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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.

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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

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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

structural STEEL

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Fundamentals of Stability for Steel Design

Session 6

Behavior and Design of Beam-Columns

Donald W. White, Ph.D.

structural STEEL

9

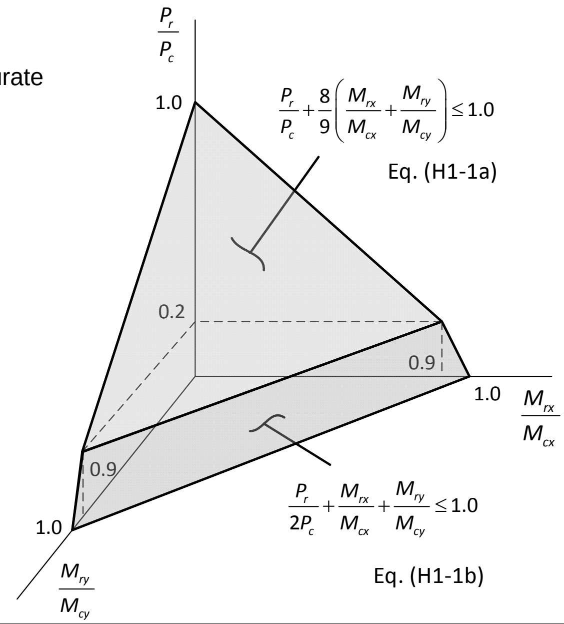
Session Outline

- **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

10

Basic AISC Interaction Equations

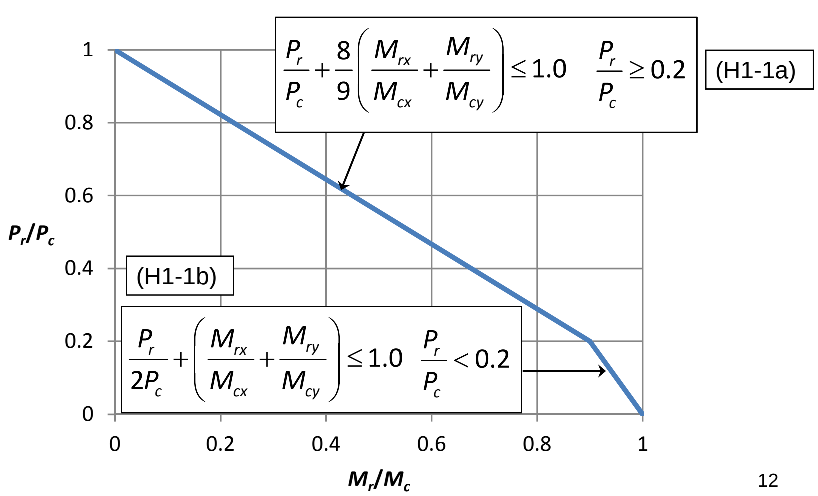
Simplistic,
… but often more accurate
than linear interaction,
… and practical …



11

Basic AISC Interaction Equations

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



12

Definition of Terms (LRFD)

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

13

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

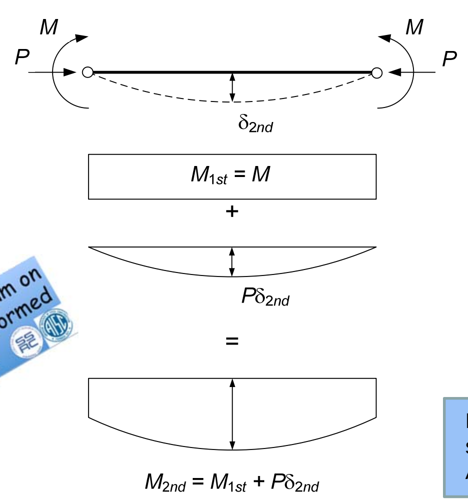
Calculation of P_r , M_{rx} & M_{ry}

- 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

15

P-δ effects

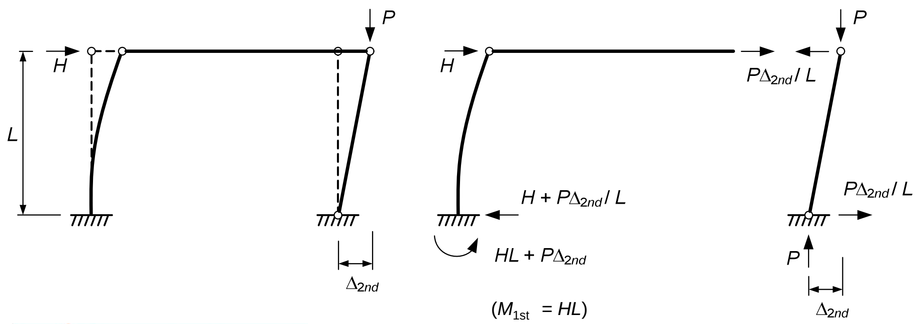


Equations for B_1 specified in AISC Appendix 8.2.1

Approximations: $\delta_{2nd} \cong B_1 \delta_{1st}$, $M_{2nd} \cong B_1 M_{1st}$

16

P-Δ effects



Equations for B_2 specified in AISC Appendix 8.2.2

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

17

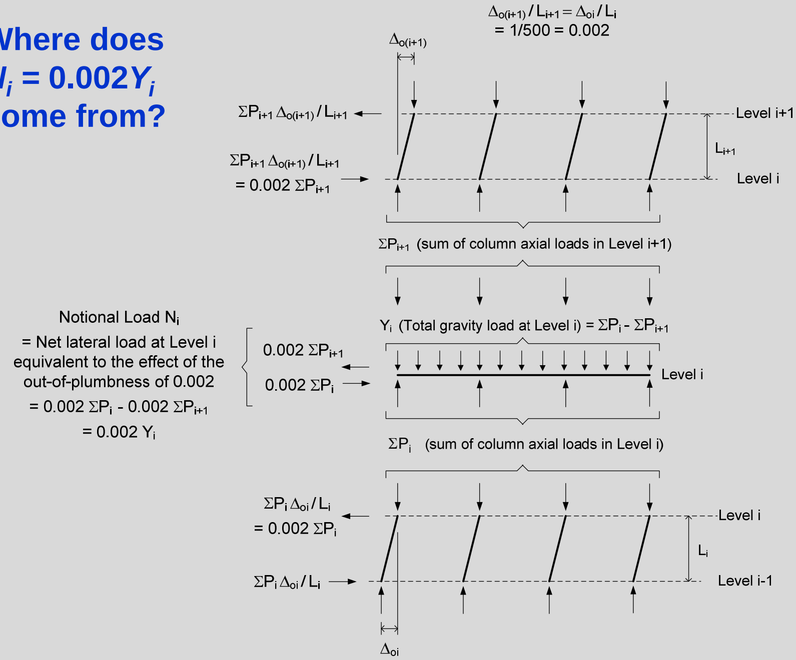
"It's the ~~Economy~~ Stupid"

