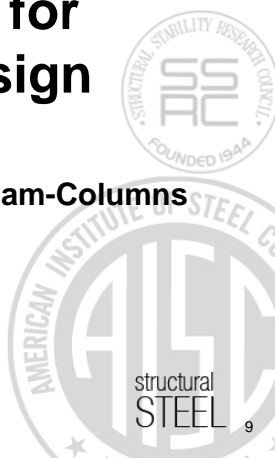
There's always a solution in steel.

There's always a solution in steel.

Fundamentals of Stability for Steel Design

Session 6 Behavior and Design of Beam-Columns

Donald W. White, Ph.D.



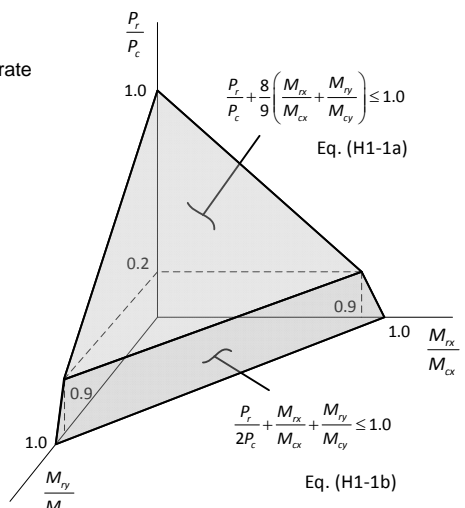
Session Outline

- **Fundamental stability behavior, key technical background & application of AISC Eqs. H1-1a & H1-1b**
 - Analysis/Demand side of the equations
 - ... design considering lateral stiffness requirements
 - Design/Capacity side of the equations
 - ... streamlined application of AISC Manual Table 6-1
- Background to and use of AISC Eq. H1-2 to account more realistically for out-of-plane strength of I-section members loaded in axial compression & major-axis bending
 - ...streamlined application of AISC Manual Table 6-1
- Modified C_b accounting for beneficial effects of concurrent axial tension on the LTB resistance of I-section members
 - ... design example

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Basic AISC Interaction Equations

Simplistic, ... but often more accurate than linear interaction, ... and practical ...



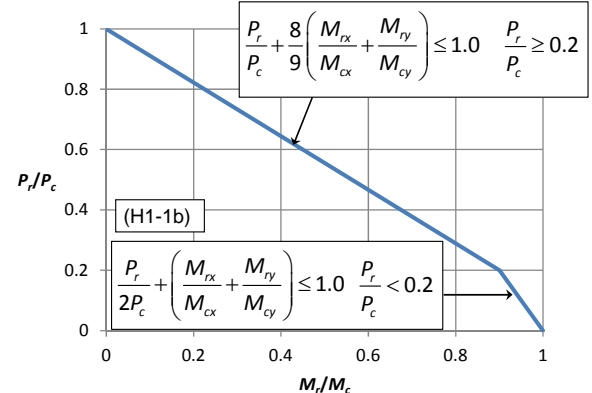
$$\frac{P_r}{P_c} + \frac{8}{9} \left(\frac{M_{rx} + M_{ry}}{M_{cx} + M_{cy}} \right) \leq 1.0$$
 Eq. (H1-1a)

$$\frac{P_r}{2P_c} + \frac{M_{rx} + M_{ry}}{M_{cx} + M_{cy}} \leq 1.0$$
 Eq. (H1-1b)

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Basic AISC Interaction Equations

AISC Sections H1.1 & H1.2: Doubly and Singly Symmetric Members in Flexure & Compression, or Flexure & Tension



$$\frac{P_r}{P_c} + \frac{8}{9} \left(\frac{M_{rx} + M_{ry}}{M_{cx} + M_{cy}} \right) \leq 1.0 \quad \frac{P_r}{P_c} \geq 0.2$$
 (H1-1a)

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx} + M_{ry}}{M_{cx} + M_{cy}} \right) \leq 1.0 \quad \frac{P_r}{P_c} < 0.2$$
 (H1-1b)

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Definition of Terms (LRFD)

$P_r = P_u =$ required axial strength (LRFD)
 $P_c = \phi_c P_n =$ design (provided) compressive strength
 $P_t = \phi_t P_n =$ design (provided) tensile strength
 $M_r = M_u =$ required flexural strength (LRFD)
 $M_c = \phi_b M_n =$ design (provided) flexural strength
 $\phi_c = \phi_b = \phi_t = 0.9$

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Calculation of P_r , M_{rx} & M_{ry} (Analysis/Demand Side of Eqs. H1-1)

- Follow AISC Chapter C requirements
- Account for significant flexural, shear & axial deformations in the structure
- Account for significant 2nd-order (P- δ & P- Δ) effects

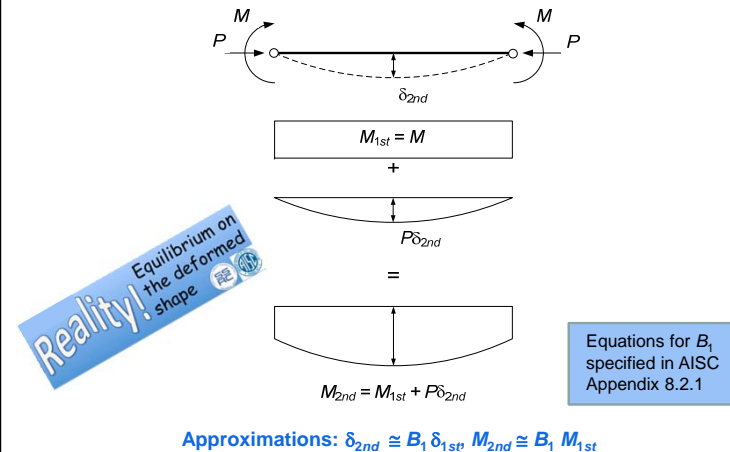
14

Calculation of P_r , M_{rx} & M_{ry}

- Follow AISC Chapter C requirements
- Account for significant flexural, shear & axial deformations in the structure
- Account for significant 2nd-order (P- δ & P- Δ) effects
- When using the Direct Analysis Method (the DM), account for:
 - Effects of geometric imperfections
 - Effects of stiffness reductions due to inelasticity
 - Uncertainty in stiffness & strength
 via stiffness reduction factors & imperfections (or notional loads) in the structural analysis

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P- δ effects



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P-Δ effects

Reality! Equilibrium on the deformed shape.

Equations for B_2 specified in AISC Appendix 8.2.2

Approximations: $\Delta_{2nd} \cong B_2 \Delta_{1st}$, $M_{2nd} \cong B_2 M_{1st}$

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~~"It's the Economy Stupid"~~ Lateral Stiffness

- As long as the structure is NOT **stability critical** (i.e., when $B_2 \leq 1.7$), the AISC Direct Analysis (DM) provisions do not require any consideration of geometric imperfections in the analysis, except for gravity-only load combinations
- For members with $P_u/P_y \leq 0.5$ (LRFD), the AISC DM provisions specify a stiffness reduction factor of $\tau_b = 1$
- Therefore, in many practical cases, the only thing "special" about the DM analysis requirements is simply the application of a stiffness reduction factor of 0.8 to all the elastic stiffnesses in the structural analysis

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Where does $N_i = 0.002 Y_i$ come from?

Notional Load N_i
= Net lateral load at Level i equivalent to the effect of the out-of-plumbness of 0.002
= $0.002 \Sigma P_i - 0.002 \Sigma P_{i+1}$
= $0.002 Y_i$

$\Delta_{\alpha(i+1)}/L_{i+1} = \Delta_{\alpha i}/L_i = 1/500 = 0.002$

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Where does $N_i = 0.002 Y_i$ come from?

