Basic Steel Design -- Session 7:
Stability Analysis and Design II

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Session Description

22.6 Stability Analysis and Design II
March 24, 2020

Part II of this 2-session lecture continues to explore the requirements of the AISC *Specification* for the design for stability. The session will discuss and demonstrate the Effective Length Method through examples. The session will then highlight the Direct Analysis Method and discuss the advantages of the method; specifically, that the Direct Analysis Method allows the of a K-factor equal to 1.

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Learning Objectives:

- List AISC *Specification* stability requirements per chapter C.
- Describe how to apply the Effective Length Method in the stability analysis and design of structural steel buildings.
- Describe how to apply the Direct Analysis Method in the stability analysis and design of structural steel buildings.
- Compare and contrast the Effective Length Method and Direct Analysis Method.

Basic Steel Design: A review of the principles of steel design according to ANSI/AISC 360-16

Night School 22

Lesson 7

Stability Analysis and Design II

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Stability Analysis and Design II

- Stability analysis and design
  - Effective length method
  - Effective length nomograph
  - Violated nomograph assumptions
  - Inelastic effective length
  - Impact of gravity only columns
  - Direct analysis
  - Stiffness reduction

7.9

AISC 360-16

- Stability

B3.7 Design for Stability

"The structure and its elements shall be designed for stability in accordance with Chapter C."

C1. General Stability Requirements

"Stability shall be provided for the structure as a whole and for each of its elements."

We discussed these requirements in Lesson 6

7.10

AISC 360-16

- C1. General Stability Requirements

  "Any rational method of design for stability that considers all of the listed effects is permitted; this includes the methods identified in Sections C1.1 and C1.2"

  C1.1 Direct Analysis Method of Design

  C1.2 Alternative Methods of Design

7.11

AISC 360-16

- Alternate Methods of Design for Stability

  App.7.1. General Stability Requirements
  - General requirements from Chapter C still apply.
  - Alternative methods are permissible, subject to limitations given in App.7.

  App.7.2. Effective Length Method

  App.7.3. First-Order Analysis Method

7.12

Effective Length Method

App.7.2.1. Limitations

(a) Structure supports gravity loads primarily through nominally vertical columns, walls or frames

(b) Second-order effects must be limited

$$B_2 = \frac{\Delta_{2nd-order}}{\Delta_{1st-order}} \leq 1.5$$

7.13

Effective Length Method

App.7.2.2. Required Strengths

– Must satisfy the general stability requirements of Section C2.1 that we have already discussed in Lesson 6.

– For out-of-plumbness must use notional loads.

– Those notional loads are applied in gravity only load cases.

Remember from Lesson 6 that these notional loads are to account for out-of-plumbness.

7.14

Effective Length Method

App.7.2.3. Available Strengths

- Calculated in accordance with the provisions of Chapters D through K.
- Effective length factors taken as $K = 1.0$ when lateral stability does not rely on flexural stiffness (braced frames). $L_c = L$
- In moment frames, K determined through a sidesway buckling analysis. $L_c = KL$

If $\Delta_{2nd-order}/\Delta_{1st-order} \leq 1.1$ then $K = 1.0$

7.15

Effective Length Method

- Design process
  - Apply notional loads, $N_i = 0.002\alpha Y_i$, in the gravity only load cases
  - Perform a second-order elastic analysis
  - Determine K from a sidesway buckling analysis

7.16

Effective Length Factor

- Every approach proposed for determination of the effective length factor, K, is an attempt to determine the exact effective length factor, K_{exact}, such that the exact critical buckling load may be determined, without the need to resort to an elastic buckling analysis.

7.17

Elastic Buckling Analysis

- Eigenvalue Analysis
  - General form of the eigenvalue problem

$$\left(\left[K_o\right] + \lambda\left[K_g\right]\right)\left\{\Delta\right\} = 0$$

$\left[K_o\right] =$ linear stiffness matrix

$\left[K_g\right] =$ geometric stiffness matrix

7.18

Elastic Buckling Analysis

- Eigenvalue Analysis
  - Reduction to standard form

$$[H]\{Y\} = \omega\{Y\}$$

$$\{Y\} = \text{eigenvector}$$

$$\omega = \text{eigenvalue}$$

$$\lambda = \frac{1}{\omega} = \text{load ratio}$$

From the load ratio you can determine the critical buckling load.

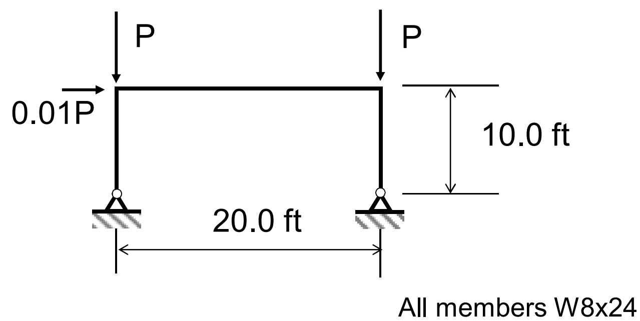
7.19

Elastic Buckling Analysis

- Eigenvalue Analysis
  - Solution techniques
    - Polynomial expansion
    - Power method
    - Iteration
  - These are embedded within your software, not something you would do by-hand.
  - Note that even though these might be called “rigorous,” they are still approximations.

7.20

Elastic Buckling Analysis



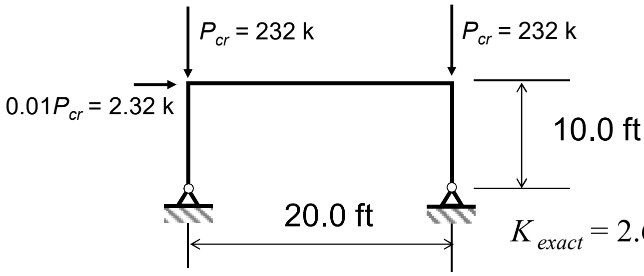
7.21

Elastic Buckling Analysis

7.22

Elastic Buckling Analysis

At this load, the structure will buckle in a sway mode.



$K_{exact} = 2.66$

The elastic buckling load is given by

$$P_{cr} = \frac{\pi^2 EI}{(KL)^2}$$

Therefore, the effective length factor is

$$K_{exact} = \frac{\pi}{L}\sqrt{\frac{EI}{P_{cr}}}$$

7.23

Effective Length Factor

- Goal
  - Develop equations that may be used to determine the effective length of columns in moment frames or braced frames without first requiring an elastic buckling analysis to determine P_{cr}.

7.24

Effective Length Factor

Assumptions employed in developing equations:

1. Behavior is purely elastic.
2. All members have a constant cross section.
3. All joints are rigid.
4. In sidesway inhibited frames (braced frames), rotations at opposite ends of beams are equal producing single curvature.
5. In sidesway permitted frames (moment frames), rotations at opposite ends of restraining beams are equal producing reverse curvature

7.25

Effective Length Factor

Assumptions:

6. Stiffness parameter $L\sqrt{P/EI}$ of all columns is equal
7. Joint restraint is distributed to column above and below in proportion to I/L
8. All columns buckle simultaneously
9. No significant axial force in girders
10. Shear deformations are neglected

7.26

Effective Length Factor

- Sidesway Inhibited (Braced Frames)

$$\frac{G_A G_B}{4}(\pi/K)^2 + \left(\frac{G_A + G_B}{2}\right)\left(1 - \frac{\pi/K}{\tan(\pi/K)}\right) + \frac{2\tan(\pi/2K)}{\pi/K} = 1 \quad \text{(C-A-7-1)}$$

- Sidesway Permitted (Moment Frames)

$$\frac{G_A G_B (\pi/K)^2 - 36}{6(G_A + G_B)} = \frac{\pi/K}{\tan(\pi/K)} \quad \text{(C-A-7-2)}$$

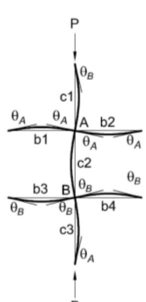
7.27

Effective Length

Nomograph or Alignment Chart for Braced Frame based on Eq C-A-7-1

At each end, A and B

$$G = \frac{\Sigma(I/L)_c}{\Sigma(I/L)_g}$$



We looked at this in Lesson 3

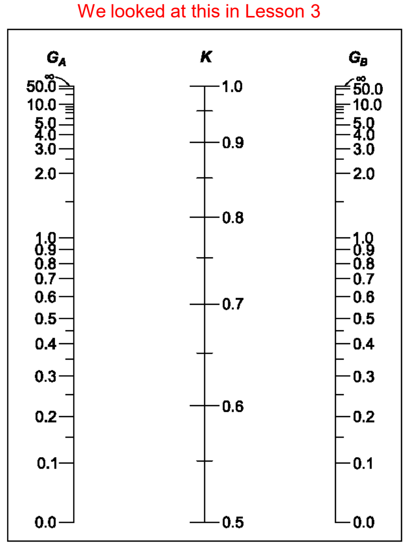
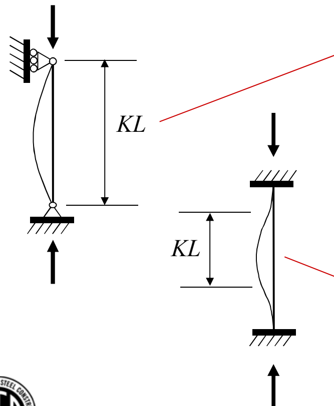