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

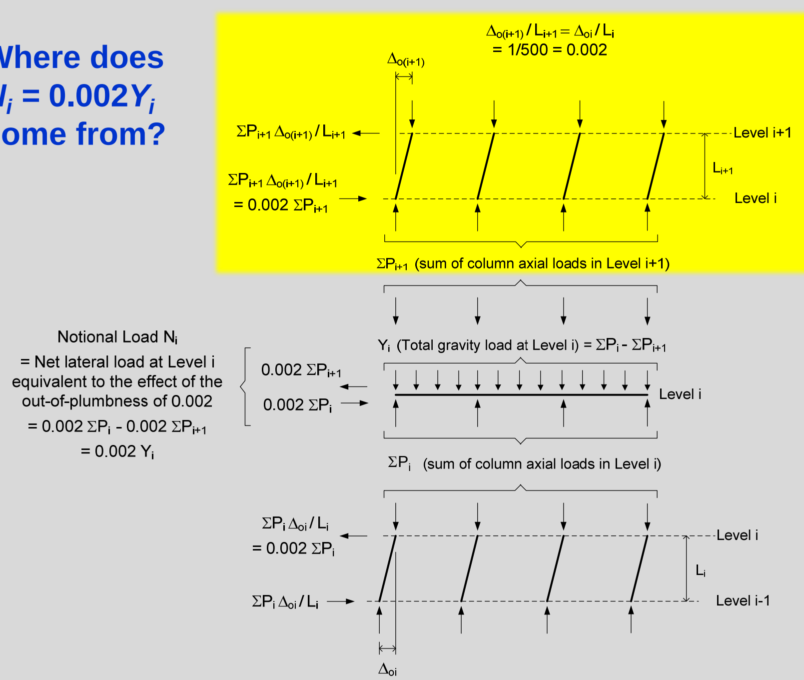
18

Where does $N_i = 0.002Y_i$ come from?



19

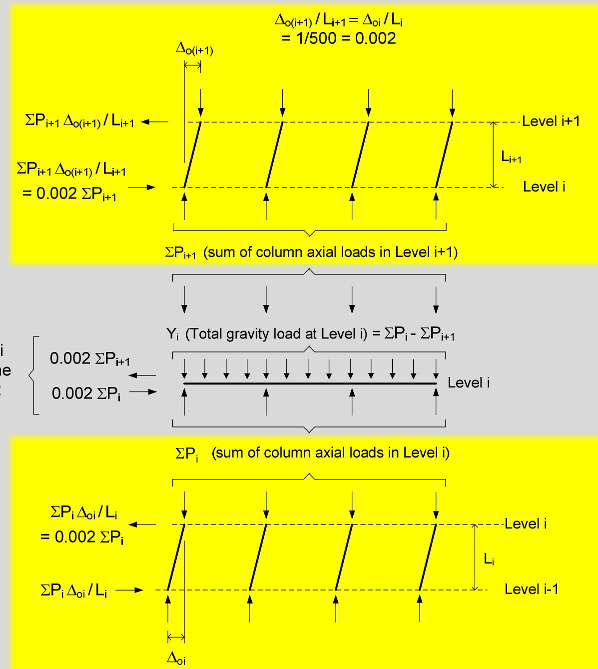
Where does $N_i = 0.002Y_i$ come from?



20

Where does $N_i = 0.002Y_i$ come from?

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



21

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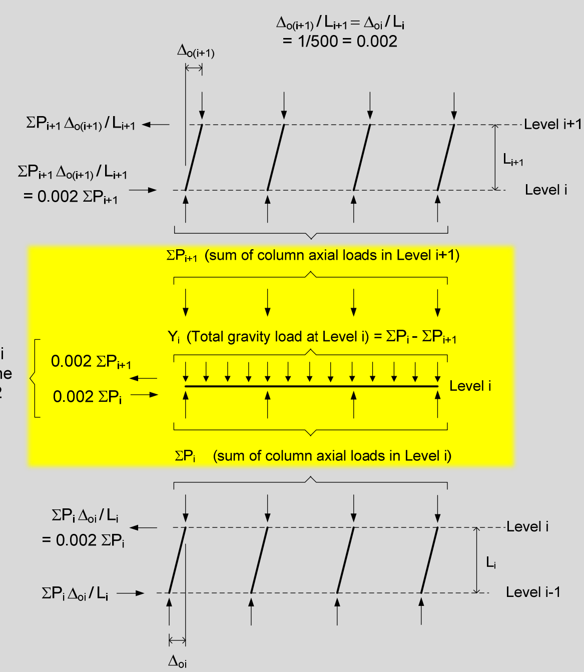
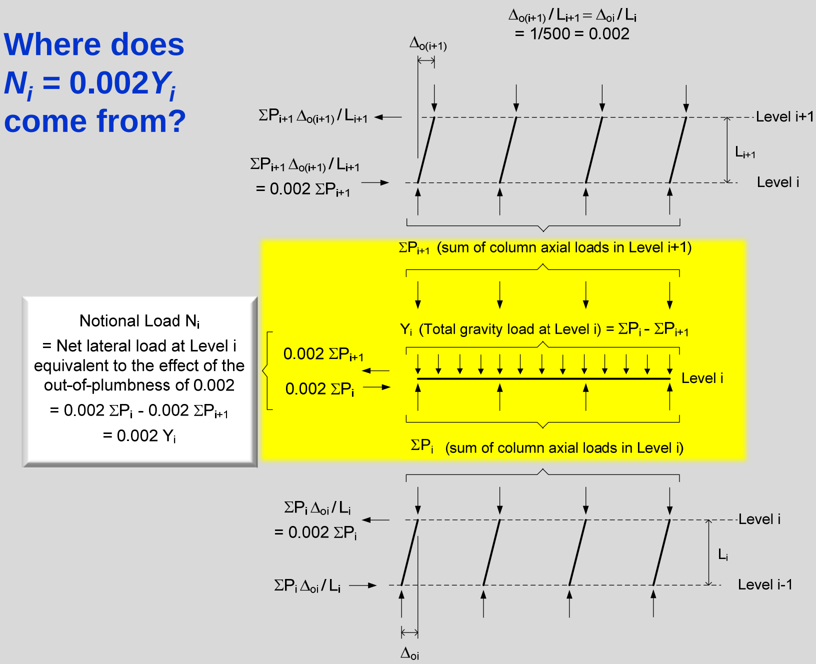


22

Where does $N_i = 0.002Y_i$ come from?