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22

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 = $0.002 Y_i$

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Design Considering Lateral Stiffness Requirements

1st-Order Service Wind Drift Limit (ψ_1) Based on Holding the 2nd-Order Service Drift to ψ_2

$$\frac{(\Delta_{2nd})_s}{L} \leq \left[\psi_2 = \frac{1}{400} \right]$$

$$B_{2s} \frac{(\Delta_{1st})_s}{L} \leq \psi_2$$

$$\frac{1}{R_M P_{Lstory}} \frac{H_s}{P_{story,s}} \leq \psi_2$$

$$\frac{1}{\frac{P_{Lstory}}{H_s} \frac{P_{story,s}}{R_M H_s}} \leq \psi_2 \Rightarrow \frac{1}{\psi_2} \frac{P_{story,s}}{R_M H_s} \leq \frac{1}{\frac{P_{Lstory}}{H_s}} \Rightarrow \psi_1 \leq \frac{1}{\frac{1}{\psi_2} + \frac{P_{story,s}}{R_M H_s}} = \frac{1}{400 + \frac{P_{story,s}}{R_M H_s}}$$

B_{2s} = sidesway amplification, service wind combo

$\frac{(\Delta_{1st})_s}{L} = \frac{H_s}{P_{Lstory}}$ or $P_{Lstory} = \frac{H_s L}{(\Delta_{1st})_s}$

P_{Lstory} = story sideways stiffness

$P_{story,s}$ = story vertical load, service wind combo

H_s = story shear, service wind combo

$R_M = 1 - 0.15(P_{mf,s} / P_{story,s})$

$P_{mf,s}$ = total service wind combo vertical load in columns of the story that are part of moment frames, in the direction being considered

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Example Calculation 1st-Order Service Wind Drift Limit (ψ_1)

$P_{story,D} = 884 \text{ k}$ $P_{story,L} = 228 \text{ k}$ $P_{story,S} = 76 \text{ k}$

ASCE 7 Service Wind Load Combination:
 $P_{story,s} = 1.0 P_{story,D} + 0.5 P_{story,L} + 0.5 P_{story,S} = 1036 \text{ k}$

$P_{mf,s} = P_{story,s} = 1036 \text{ k}$ $R_M = 1 - 0.15 P_{mf,s} / P_{story,s} = 0.85$

... all columns are part of the moment frame in this example

From ASCE 7 10 Year Service Wind of 0.44 W: $H_s = 49.2 \text{ k}$

2nd-Order Wind Drift Limit of $1/\psi_2 = 1/400$

$$\psi_1 \leq \frac{1}{\frac{1}{\psi_2} + \frac{P_{story,s}}{R_M H_s}} = \frac{1}{400 + \frac{1036 \text{ k}}{0.85 \times 49.2 \text{ k}}} = \frac{1}{425}$$

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Estimating B_2 from Service Wind Drift, Seismic Drift & Seismic $P-\Delta$ Limits

Lower-bound story lateral stiffness:

$$P_{Lstory,min} = \max \left(\frac{H_s}{\psi_1}, \frac{V_x h_{sx}}{\Delta_a I_e / C_d}, \frac{P_x}{\theta_{max}} \right)$$

V_x = story seismic shear force

h_{sx} = story height

Δ_a = allowable story seismic drift

I_e = importance factor

C_d = seismic inelastic deflection factor

P_x = story unfactored vertical load

θ_{max} = max allowable stability coeff.

Upper-bound sidesway amplification (DM, taking $\tau_b = 1$):

$$B_{2,max} = \frac{1}{1 - \frac{\alpha P_{story}}{0.8 R_M P_{Lstory,min}}}$$

$\alpha = 1.0$ (LRFD), 1.6 (ASD)

P_{story} = total vertical load supported by the story using LRFD or ASD load combinations

$R_M = 1 - 0.15(P_{mf} / P_{story})$

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Example Calculation Lower-Bound Story Stiffness $P_{Lstory,min}$

From Service Wind Drift Limit: $\frac{H_s}{\psi_1} = \frac{49.2 \text{ k}}{1/425} = 20,910 \frac{\text{kip}}{\text{rad}}$

From ASCE 7 Seismic Drift Limit: $V_x = 110 \text{ k}$ $I_e = 1.0$ $\Delta_a = 0.025 h_{sx}$ $C_d = 3$

$$\frac{V_x h_{sx}}{\Delta_a I_e / C_d} = \frac{110 \text{ k}}{0.025 \times 1.0 / 3} = 13,200 \frac{\text{kip}}{\text{rad}}$$

From ASCE 7 Seismic $P-\Delta$ Limit: Assume $\theta_{max} = 0.25$ $\theta = \frac{P_x \Delta I_e}{V_x h_{sx} C_d}$

$P_x = 1.0 P_{story,D} + 0.5 P_{story,L} = 1.0 \times 884 \text{ k} + 0.5 \times 228 \text{ k} = 998 \text{ k}$

$$\frac{P_x}{\theta_{max}} = \frac{998 \text{ k}}{0.25} = 3,992 \frac{\text{kip}}{\text{rad}}$$

$$P_{Lstory,min} = \max \left(\frac{H_s}{\psi_1}, \frac{V_x h_{sx}}{\Delta_a I_e / C_d}, \frac{P_x}{\theta_{max}} \right) = 20,910 \frac{\text{kip}}{\text{rad}}$$

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Example Calculation Upper-Bound Story Sidesway Amplifier

ASCE 7 Load Combination 2, AISC Direct Analysis Method ($\tau_b = 1.0$):

$$P_{story} = 1.2P_{story,D} + 1.6P_{story,L} + 0.5P_{story,S}$$

$$= 1.2 \times 884 \text{ k} + 1.6 \times 228 \text{ k} + 0.5 \times 76 \text{ k} = 1464 \text{ k}$$

$$\alpha = 1.0$$

$$B_{2,max} = \frac{1}{1 - \frac{\alpha P_{story}}{0.8R_M P_{Lstory,min}}} = \frac{1}{1 - \frac{1.0 \times 1464 \text{ k}}{0.8 \times 0.85 \times 20,910}} = 1.12$$

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Calculation of P_n , M_{nx} & M_{ny} (Design/Capacity Side of Eqs. H1-1)

- For flexure, determine M_n using the AISC Specification Chapter F provisions
- For axial tension, determine P_n using the AISC Specification Chapter D provisions
- For axial compression, determine P_n using the AISC Specification Chapter E provisions

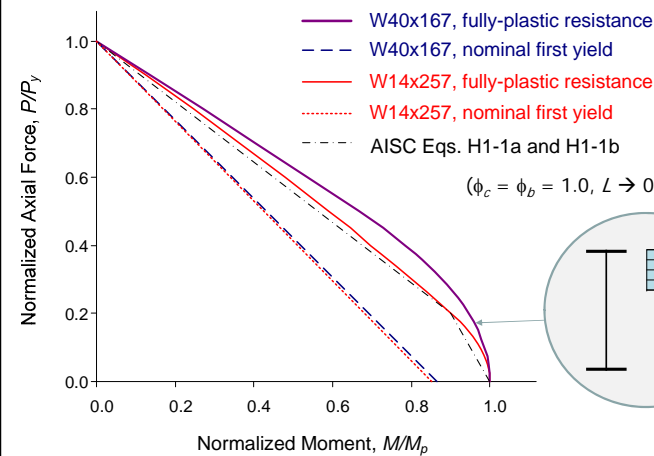
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Calculation of P_n , M_{nx} & M_{ny}