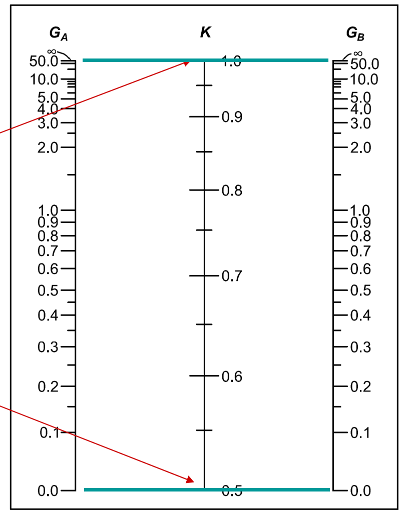
Fig. C-A-7.1

7.28

Effective Length Factor

Braced frame members: ends do not sway relative to each other





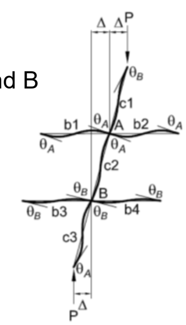
7.29

Effective Length

Nomograph or Alignment Chart for Moment Frame based on Eq. C-A-7-2

At each end, A and B

$$G = \frac{\Sigma(I/L)_c}{\Sigma(I/L)_g}$$



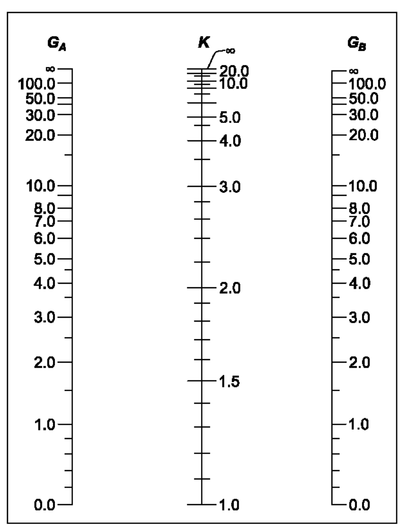


Fig. C-A-7.2

7.30

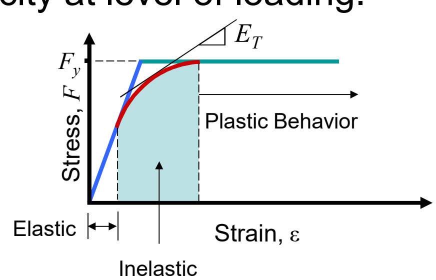
Effective Length Factor

- Problems with nomograph
  - Real structures rarely satisfy the assumptions made in developing these equations.
    - Gravity only columns
    - Stiffness parameters not usually the same
    - All columns don't buckle simultaneously
    - Different end conditions means I/L is not a good measure of stiffness at a joint
    - Columns may not behave elastically

7.31

Effects of Inelasticity

- From Lesson 6
  - When the stress-strain relationship is no longer linear, use the Tangent Modulus of Elasticity at level of loading.



$$\tau_b = \frac{E_T}{E}$$

7.32

Effects of Inelasticity

- Look at the stiffness ratio at each end of the column. The derivation had originally included E.

$$G_{elastic} = \frac{\Sigma(EI/L)_c}{\Sigma(EI/L)_g} = \frac{\Sigma(I/L)_c}{\Sigma(I/L)_g}$$

- If E_T is used for the columns and it is assumed that all columns at the joint have the same E_T, then

$$G_{inelastic} = \frac{\Sigma(E_T I/L)_c}{\Sigma(EI/L)_g} = \frac{E_T}{E}\frac{\Sigma(I/L)_c}{\Sigma(I/L)_g} = \tau_b G_{elastic}$$

7.33

Effects of Inelasticity

- Depends on the level of axial stress in the member given by ratio of required strength to maximum strength:

when $\alpha P_r/P_{ns} \le 0.5;\quad \tau_b = 1.0$ (C2-2a)

when $\alpha P_r/P_{ns} > 0.5;\quad \tau_b = 4\left[\frac{\alpha P_r}{P_{ns}}\left(1-\frac{\alpha P_r}{P_{ns}}\right)\right]$ (C2-2b)

$$P_{ns} = F_y A_g \quad \text{or} \quad P_{ns} = F_y A_e$$

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

7.34

Effects of Inelasticity

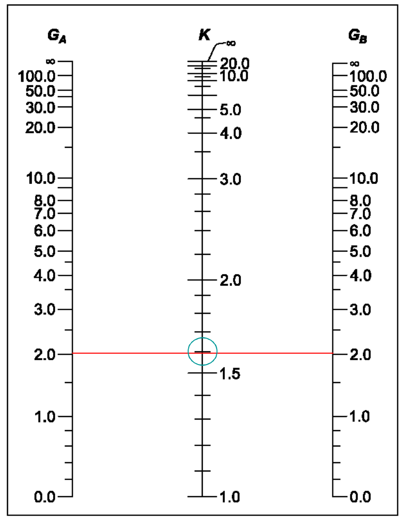
Moment Frame Column

Beams W16x36, L= 24 ft, I_x = 448 in.^{3}
Columns W10x88, L = 14 ft, I_x = 534 in.^{3}

At each end, A and B

$$G_{elastic} = \frac{\Sigma(I/L)_c}{\Sigma(I/L)_g} = \frac{2\left(\frac{534}{14}\right)}{2\left(\frac{448}{24}\right)} = 2.04$$

$$K = 1.6$$



7.35

Effects of Inelasticity

Moment Frame Column

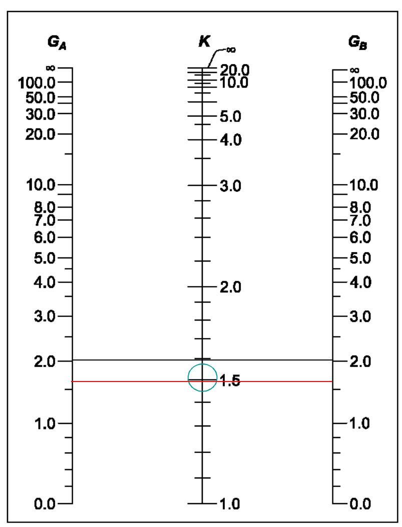
If the LRFD load on the column is P_u = 950 kips and P_{ns} = P_y

$$\frac{\alpha P_r}{P_y} = \frac{950}{50(26.0)} = 0.731 > 0.5$$

$$\tau_b = 4\left[\frac{\alpha P_r}{P_y}\left(1 - \frac{\alpha P_r}{P_y}\right)\right]$$
$$= 4\left[0.731(1 - 0.731)\right] = 0.786$$

$$G_{inelastic} = 0.786(2.04) = 1.60$$

$$K = 1.5$$



7.36

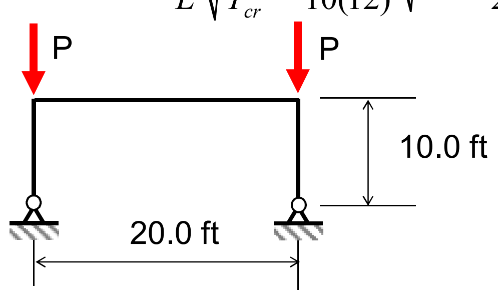
Effective Length Factor

- Problems with nomograph
  - Real structures rarely satisfy assumptions
    - Gravity only columns
    - Stiffness parameters not usually the same
    - All columns don't buckle simultaneously
    - Different end conditions means I/L is not a good measure of stiffness at a joint
    - Columns may not behave elastically

7.37

Buckling vs. Nomograph

$$K_{exact} = \frac{\pi}{L}\sqrt{\frac{EI}{P_{cr}}} = \frac{\pi}{10(12)}\sqrt{\frac{29,000(82.7)}{232}} = 2.66$$



All members W8x24

$P_{cr} = 232$ kips

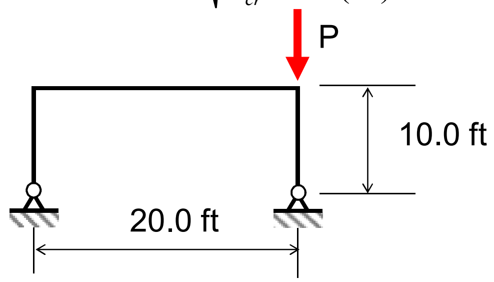
Nomograph
$K = 2.63$

The structure and loading are close to satisfying the derivation assumptions. Thus, the results are close.

7.38

Buckling vs. Nomograph

$$K_{exact} = \frac{\pi}{L}\sqrt{\frac{EI}{P_{cr}}} = \frac{\pi}{10(12)}\sqrt{\frac{29,000(82.7)}{460}} = 1.89$$



Nomograph
$K = 2.63$

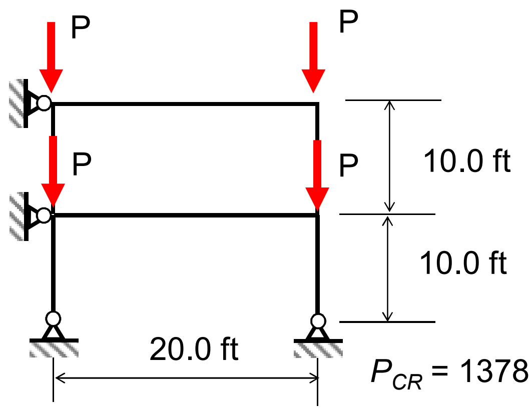
All members
W8x24

$P_{cr} = 460$ kips

The structure and loading are far from satisfying the derivation assumptions. Thus, the results are quite different.