23

Design Considering Lateral Stiffness Requirements

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

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

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

$$\frac{1}{1-\dfrac{P_{story.s}}{R_M P_{Lstory}}}\frac{H_s}{P_{Lstory}} \le \psi_2$$

$$\frac{1}{\dfrac{P_{Lstory}}{H_s}-\dfrac{P_{story.s}}{R_M H_s}} \le \psi_2 \quad\Rightarrow\quad \frac{1}{\dfrac{1}{\psi_1}-\dfrac{P_{story.s}}{R_M H_s}} \le \psi_2 \quad\Rightarrow\quad \psi_1 \le \frac{1}{\dfrac{1}{\psi_2}+\dfrac{P_{story.s}}{R_M H_s}} = \frac{1}{400+\dfrac{P_{story.s}}{R_M H_s}}$$

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

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

P_{Lstory} = story sidesway stiffness

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

H_s = story shear, service wind combo

$R_M = 1-0.15\left(P_{mf.s}/P_{story.s}\right)$

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

25

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

$P_{story.D}$ = 884 k $P_{story.L}$ = 228 k $P_{story.S}$ = 76 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 k

$P_{mf.s}$ = $P_{story.s}$ = 1036 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 k

2nd-Order Wind Drift Limit of 1/ψ_2 = 1/400

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

26

Estimating B_2 from Service Wind Drift, Seismic Drift & Seismic P-Δ 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 τ_b = 1):

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

α = 1.0 (LRFD), 1.6 (ASD)
P_{story} = total vertical load supported by the story using LRFD or ASD load combinations
$R_M = 1 - 0.15\left(P_{mf} / P_{story}\right)$

27

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

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

From ASCE 7 Seismic Drift Limit: V_x = 110 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-Δ Limit: Assume θ_{max} = 0.25 $\theta = \dfrac{P_x \Delta I_e}{V_x h_{sx} C_d}$

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

$$\frac{P_x}{\theta_{max}} = \frac{998\text{ k}}{0.25} = 3{,}992\ \frac{\text{kip}}{\text{rad}} \qquad P_{Lstory.\min} = \max\left(\frac{H_s}{\psi_1}, \frac{V_x h_{sx}}{\Delta_a I_e / C_d}, \frac{P_x}{\theta_{max}}\right) = 20{,}910\frac{\text{kip}}{\text{rad}}$$

28

Example Calculation
Upper-Bound Story Sidesway Amplifier

ASCE 7 Load Combination 2, AISC Direct Analysis Method (τ_b = 1.0):