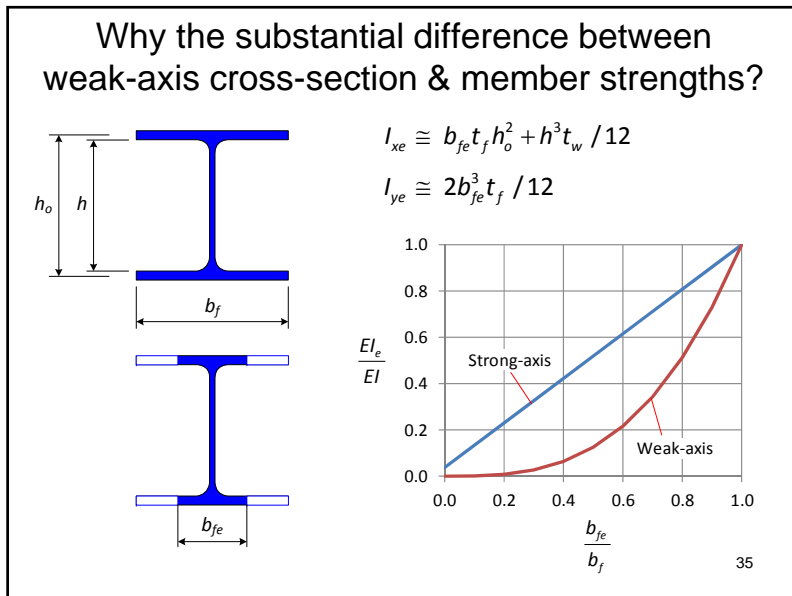
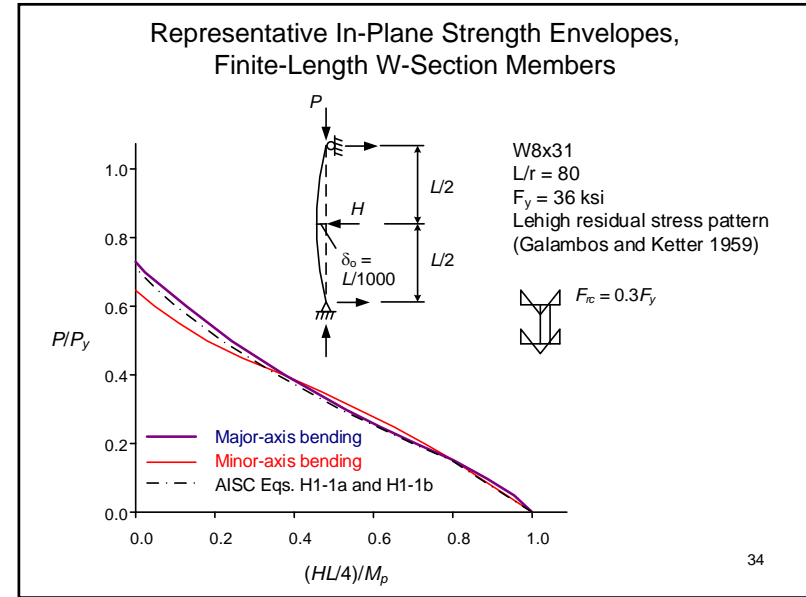
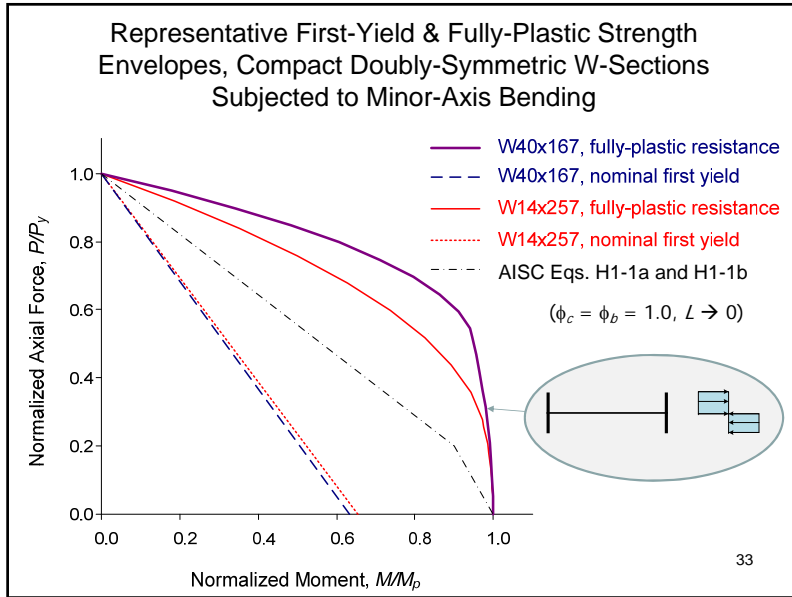
- For flexure, M_n is obtained using the AISC Specification Chapter F provisions
- For axial tension, P_n is obtained using the AISC Specification Chapter D provisions
- For axial compression, P_n is obtained using the AISC Specification Chapter E provisions
 - When using the Effective Length Method (ELM), account for geometric imperfection & stiffness reduction effects by using $KL > L$ in calculating P_n (for cases involving sidesway stability)
 - When using the DM, use $KL = L$ for routine design
 - Account for uncertainties in stiffness & strength in both the DM & the ELM via ϕ or Ω factors

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Representative First-Yield & Fully-Plastic Strength Envelopes, Compact Doubly-Symmetric W-Sections Subjected to Major-Axis Bending

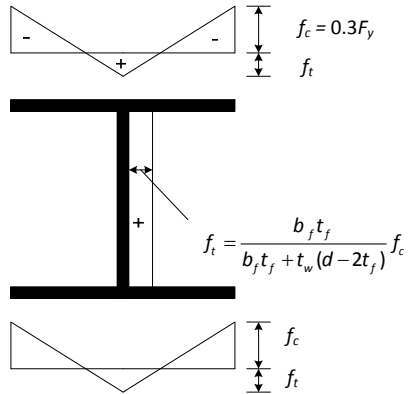


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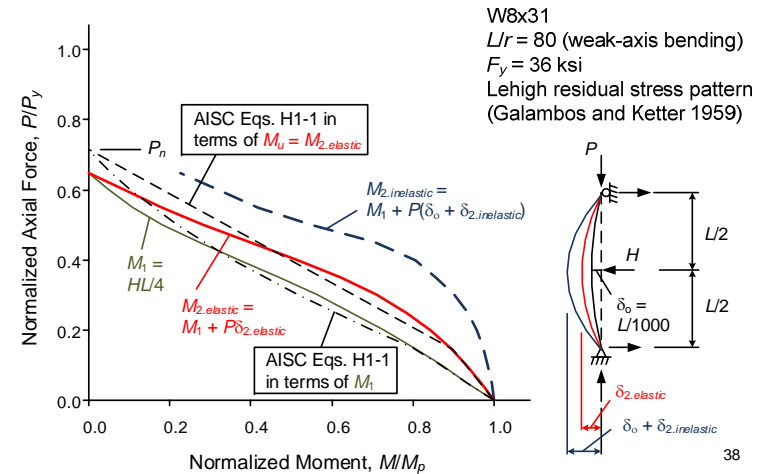
- ### Distributed Plasticity Analysis (i.e., Test Simulation)
- Basic requirements and guidelines provided by AISC Specification Appendix 1
 - Refined method of determining design capacities
 - Typical calculations include:
 - Sinusoidal out-of-straightness with a maximum amplitude of $\delta_o = L/1000$
 - Out-of-plumbness of $\Delta_o = L/500$
 - Lehigh residual stress pattern (although other patterns are more appropriate in many situations)
 - Elastic-perfectly plastic stress-strain response
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Lehigh Residual Stress Pattern



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Beam-Column Strength Envelopes in Terms of Different Moments



Equations H1-1

- Established in large part by curve fitting to the results from a large number of compact I-section beam-column solutions similar to those illustrated in the previous slide

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Equations H1-1

- Established in large part by curve fitting to the results from a large number of compact I-section beam-column solutions similar to those illustrated in the previous slide
- The **bilinear form** gives an accurate to conservative representation of the beam-column strengths plotted in terms of **P vs. $M_{2,elastic}$**

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Equations H1-1

- Established in large part by curve fitting to the results from a large number of compact I-section beam-column solutions similar to those illustrated in the previous slide
- The **bilinear form** gives an accurate to conservative representation of the beam-column strengths plotted in terms of **P vs. $M_{2,elastic}$**
- The accuracy is improved, for the cases where the design calculations under-predict $M_{2,elastic}$ the most, by using $0.8E$ in the structural analysis

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Equations H1-1 – Key Attributes

- Clear separation between:
 - Calculation of 2nd-order forces from structural analysis &
 - Calculation of member resistances

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Equations H1-1 – Key Attributes

- Clear separation between:
 - Calculation of 2nd-order forces from structural analysis &
 - Calculation of member resistances
- Combined consideration of “member strength” & “member stability”

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Equations H1-1 – Key Attributes

- Clear separation between:
 - Calculation of 2nd-order forces from structural analysis &
 - Calculation of member resistances
- Combined consideration of “member strength” & “member stability”
- For $0 \leq L/r < 100$, the bilinear form gives a “superb” fit for W-sections in major-axis bending

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Equations H1-1 – Key Attributes

- Clear separation between:
 - Calculation of 2nd-order forces from structural analysis &
 - Calculation of member resistances
- Combined consideration of “member strength” & “member stability”
- For $0 \leq L/r < 100$, the bilinear form gives a “superb” fit for W-sections in major-axis bending
- For $L/r < 40$, the bilinear form tends to be measurably conservative for W-sections in minor-axis bending

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Equations H1-1 – Key Attributes

- Clear separation between:
 - Calculation of 2nd-order forces from structural analysis &
 - Calculation of member resistances
- Combined consideration of “member strength” & “member stability”
- For $0 \leq L/r < 100$, the bilinear form gives a “superb” fit for W-sections in major-axis bending
- For $L/r < 40$, the bilinear form tends to be measurably conservative for W-sections in minor-axis bending
- For $L/r > 120$, the bilinear form is moderately conservative for both axes

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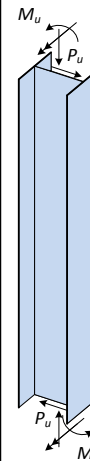
Design Example – Axial Compression & Bending of a W-Section Beam-Column



- Sway-Frame Column
- Story height = 13.5 ft
- Preliminary P_u & M_{ux} determined using
 - Basic gravity load takedowns
 - Portal frame analysis for lateral load
 - Use of wind & seismic drift limits & stiffness reduction of $0.8E$ in determining B_1 & B_2
 - Second-order service wind drift limit = $1/400$
- Design by Direct Analysis Method ... based solely on in-plane resistance