7.39

Buckling vs. Nomograph



All members
W8x24

P_{CR} = 1378 kips

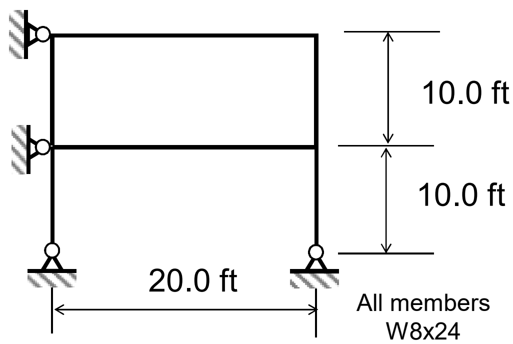
Sidesway Prevented

Upper column carries 1378 kips
Lower column carries 2756 kips

7.40

Buckling vs. Nomograph

All members W8 x 24



Nomograph

K_{upper} = 0.88

K_{lower} = 0.95

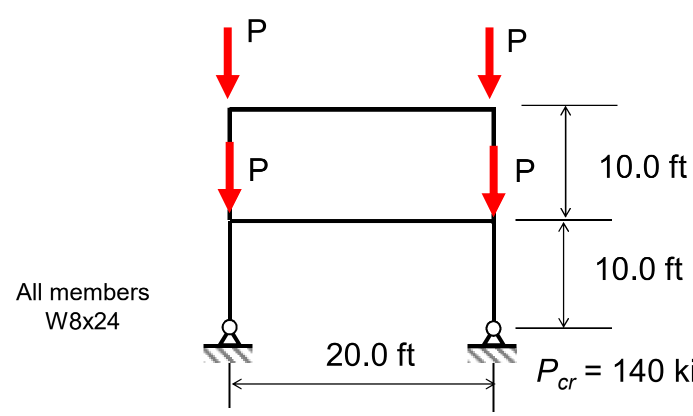
Elastic Buckling

K_{upper} = 1.09

K_{lower} = 0.77

7.41

Buckling vs. Nomograph

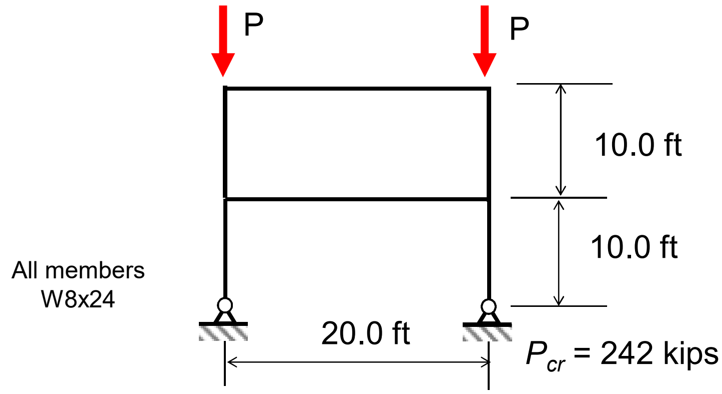


P_{cr} = 140 kips

Sidesway Permitted

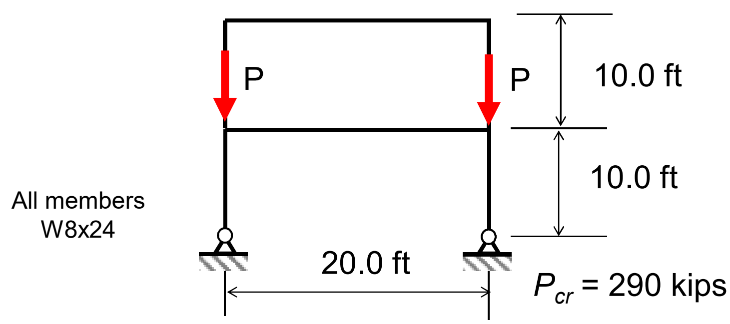
7.42

Buckling vs. Nomograph



7.43

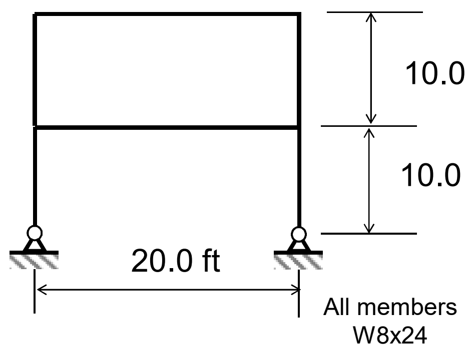
Buckling vs. Nomograph



7.44

Buckling vs. Nomograph

All members W8 x 24



Nomograph

$K_{\text{upper}} = 1.79$

$K_{\text{lower}} = 3.18$

7.45

Buckling vs. Nomograph

- Elastic Buckling compared to Nomograph
  - From Elastic Buckling, each load pattern results in a different critical buckling load.
  - Both stories loaded,

    $K_{\text{upper}} = 3.43,\ K_{\text{lower}} = 2.42$
  - Upper story loaded,

    $K_{\text{upper}} = 2.61,\ K_{\text{lower}} = 2.61$
  - Lower story loaded,

    $K_{\text{upper}} = ?,\quad K_{\text{lower}} = 2.38$

Nomograph

$K_{\text{upper}} = 1.79$

$K_{\text{lower}} = 3.18$

7.46

Effective Length Factor

- Problems with nomograph
  - Real structures rarely satisfy assumptions
    - Gravity only columns
    - Stiffness parameters not usually the same
    - All columns don't buckle simultaneously
    - Different end conditions means I/L is not a good measure of stiffness at a joint
    - Columns may not behave elastically

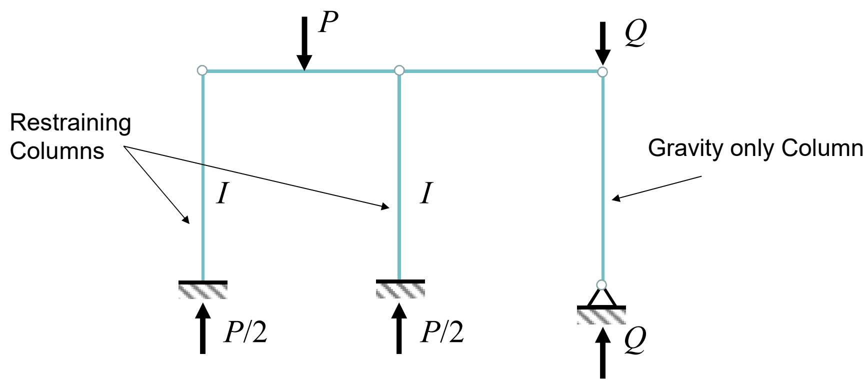
7.47

Gravity only Columns

- What are gravity only columns?
  - Columns that do not contribute to the lateral load resistance of the structure.
  - They rely on the remaining portion of the structure to provide their lateral restraint.
  - Also called leaning columns because they “lean on” the other parts of the structure for stability.
  - Designed with $K = 1.0$.

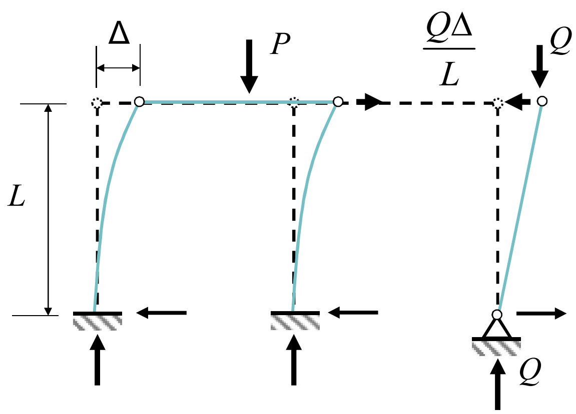
7.48

Buckling with Gravity Only Columns



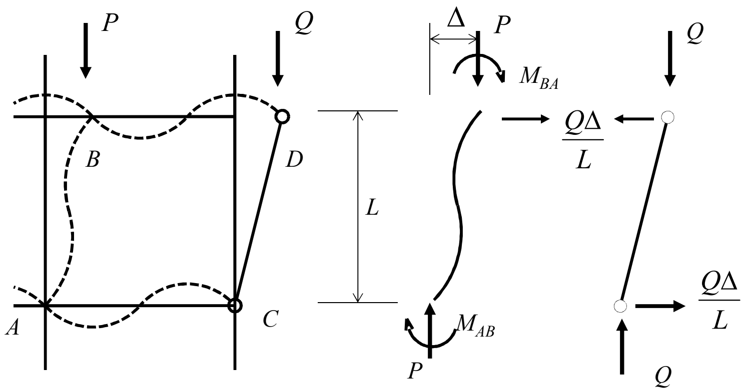
7.49

Buckling with Gravity Only Columns



7.50

Buckling with Gravity Only Columns



Using the same model as that used for developing the nomographs

7.51

Buckling with Gravity Only Columns

"Nomograph" type equation for sidesway uninhibited including gravity only columns

$$\frac{G_A G_B (\pi/K)^2 - 36}{6(G_A + G_B)}\left(1+\frac{Q}{P}\right) - \frac{\pi/K}{\tan(\pi/K)}\left(1+\frac{Q}{P}\right) + \frac{6\tan(\pi/2K)}{(G_A+G_B)(\pi/2K)}\left(\frac{Q}{P}\right) + \left(\frac{Q}{P}\right) = 0$$