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

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

$\alpha = 1.0$

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

29

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

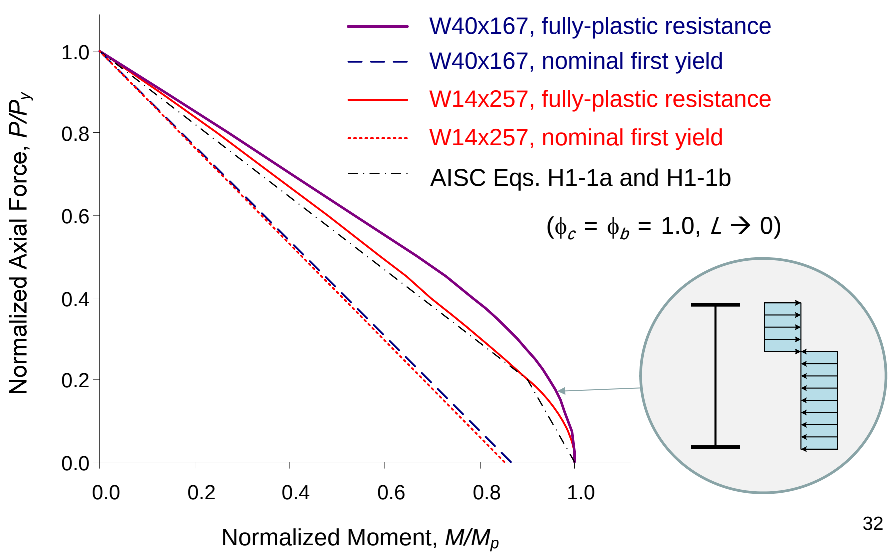
30

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

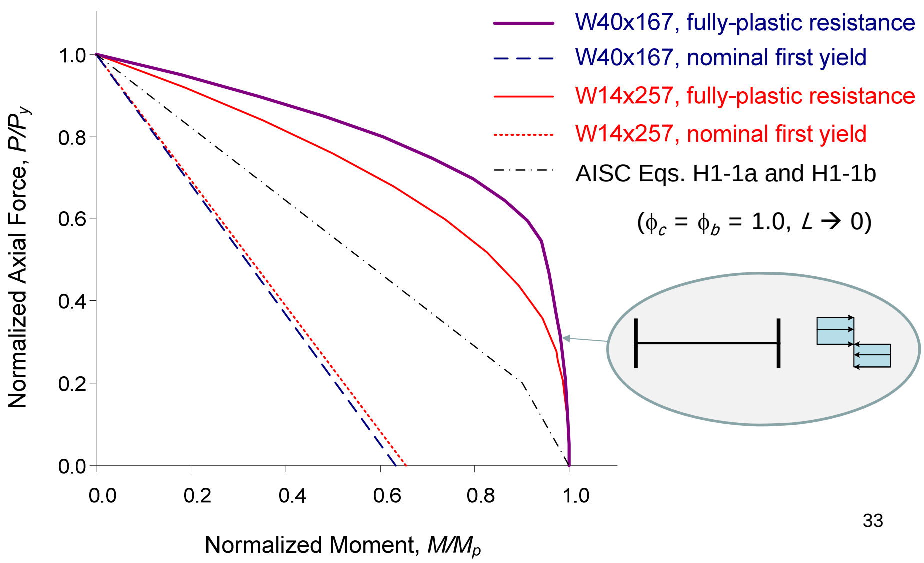
31

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



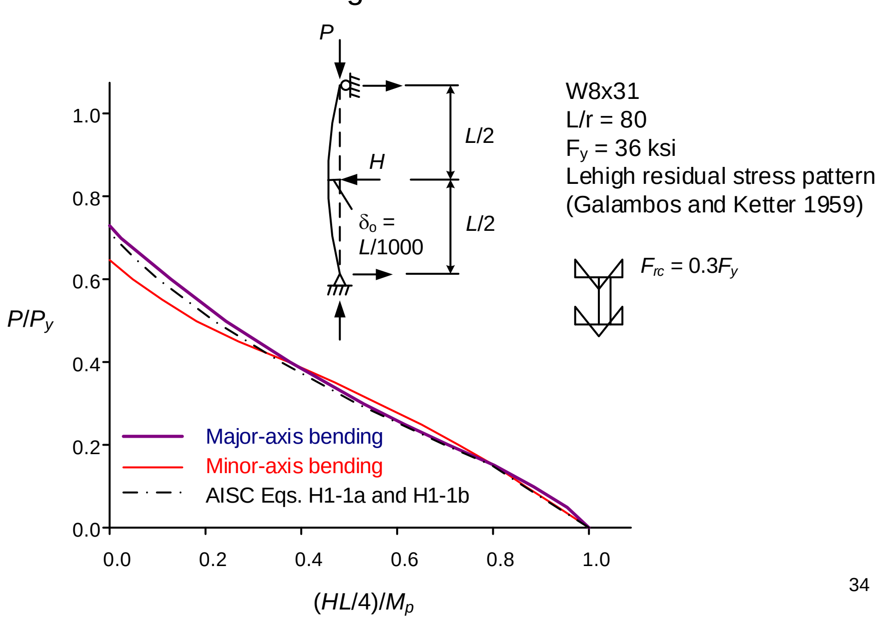
32

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



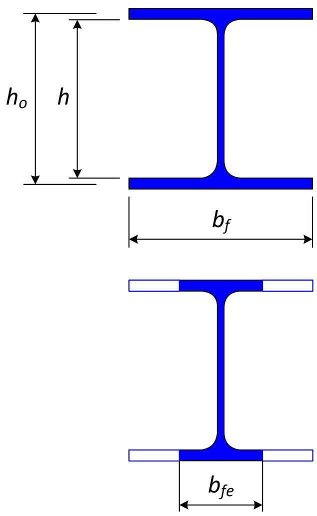
33

Representative In-Plane Strength Envelopes, Finite-Length W-Section Members



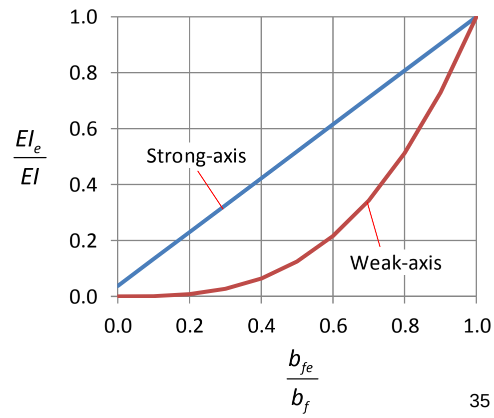
34

Why the substantial difference between weak-axis cross-section & member strengths?



$$I_{xe} \cong b_{fe} t_f h_o^2 + h^3 t_w / 12$$

$$I_{ye} \cong 2 b_{fe}^3 t_f / 12$$



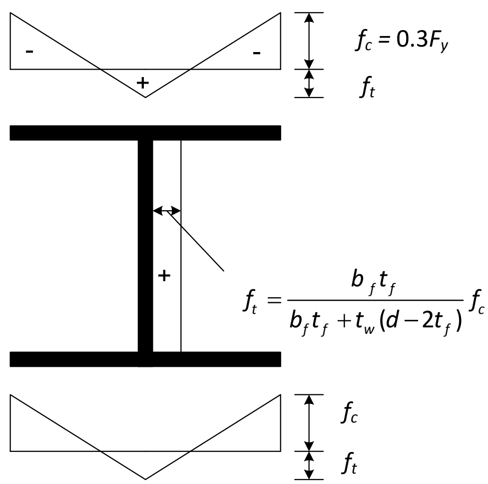
35

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

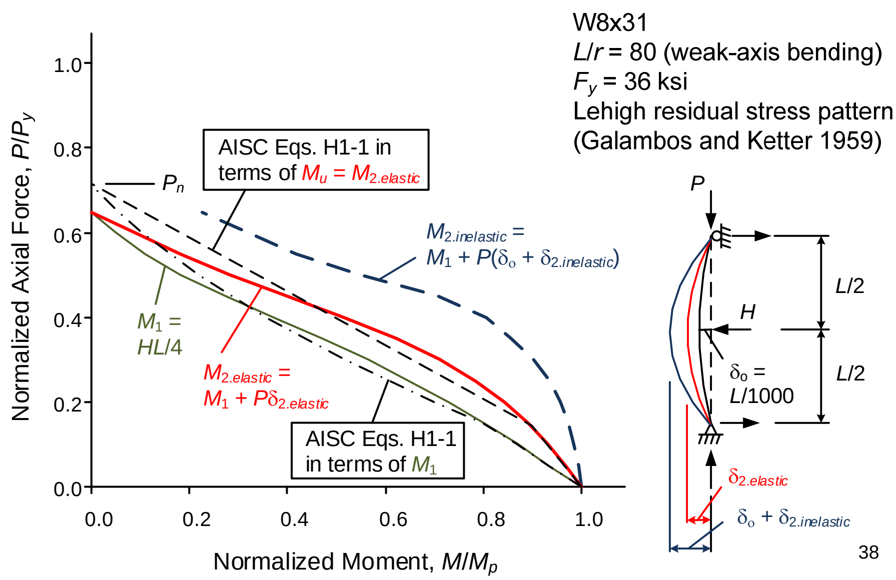
36

Lehigh Residual Stress Pattern



37

Beam-Column Strength Envelopes in Terms of Different Moments



38

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

39

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. $M_{2.elastic}$**

40

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 $\boldsymbol{P}$ **vs.** $\boldsymbol{M_{2.elastic}}$
- 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

41

Equations H1-1 – Key Attributes

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

42

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”

43

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

44

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

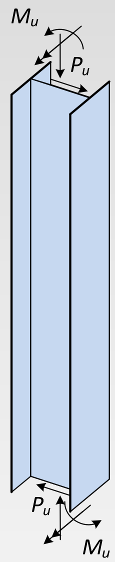
45

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

46

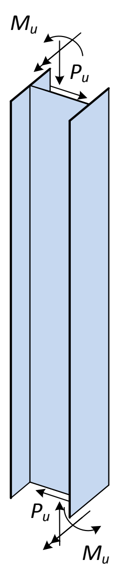
Design Example – Axial Compression & Bending of a W-Section Beam-Column



- 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

47

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

48

Table 6-1 of AISC Manual

Equations H1-1 (LRFD):

For $\dfrac{P_u}{\phi_c P_n} \geq 0.2$ $\quad \dfrac{P_u}{\phi_c P_n} + \dfrac{8}{9}\dfrac{M_u}{\phi_c M_n} \leq 1$ (H1-1a)

For $\dfrac{P_u}{\phi_c P_n} < 0.2$ $\quad \dfrac{P_u}{2\phi_c P_n} + \dfrac{M_u}{\phi_c M_n} \leq 1$ (H1-1b)

Equations H1-1, alternate format (LRFD):

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

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

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

49

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

F_y = 50 ksi

W14

Shape	W14× 43[c]				W14× 38[c]				W14× 34[c]			
Design	$p \times 10^3$ (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)$^{-1}$	
Effective length, KL (ft), with respect to least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	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