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Required Strengths (LRFD) AISC Specification Appendix 8



$$P_u = P_{nt} + B_2 P_{lt} \quad (\text{kips})$$

$$M_u = 1.0 M_{nt} + B_2 M_{lt} \quad (\text{kip-ft})$$

Use $B_1 = 1$ for preliminary design of typical moment frame columns

Combo	P_{nt}	B_{2x}	P_{lt}	P_u	M_{ntx}	M_{ltx}	M_{ux}
LC2	91.5	1.12	0.12	91.6	124	1.2	125
LC4	75.8	1.09	6.2	82.6	89.5	47.6	141

B_2 factors calculated as shown in previous example (preliminary design)

Select the lightest W14 section that will work

Design is via the Direct Analysis Method, so
 $KL_x = KL_y = KL_z = L_b = 13.5$ ft

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Table 6-1 of AISC Manual

Equations H1-1 (LRFD): For $\frac{P_u}{\phi_c P_n} \geq 0.2$ $\frac{P_u}{\phi_c P_n} + \frac{8}{9} \frac{M_u}{\phi_c M_n} \leq 1$ (H1-1a)
For $\frac{P_u}{\phi_c P_n} < 0.2$ $\frac{P_u}{2\phi_c P_n} + \frac{M_u}{\phi_c M_n} \leq 1$ (H1-1b)

Equations H1-1, alternate format (LRFD):

For $P_u/\phi_c P_n \geq 0.2$ $\frac{pP_u}{1000} + \frac{b_x M_{ux}}{1000} \leq 1$ (H1-1a)

where $p = \frac{1000}{\phi_c P_n}$ & $b_x = \frac{8000}{9\phi_b M_n}$

For $P_u/\phi_c P_n < 0.2$ $\frac{p}{2000} P_u + \frac{9b_x}{8000} M_{ux} \leq 1$ (H1-1b)

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Table 6-1 (continued)
Combined Flexure
and Axial Force
W-Shapes



Effective length, KL (ft), with respect to least radius of gyration, r_y , or Unbraced Length, L_b (ft), for X-X axis bending

$F_y = 50$ ksi

Shape	W14x											
	43 ^c				38 ^c				34 ^c			
	$p \times 10^3$ (kips) ⁻¹		$b_x \times 10^3$ (kip-ft) ⁻¹		$p \times 10^3$ (kips) ⁻¹		$b_x \times 10^3$ (kip-ft) ⁻¹		$p \times 10^3$ (kips) ⁻¹		$b_x \times 10^3$ (kip-ft) ⁻¹	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	2.68	1.78	5.12	3.41	3.06	2.04	5.79	3.85	3.50	2.33	6.53	4.34
6	2.95	1.96	5.12	3.41	3.51	2.34	5.90	3.93	4.02	2.67	6.67	4.44
7	3.06	2.04	5.17	3.44	3.70	2.46	6.12	4.07	4.23	2.81	6.94	4.61
8	3.20	2.13	5.31	3.54	3.95	2.63	6.36	4.23	4.49	2.99	7.22	4.80
9	3.37	2.24	5.47	3.64	4.25	2.83	6.61	4.40	4.81	3.20	7.53	5.01
10	3.56	2.37	5.64	3.75	4.62	3.08	6.89	4.58	5.24	3.48	7.87	5.23
11	3.79	2.52	5.82	3.87	5.07	3.37	7.19	4.78	5.76	3.83	8.24	5.48
12	4.05	2.70	6.01	4.00	5.61	3.73	7.52	5.00	6.38	4.25	8.64	5.75
13	4.36	2.90	6.21	4.13	6.25	4.16	7.88	5.24	7.14	4.75	9.09	6.05
14	4.72	3.14	6.42	4.27	7.04	4.68	8.27	5.50	8.07	5.37	9.58	6.37
15	5.15	3.42	6.66	4.43	8.01	5.33	8.71	5.80	9.21	6.13	10.1	6.74

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Determining "Required Resistances"

- 1) Assuming Eq. (H1-1a) governs & P_r/P_c dominates the Unity Check ... estimate b_x & solve for p_{req}

$$p_{req} \leq \left(1 - \frac{b_x M_{ux}}{1000}\right) \frac{1000}{P_u}$$

- 2) Assuming Eq. (H1-1b) governs ... estimate p & solve for $b_{x,req}$

$$b_{x,req} \leq \left(1 - \frac{pP_u}{2000}\right) \frac{8000}{9M_{ux}}$$

- 3) Assuming Eq. (H1-1a) governs & M_r/M_c dominates the Unity Check ... estimate p & solve for $b_{x,req}$

$$b_{x,req} \leq \left(1 - \frac{pP_u}{1000}\right) \frac{1000}{M_{ux}}$$

51

Where to start?

- Your best SWAG (Scientific Wild Angled Guess)
- For our case, let's guess that LC4 & Eq. (H1-1a) governs, & that M_r/M_c dominates the Unity Check
- Let's SWAG a value of $pP_u/1000 = 0.3$

$$b_{x,req} \leq (1 - 0.3) \frac{1000}{141} = 5.0$$

- Select a section for in-plane strength using $L_b = 0$ & $KL = KL_x / (r_x / r_y)$... we'll check out-of-plane strength later
- Note: C_b is typically large in moment frame columns, so the "plateau strength" is likely to govern the flexural resistance
∴ even if checking out-of-plane strength w/ Eq. (H1-1a), select b_x based on $L_b = 0$

52



Effective length, KL (ft), with respect to least radius of gyration, r_y , or Unbraced Length, L_y (ft), for X-X axis bending

Shape	W14x											
	43°				38°				34°			
	$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
Design	(kips) ⁻¹		(kip-ft) ⁻¹		(kips) ⁻¹		(kip-ft) ⁻¹		(kips) ⁻¹		(kip-ft) ⁻¹	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	2.68	1.78	5.12	3.41	3.06	2.04	5.79	3.85	3.50	2.33	6.53	4.34
6	2.95	1.96	5.12	3.41	3.51	2.34	5.90	3.93	4.02	2.67	6.67	4.44
7	3.06	2.04	5.17	3.44	3.70	2.46	6.12	4.07	4.23	2.81	6.94	4.61
8	3.20	2.13	5.31	3.54	3.95	2.63	6.36	4.23	4.49	2.99	7.22	4.80
9	3.37	2.24	5.47	3.64	4.25	2.83	6.61	4.40	4.81	3.20	7.53	5.01
10	3.56	2.37	5.64	3.75	4.62	3.08	6.89	4.58	5.24	3.48	7.87	5.23
$b_y \times 10^3$, (kip-ft) ⁻¹	20.6		13.7		29.4		19.6		33.6		22.4	
$t_y \times 10^3$, (kips) ⁻¹	2.65		1.76		2.98		1.98		3.34		2.22	
$t_r \times 10^3$, (kips) ⁻¹	3.26		2.17		3.66		2.44		4.10		2.74	
r_x/r_y	3.08				3.79				3.81			
r_y , in.	1.89				1.55				1.53			

Shape is slender for compression with $F_y = 50$ ksi. ← Tabulated values reflect this!
 Note: Heavy line indicates KL/r_y equal to or greater than 200.

$KL_x / (r_x / r_y) = 13.5 / 3.81 = 3.5$ ft $p_x = 2.33 + (2.67 - 2.33)(3.5/6) = 2.53$

$b_x \leq \left(1 - \frac{p_x P_u}{1000}\right) \frac{1000}{M_{ux}} = \left(1 - \frac{2.53 \times 82.6}{1000}\right) \frac{1000}{141} = (1 - 0.209) \frac{1000}{141} = 5.6$ 53

Effective length, KL (ft), with respect to least radius of gyration, r_y , or Unbraced Length, L_y (ft), for X-X axis bending

Shape	W14x											
	30°				26°				22°			
	$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
Design	(kips) ⁻¹		(kip-ft) ⁻¹		(kips) ⁻¹		(kip-ft) ⁻¹		(kips) ⁻¹		(kip-ft) ⁻¹	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	4.02	2.68	7.53	5.01	4.73	3.15	8.86	5.90	5.82	3.87	10.7	7.14
6	4.63	3.08	7.76	5.16	6.18	4.11	10.0	6.67	7.65	5.09	12.4	8.24
7	4.89	3.25	8.09	5.38	6.85	4.56	10.7	7.10	8.52	5.67	13.3	8.83
8	5.20	3.46	8.44	5.62	7.75	5.16	11.4	7.59	9.70	6.45	14.3	9.51
9	5.59	3.72	8.83	5.88	9.02	6.00	12.3	8.15	11.3	7.54	15.5	10.3
10	6.07	4.04	9.26	6.16	10.7	7.13	13.2	8.80	13.6	9.08	16.9	11.2
$b_y \times 10^3$, (kip-ft) ⁻¹	39.6		26.4		64.3		42.8		81.2		54.0	
$t_y \times 10^3$, (kips) ⁻¹	3.77		2.51		4.34		2.89		5.15		3.42	
$t_r \times 10^3$, (kips) ⁻¹	4.64		3.09		5.33		3.56		6.32		4.21	
r_x/r_y	3.85				5.23				5.33			
r_y , in.	1.49				1.08				1.04			

Shape is slender for compression with $F_y = 50$ ksi.
 Note: Heavy line indicates KL/r_y equal to or greater than 200.