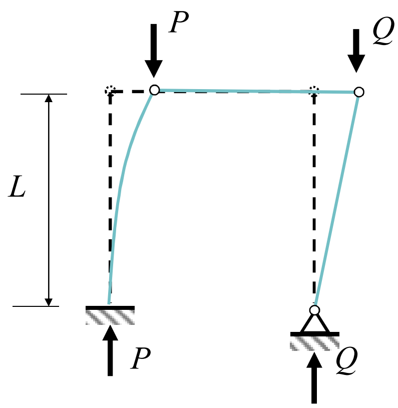
Q represents the sum of the load on all of the gravity only columns attributed to the frame

P represents the sum of the load on all of the restraining columns attributed to the frame

7.52

Buckling with Gravity Only Columns

$$\frac{G_A G_B (\pi/K)^2 - 36}{6(G_A + G_B)}\left(1+\frac{Q}{P}\right) - \frac{\pi/K}{\tan(\pi/K)}\left(1+\frac{Q}{P}\right) + \frac{6\tan(\pi/2K)}{(G_A+G_B)(\pi/2K)}\left(\frac{Q}{P}\right) + \left(\frac{Q}{P}\right) = 0$$



No load on gravity only column
$Q = 0$, $K = 2.0$

Equal loads on restraining and gravity only column
$Q/P = 1$, $K = 2.7$

Other combinations
$Q/P = 2$, $K = 3.25$

$Q/P = 10$, $K = 6.07$

What would you do if there were no load P?

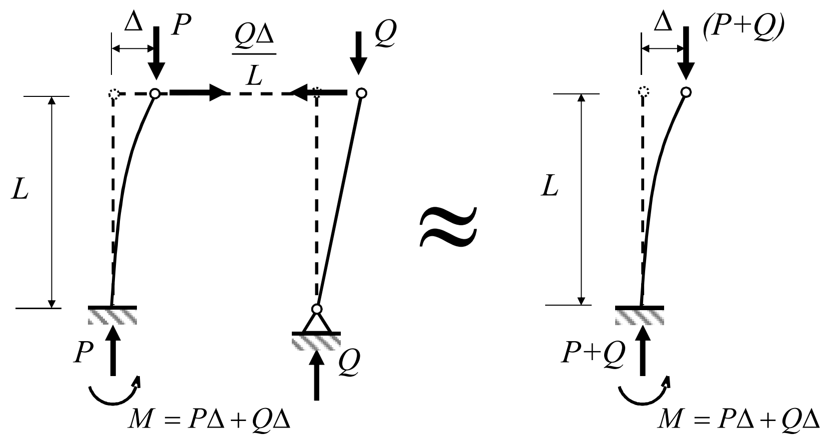
7.53

Buckling with Gravity Only Columns

- Develop a simplified approach for determination of *K* considering gravity only columns.
  - Either design the column for a larger load, or
  - Design the column with a larger *K*-factor

7.54

Buckling with Gravity Only Columns

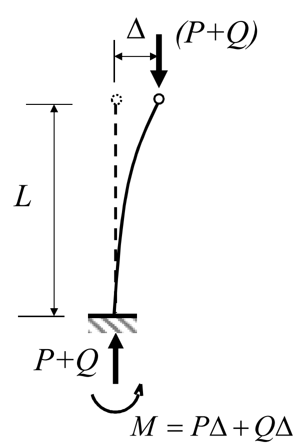


7.55

Buckling with Gravity Only Columns

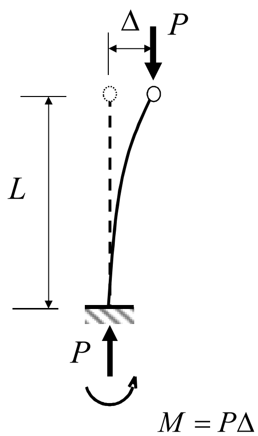
Design this column for the load $(P+Q)$ using the nomograph effective length factor, K_o.

$$\left(P+Q\right)=\frac{\pi^2 EI}{\left(K_o L\right)^2}$$



7.56

Buckling with Gravity Only Columns



Or, design this column for the load P using the modified nomograph effective length factor, K_n.

$$P = \frac{\pi^2 EI}{(K_n L)^2}$$

7.57

Buckling with Gravity Only Columns

- Solve both equations for $\frac{\pi^2 EI}{L^2}$

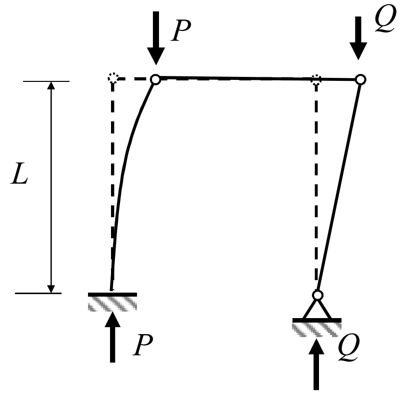
$$\frac{\pi^2 EI}{L^2} = K_o^2 (P+Q) \text{ and } \frac{\pi^2 EI}{L^2} = K_n^2 P$$

- Set equal and solve for K_n

$$K_n = K_o \sqrt{\frac{P+Q}{P}} = K_o \sqrt{1+\frac{Q}{P}}$$

7.58

Buckling with Gravity Only Columns



$$K_n = K_o\sqrt{1+\frac{Q}{P}}$$

No load on gravity only column
$Q = 0, K = 2.0$

Equal loads on restraining and gravity only column
$Q/P = 1, K = 2.8$

Other combinations
$Q/P = 2, K = 3.46$

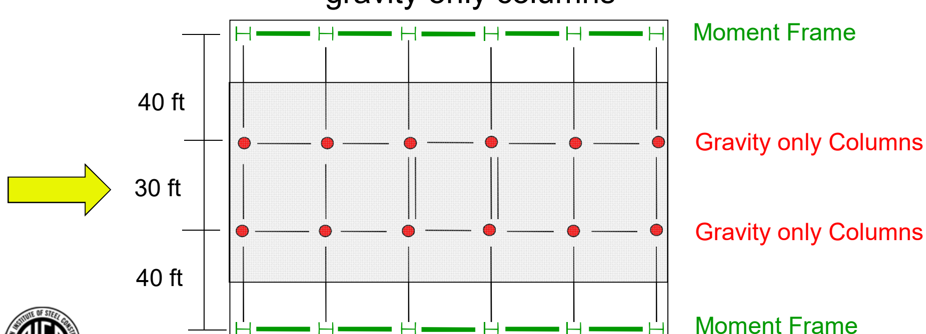
$Q/P = 10, K = 6.63$

What would you do if there were no load P?

7.59

Gravity only Columns

In the left to right direction, this structure shows 12 gravity only columns, 50%, but the relationship of loading based on tributary area is more like 64% on the gravity only columns



7.60

Example 1 (ASD)

Check a W14x120 exterior column as part of a moment frame with an effective length of 12.5 ft by the effective length method using the approximate first-order analysis of Appendix 8

Load Combination $= D + 0.75L + 0.75(0.6W)$

$$P_{nt} + P_{lt} = 378 + 46 = 424 \text{ kips}$$

$$M_{Ant} + M_{Alt} = 87.8 + 72.0 = 160 \text{ ft-kips}$$

$$M_{Bnt} + M_{Blt} = 43.9 + 36.1 = 80 \text{ ft-kips}$$

7.61

Example 1 (ASD)

Determine second-order moments

No translation, M_{nt}

Since the member end moments are both positive, the member bends in reverse curvature.

$$C_m = 0.6 - 0.4(M_1/M_2)$$

$$C_m = 0.6 - 0.4(43.9/87.8) = 0.4$$

7.62

Example 1 (ASD)

$$P_{e1} = \frac{\pi^2 EI}{\left(L_{c1}\right)^2}$$

$$P_{e1} = \frac{\pi^2 (29{,}000)(1380)}{\left(1.0\left(12.5\right)(12)\right)^2} = 17{,}550 \text{ kips}$$

7.63

Example 1 (ASD)

$$B_1 = \frac{C_m}{1 - \dfrac{\alpha P_r}{P_{e1}}} \geq 1.0$$

$$B_1 = \frac{0.4}{1 - \dfrac{1.6\left(424\right)}{17{,}550}} = 0.42 \not> 1.0 \therefore \; B_1 = 1.0$$

7.64

Example 1 (ASD)

Determine second-order moments

Translation, $M_{lt.}$

For design, the frame deflection will be limited to

$$\Delta_H = L/400$$

with $H = 150$ kips (service load to cause drift limit)

7.65

Example 1 (ASD)

- For a moment frame system where 36% of the gravity load is carried by the moment frames, as illustrated in an earlier framing plan,

$$\frac{P_{mf}}{P_{story}} = 0.36$$

$$R_M = 1 - 0.15(0.36) = 0.946$$

7.66

Example 1 (ASD)

thus,

$$P_{e\ story} = 0.946\frac{HL}{\Delta_H} = 0.946(150)(400) = 56,800 \text{ kips}$$

This is a measure of the frame sway buckling strength

7.67

Example 1 (ASD)

For the entire frame at this story

$$P_{story} = 2270 \text{ kips (total gravity load)}$$

- Sway amplification factor

$$B_2 = \frac{1}{1-\frac{\alpha P_{story}}{P_{e\ story}}} = \frac{1}{1-\frac{1.6(2270)}{56,800}} = 1.07$$