50

Determining "Required Resistances"

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

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

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

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

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

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

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 $pP_u / 1000 = 0.3$

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

- 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
  $\therefore$ even if checking out-of-plane strength w/ Eq. (H1-1a), select b_x based on $L_b = 0$

52

Shape		W14×											
		43c				38c				34c			
Design		$p \times 10^3$ (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)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, KL (ft), with respect to least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	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			

c 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

$$b_x \leq \left(1-\frac{p_x P_u}{1000}\right)\frac{1000}{M_{ux}} = \left(1-\frac{2.53x82.6}{1000}\right)\frac{1000}{141} = \left(1-0.209\right)\frac{1000}{141} = 5.6$$

53

Shape		W14×											
		30c				26c				22c			
Design		$p \times 10^3$ (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)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, KL (ft), with respect to least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	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			

c 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

$$b_x \leq \left(1-\frac{p_x P_u}{1000}\right)\frac{1000}{M_{ux}} = \left(1-\frac{2.91x82.6}{1000}\right)\frac{1000}{141} = \left(1-0.240\right)\frac{1000}{141} = 5.4$$

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

$$\frac{p_x P_u}{1000} = \frac{2.91x91.6}{1000} = 0.267$$

$$\frac{p_x P_u}{1000} + \frac{b_x M_{ux}}{1000} = 0.267 + \frac{5.01x125}{1000} = 0.89$$ OK Eq. (H1-1a)

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

55

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

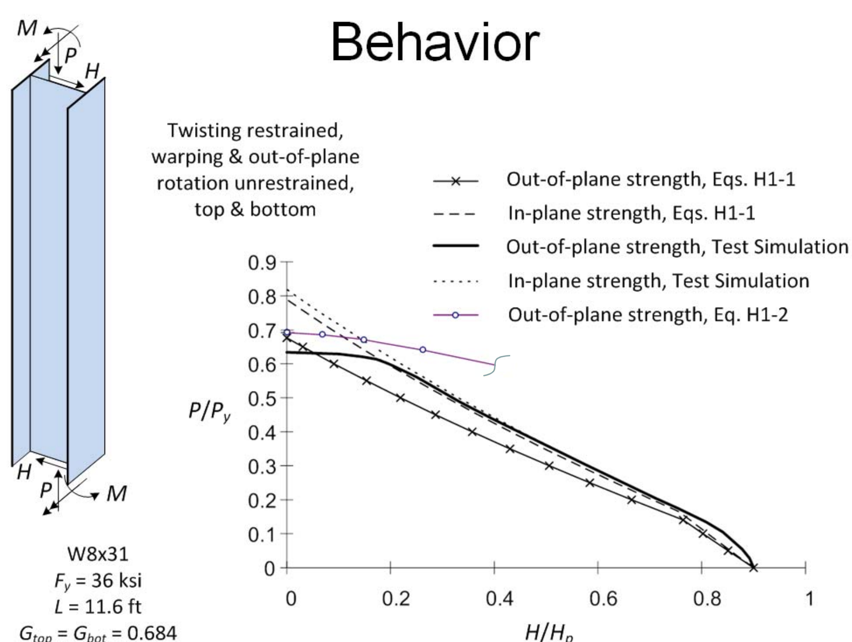
56

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

57

Behavior



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

58

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

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

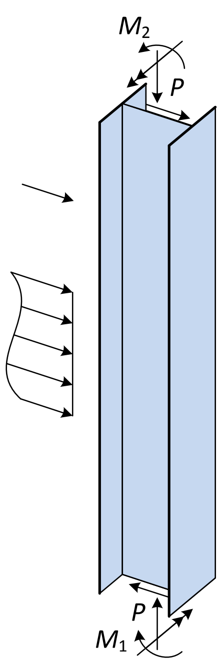
… or, expressed in terms of the flexural capacity of a member subjected to axial compression…

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

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

59

Analytical Basis



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

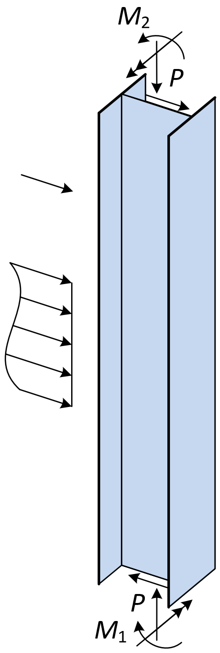
C_b = moment gradient factor

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

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

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

60



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

61

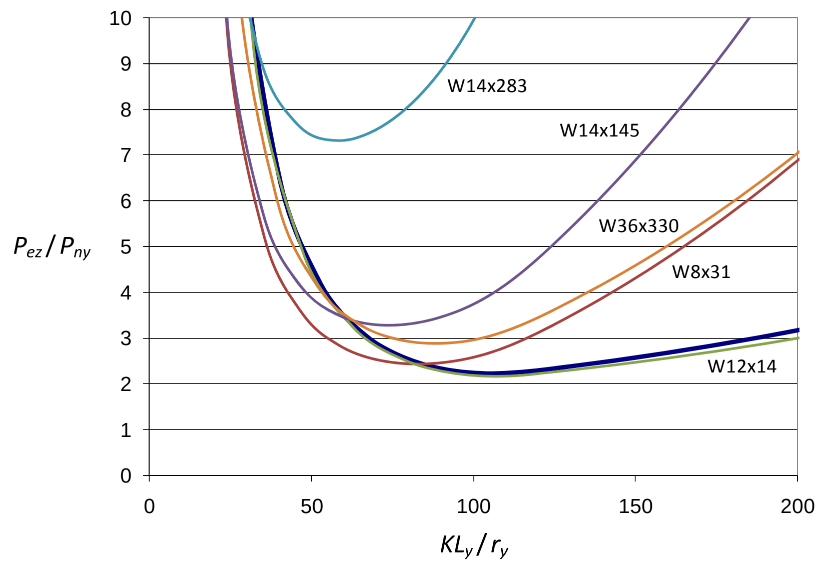
"Mapping" to Nominal Strength Equation

- 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 $KL_z \le KL_y$)
- Set P_{ny} to P_{cy} & $M_{nx(Cb=1)}$ to $M_{cx(Cb=1)}$