$KL_x / (r_x / r_y) = 13.5 / 3.85 = 3.5$ ft $p_x = 2.68 + (3.08 - 2.68)(3.5/6) = 2.91$

$b_x \leq \left(1 - \frac{p_x P_u}{1000}\right) \frac{1000}{M_{ux}} = \left(1 - \frac{2.91 \times 82.6}{1000}\right) \frac{1000}{141} = (1 - 0.240) \frac{1000}{141} = 5.4$ 54

The W14x30 works for LC4 Check LC2

- $p_x = 2.91$, $b_x = 5.01$
- $P_u = 91.6$ k, $M_u = 125$ kip-ft

$$\frac{p_x P_u}{1000} = \frac{2.91 \times 91.6}{1000} = 0.267$$

$$\frac{p_x P_u}{1000} + \frac{b_x M_u}{1000} = 0.267 + \frac{5.01 \times 125}{1000} = 0.89 \quad \text{OK} \quad \text{Eq. (H1-1a)}$$

... the structural analysis should be repeated and the members re-checked, given these member sizes

Polling Question

True or False:

AISC Eqs. H1-1 give the maximum second-order inelastic internal moments attained in a member at its ultimate strength condition

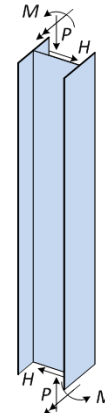


Session Outline

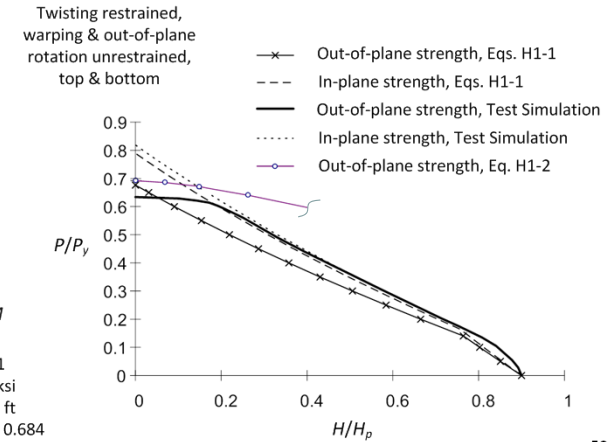
- Fundamental stability behavior, key technical background & application of AISC Eqs. H1-1a & H1-1b
 - Analysis/Demand side of the equations
 - ... design considering lateral stiffness requirements
 - Design/Capacity side of the equations
 - ... streamlined application of AISC Manual Table 6-1
- Background to and use of AISC Eq. H1-2 to account more realistically for out-of-plane strength of I-section members loaded in axial compression & major-axis bending**
 - ...streamlined application of AISC Manual Table 6-1
- Modified C_b accounting for beneficial effects of concurrent axial tension on the LTB resistance of I-section members
 - ... design example

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Behavior



W8x31
 $F_y = 36$ ksi
 $L = 11.6$ ft
 $G_{top} = G_{bot} = 0.684$



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Out-of-Plane Strength, Eq. H1-2

$$\frac{P_r}{P_{cy}} \left(1.5 - 0.5 \frac{P_r}{P_{cy}} \right) + \left(\frac{M_{rx}}{C_b M_{cx}(Cb=1)} \right)^2 \leq 1.0$$

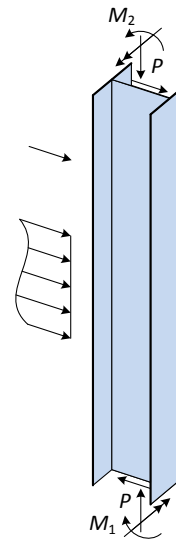
... or, expressed in terms of the flexural capacity of a member subjected to axial compression...

$$M_{rx} \leq C_b M_{cx}(Cb=1) \sqrt{1 - \frac{P_r}{P_{cy}} \left(1.5 - 0.5 \frac{P_r}{P_{cy}} \right)}$$

Applicability: Axial compression + major-axis bending of "compact" I-section members with $(KL)_z \leq (KL)_y$

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Analytical Basis



$$\frac{M_{max}^2}{C_b^2 r_o^2 P_{ey} P_{ez}} = \left(1 - \frac{P}{P_{ey}} \right) \left(1 - \frac{P}{P_{ey}} \frac{P_{ey}}{P_{ez}} \right)$$

C_b = moment gradient factor

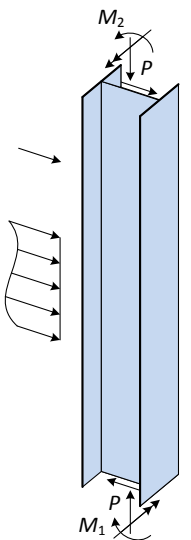
$$r_o = \sqrt{\frac{I_x + I_y}{A_g}} \quad P_{ey} = \frac{\pi^2 E I_y}{(KL_y)^2} = \frac{\pi^2 E}{(KL_y / r_y)^2} A_g$$

$$P_{ez} = \left[\frac{\pi^2 E C_w}{(KL_z)^2} + GJ \right] \frac{A_g}{I_x + I_y}$$

$$M_e = \sqrt{C_b^2 r_o^2 P_{ey} P_{ez}} = C_b \frac{\pi}{L} \sqrt{\left(\frac{\pi E}{L} \right)^2 I_y C_w + E I_y GJ}$$

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Analytical Basis



$$\left(\frac{M_{max}}{C_b M_{e(Cb=1)}} \right)^2 = \left(1 - \frac{P}{P_{ey}} \right) \left(1 - \frac{P}{P_{ey}} \frac{P_{ey}}{P_{ez}} \right) \quad \text{Eq. (A)}$$

... or ...

$$M_{max} = C_b M_{e(Cb=1)} \sqrt{\left(1 - \frac{P}{P_{ey}} \right) \left(1 - \frac{P}{P_{ey}} \frac{P_{ey}}{P_{ez}} \right)}$$

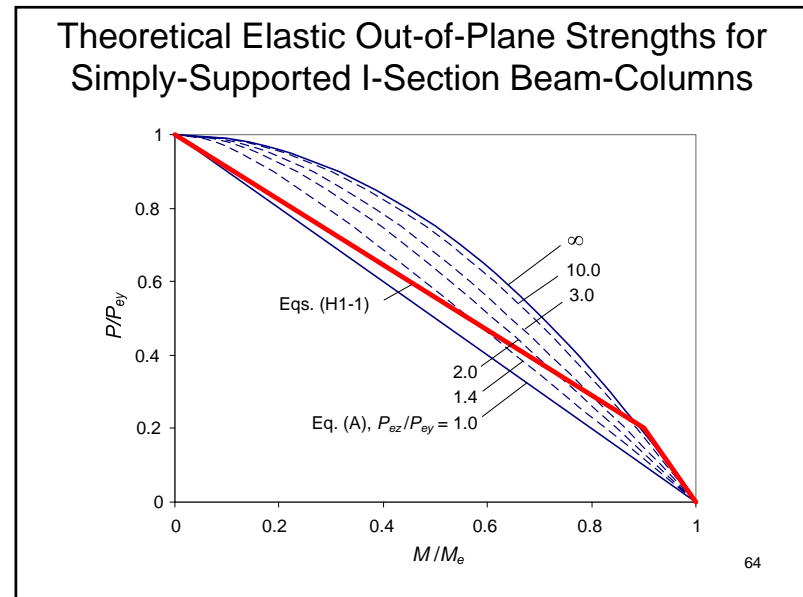
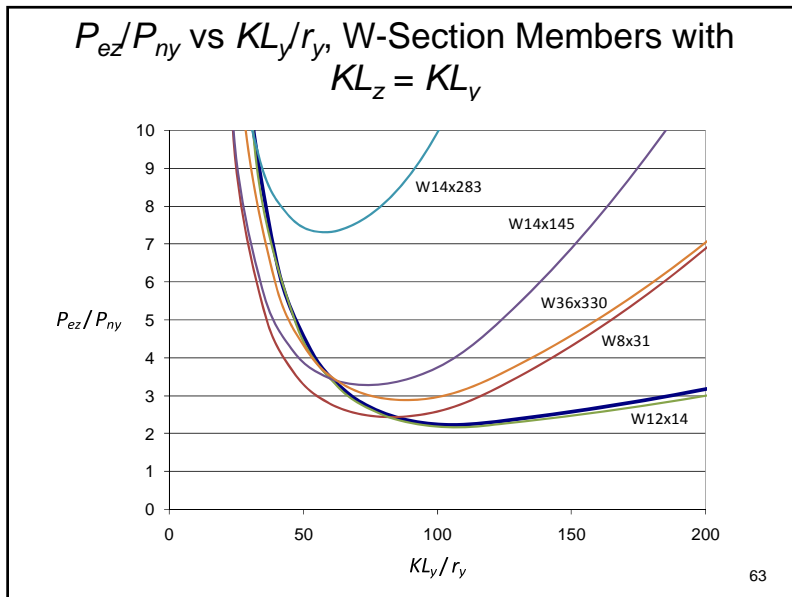
where $M_{e(Cb=1)} = \frac{\pi}{L} \sqrt{\left(\frac{\pi E}{L} \right)^2 I_y C_w + E I_y GJ}$