7.68

Example 1 (ASD)

Second-order moment

$$M_r = B_1 M_{nt} + B_2 M_{lt}$$

$$M_a = 1.0(87.8) + 1.07(72.0) = 165 \text{ ft-kips}$$

Second-order force

$$P_r = P_{nt} + B_2 P_{lt}$$

$$P_a = (378) + 1.07(46.0) = 427 \text{ kips}$$

7.69

Example 1 (ASD)

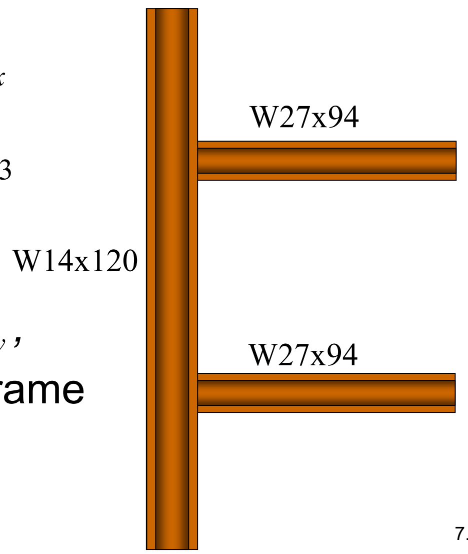
Effective length, K_x

$$G_T = G_B = \frac{2\left(\frac{1380}{12.5}\right)}{\left(\frac{3270}{30.0}\right)} = 2.03$$

$$K_x = 1.60$$

Effective length, K_y, assumed braced frame

$$K_y = 1.0$$



7.70

Example 1 (ASD)

- If the load on the gravity only columns is equal to 64% of the total load on all columns

$$K = K_o\sqrt{1+\frac{Q}{P}} = K_o\sqrt{1+\frac{\text{gravity only column load}}{\text{restraining column load}}}$$

$$= 1.60\sqrt{1+\frac{0.64(2270)}{0.36(2270)}} = 2.67$$

7.71

Example 1 (ASD)

- Consider impact of inelastic buckling

$$\frac{\alpha P_r}{P_{ns}} = \frac{1.6(427)}{50(35.3)} = 0.387 < 0.5$$

$$\tau_b = 1.0$$

The column is not behaving inelastically.
Thus, use the elastic effective length factor.

7.72

Example 1 (ASD)

Nominal compressive strength

In-plane effective length

$$\frac{L_{cx}}{r_x/r_y} = \frac{KL_x}{r_x/r_y} = \frac{2.67(12.5)}{1.67} = 20.0$$

$$L_{cy} = KL_y = 12.5$$

$$\frac{P_n}{\Omega_c} = 782 \text{ kips}$$

7.73

Example 1 (ASD)

Determine which interaction equation to use

$$\frac{P_a}{P_n/\Omega_c} = \frac{427}{782} = 0.55 > 0.2 \therefore \text{ use H1-1a}$$

$$\frac{P_a}{P_n/\Omega_c} + \frac{8}{9}\left(\frac{M_{ax}}{M_{nx}/\Omega_b}\right) \leq 1.0$$

7.74

Example 1 (ASD)

- Moment strength

$$\frac{M_n}{\Omega} = 529 \text{ ft-kips}$$

- Interaction Equation H1-1a

$$0.55 + \frac{8}{9}\left(\frac{165}{529}\right) = 0.83 \leq 1.0$$

7.75

Example 1 (ASD)

Thus, the W14x120, $F_y = 50$ will work for this loading combination

Now it should be checked for any other load combination, such as D + L plus the minimum lateral notional load

7.76

Example 1 (LRFD)

Check a W14x120 exterior column as part of a moment frame with an effective length of 12.5 ft by the effective length method

Load Combination $= 1.2D + 0.5L + 1.0W$

$$P_{nt} + P_{lt} = 408 + 98 = 506 \text{ kips}$$

$$M_{Ant} + M_{Alt} = 94.5 + 154.5 = 249 \text{ ft-kips}$$

$$M_{Bnt} + M_{Blt} = 47.3 + 77.7 = 125 \text{ ft-kips}$$

7.77

Example 1 (LRFD)

Determine second-order moments

No translation, M_{nt}

Since the member end moments are both positive, the member bends in reverse curvature.

$$C_m = 0.6 - 0.4\left(M_1/M_2\right)$$

$$C_m = 0.6 - 0.4\left(47.3/94.5\right) = 0.4$$

7.78

Example 1 (LRFD)

$$P_{e1} = \frac{\pi^2 EI}{(K_1 L)^2}$$

$$P_{e1} = \frac{\pi^2 (29,000)(1380)}{\left(1.0(12.5)(12)\right)^2} = 17,550 \text{ kips}$$

7.79

Example 1 (LRFD)

$$B_1 = \frac{C_m}{1 - \frac{\alpha P_r}{P_{e1}}} \geq 1.0$$

$$B_1 = \frac{0.4}{1 - \frac{506}{17,550}} = 0.41 \ngtr 1.0 \therefore\ B_1 = 1.0$$

7.80

Example 1 (LRFD)

Determine second-order moments

Translation, $M_{lt.}$

For design, the frame deflection will be limited to

$$\Delta_H = L/400$$

with $H = 150$ kips (service load to cause drift limit)

7.81

Example 1 (LRFD)

- For a moment frame system where 36% of the gravity load is carried by the moment frames, as illustrated in an earlier framing plan,

$$\frac{P_{mf}}{P_{story}} = 0.36$$

$$R_M = 1 - 0.15(0.36) = 0.946$$

7.82

Example 1 (LRFD)

thus,

$$P_{e\ story} = 0.946\frac{HL}{\Delta_H} = 0.946(150)(400) = 56{,}800 \text{ kips}$$

This is a measure of the frame sway buckling strength

7.83

Example 1 (LRFD)

For the entire frame at this story

$P_{story} = 2445$ kips (total gravity load)

- Sway amplification factor

$$B_2 = \frac{1}{1-\frac{\alpha P_{story}}{P_{e\ story}}} = \frac{1}{1-\frac{1.0(2445)}{56{,}800}} = 1.04$$

7.84

Example 1 (LRFD)

Second-order moment

$$M_r = B_1 M_{nt} + B_2 M_{lt}$$

$$M_u = 1.0(94.5) + 1.04(154.5) = 255 \text{ ft-kips}$$

Second-order force

$$P_r = P_{nt} + B_2 P_{lt}$$

$$P_u = (408) + 1.04(98) = 510 \text{ kips}$$

7.85

Example 1 (LRFD)

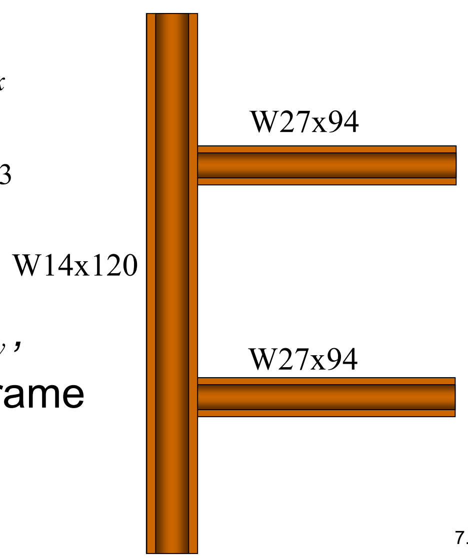
Effective length, K_x

$$G_T = G_B = \frac{2\left(\frac{1380}{12.5}\right)}{\left(\frac{3270}{30.0}\right)} = 2.03$$

$$K_x = 1.60$$

Effective length, K_y, assumed braced frame

$$K_y = 1.0$$



7.86

Example 1 (LRFD)

- Consider impact of inelastic buckling

$$\frac{\alpha P_r}{P_y} = \frac{1.0(510)}{50(35.3)} = 0.289 < 0.5$$

$$\tau_b = 1.0$$

The column is not behaving inelastically.
Thus, use the elastic effective length factor.

7.87

Example 1 (LRFD)

- If the load on the gravity only columns is equal to 64% of the total load on all columns

$$K = K_o\sqrt{1+\frac{Q}{P}} = K_o\sqrt{1+\frac{\text{gravity only column load}}{\text{restraining column load}}}$$

$$= 1.60\sqrt{1+\frac{0.64(2445)}{0.36(2445)}} = 2.67$$

7.88

Example 1 (LRFD)

Nominal compressive strength

In-plane effective length

$$\frac{KL_x}{r_x / r_y} = \frac{2.67(12.5)}{1.67} = 20.0$$ ☆

$$KL_y = 12.5 \qquad \phi_c P_n = 1180 \text{ kips}$$

7.89

Example 1 (LRFD)

Determine which interaction equation to use

$$\frac{P_u}{\phi_c P_n} = \frac{510}{1180} = 0.43 > 0.2 \therefore \text{ use H1-1a}$$

$$\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}}\right) \le 1.0$$

7.90

Example 1 (LRFD)

- Moment strength

$$\phi M_n = 795 \text{ ft-kips}$$

- Interaction Equation H1-1a

$$0.43 + \frac{8}{9}\left(\frac{255}{795}\right) = 0.72 \leq 1.0$$

7.91

Example 1 (LRFD)

Thus, the W14x120, $F_y = 50$ will work for this loading combination.