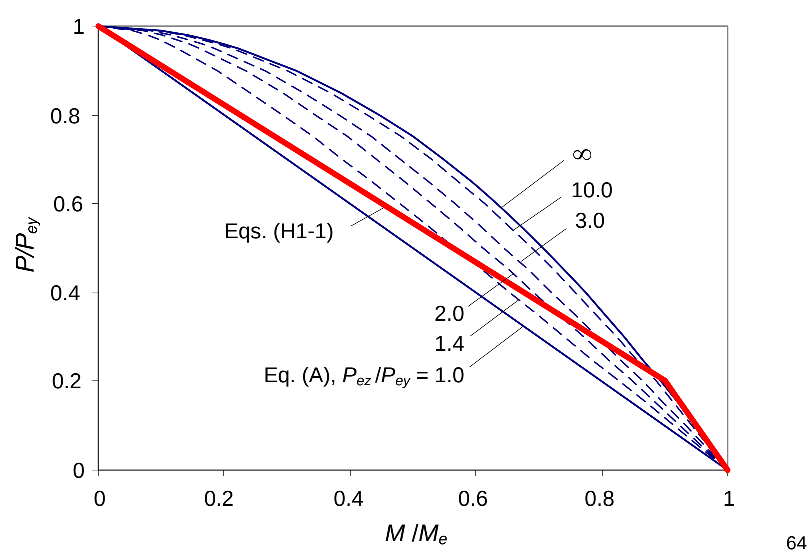
62

P_{ez}/P_{ny} vs KL_y/r_y, W-Section Members with $KL_z = KL_y$



63

Theoretical Elastic Out-of-Plane Strengths for Simply-Supported I-Section Beam-Columns



64

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

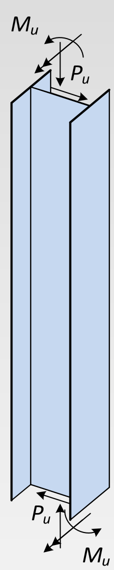
$$\frac{P_r}{P_{cy}}\left(1.5-0.5\frac{P_r}{P_{cy}}\right)+\left(\frac{M_{rx}}{C_b M_{cx(Cb=1)}}\right)^2 \leq 1.0$$

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

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

65

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

66

Check Out-of-Plane Resistance

- Alternate format of Eq. (H1-2)

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

- 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):

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

67

Shape		W14×											
		30^c				26^c				22^c			
Design		$p \times 10^3$ (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)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, KL (ft), with respect to least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	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

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

$$\therefore \text{ for LC4: } \frac{p_y P_u}{1000}\left(1.5-0.5\frac{p_y P_u}{1000}\right)+\left(\frac{9b_x M_{ux}}{8000C_b}\right)^2=\frac{5.98x82.6}{1000}\left(1.5-0.5\frac{5.98x82.6}{1000}\right)+\left(\frac{9x7.43x141.4}{8000x2.3}\right)^2=0.89\leq 1.0$$

$$\text{\& for LC2: } \frac{p_y P_u}{1000}\left(1.5-0.5\frac{p_y P_u}{1000}\right)+\left(\frac{9b_x M_{ux}}{8000C_b}\right)^2=\frac{5.98x91.6}{1000}\left(1.5-0.5\frac{5.98x91.6}{1000}\right)+\left(\frac{9x7.43x125.3}{8000x2.3}\right)^2=0.88\leq 1.0$$

68

Check W14x30 Web Compactness

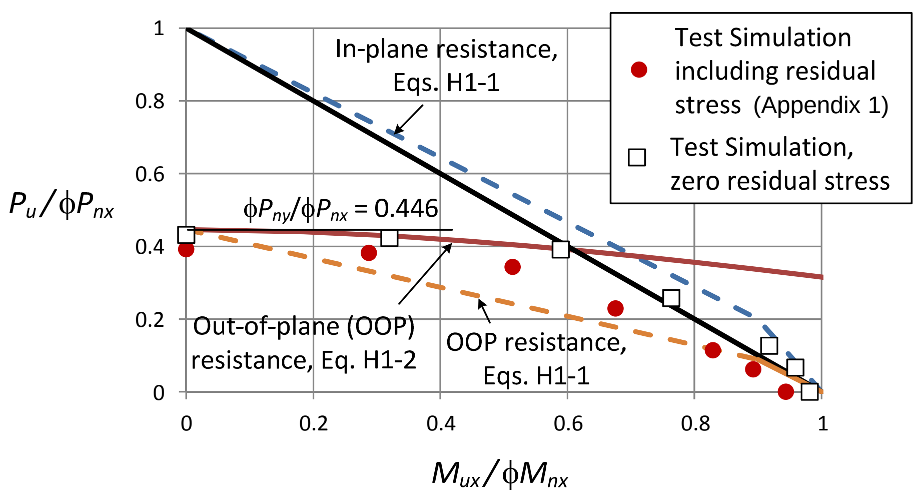
- Flange is compact since Table 6-1 does not provide a footnote about the flange compactness
- Use Appendix 1, Section 1.2.2 to check web compactness