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“Mapping” to Nominal Strength Equation

- Replace P_{ey} by P_{ny}
- Replace $M_{e(Cb=1)}$ by $M_{nx(Cb=1)}$
- Set P_{ez} to $2.0P_{ny}$
 (lower-bound for all W-Section members with $KL_z \leq KL_y$)
- Set P_{ny} to P_{cy} & $M_{nx(Cb=1)}$ to $M_{cx(Cb=1)}$

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Out-of-Plane Strength, Eq. H1-2

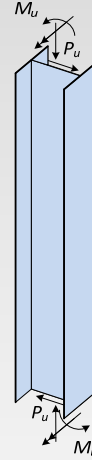
$$\frac{P_r}{P_{cy}} \left(1.5 - 0.5 \frac{P_r}{P_{cy}} \right) + \left(\frac{M_{rx}}{C_b M_{cx}(Cb=1)} \right)^2 \leq 1.0$$

... or, expressed in terms of the flexural capacity of a member subjected to axial compression...

$$M_{rx} \leq C_b M_{cx}(Cb=1) \sqrt{1 - \frac{P_r}{P_{cy}} \left(1.5 - 0.5 \frac{P_r}{P_{cy}} \right)}$$

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Design Example – Axial Compression & Bending of a W-Section Beam-Column



- Sway-Frame Column
- Story height = 13.5 ft
- Preliminary \$P_u\$ & \$M_{ux}\$ determined using
 - Basic gravity load takedowns
 - Portal frame analysis for lateral load
 - Use of wind & seismic drift limits & stiffness reduction of 0.8E in determining \$B_1\$ & \$B_2\$
 - Second-order service wind drift limit = 1/400
- Check out-of-plane resistance

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Check Out-of-Plane Resistance

- Alternate format of Eq. (H1-2)

$$\frac{p_y P_u}{1000} \left(1.5 - 0.5 \frac{p_y P_u}{1000} \right) + \left(\frac{9 b_x M_{ux}}{8000 C_b} \right)^2 \leq 1.0$$

- For typical columns in multi-story buildings (moment diagrams close to linear), use AISC Eq. (C-F1-1):

$$C_b = 1.75 + 1.05 M_1/M_2 + 0.3 (M_1/M_2)^2 \leq 2.3$$

- For our case (fully-reversed curvature bending):

$$C_b = 1.75 + 1.05 (1) + 0.3 (1)^2 = 3.1 \quad (\text{use } C_b = 2.3)$$

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Shape	W14x											
	30c				26c				22c			
	\$p \times 10^3\$		\$b_x \times 10^3\$		\$p \times 10^3\$		\$b_x \times 10^3\$		\$p \times 10^3\$		\$b_x \times 10^3\$	
Design	(kips) ⁻¹	(kip-ft) ⁻¹	(kips) ⁻¹	(kip-ft) ⁻¹	(kips) ⁻¹	(kip-ft) ⁻¹	(kips) ⁻¹	(kip-ft) ⁻¹	(kips) ⁻¹	(kip-ft) ⁻¹	(kips) ⁻¹	(kip-ft) ⁻¹
0	4.02	2.68	7.53	5.01	4.73	3.15	8.86	5.90	5.82	3.87	10.7	7.14
6	4.63	3.08	7.76	5.16	6.18	4.11	10.0	6.67	7.65	5.09	12.4	8.24
7	4.89	3.25	8.09	5.38	6.85	4.56	10.7	7.10	8.52	5.67	13.3	8.83
8	5.20	3.46	8.44	5.62	7.75	5.16	11.4	7.59	9.70	6.45	14.3	9.51
9	5.59	3.72	8.83	5.88	9.02	6.00	12.3	8.15	11.3	7.54	15.5	10.3
10	6.07	4.04	9.26	6.16	10.7	7.13	13.2	8.80	13.6	9.08	16.9	11.2
11	6.70	4.46	9.74	6.48	12.9	8.60	14.4	9.56	16.5	11.0	19.2	12.8
12	7.47	4.97	10.3	6.83	15.4	10.2	16.5	11.0	19.7	13.1	22.3	14.8
13	8.41	5.60	10.8	7.21	18.1	12.0	18.7	12.4	23.1	15.3	25.4	16.9
14	9.56	6.36	11.5	7.65	20.9	13.9	20.9	13.9	26.8	17.8	28.5	19.0

Effective length, \$K_L\$ (ft), with respect to least radius of gyration, \$r_y\$, or Unbraced Length, \$L_y\$ (ft), for X-X axis bending

\$p_y = (5.60 + 6.36) / 2 = 5.98\$ \$b_x = (7.21 + 7.65) / 2 = 7.43\$

∴ for LC4: $\frac{p_y P_u}{1000} \left(1.5 - 0.5 \frac{p_y P_u}{1000} \right) + \left(\frac{9 b_x M_{ux}}{8000 C_b} \right)^2 = \frac{5.98 \times 82.6}{1000} \left(1.5 - 0.5 \frac{5.98 \times 82.6}{1000} \right) + \left(\frac{9 \times 7.43 \times 141.4}{8000 \times 2.3} \right)^2 = 0.89 \leq 1.0$

& for LC2: $\frac{p_y P_u}{1000} \left(1.5 - 0.5 \frac{p_y P_u}{1000} \right) + \left(\frac{9 b_x M_{ux}}{8000 C_b} \right)^2 = \frac{5.98 \times 91.6}{1000} \left(1.5 - 0.5 \frac{5.98 \times 91.6}{1000} \right) + \left(\frac{9 \times 7.43 \times 125.3}{8000 \times 2.3} \right)^2 = 0.88 \leq 1.0$

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Check W14x30 Web Compactness

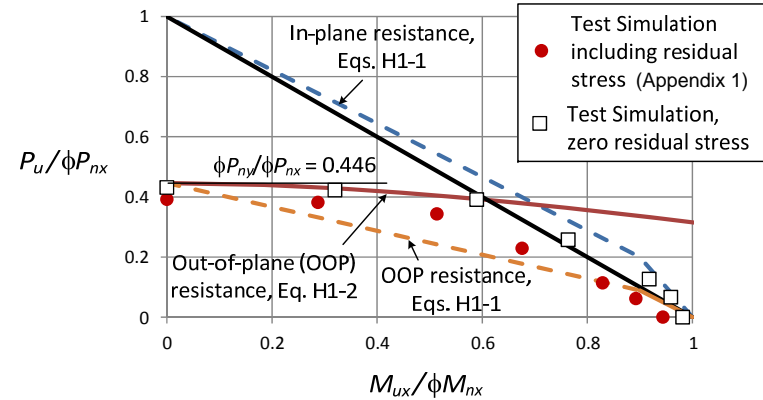
- Flange is compact since Table 6-1 does not provide a footnote about the flange compactness
- Use Appendix 1, Section 1.2.2 to check web compactness

$$P_u / \phi_c P_y = \rho_{(KL=0)} P_u / 1000 = 2.68 (91.6 \text{ k}) / 1000 = 0.245$$

$$[\lambda_w = h/t_w = 45.4] < \left[1.12 \sqrt{\frac{E}{F_y}} \left(2.33 - \frac{P_u}{\phi_c P_y} \right) = 2.34 \sqrt{\frac{E}{F_y}} = 56.2 \right] \text{ OK}$$