Now it should be checked for any other load combination, such as 1.2D+1.6L plus the minimum lateral notional load

7.92

Effective Length vs. Actual Length

- 1961 AISC *Specification* introduced "effective length" concept.
- 1963 AISC *Specification* introduced "effective length factor," K.
- 2005 AISC *Specification* introduced the concept of using $K = 1$ for moment frames.
  - This is the Direct Analysis Method
- 2016 AISC *Specification* replaced KL with L_c

7.93

Direct Analysis

- There are no limitations on the use of the Direct Analysis method.
- C2. Calculation of Required Strengths
  - Must satisfy the general stability requirements of Section C2.1.
  - For out-of-plumbness use notional loads or model out-of-plumbness.
  - Structure stiffness must be reduced.

7.94

Stiffness Reduction

- C2.3 Adjustments to Stiffness
  a) reduce all stiffnesses by 0.80
  b) reduce all flexural stiffnesses, if they contribute to stability, by τ_b

The adjustment given in (a) is the unique part of design by the Direct Analysis method and what permits the use of *K*=1.0 for all members

7.95

Stiffness Reduction

Thus,

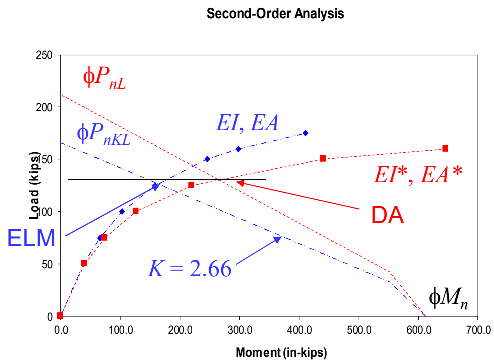
$$EI^* = 0.8\tau_b EI$$

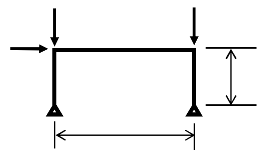
$$EA^* = 0.8EA$$

b) Could use $\tau_b = 1.0$ and apply a notional load instead but this has some drawbacks.

7.96

Effective Length vs. Actual Length





For the effective length method use EI and AE and ϕP_{nKL}, ϕM_n.

For the direct analysis method, use a reduced stiffness EI^* and EA^* and ϕP_{nL}, ϕM_n.

7.97

Direct Analysis

- C3. Calculation of Available Strengths
  - Follow provisions of Chapters D through K with no further consideration of overall structure stability.
  - Take the effective length factor, $K = 1.0$, unless a smaller value can be justified by rational analysis. This means that $L_c = L$.

7.98

Direct Analysis

- Design process
  - Perform second-order analysis
    - Use reduced stiffness, EI^* and EA^*
  - Use nominal geometry or out-of-plumb geometry
    - If using nominal geometry.
      - Apply notional loads, $N_i = 0.002\alpha Y_i$
      - As a minimum lateral load if $\Delta_{2nd-order}/\Delta_{1st-order} \leq 1.7$
      - As an additional lateral load if $\Delta_{2nd-order}/\Delta_{1st-order} > 1.7$
  - Design members using $K=1$ for compression

7.99

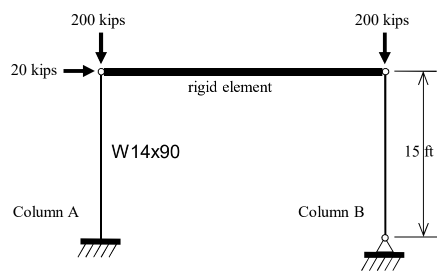
Direct Analysis

- We will concentrate on what to do for the analysis part.
  - Reduced stiffness
  - Notional load
  - Approximate second-order analysis
- Design is as you would design any column.

7.100

Example 2 (LRFD)

- Design by Direct Analysis



$P_D = 75.0$ kips
$P_L = 220$ kips
$W = 20.0$ kips

$$1.2D + 0.5L + 1.0W$$

7.101

Example 2 (LRFD)

- Design by Direct Analysis
  - Notional load

$$Y_i = (200 + 200) = 400 \text{ kips}$$

$$N_i = 0.002(1.0)(400) = 0.8 \text{ kips}$$

  - Assume

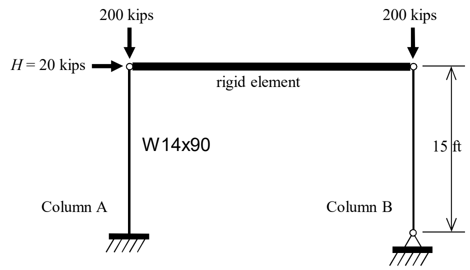
$$B_2 \leq 1.7$$

  - Therefore, since notional load is less than applied lateral load, there is no need to add notional load

7.102

Example 2 (LRFD)

- First-order elastic analysis with reduced stiffness



$P_{nt} = 200$ kips

$P_{lt} = 0$ kips

$M_{ntx} = 0$ ft-kips

$M_{ltx} = 300$ ft-kips

$K_x = 1.0$

$K_y = 1.0$

$L_b = 15$ ft

$$\Delta_{1st} = \frac{Hl^3}{3EI^*} = \frac{20(15)^3(1728)}{3(0.8)(1.0)(29{,}000)(999)} = 1.68 \text{ in.}$$

These are the values that make it the Direct Analysis method.

7.103

Example 2 (LRFD)

- Stiffness reduction $\tau_b = 1.0$ for $\frac{\alpha P_r}{P_{ns}} \le 0.5$

$$= 4\left[\frac{\alpha P_r}{P_{ns}}\left(1 - \frac{\alpha P_r}{P_{ns}}\right)\right] \text{ for } \frac{\alpha P_r}{P_{ns}} > 0.5$$

$$\frac{\alpha P_r}{P_{ns}} = \frac{1.0(200)}{(50 \text{ ksi})(26.5 \text{ in.}^2)} = 0.15 < 0.5$$

Thus

$$\tau_b = 1.0$$

No need to recalculate Δ

7.104

Example 2 (LRFD)

- Design by Direct Analysis
  - Amplify first-order analysis; Appendix 8 member effect.
    - Since there are no moments in this example without lateral displacement, there is no need to calculate B_1

7.105

Example 2 (LRFD)

- Design by Direct Analysis
  - Amplify first-order analysis; Appendix 8 sway effect

$$B_2 = \frac{1}{1 - \dfrac{\alpha P_{story}}{P_{e\ story}}} \geq 1.0$$

$$P_{story} = 400 \text{ kips}$$

$$H = 20 \text{ kips}$$

$$\Delta_H = 1.68 \text{ in.}$$

Must be matched (H and Δ_H)

Remember also that this sway is with reduced stiffness

7.106

Example 2 (LRFD)

- Design by Direct Analysis
  - Amplify first-order analysis; Appendix 8 sway effect

$$R_M = 1 - 0.15\left(\frac{P_{mf}}{P_{story}}\right) = 1 - 0.15\left(\frac{200}{400}\right) = 0.925$$

$$P_{e\ story} = R_M \frac{HL}{\Delta_H} = 0.925 \frac{(20)(15.0(12))}{1.68} = 1{,}980 \text{ kips}$$

7.107

Example 2 (LRFD)

- Design by Direct Analysis
  - Amplify first-order analysis; Appendix 8 sway effect

$$B_2 = \frac{1}{1 - \frac{(1.0)(400)}{1{,}980}} = 1.25$$

When using the reduced stiffness, the limit on B_2 for application of notional loads as minimum loads is 1.7.

There are no restrictions on this method

7.108

Example 2 (LRFD)

- Second-order moment

$$M_r = B_1 M_{nt} + B_2 M_{lt} \qquad \text{(A-8-1)}$$

$$M_u = (0.0) + 1.25(300) = 375 \text{ ft-kips}$$

- Second-order force

$$P_r = P_{nt} + B_2 P_{lt} \qquad \text{(A-8-2)}$$

$$P_u = (200) + 1.25(0.0) = 200 \text{ kips}$$

7.109

Example 2 (LRFD)

- Determine member strength

$L_{cx} = K_x L = 15.0$ ft $\qquad \phi_c P_n = 1000$ kips

$L_{cy} = K_y L = 15.0$ ft

$L_b = 15.0$ ft $\qquad \phi_b M_n = 574$ ft-kips

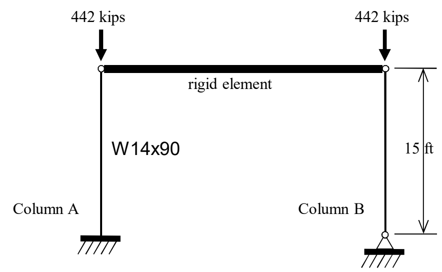
- Interaction Eq. H1-1a

$$\frac{200}{1,000} + \frac{8}{9}\left(\frac{375}{574}\right) = 0.781 < 1.0 \ \therefore \text{ ok}$$

7.110

Example 3 (LRFD)

- Design by Direct Analysis



$P_D = 75.0$ kips
$P_L = 220$ kips
$W = 20.0$ kips

$$1.2D + 1.6L$$

7.111

Example 3 (LRFD)

- Design by Direct Analysis
  - Notional load

$$Y_i = (442 + 442) = 884 \text{ kips}$$

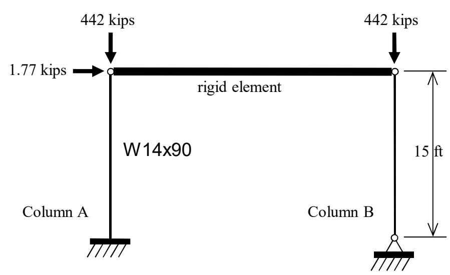
$$N_i = 0.002(1.0)(884) = 1.77 \text{ kips}$$

  - Since there is no lateral load, the notional load must be applied, regardless of the magnitude of B_2.