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

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

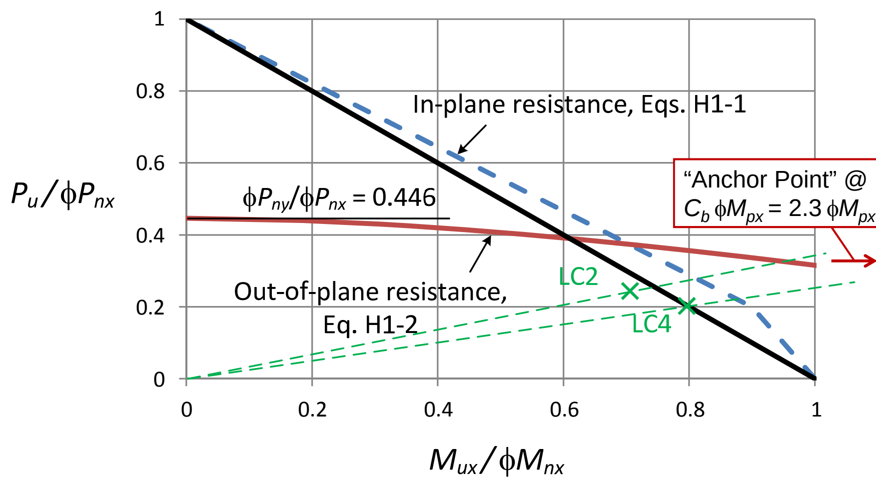
69

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



70

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

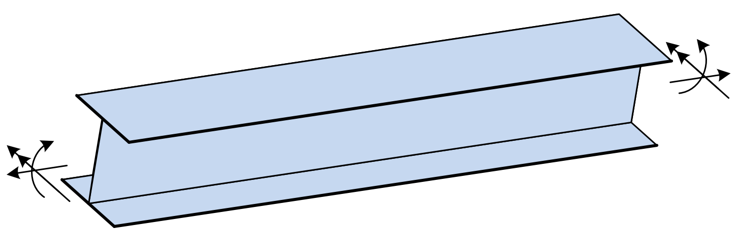
72

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

73

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



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

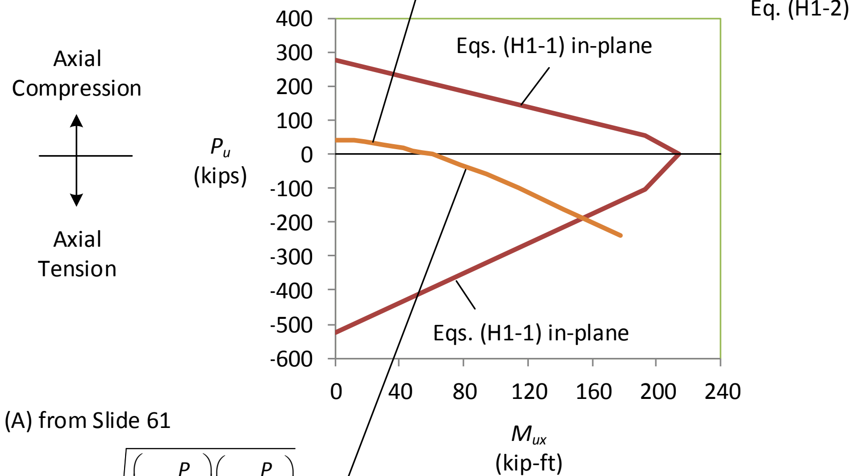
74

Mechanics Based P-M Interaction

Specific calculated values determined in subsequent example

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

Eq. (H1-2)



Eq. (A) from Slide 61

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

75

Axial Tension & Bending – Derivation of AISC Modified C_b Factor

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

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

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

… 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)

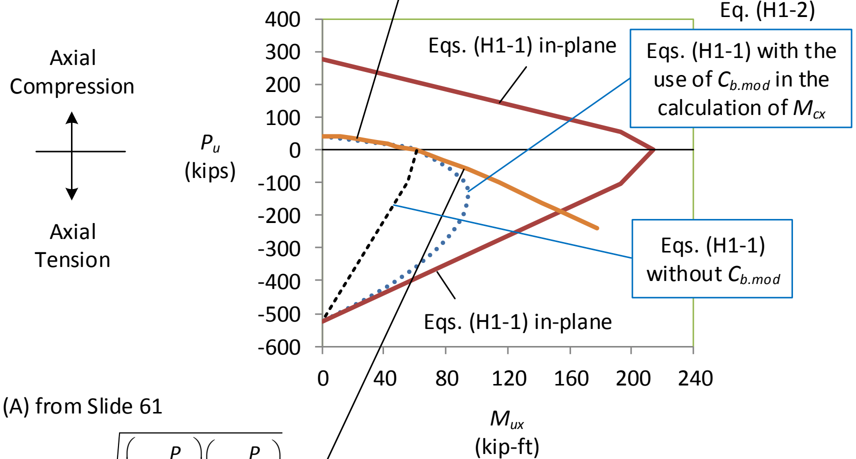
$$C_{b.mod}=C_b\sqrt{\left(1+\frac{P_u}{P_{ey}}\right)}$$

76

Comparison of Equations

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

Eq. (H1-2)



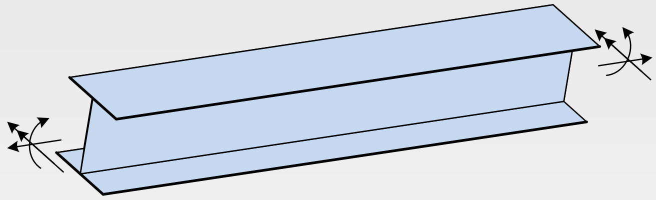
Eq. (A) from Slide 61

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

77

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

78

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

$KL_x := 40ft$ $KL_y := KL_x$ $KL_z := KL_x$ $L_b := KL_x$

$F_y := 50ksi$ $E := 29000ksi$

$A_g := 11.7in^2$ $\phi P_y := 0.9 \cdot F_y \cdot A_g = 526.5\,kip$

$I_y := 44.1in^4$ $P_{ey} := \dfrac{\pi^2 \cdot E \cdot I_y}{(KL_y)^2} = 54.8\,kip$

$L_r := 21.1ft$ $h_o := 11.4in$ $S_x := 51.5in^3$ $J := 0.906in^4$ $r_{ts} := 2.21in$

$$X := \sqrt{\frac{S_x \cdot h_o}{J}} = 25.5 \qquad \phi M_{n1} := 0.9\,\frac{\pi^2 \cdot E \cdot S_x}{\left(\frac{L_b}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{1}{X^2} \cdot \left(\frac{L_b}{r_{ts}}\right)^2} = 60.6 \cdot kip \cdot ft$$

79

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

$P_u := 160kip$ $M_u := 100kip \cdot ft$

$C_b := \sqrt{1 + \dfrac{P_u}{P_{ey}}} = 1.98$ $\phi M_p := 214kip \cdot ft$ from Table 3-2

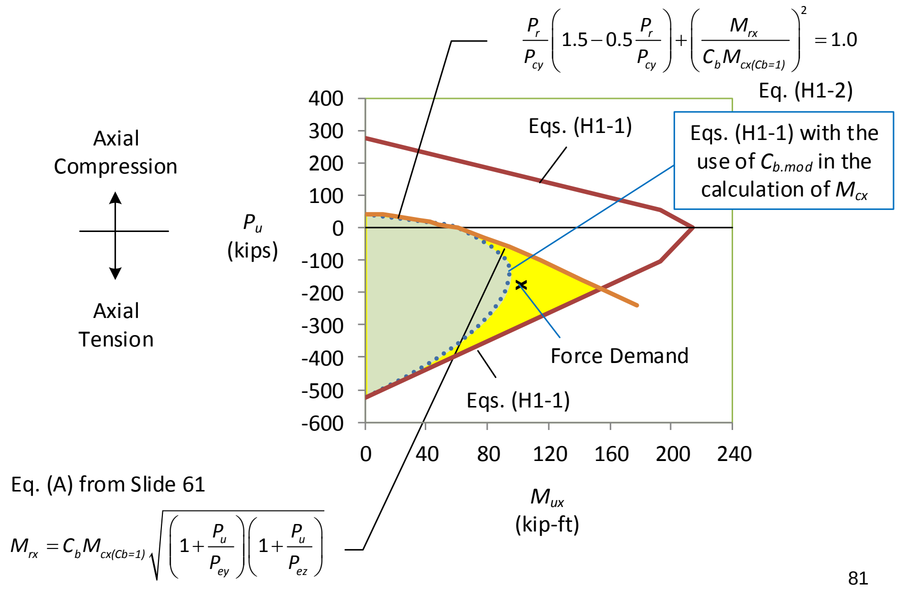
$\phi M_n := \min\left(C_b \cdot \phi M_{n1}, \phi M_p\right) = 120 \cdot kip \cdot ft$

$\dfrac{P_u}{\phi P_y} = 0.304$ $\dfrac{P_u}{\phi P_y} + \dfrac{8}{9} \cdot \dfrac{M_u}{\phi M_n} = 1.045$ NG

Almost … but No Gravy!