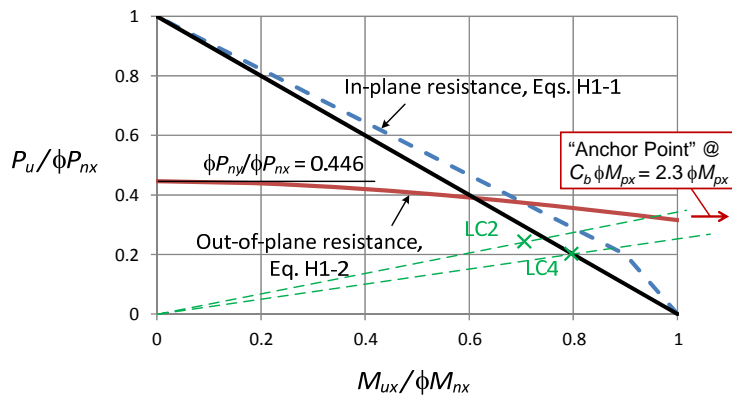
69

In-Plane & Out-of-Plane Strengths for Selected W14x30



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In-Plane & Out-of-Plane Strengths for Selected W14x30



71

Polling Question

True or False:
 For vertically oriented members in building frames, the out-of-plane member resistance from Eq. H1-2 typically governs relative to the in-plane resistance from Eqs. H1-1

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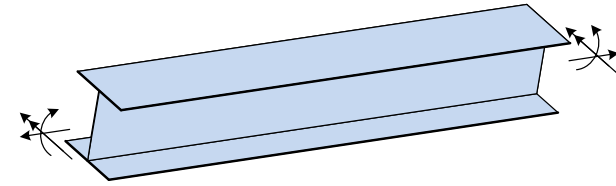


Session Outline

- Fundamental stability behavior, key technical background & application of AISC Eqs. H1-1a & H1-1b
 - Analysis/Demand side of the equations
 - ... design considering lateral stiffness requirements
 - Design/Capacity side of the equations
 - ... streamlined application of AISC Manual Table 6-1
- Background to and use of AISC Eq. H1-2 to account more realistically for out-of-plane strength of I-section members loaded in axial compression & major-axis bending
 - ...streamlined application of AISC Manual Table 6-1
- C_b modifier accounting for beneficial effects of concurrent axial tension on the LTB resistance of I-section members
 - ... design example

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Increase in M_n (LTB) due to the Presence of Axial Tension

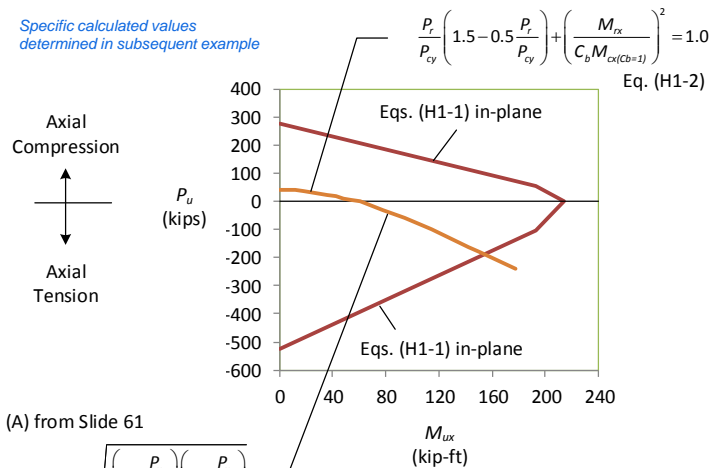


Twisting restrained, warping
 & out-of-plane rotation
 unrestrained at ends

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Mechanics Based P-M Interaction

Specific calculated values
 determined in subsequent example



Eq. (A) from Slide 61

$$M_{rx} = C_b M_{cx(Cb=1)} \sqrt{\left(1 + \frac{P_u}{P_{ey}}\right) \left(1 + \frac{P_u}{P_{ez}}\right)}$$

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Axial Tension & Bending – Derivation of AISC Modified C_b Factor

$$M_{rx} = C_b M_{cx(Cb=1)} \sqrt{\left(1 + \frac{P_u}{P_{ey}}\right) \left(1 + \frac{P_u}{P_{ez}}\right)}$$

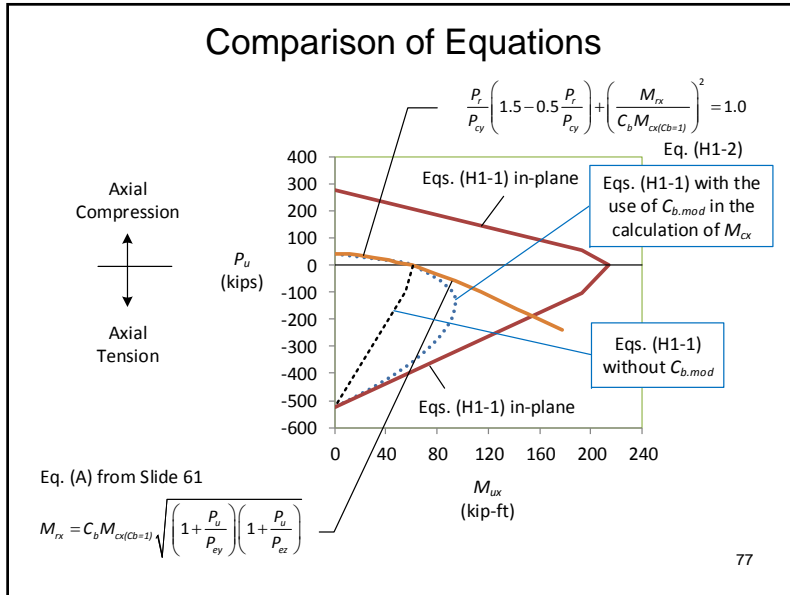
... Conservatively (for axial tension) assume P_{ez} is infinite compared to P_u

$$M_{rx} = C_b M_{cx(Cb=1)} \sqrt{\left(1 + \frac{P_u}{P_{ey}}\right)}$$

... incorporate the square root term as a modification to C_b ... but then ... "launder" this back through the bilinear strength interaction Eqs. (H1-1a) and (H1-1b)

$$C_{b,mod} = C_b \sqrt{\left(1 + \frac{P_u}{P_{ey}}\right)}$$

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Design Example - Axial Tension & Bending of a W-Section Beam-Column

- Braced W12x40 Member
- $KL_x = KL_y = KL_z = 40$ ft
- Axial tension $P_{ux} = 160$ k
- Uniform major-axis bending $M_{ux} = 100$ kip-ft
- Check the adequacy of the member
- Assume tension yielding governs for the axial tension resistance

Twisting restrained, warping & out-of-plane rotation unrestrained at ends

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Check using AISC Modified C_b & Eqs. (H1-1)

$KL_x := 40$ ft $KL_y := KL_x$ $KL_z := KL_x$ $L_b := KL_x$
 $F_y := 50$ ksi $E := 29000$ ksi
 $A_g := 11.7$ in² $\phi P_y := 0.9 \cdot F_y \cdot A_g = 526.5$ kip
 $I_y := 44.1$ in⁴ $P_{ey} := \frac{\pi^2 \cdot E \cdot I_y}{(KL_y)^2} = 54.8$ kip
 $L_r := 21.1$ ft $h_o := 11.4$ in $S_x := 51.5$ in³ $J := 0.906$ in⁴ $r_{ts} := 2.21$ in
 $X := \sqrt{\frac{S_x \cdot h_o}{J}} = 25.5$ $\phi M_{n1} := 0.9 \cdot \frac{\pi^2 \cdot E \cdot S_x}{\left(\frac{L_b}{r_{ts}} \right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{1}{X^2} \cdot \left(\frac{L_b}{r_{ts}} \right)^2} = 60.6$ kip-ft