7.112

Example 3 (LRFD)

- First-order elastic analysis with reduced stiffness



$P_{nt} = 442$ kips

$P_{lt} = 0$ kips

$M_{ntx} = 0$ ft-kips

$M_{ltx} = 1.77(15) = 26.6$ ft-kips

$K_x = 1.0$

$K_y = 1.0$

$L_b = 15$ ft

$$\Delta_{1st} = \frac{Hl^3}{3EI^*} = \frac{1.77(15)^3(1,728)}{3(0.8)(1.0)(29,000)(999)} = 0.148 \text{ in.}$$

7.113

Example 3 (LRFD)

- Stiffness reduction $\tau_b = 1.0$ for $\dfrac{\alpha P_r}{P_{ns}} \leq 0.5$

$$= 4\left[\frac{\alpha P_r}{P_{ns}}\left(1 - \frac{\alpha P_r}{P_{ns}}\right)\right] \text{ for } \frac{\alpha P_r}{P_{ns}} > 0.5$$

$$\frac{\alpha P_r}{P_{ns}} = \frac{1.0(442)}{(50 \text{ ksi})(26.5 \text{ in.}^2)} = 0.33 < 0.5$$

Thus

$$\tau_b = 1.0$$

No need to recalculate Δ

7.114

Example 3 (LRFD)

- Design by Direct Analysis
  – Amplify first-order analysis; member effect.
    - Again, no moment without lateral displacement so no need to use B_1.

7.115

Example 3 (LRFD)

- Design by Direct Analysis
  – Amplify first-order analysis; structure effect

$$B_2 = \frac{1}{1 - \frac{\alpha P_{story}}{P_{e\ story}}} \geq 1.0$$

$$P_{story} = 884 \text{ kips}$$

$$H = 1.77 \text{ kips}$$

$$\Delta_H = 0.148 \text{ in.}$$

Must be matched (H and Δ_H)

7.116

Example 3 (LRFD)

- Design by Direct Analysis
  - Amplify first-order analysis; structure effect

$$R_M = 1 - 0.15\left(\frac{P_{mf}}{P_{story}}\right) = 1 - 0.15\left(\frac{442}{884}\right) = 0.925$$

$$P_{e\ story} = R_M \frac{HL}{\Delta_H} = 0.925 \frac{(1.77)(15.0(12))}{0.148} = 1,990 \text{ kips}$$

Difference from DLW load combination is due to rounding of displacement, it was 1,980.

7.117

Example 3 (LRFD)

- Design by Direct Analysis
  - Amplify first-order analysis; structure effect

$$B_2 = \frac{1}{1 - \frac{(1.0)(884)}{1,990}} = 1.80$$

When using the reduced stiffness, the limit on B_2 for application of notional loads as minimum loads is 1.7.

We have already included the notional load. Note also that B_2 is larger than previously, due to increased gravity load.

7.118

Example 3 (LRFD)

- Second-order moment

$$M_r = B_1 M_{nt} + B_2 M_{lt} \qquad \text{(A-8-1)}$$

$$M_u = (0.0) + 1.80(26.6) = 47.9 \text{ ft-kips}$$

- Second-order force

$$P_r = P_{nt} + B_2 P_{lt} \qquad \text{(A-8-2)}$$

$$P_u = (442) + 1.80(0.0) = 442 \text{ kips}$$

7.119

Example 3 (LRFD)

- Determine member strength

$L_{cx} = K_x L = 15.0$ ft $\phi_c P_n = 1{,}000$ kips

$L_{cy} = K_y L = 15.0$ ft

$L_b = 15.0$ ft $\phi_b M_n = 574$ ft-kips

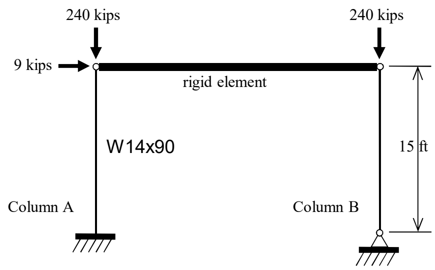
- Interaction Eq. H1-1a

$$\frac{442}{1{,}000} + \frac{8}{9}\left(\frac{47.9}{574}\right) = 0.516 < 1.0 \;\; \therefore \text{ ok}$$

7.120

Example 2 (ASD)

- Design by Direct Analysis



$P_D = 75.0$ kips

$P_L = 220$ kips

$W = 20.0$ kips

$$D + 0.75L + 0.75(0.6W)$$

7.121

Example 2 (ASD)

- Design by Direct Analysis
  - Notional load

$$Y_i = (240 + 240) = 480 \text{ kips}$$

$$N_i = 0.002(1.0)(480) = 0.96 \text{ kips}$$

Although α is in the equation for notional load. It is not used unless a rigorous second-order analysis is conducted.

  - Assume

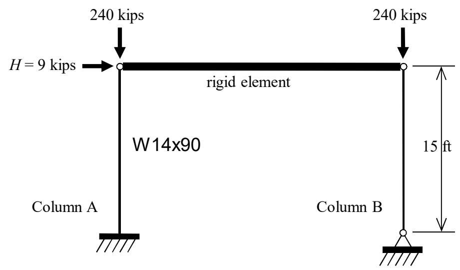
$$B_2 \leq 1.7$$

  - Therefore, since notional load is less than applied lateral load, there is no need to add notional load

7.122

Example 2 (ASD)

- First-order elastic analysis with reduced stiffness



$P_{nt} = 240$ kips

$P_{lt} = 0$ kips

$M_{ntx} = 0$ ft-kips

$M_{ltx} = 135$ ft-kips

$K_x = 1.0$

$K_y = 1.0$

$L_b = 15$ ft

$$\Delta_{1st} = \frac{Hl^3}{3EI^*} = \frac{9(15)^3(1728)}{3(0.8)(1.0)(29,000)(999)} = 0.755 \text{ in.}$$

7.123

Example 2 (ASD)

- Stiffness reduction $\tau_b = 1.0$ for $\frac{\alpha P_r}{P_{ns}} \le 0.5$

$$= 4\left[\frac{\alpha P_r}{P_{ns}}\left(1 - \frac{\alpha P_r}{P_{ns}}\right)\right] \text{ for } \frac{\alpha P_r}{P_{ns}} > 0.5$$

$$\frac{\alpha P_r}{P_{ns}} = \frac{1.6(240)}{(50 \text{ ksi})(26.5 \text{ in.}^2)} = 0.29 < 0.5$$

Thus

$$\tau_b = 1.0$$

No need to recalculate Δ

7.124

Example 2 (ASD)

- Design by Direct Analysis
  - Amplify first-order analysis; Appendix 8 member effect.
    - Since there are no moments in this example without lateral displacement, there is no need to calculate B_1.

7.125

Example 2 (ASD)

- Design by Direct Analysis
  - Amplify first-order analysis; Appendix 8 sway effect

$$B_2 = \frac{1}{1 - \frac{\alpha P_{story}}{P_{e\ story}}} \geq 1.0$$

$$P_{story} = 480 \text{ kips}$$

$$H = 9 \text{ kips}$$

$$\Delta_H = 0.755 \text{ in.}$$

Must be matched (H and Δ_H)

7.126

Example 2 (ASD)

- Design by Direct Analysis
  - Amplify first-order analysis; Appendix 8 sway effect

$$R_M = 1 - 0.15\left(\frac{P_{mf}}{P_{story}}\right) = 1 - 0.15\left(\frac{240}{480}\right) = 0.925$$

$$P_{e\ story} = R_M \frac{HL}{\Delta_H} = 0.925 \frac{(9)(15.0(12))}{0.755} = 1,980 \text{ kips}$$

Note that this is the same as determined for the LRFD solution.

7.127

Example 2 (ASD)

- Design by Direct Analysis
  - Amplify first-order analysis; Appendix 8 sway effect

$$B_2 = \frac{1}{1 - \frac{(1.6)(480)}{1,980}} = 1.63$$

When using the reduced stiffness, the limit on B_2 for application of notional loads as minimum loads is 1.7.