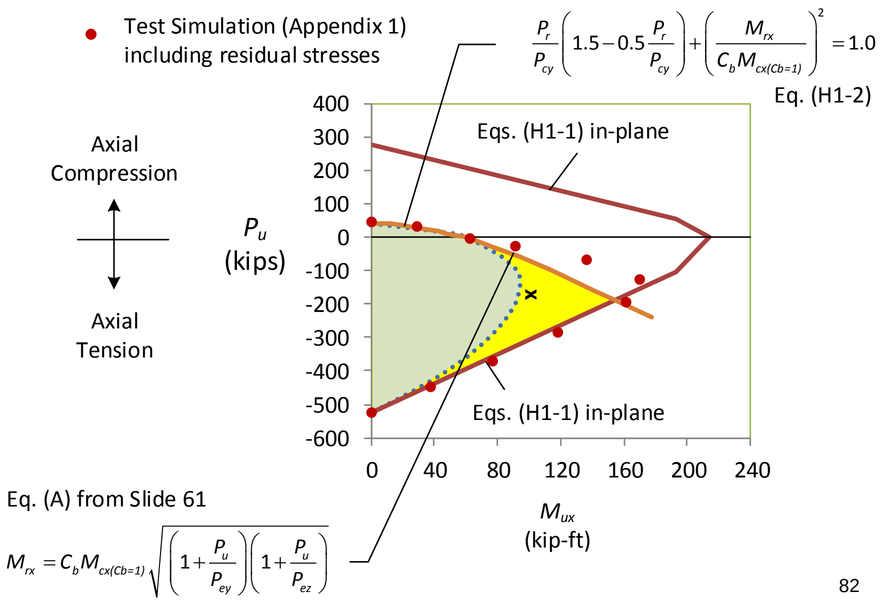
80

Graphical Illustration of Resistance Checks



81

Comparison to Test Simulation Results



82

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

83

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

84

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.
- Password: Same as AISC website password.

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.

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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.

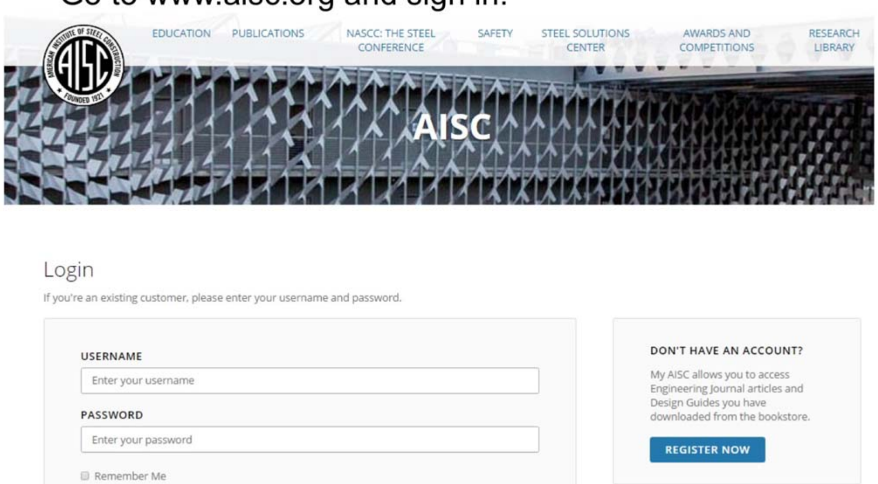
CEUs/PDHS – If you watch a recorded session you must take AND PASS the quiz for CEUs/PDHs.

Night School Resources for 8-session package Registrants

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

Night School Resources for 8-session package Registrants

Go to www.aisc.org and sign in.



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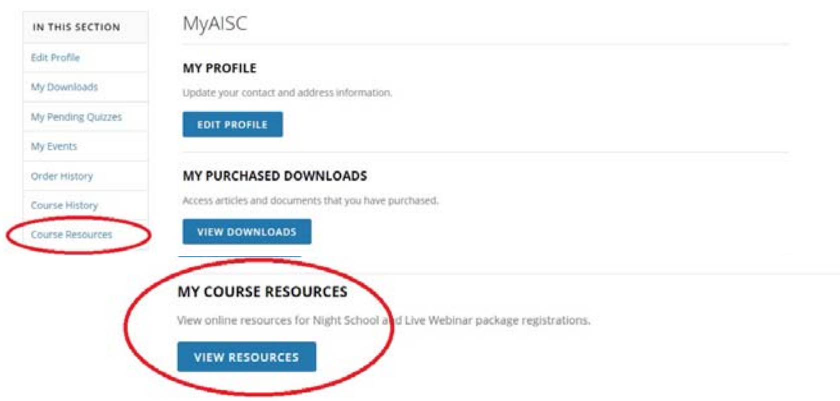
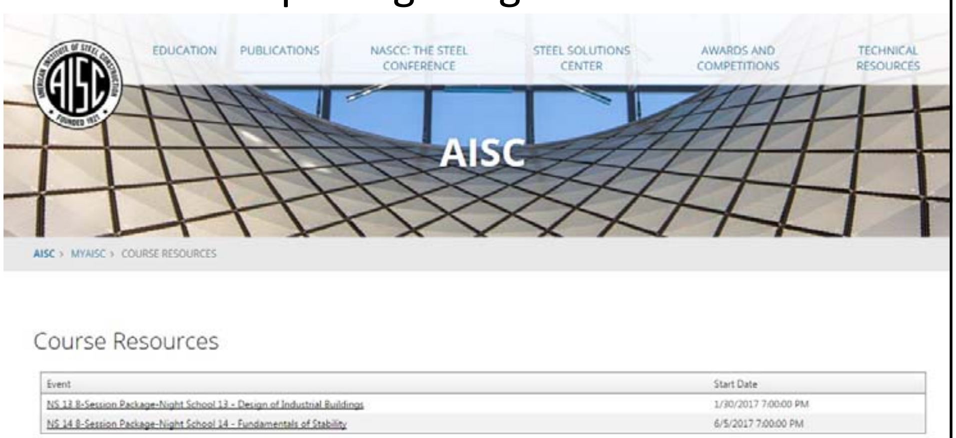


Night School Resources for 8-session package Registrants



Night School Resources for 8-session package Registrants

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	View Passcode: 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 Girder 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 Girder & Longitudinal Bldg Bracing Dsn	3/27/2017 7:00:00 PM	Handouts	Available 03/29/2017 5pm EST	Available 03/29/2017 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

There's always a solution in steel.

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

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