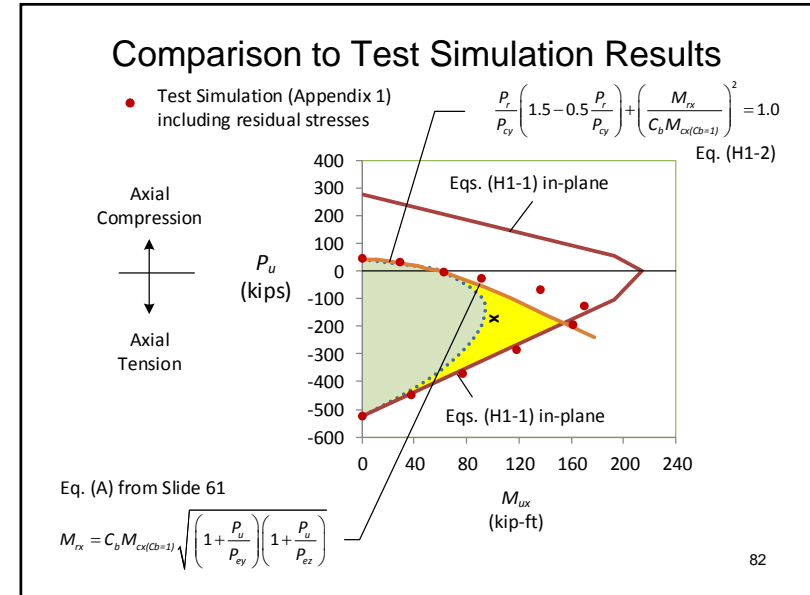
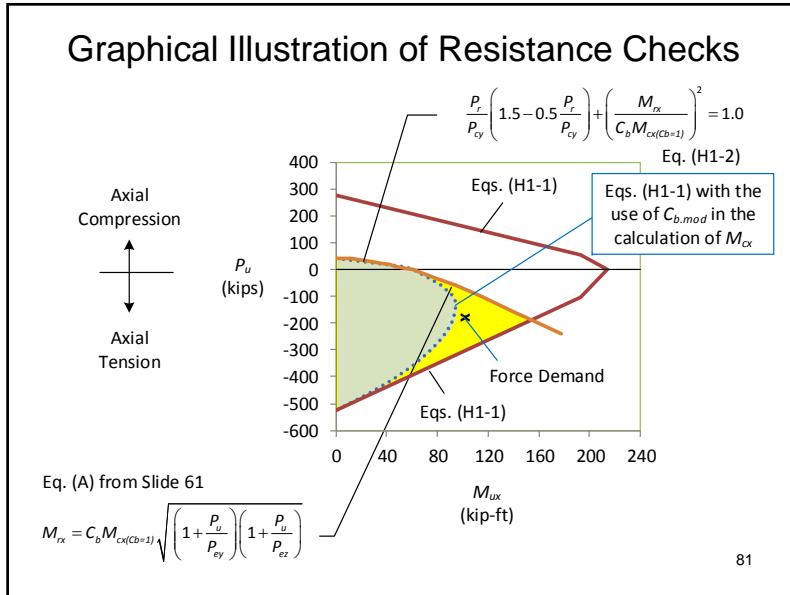
79

Check using AISC Modified C_b & Eqs. (H1-1)

$P_u := 160$ kip $M_u := 100$ kip-ft
 $C_b := \sqrt{1 + \frac{P_u}{P_{ey}}} = 1.98$ $\phi M_p := 214$ kip-ft from Table 3-2
 $\phi M_n := \min(C_b \cdot \phi M_{n1}, \phi M_p) = 120$ kip-ft
 $\frac{P_u}{\phi P_y} = 0.304$ $\frac{P_u}{\phi P_y} + \frac{8}{9} \cdot \frac{M_u}{\phi M_n} = 1.045$ NG
Almost ... but No Gravy!

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Polling Question

True or False:
Equations H1-1a and H1-1b, without the modified C_b equation, tend to be grossly conservative for evaluation of the LTB resistance of wide-flange members subjected to flexure and concurrent axial tension

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- ### Summary
- AISC Eqs. H1-1 provide an accurate to conservative estimate of the strength envelope for wide-flange members
 - The sidesway amplification factor (B_2) can be estimated readily in preliminary design by using typical lateral stiffness design criteria
 - The AISC Manual Table 6-1, when used well, nicely streamlines the design selection of wide-flange beam-columns
 - AISC Eq. H1-2 gives significantly larger out-of-plane resistances, compared to Eqs. H1-1, for typical columns in building moment frames
 - The AISC Manual Table 6-1 can be used efficiently with Eq. H1-2 by first designing for in-plane strength with Eqs. H1-1, then checking out-of-plane strength with Eq. H1-2
 - The modified C_b factor in AISC Section H1.2 recognizes the potential substantial increase in the LTB strength of wide-flange section members due to concurrent axial tension
 - AISC Appendix 1 (Design by test simulation) allows for “high-end” solutions that allow rigorous characterization of member resistances
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Up Next...

Session 7: July 31 –

Fundamental Concepts of Bracing Compression and Flexural Members

by T.A. Helwig, PE, Ph.D.

This lecture will focus on the fundamental behavior related to bracing of compression and flexural members. The dual criteria of necessary stiffness and strength will be covered. The effects of imperfections on brace forces will be addressed, along with the impact of connection flexibility and cross-sectional distortion on the effectiveness of the bracing. An overview of the different classifications of bracing including relative, nodal, continuous, and lean-on bracing will be provided.

85

Individual Webinar Registrants

CEU/PDH Certificates

Within 2 business days...

- You will receive an email on how to report attendance from: registration@aisc.org.
- Be on the lookout: Check your spam filter! Check your junk folder!
- Completely fill out online form. Don't forget to check the boxes next to each attendee's name!



Individual Webinar Registrants

CEU/PDH Certificates

Within 2 business days...

- New reporting site (URL will be provided in the forthcoming email).
- Username: Same as AISC website username.
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8-Session Registrants

CEU/PDH Certificates

One certificate will be issued at the conclusion of all 8 sessions.



8-Session Registrants

Access to the quiz: Information for accessing the quiz will be emailed to you by Wednesday. It will contain a link to access the quiz. EMAIL COMES FROM NIGHTSCHOOL@AISC.ORG

Quiz and Attendance records: Posted Tuesday mornings.
www.aisc.org/nightschool - click on Current Course Details.

Reasons for quiz:

- EEU – must take all quizzes and final to receive EEU
- CEUs/PDHS – If you watch a recorded session you must take quiz for CEUs/PDHS.
- REINFORCEMENT – Reinforce what you learned tonight. Get more out of the course.

NOTE: If you attend the live presentation, you do not have to take the quizzes to receive CEUs/PDHS.



8-Session Registrants

Access to the recording: Information for accessing the recording will be emailed to you by this Wednesday. The recording will be available for three weeks. For 8-session registrants only. EMAIL COMES FROM NIGHTSCHOOL@AISC.ORG.

CEUs/PDHS – If you watch a recorded session you must take AND PASS the quiz for CEUs/PDHS.



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Find all your handouts, quizzes and quiz scores, recording access, and attendance information all in one place!



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Course Resources

Event	Start Date
NS 13 8-Session Package-Night School 13 - Design of Industrial Buildings	1/30/2017 7:00:00 PM
NS 14 8-Session Package-Night School 14 - Fundamentals of Stability	6/5/2017 7:00:00 PM

Night School Resources for 8-session package Registrants

Navigation: EDUCATION, PUBLICATIONS, NASCC: THE STEEL CONFERENCE, SAFETY, STEEL SOLUTIONS CENTER, AWARDS AND COMPETITIONS, RESEARCH LIBRARY

Header: AISC

Breadcrumbs: AISC > MYAISC > NIGHT SCHOOL RESOURCES > NS13 8-SESSION PACKAGE RESOURCES

Night School 13: Design of Industrial Buildings

8-SESSION PACKAGE RESOURCES

Event	Date	Handouts	Video	Quiz	Attendance
NS13 - Design Criteria	1/30/2017 7:00:00 PM	Handouts	Video Procedure: NS13DSN	Pass Score: 80	Pending
NS13 - Economic Considerations	2/6/2017 7:00:00 PM	Handouts	Available 02/08/2017 5pm EST	Available 02/08/2017 5pm EST	Pending
NS13 - Lateral Load Systems and Details	2/13/2017 7:00:00 PM	Handouts	Available 02/15/2017 5pm EST	Available 02/15/2017 5pm EST	Pending
NS13 - Preliminary Design Procedures	2/27/2017 7:00:00 PM	Handouts	Available 03/01/2017 5pm EST	Available 03/01/2017 5pm EST	Pending
NS13 - Crane Order Design and Frame Analysis	3/6/2017 7:00:00 PM	Handouts	Available 03/08/2017 5pm EST	Available 03/08/2017 5pm EST	Pending
NS13 - Frame Member and Connection Design	3/13/2017 7:00:00 PM	Handouts	Available 03/15/2017 5pm EST	Available 03/15/2017 5pm EST	Pending
NS13 - Transfer Crane Order & Longitudinal Bolt Bracing Design	3/27/2017 7:00:00 PM	Handouts	Available 03/28/2017 5pm EST	Available 03/28/2017 5pm EST	Pending
NS13 - Building Envelope and Bracing Design	4/3/2017 7:00:00 PM	Handouts	Available 04/05/2017 5pm EST	Available 04/05/2017 5pm EST	Pending
NS13 - Final Exam	4/10/2017 7:00:00 PM			Available 04/12/2017 5pm EST	

Night School Resources for 8-session package Registrants

- Weekly “quiz and recording” email.
- Weekly updates of the master Quiz and Attendance record found at www.aisc.org/nightschool. Scroll down to Quiz and Attendance records.
 - Updated on Tuesday mornings.



Night School Resources for 8-session package Registrants

- Webinar connection information:
 - Found in your registration confirmation/receipt.
 - Reminder email sent out Monday mornings.
- Link to handouts also found here.



Thank You

Please give us your feedback!
Survey at conclusion of webinar.

There's always a solution in steel.