There are no restrictions on this method

7.128

Example 2 (ASD)

- Second-order moment

$$M_r = B_1 M_{nt} + B_2 M_{lt} \qquad \text{(A-8-1)}$$

$$M_u = (0.0) + 1.63(135) = 220 \text{ ft-kips}$$

- Second-order force

$$P_r = P_{nt} + B_2 P_{lt} \qquad \text{(A-8-2)}$$

$$P_u = (240) + 1.63(0.0) = 240 \text{ kips}$$

7.129

Example 2 (ASD)

- Determine member strength

$L_{cx} = K_x L = 15.0$ ft

$L_{cy} = K_y L = 15.0$ ft

$L_b = 15.0$ ft

$P_n / \Omega_c = 667$ kips

$M_n / \Omega_b = 382$ ft-kips

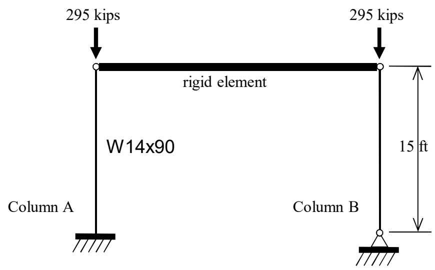
- Interaction Eq. H1-1a

$$\frac{240}{667} + \frac{8}{9}\left(\frac{220}{382}\right) = 0.872 < 1.0 \quad \therefore \text{ ok}$$

7.130

Example 3 (ASD)

- Design by Direct Analysis



$P_D = 75.0$ kips
$P_L = 220$ kips
$W = 20.0$ kips

$D + L$

7.131

Example 3 (ASD)

- Design by Direct Analysis
  - Notional load

$$Y_i = (295 + 295) = 590 \text{ kips}$$
$$N_i = 0.002(1.0)(590) = 1.18 \text{ kips}$$

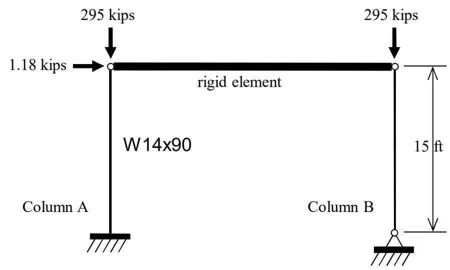
Although α is in the equation for notional load. It is not used unless a rigorous second-order analysis is conducted.

  - Since there is no lateral load, the notional load must be applied, regardless of the magnitude of B_2.

7.132

Example 3 (ASD)

- First-order elastic analysis with reduced stiffness



$P_{nt} = 295$ kips

$P_{lt} = 0$ kips

$M_{ntx} = 0$ ft-kips

$M_{ltx} = 1.18(15) = 17.7$ ft-kips

$K_x = 1.0$

$K_y = 1.0$

$L_b = 15$ ft

$$\Delta_{1st} = \frac{Hl^3}{3EI^*} = \frac{1.18(15)^3(1{,}728)}{3(0.8)(1.0)(29{,}000)(999)} = 0.099 \text{ in.}$$

7.133

Example 3 (ASD)

- Stiffness reduction $\tau_b = 1.0$ for $\frac{\alpha P_r}{P_{ns}} \le 0.5$

$$= 4\left[\frac{\alpha P_r}{P_{ns}}\left(1 - \frac{\alpha P_r}{P_{ns}}\right)\right] \text{ for } \frac{\alpha P_r}{P_{ns}} > 0.5$$

$$\frac{\alpha P_r}{P_{ns}} = \frac{1.6(295)}{(50 \text{ ksi})(26.5 \text{ in.}^2)} = 0.36 < 0.5$$

Thus

$$\tau_b = 1.0$$

No need to recalculate Δ

7.134

Example 3 (ASD)

- Design by Direct Analysis
  - Amplify first-order analysis; member effect.
    - Again, no moment without lateral displacement so no need to calculate B_1.

7.135

Example 3 (ASD)

- Design by Direct Analysis
  - Amplify first-order analysis; structure effect

$$B_2 = \frac{1}{1 - \frac{\alpha P_{story}}{P_{e\ story}}} \geq 1.0$$

$$P_{story} = 590 \text{ kips}$$

$$H = 1.18 \text{ kips}$$

$$\Delta_H = 0.099 \text{ in.}$$

Must be matched (H and Δ_H)

7.136

Example 3 (ASD)

- Design by Direct Analysis
  - Amplify first-order analysis; structure effect

$$R_M = 1 - 0.15\left(\frac{P_{mf}}{P_{story}}\right) = 1 - 0.15\left(\frac{295}{590}\right) = 0.925$$

$$P_{e\ story} = R_M \frac{HL}{\Delta_H} = 0.925 \frac{(1.18)(15.0(12))}{0.099} = 1{,}980 \text{ kips}$$

Note that this is the same as determined for the LRFD solution.

7.137

Example 3 (ASD)

- Design by Direct Analysis
  - Amplify first-order analysis; structure effect

$$B_2 = \frac{1}{1 - \frac{(1.6)(590)}{1{,}980}} = 1.91$$

When using the reduced stiffness, the limit on B_2 for application of notional loads as minimum loads is 1.7.

We have already included the notional load.

7.138

Example 3 (ASD)

- Second-order moment

$$M_r = B_1 M_{nt} + B_2 M_{lt} \qquad \text{(A-8-1)}$$

$$M_u = (0.0) + 1.91(17.7) = 33.8 \text{ ft-kips}$$

- Second-order force

$$P_r = P_{nt} + B_2 P_{lt} \qquad \text{(A-8-2)}$$

$$P_u = (295) + 1.91(0.0) = 295 \text{ kips}$$

7.139

Example 3 (ASD)

- Determine member strength

$L_{cx} = K_x L = 15.0$ ft

$L_{cy} = K_y L = 15.0$ ft

$L_b = 15.0$ ft

$P_n/\Omega_c = 667$ kips

$M_n/\Omega_b = 382$ ft-kips

- Interaction Eq. H1-1a

$$\frac{295}{667} + \frac{8}{9}\left(\frac{33.8}{382}\right) = 0.521 < 1.0 \;\therefore \text{ ok}$$

7.140

Summary

- Considered the effective length method of analysis and design.
- Discussed the problems associated with determining the effective length.
- Introduced the direct analysis method.
- Used the stiffness reduction which permits the use of an effective length factor equal to one.
- Illustrated application of the notional load to account for out-of-plumbness.

7.141

Lesson 8

- The last of our lessons will address the design of composite flexural members
- Composite beams and filled and encased composite flexural members will be investigated.
- We will concentrate on Chapter I of the Specification and Part 3 of the Manual

7.142

Thank You

American Institute of Steel Construction
130 East Randolph, Suite 2000
Chicago, IL 60601

7.143

Individual Session Registrants

PDH Certificates

- 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 Session Registrants

PDH Certificates

- 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

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 Thursday. It will contain a link to access the quiz. EMAIL COMES FROM NIGHTSCHOOL@AISC.ORG.

Quiz and attendance records

Posted Thursday mornings. www.aisc.org/nightschool -- Click on Current Course Details.

Reasons for quiz

- EEU – You must take all quizzes and the final exam to receive EEU.
- PDHs – If you watch a recorded session, you must pass quiz for PDHs.
- REINFORCEMENT – Reinforce what you learn tonight. Get more out of the course.

Note: If you attend the live presentation, you do not have to take the quizzes to receive PDHs

8-Session Registrants

Access to the recording

Information for accessing the recording will be emailed to you by Thursday. The recording will be available for four weeks. (For 8-session registrants only.) EMAIL COMES FROM NIGHTSCHOOL@AISC.ORG.

PDHs via recording

If you watch a recorded session, you must take *and pass* the quiz for PDHs.

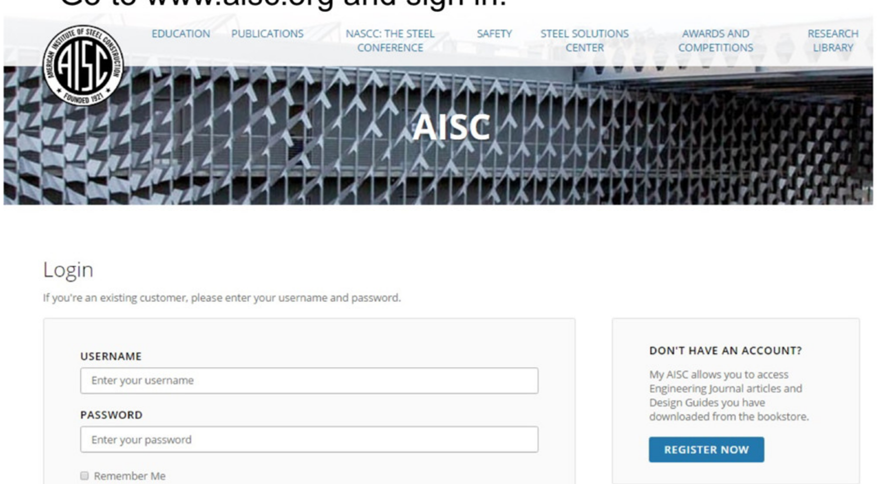
8-Session Registrants

Night School Resources

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.



Night School Resources for 8-session package Registrants

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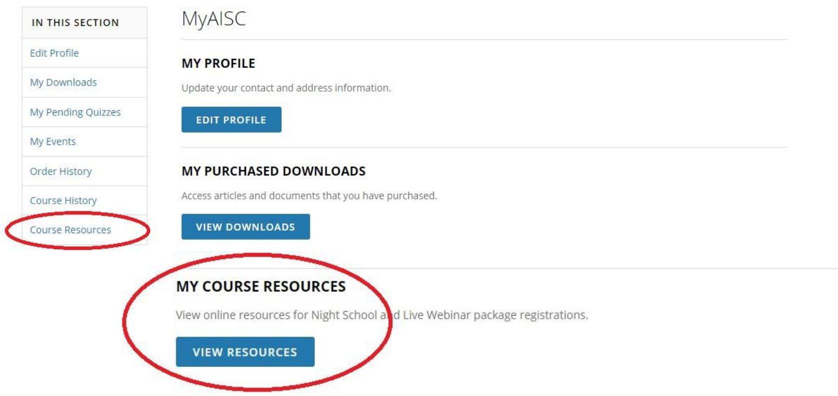

Please fill out our brief survey at the conclusion of the webinar. We greatly appreciate your feedback.

AISC | Thank you

Smarter.
Stronger.
Steel.